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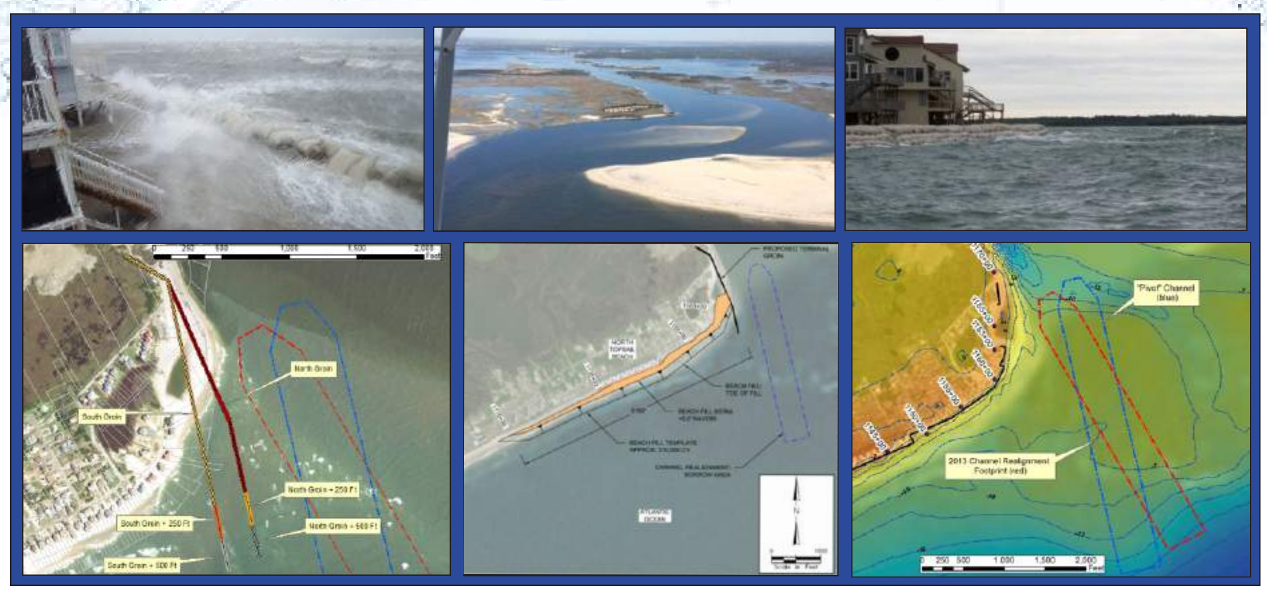
The Town of
North Topsail Beach

Onslow County

New River Inlet Management Master Plan Engineering and Modeling Report

New River Inlet
Onslow County, North Carolina



Submitted to:

U.S. Army Corps of Engineers
Wilmington District, Regulatory Branch
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1.0 INTRODUCTION

This document presents the alternatives and modeling analysis for an inlet master plan project that includes shore protection and navigation components at New River Inlet (NRI). The Town of North Topsail Beach (also referred to herein as the Town) is positioned to the southwest of NRI, with Onslow Beach to the northeast. Both North Topsail Beach and Onslow Beach are located within Onslow County, North Carolina (Figure 1-1).

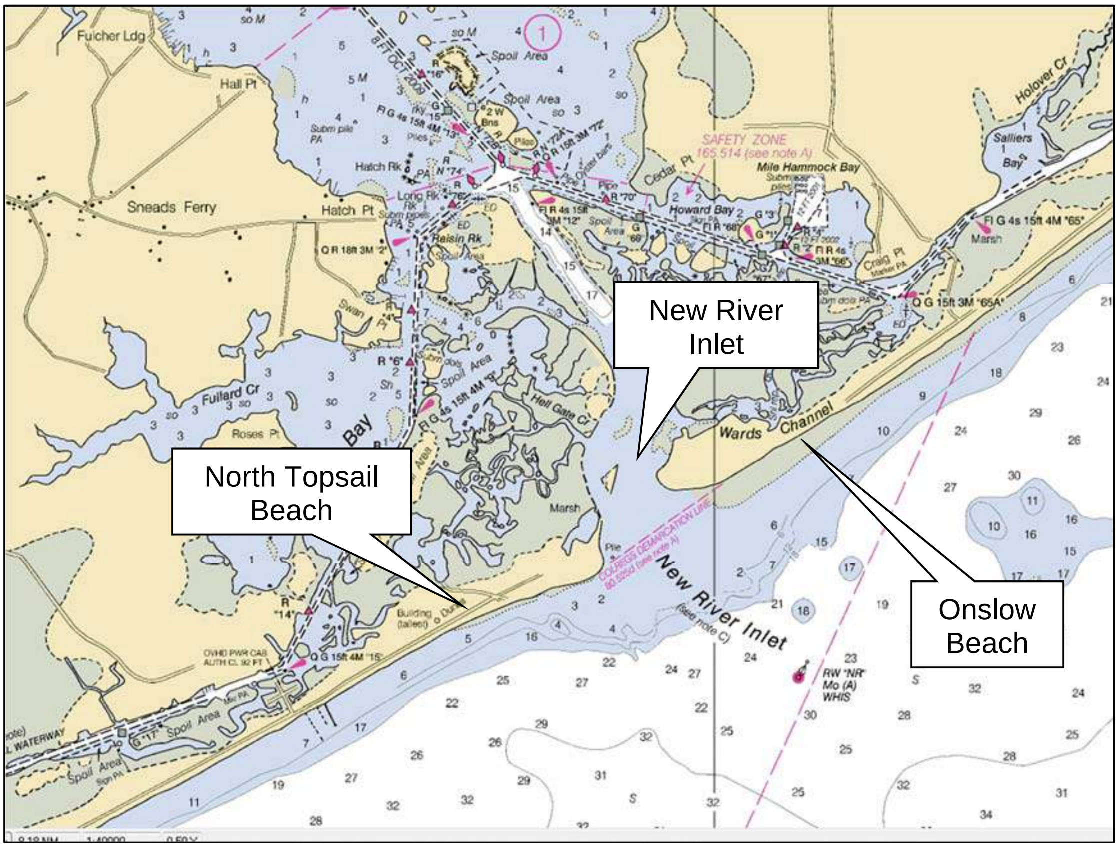


Figure 1-1. Project Location Map of North Topsail Beach, Onslow Beach, and New River Inlet, NC (NOAA Chart 11542)

The northeast end of North Topsail Beach (NTB) continues to experience consistent, relatively severe erosional conditions. Figures 1-2 and 1-3 present 2011 North Carolina Division of Coastal Management (DCM) long-term erosion rate maps of NTB and Onslow Beach (these maps were developed in 2011 and effective in 2013).

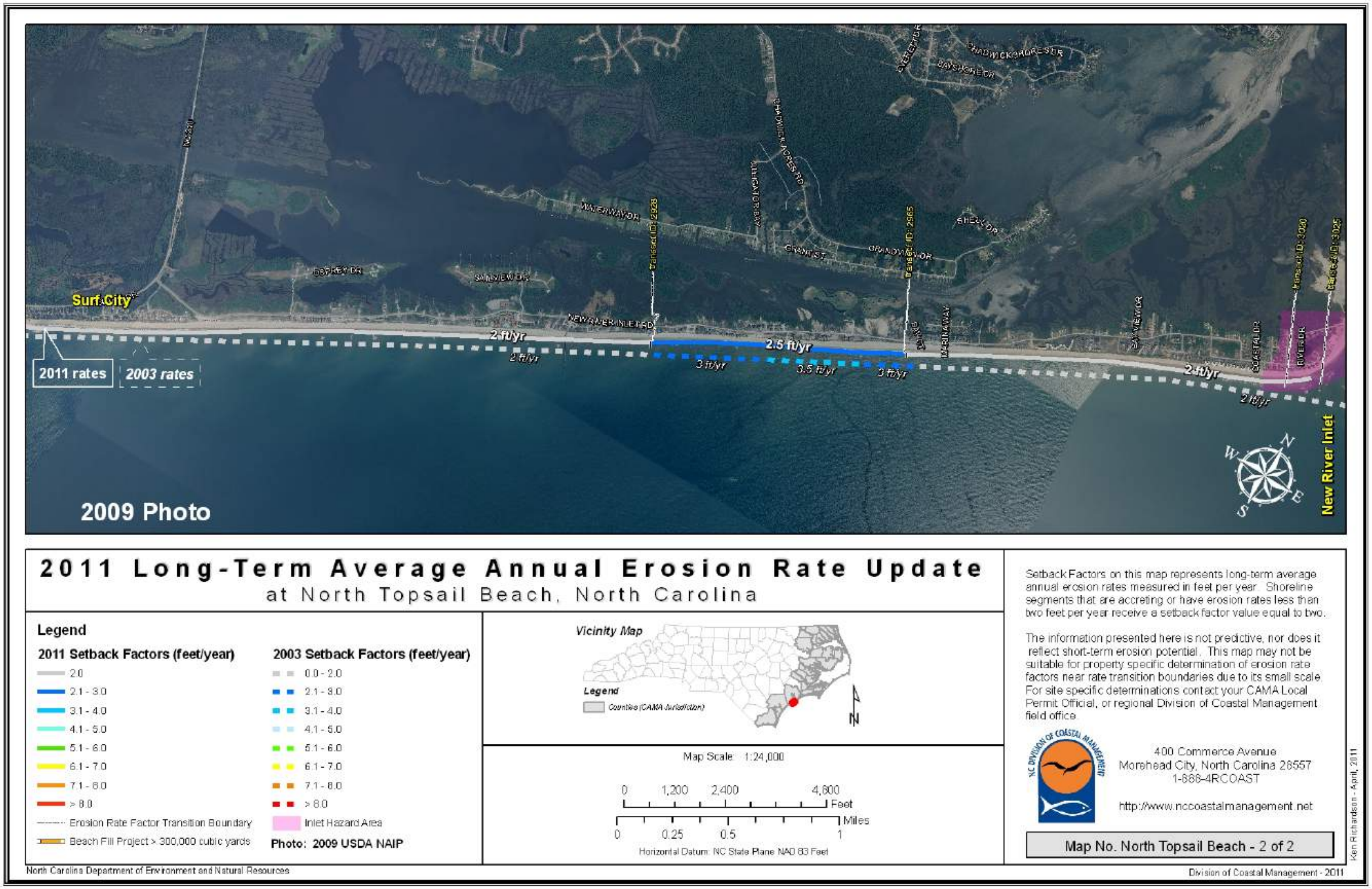


Figure 1-2. 2011 North Carolina Division of Coastal Management Long-Term Erosion Rate Map of North Topsail Beach

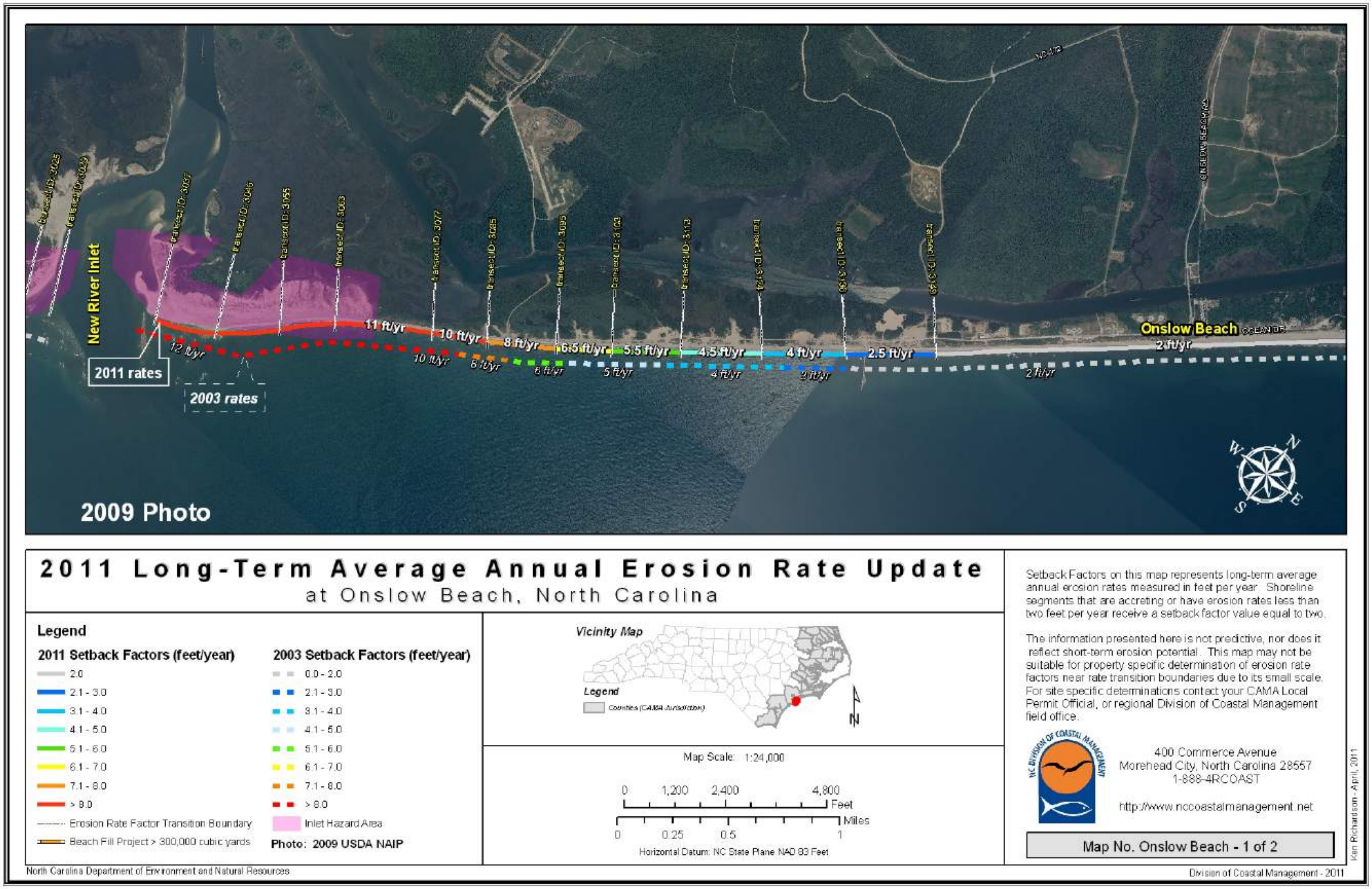


Figure 1-3. 2011 North Carolina Division of Coastal Management Long-Term Erosion Rate Map of Onslow Beach

DCM's official long-term erosion rates are very low for the NTB shoreline adjacent to NRI, whereas they are -11 feet per year (ft/yr) on the Onslow Beach side. However, note that DCM states "The area of North Topsail adjacent to New River Inlet is experiencing the highest erosion" on the entire Island of Topsail Beach (which includes NTB, Surf City and Topsail Beach). Figure 1-4 presents DCM's long-term shoreline analysis, where erosion is much higher along the project area shoreline. While the southwestern inlet-influenced portion of Onslow Beach is also erosional, Camp Lejeune owns this undeveloped reach of shoreline.

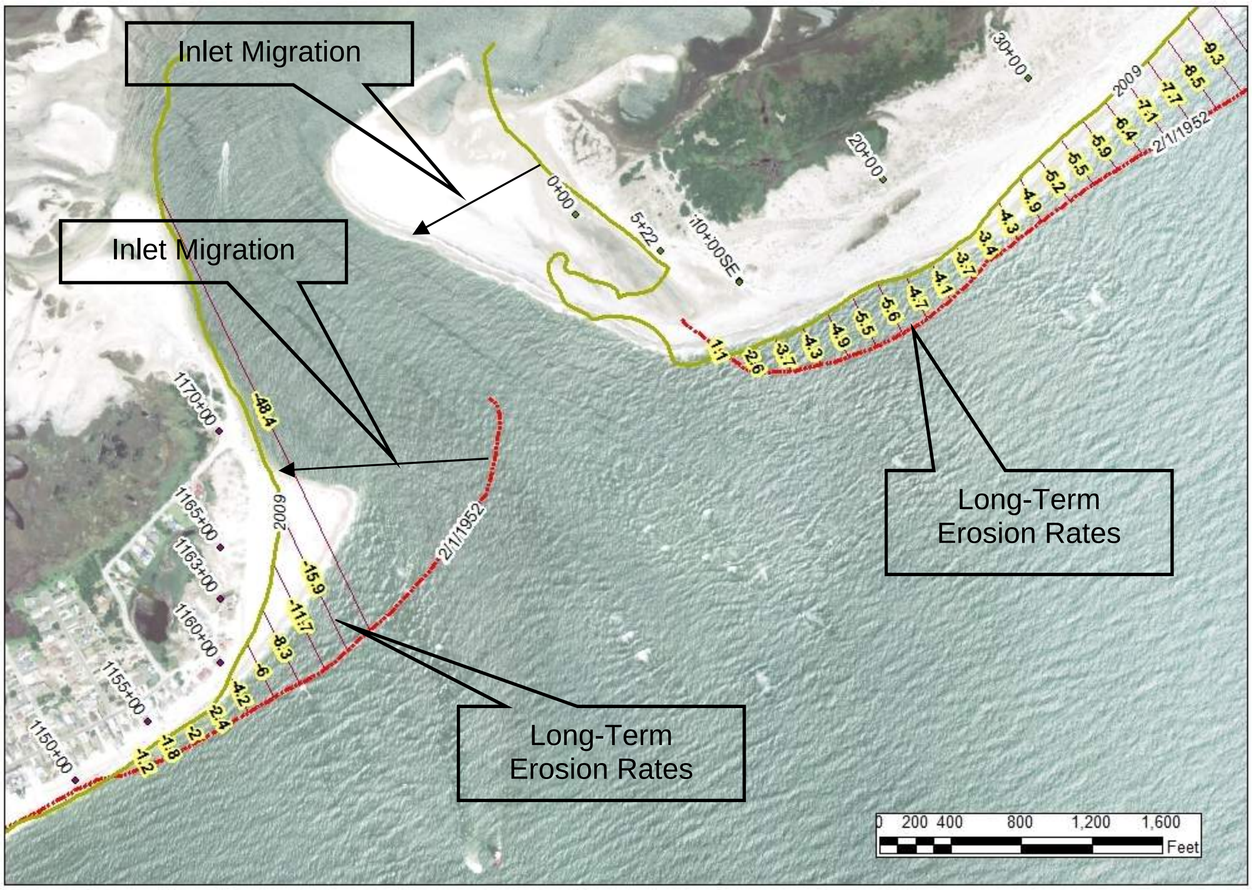


Figure 1-4. DCM Shoreline Erosion Analysis (ft/yr) along the NRI Shoreline Indicates Higher Erosion Rates, However These Higher Rates are not Included in Official Setback Factor (SBF) Rates along the NTB Shoreline. 2009 and 1952 Shorelines are shown on a 2016 Aerial.

Long-term erosion along the northern end of North Topsail Beach and along the inlet shoreline is generally between 10 and 15 ft/yr. Short-term erosion can be much more extreme. For example, erosion rates seaward of Topsail Reef Condos in 2013 and 2014 exceeded 100 ft/yr.

Dune breaching, overwashing and flooding has occurred relatively frequently along the northeastern end of NTB for the last few years. These events have required the installation of approximately 3,600 feet of sandbag revetment to prevent the loss of homes and infrastructure (Figure 1-5).

Figure 1-5. Geotube Sandbag Revetment (black solid line) along approximately 2,000 feet of NTB Shoreline

The geotube sandbag revetment requires constant maintenance (restacking, refilling, etc.) to protect more than 35 structures, including 8 large condominium structures (i.e., Topsail Reef Condos at 24 units per building) and at least 10 duplexes.

Figure 1-6 shows overtopping of the sandbag revetment during Hurricane Matthew. Figures 1-7 and 1-8 are recent photos of the sandbag revetment, which is currently in need of restacking in some areas.

Figure 1-6. Hurricane Matthew Sandbag Revetment Overtopping

Figure 1-7. March 1, 2018 Photo of The Sandbag Revetment Weak Spot

Figure 1-8. Sandbag Revetment Looking Southeast (March 1, 2018 Photo). No Dune Currently Exists in this Area.

A recent nourishment by the Town attempted to relieve this erosion; however, the intermittent fill placement provided only a short-term benefit for the northeast end. A more long-term solution is required to help reduce the large fluctuations that occur along the shoulder of the NRI.

In addition to shoreline management, another important element of this inlet management project is improving NRI navigation. NRI is notoriously difficult to navigate under even mild weather conditions. The U.S. Coast Guard (USCG) Coast Pilot states that navigating NRI "*is considered dangerous by local pilots, and entrance should not be attempted except under the most favorable conditions*." Navigation buoys are consistently removed from the inlet and shoaling conditions are ever-changing. Potential improvements to navigation include dredging and hardened structures. NRI connects the Atlantic Ocean to the New River Estuary (NRE), as well as the Atlantic Intracoastal Waterway (AIWW) (Figure 1-9). NRI is the primary access to the Atlantic Ocean for a large portion of Onslow County recreational and commercial boaters.

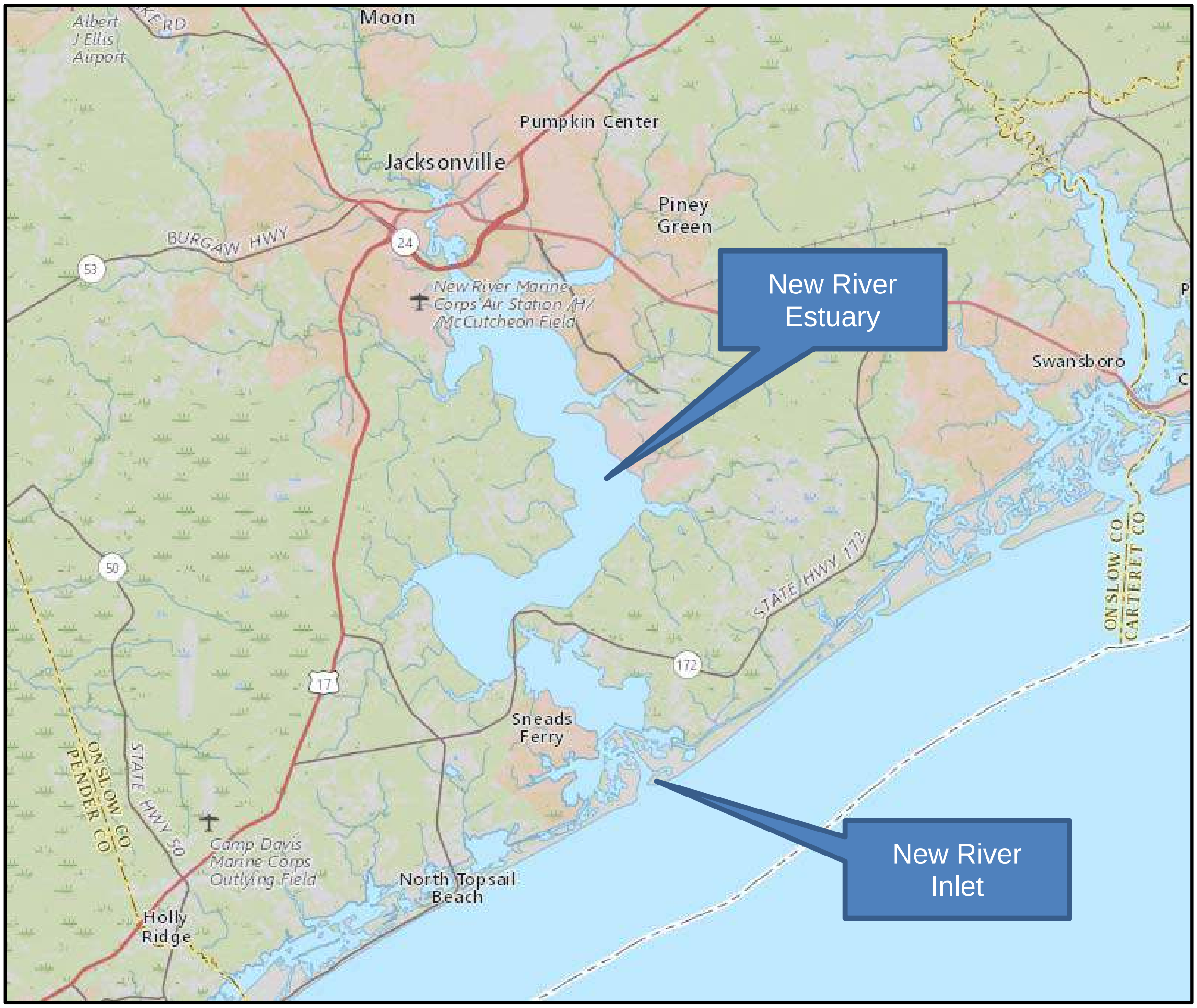


Figure 1-9. New River Inlet Access to the Atlantic Ocean for Onslow County Commercial and Recreational Boaters

2.0 PURPOSE AND NEED

The purpose of the proposed project has two main elements: 1) beach restoration, and 2) inlet navigation improvements. The beach/dune restoration will provide short-term and long-term protection for threatened residential structures, Town infrastructure, and recreational assets, including beach area, public parking, and public beach access, along the northeast end of NTB. The navigation improvements to NRI will allow safer and more consistent use of this inlet by commercial and residential boaters from the Onslow County area.

The proposed project seeks to satisfy following needs:

1 Ensure dependable navigation across the ocean bar channel for commercial, recreational, and military vessels using NRI;
2 Maintain permitted authorized depths and dimensions of NRI as a renewable sand resource for the beneficial use in beach nourishment;
3 Provide long-term stabilization of approximately 1.0 mile of oceanfront shoreline located immediately south of NRI;
4 Provide long-term protection to the imminently threatened residential structures; and
5 Develop a cost-effective management strategy to reduce maintenance and costs.

2.1 PROJECT AREA

The project area for this study (Figure 2-1) is generally related to the NRI system and the adjacent shorelines, which are significantly influenced by it. The project area includes the northeastern section of NTB shoreline, the NRI, and the southwestern section of Onslow Beach. Cedar Bush Cut is the section of New River just inland of the inlet. For NTB, the project area includes the "Phase 1" reach of shoreline from Station 1170+00 (at the Inlet) to Station 1090+00. For Onslow Beach, the project area includes up to Station 60+00. Stations are labeled by distances (e.g., between 1170+00 and 1090+00 is 80+00 or 8,000 feet). The estuarine shorelines along NRI and Cedar Bush Cut are also included in the project area.

Oblique aerials of some of the project area are provided in Figures 2-2 through 2-4. The entire project area and associated shorelines can generally be characterized as inlet influenced.

Figure 2-1. Project Area (outline in red)

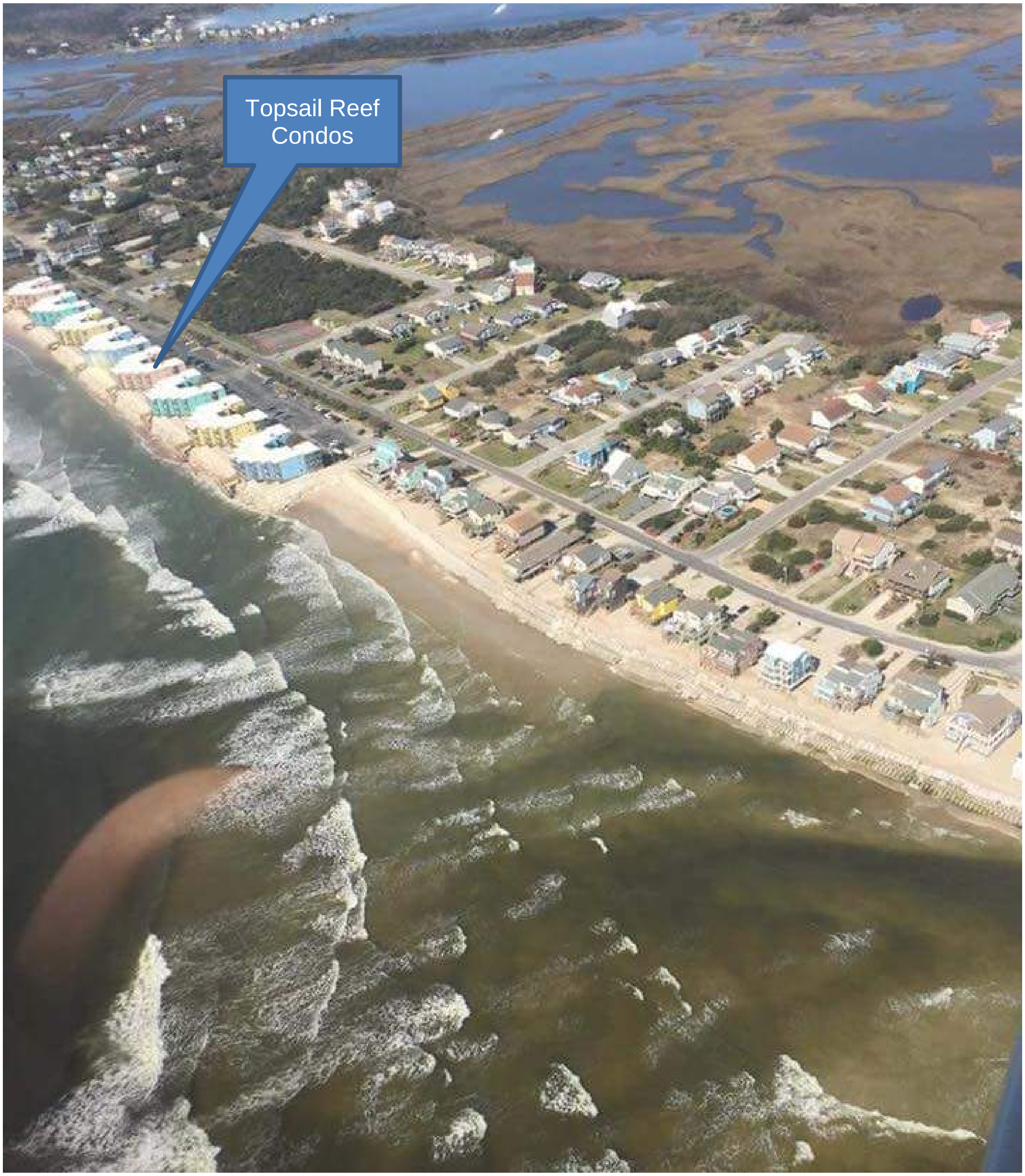


Figure 2-2. Oblique Aerial of Northern NTB (March 20, 2016; source: NTB Facebook page)

Figure 2-3. Oblique Aerial of New River Inlet (March 20, 2016; source: NTB Facebook page)

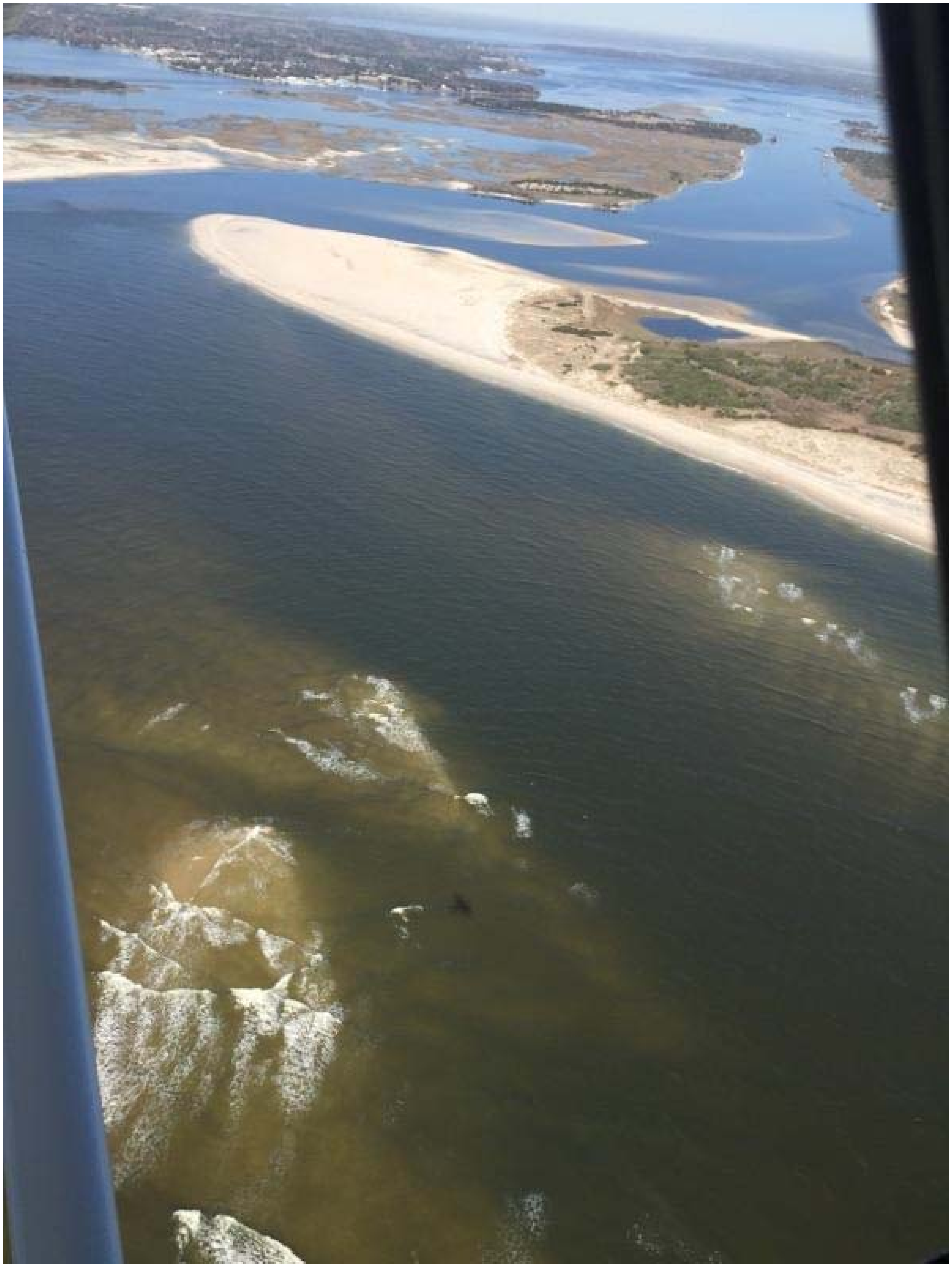


Figure 2-3. Oblique Aerial of Southwestern Tip of Onslow Beach (March 20, 2016; source: NTB Facebook page)

3.0 INDEPENDENT BEACH MANAGEMENT ACTIVITIES

In addition to seeking a permit for the project proposed herein, the Town and the U.S. Army Corps of Engineers (USACE) have several other ongoing or planned nourishment/dredging projects.

Five reaches of beach have been established on NTB (Figure 3-1), identified as Phase 1 to Phase 5. This document will focus on Phase 1 (closest to NRI). The Phase 1 reach of shoreline is strongly influenced by NRI processes.

Both the Town and USACE perform nourishment activities on NTB as the primary sponsor, although USACE fill activities are rather rare and are relatively small. The Town has historically funded the entirety of its nourishment projects, from permitting through design, construction, and monitoring.

The USACE Phase 1 reach beneficial nourishment projects have occurred infrequently for decades and are navigation related. An example of USACE placement is a 2010 project where the USACE dredged and placed approximately 45,000 cubic yards (cy) of material from Cedar Bush Cut and the AIWW/New River. USACE's primary concern is navigation, and Phase 1 has benefitted from the beach-compatible dredged material placed there. The USACE also frequently uses a side-caster dredge or shallow-draft split-hull hopper dredge for the NRI outer ebb shoal when material is not placed on the beach (refer to Section 4 for more discussion).

The USACE included NTB's most southern reach (Phase 5) as a component of a 50-year coastal storm damage reduction (CSDR) project that includes neighboring Surf City. This project has not yet been funded.

The Town has its own nourishment program for Phases 2, 3, and 4, which are largely located in Coastal Barrier Resources Act (COBRA) zones that prevent federal investment, including USACE projects, Federal Emergency Management Agency (FEMA) projects and U.S. Department of Agriculture (USDA) funding mechanisms (refer to Section 4 for more discussion).

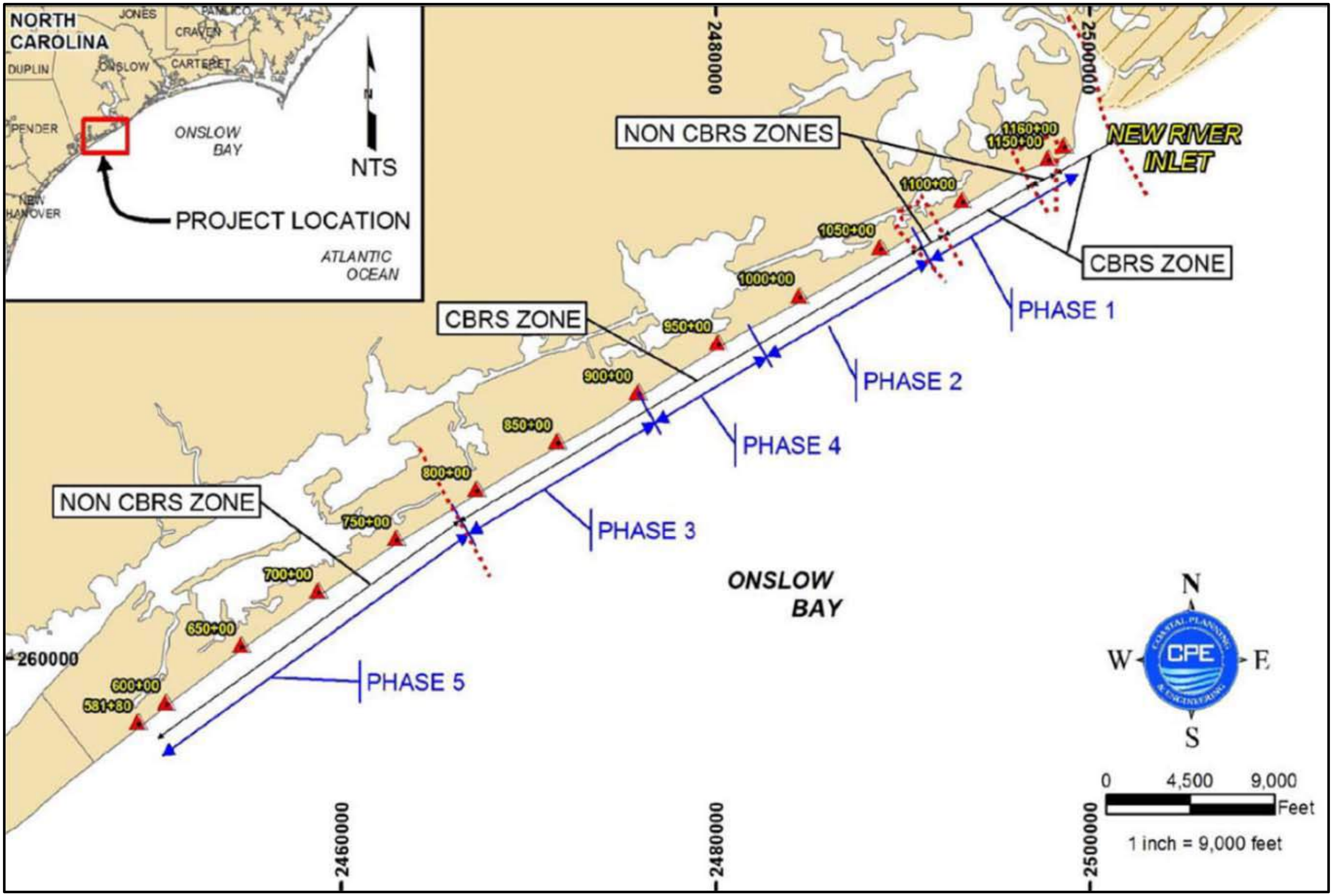


Figure 3-1. 2009 EIS Figure Depicting the 5 Phases, or Shoreline Reaches, of North Topsail Beach

3.1 PLANNED THIRD EVENT PROJECT

An upcoming Town nourishment is planned for 2018/2019 that will utilize the outer ebb channel as a borrow area and place material in the Phase 1 and Phase 2 shoreline reaches. This is known as the 3rd Event and is being permitted under the USACE Permit No. SAW 2005-00344 (issued on May 27, 2011). Figure 3-2 presents an overview of this project.

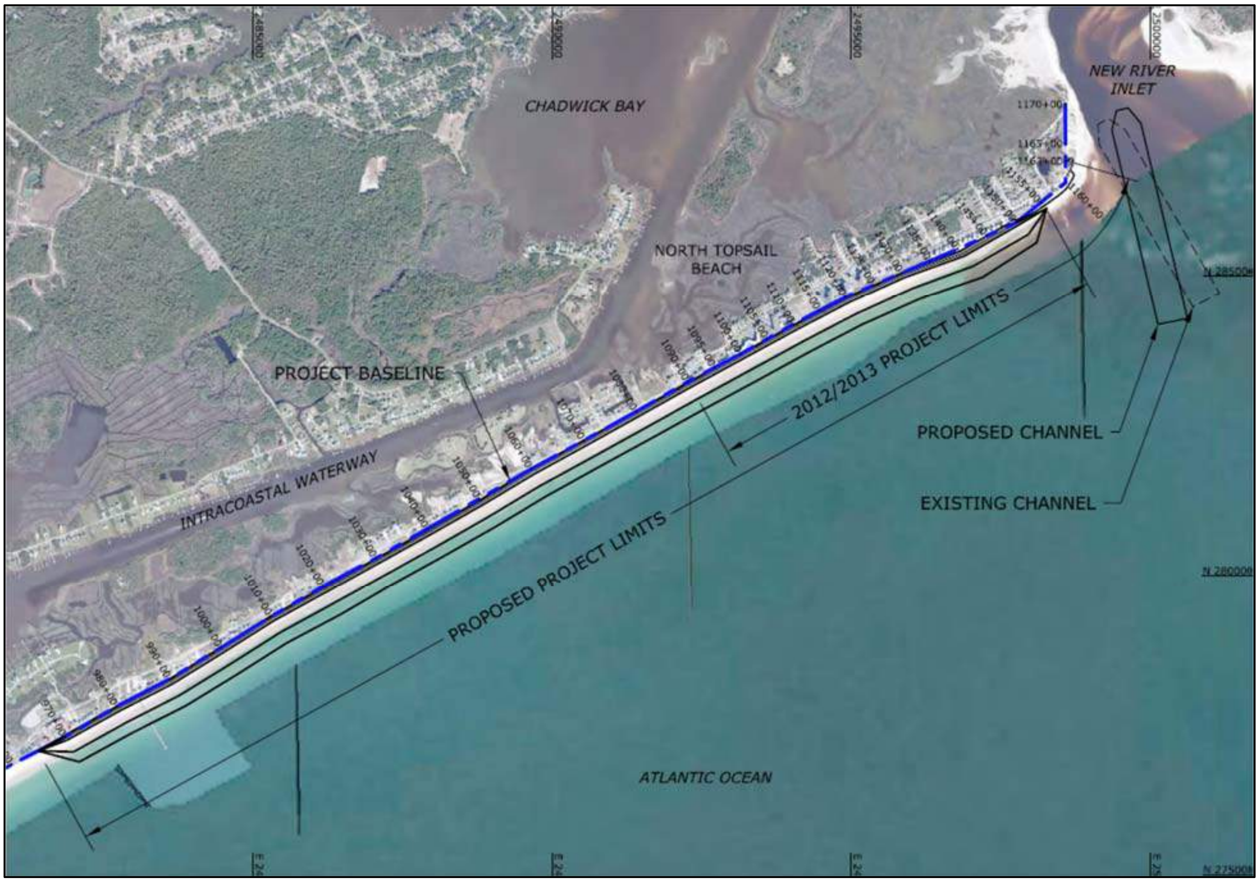


Figure 3-2. 2018/2019 3rd Event Planned Project (source: APTIM)

Special Condition #3(a) of the USACE permit specifies that no maintenance event can be initiated unless one or both of the following thresholds, as stipulated in the Final 2009 EIS, have occurred:

1. Only if shoaling of the new channel totals 85 percent of the actual dredged volume of the initial construction, and/or
2. Only if the channel thalweg migrates outside the 500-foot wide corridor established during the initial construction.

The channel thalweg is the deepest part of the channel (or channel centerline). The federal permit for the project does not allow channel maintenance to be performed more frequently than once every 4 years. The project plans to place up to 700,000 cy (depending on future sedimentation) along 18,200 feet of shoreline (approximately 40 cy/ft). The project stations (not including tapers) extend from 1152+00 down to 970+00.

3.2 RECENT AND PLANNED BEACH FILLS

Table 3-1 presents recent and planned beach fills along all of NTB. More discussion on these events is provided in Sections 4 and 5.

The sandbag revetment is included in Table 3-1, although technically it is not a beach fill. The Topsail Reef condominiums led an independent effort in constructing its revetment.

Two projects are proposed for the 2018 and 2019 environmental window, including the 3rd Event and the FEMA truck haul. The 3rd Event will place material on the Phase 1 shoreline. The FEMA truck haul will only place material on the Phase 5 shoreline as mitigation for Hurricane Matthew on the 2nd Event nourishment.

Table 3-1. Recent and Planned Nourishment Town Activities along all NTB Shorelines.

Date Completed	Project Name	Volume (cy)	Length (ft)	Volume (cy/ft)	Borrow Area	Stations		Cost (Total)	Cost ($/cy)
2009	EIS included all 5 shoreline reaches	-	-	-	-	-	-	-	-
March 2013	Channel Realignment 1	600,000	7,300	82.2	NRI Ebb Channel	1163	1090	$5,600,000	$9.33
2014	Sandbag Revetment (including geotube)	50,000	3,600	13.9	NTB “Spit”	1170	1135	$3,500,000	$70.00
April 2016	Cedar Bush Cut (CBC)	130,000	1,100	118.2	CBC	1163	1152	$2,450,000	$18.85
June 2016	Phase 5 (2nd Event)	1,300,000	18,500	70.3	Offshore	582	767	$16,800,000	$12.92
2018/2019	FEMA Truck Haul - Phase 5 Matthew Related	160,000	18,500	8.6	Upland	582	767	$5,300,000	$33.13
2018/2019	Channel Realignment 2 (Pivot Channel). Phase 1 and 2 shorelines (3rd Event)	700,000	18,200	38.5	NRI Ebb Channel	970	1152	-	-

4.0 PROJECT SITE HISTORY

4.1 PREVIOUS STUDIES

Numerous studies have documented accretion and erosion patterns in the NRI vicinity, including the following (in chronological order):

- 1999 – Hurricane impacts and beach recovery in southeastern North Carolina: the role of the geologic framework: Cleary.
- 2000 – Hurricane Fran effects on communities with and without shore protection: a case study at six North Carolina beaches: USACE.
- 2003 – Inlet-Induced shoreline changes: linkage between channel migration and ebb-tidal delta reconfiguration, Bogue and NRIs, North Carolina, Coastal Sediments 03, American Society of Civil Engineers, 15p. Cleary, et al.
- 2008 – Proposed Redelineation of North Carolina's Twelve Developed Inlet Hazard Areas. North Carolina Division of Coastal Management, Document CRC 07-09: Warren et al.
- 2009 – North Topsail Beach Environmental Impact Statement (EIS). CPE (now APTIM).
- 2016 – North Topsail Beach Terminal Groin Feasibility and Modeling. CPE (now APTIM).

NRI and the adjacent shorelines of NTB and Onslow Beach have been studied extensively from a shoreline change and sediment transport perspective since the 1990s. Many of these studies include shoreline change and inlet movement analyses dating back to the mid-1800s.

4.1.1 2009 EIS

The 2009 NTB environmental impact statement (EIS) represents a significant research, analysis and permitting effort that summarizes recent and historical erosional issues and developed a long-term nourishment and inlet management plan. The following is a relevant excerpt of the 2009 EIS:

> *The instability of the shoreline immediately southwest of New River Inlet poses the most immediate shoreline management concern. During the past year, 17 duplex structures located at the extreme north end of Town, which have a total tax value of over $17 million, have become imminently threatened. Attempts have been made by individual property owners to protect the threatened duplexes with sandbag revetments; however, most of the sandbag revetments*

have failed to provide any substantial degree of protection. Two of the imminently threatened duplexes were relocated to other parts of North Topsail Beach at the expense of the property owners. Some of the remaining duplexes have been declared uninhabitable due to the loss of water, sewer, and electrical connections and could eventually be ordered to be removed or demolished.

At that time, terminal groins were not permitted to be constructed in North Carolina. Therefore, the terminal groin alternative was not analyzed in any depth:

Alternative 7 – Terminal Groin. A terminal groin would be constructed on the south shoulder of New River Inlet to protect the extreme northeast end of the Town's shoreline from the impacts of New River Inlet. Since the State of North Carolina has adopted legislation prohibiting the use of this type of structure, the terminal groin alternative was not critically evaluated in the engineering analysis.

The proposed terminal groin project will build upon the 2009 EIS analysis, as well as more recent APTIM modeling and terminal groin analyses (APTIM, 2016), (refer to Sections 7 and 8).

4.2 NEW RIVER INLET

The New River Inlet connects New River Estuary and the AIWW to the Atlantic Ocean. While the New River Estuary and other bays/sounds in the area are naturally occurring features, the AIWW in this area was constructed around 1940. Figure 4-1 presents a 1938 historical aerial showing the New River (inland on of the inlet) meandering more and not as large and wide as it is today. Some AIWW dredge disposal areas are visible in the top left corner of the image.

Figure 4-1. 1938 Aerial of New River Inlet

Figure 4-2 presents a comparison between a 1958 aerial and a 2016 aerial. A 2009 estuarine shoreline (digitized by DCM) and tax parcels (downloaded from the Onslow County website) are provided for comparison. Additionally, the longest “north” and “south” groin alternatives as well as the ebb shoal borrow areas (2016 dredge footprint and “pivot” channel dredge footprint that is planned for 2018/2019) are shown for reference purposes.

Significant erosion is seen on Onslow Beach whereas New River meanders to the southwest, just landward of the inlet. In general, New River in the Cedar Bush Cut area is much wider. In addition, NRI has migrated to the southwest.

Figure 4-3 presents a comparison of a 1993 aerial with a 2016 aerial. As expected, the same general trends as observed in Figure 4-2 are seen. The NTB ocean shoreline from 1993 is wider than either the 1958 or 2016 aerial shorelines.

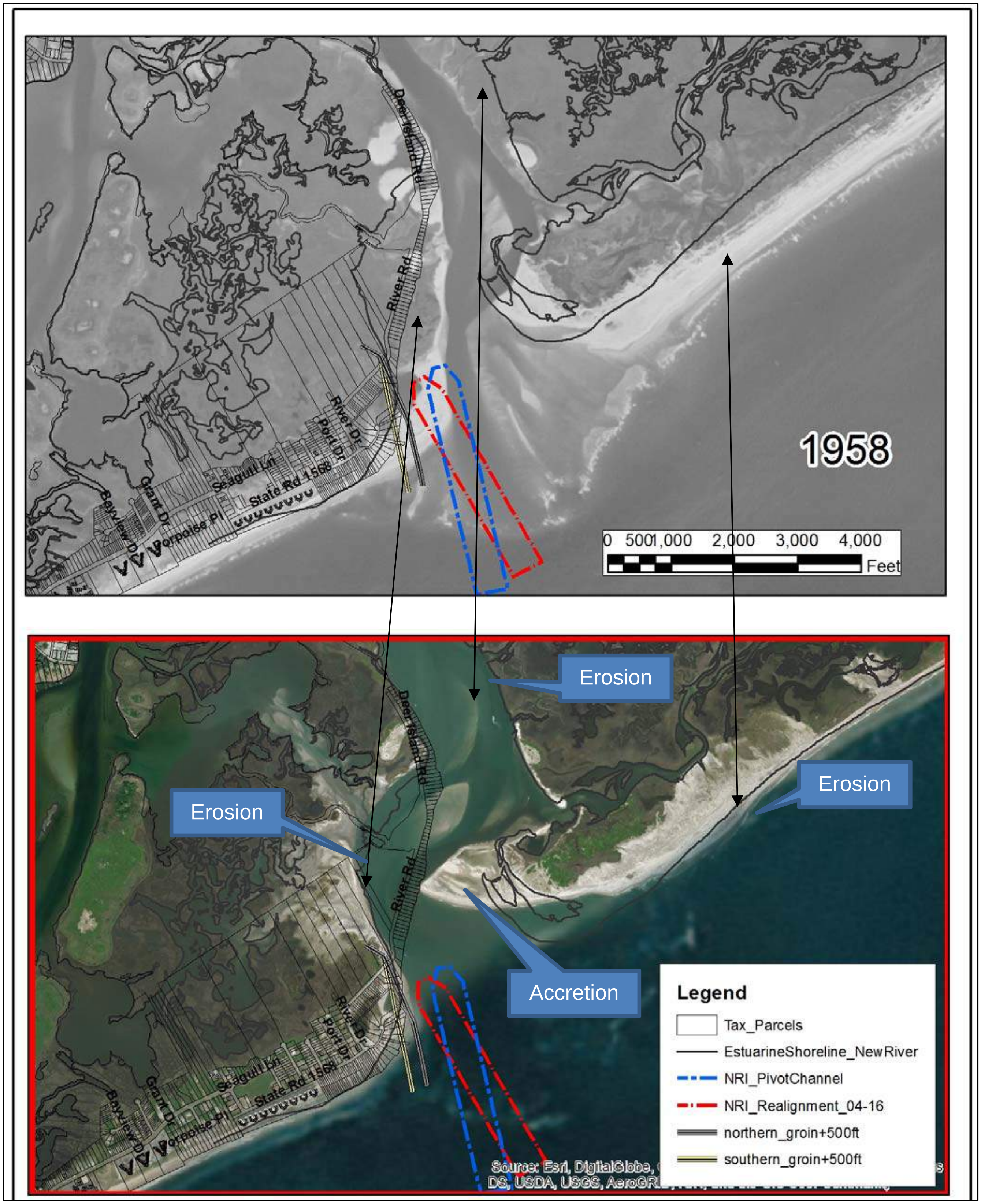


Figure 4-2. Comparison of 1958 and 2016 aerials at New River Inlet

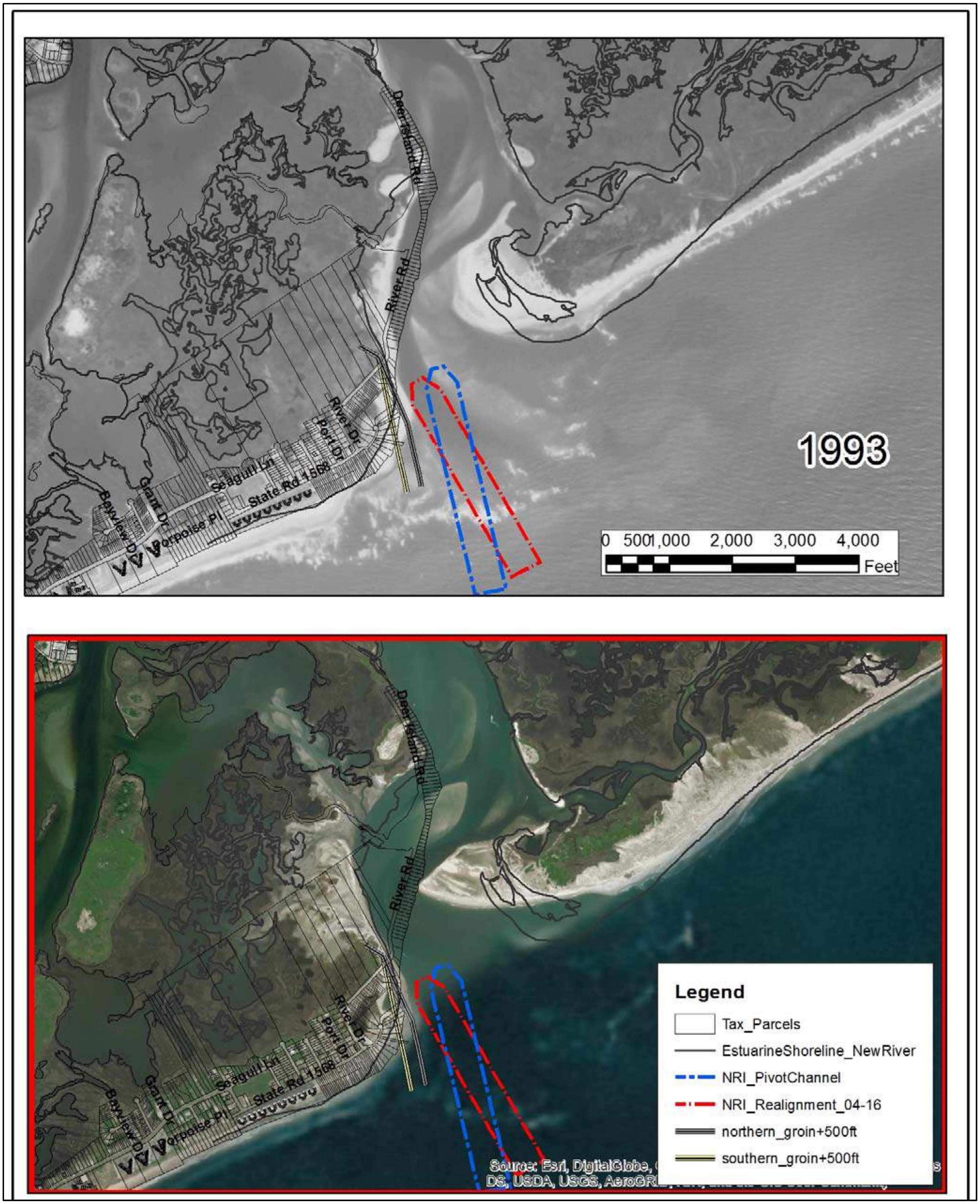


Figure 4-3. Comparison of 1993 and 2016 aerials at New River Inlet

4.2.1 NEW RIVER INLET MOVEMENT

Cleary and Marden (2001) conducted an assessment of NRI movement that was included in DCM Inlet Hazard Area (IHA) reports. Relevant excerpts include:

- Historical coastal charts indicate that the inlet has migrated within a two-kilometer zone since 1856; the migration zone width is controlled by the ancestral channel of the New River, the majority of which is located on the Onslow Beach (northeastern) shoulder of the inlet
- In recent history, the inlet's width has varied considerably ranging from 66 m (217 ft) in 1938 (prior to dredging) to a maximum width in 1987 of 304 m (997 ft)
- Although the inlet has generally moved southwest, it has periodically reversed direction (this northeastward movement was directly related to the enlargement of the marginal flood channel on the North Topsail Beach shoulder)

Note that this analysis focused on the inlet and not the river. Cleary also performed additional research related to Topsail Island geology, which is presented in Section 4.2.2.

4.2.2 TOPSAIL ISLAND GEOLOGY

Cleary (1996) presents a geologic framework for all of Topsail Island, including NRI. Figure 4-4 presents a schematic of 1938 and 1986 shorelines at the project site.

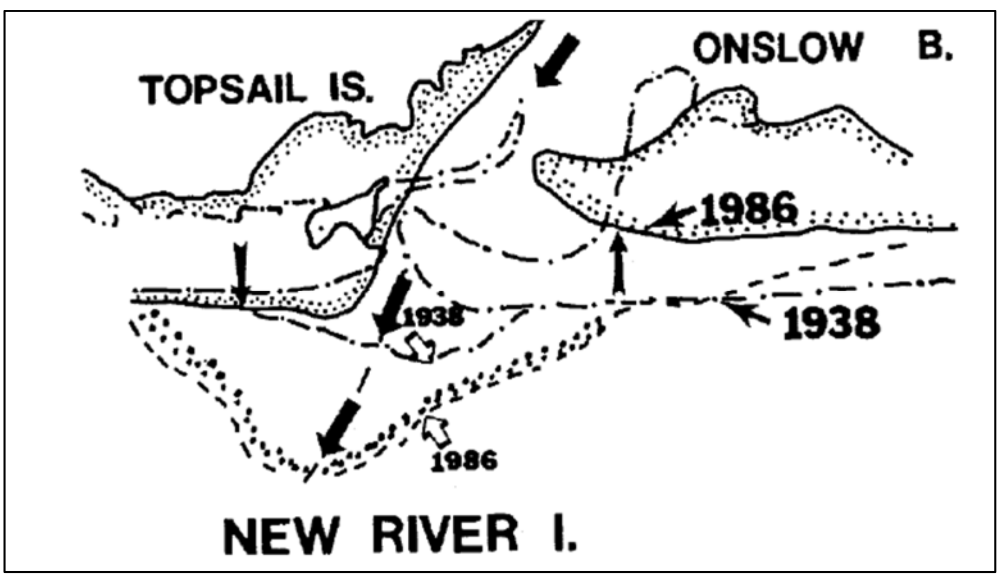


Figure 4-4. Schematic Excerpted from Cleary (1996), with 1938 and 1986 Shorelines Shown

Cleary (1996) conjectured that increased USACE dredging in the 1960s promoted an increase in tidal prism (40 percent) and the retention capacity of the ebb tidal delta.

Relevant excerpts by Cleary (1996) include the following.

- *Studies by the US Army Corps of Engineers have shown that, between 1856 and 1933, the northern half of the island was eroding at an average rate of 0.40 m/yr while the southern portion had alternating sections of accretion and erosion. Data from the period 1933 to 1980 indicate a slight increase in the erosion rate (0.70 m/yr) for the northern half, while the southern segment was characterized by sections of both accretion and erosion. Topsail Island is situated in a severe or chronic overwash zone (Fig. 12 A-E). Storms during the period 1944 to 1962 and in the late 1980's were particularly devastating to the island. Hurricane Hazel (1954) and the Ash Wednesday storm (1962) caused significant damage. Hurricane Hazel destroyed 210 out of 230 buildings and generated a 2.9 m above MSL flood level on an island whose average elevation is 2.7 m MSL.*

- *The hydrodynamics of this inlet were changed considerably by the dredging of Atlantic Intracoastal Waterway (AIWW) and the channels connecting the estuary with the open ocean. The earliest photographs [1938) and charts indicate the inlet and the main channels were clogged due to reduced tidal flow. In 1940 a 3.7 km long navigation channel was dredged connecting the AIWW with the inlet. The early 1960's marked the advent of sidecast dredging of the throat and outer bar channel for navigation purposes. Following dredging operations, the once small ebb tidal delta increased in areal extent from approximately 140,000 m2 to 700,000 m2. Due to the increased tidal prism, the volume of sand retained in the ebb delta increased by almost 50 percent (Fig 14 A&B).*

Cleary's work generally agrees with historical aerials, where the widening of the NRI in the Cedar Bush Cut is apparent (Figure 4-2 and 4-3).

4.2.3 SEDIMENT TRANSPORT OVERVIEW

Inlet sediment transport processes can be complex and vary from inlet to inlet, however, some major features are common. Figure 4-5 presents a general schematic of inlet sediment transport processes, and Figure 4-6 presents a schematic of these processes at NRI.

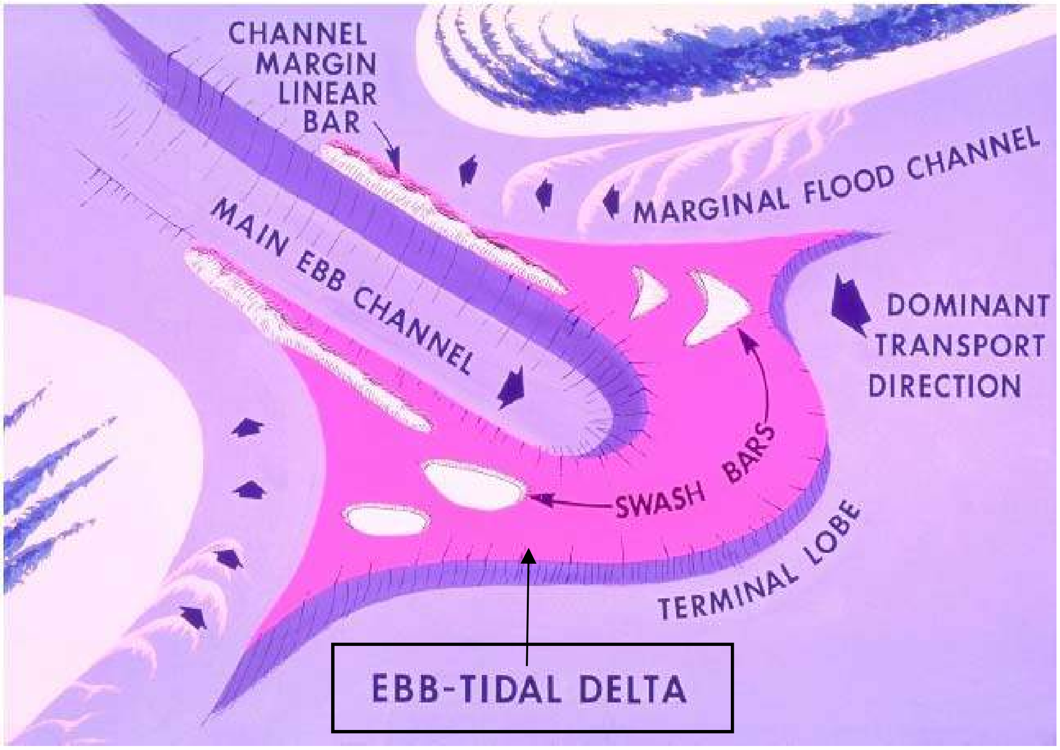


Figure 4-5. Inlet Sediment Transport Schematic (source: Hayes, 1994). The Ebb Tidal Delta is also known as the ebb shoal.

Figure 4-6. New River Inlet Sediment Transport Schematic

Sediment transport processes at NRI are influenced by waves and tidally induced currents. Currents induced by flood and ebb tides are fundamentally different in terms of hydrodynamics. Flood-tide currents push into the inlet in a much more uniform manner and can create marginal flood channels that run along the inlet shorelines. Flood tides push material into the inlet. Ebb-tide currents are more channelized when passing out of the inlet and typically form either one or possibly two main channels where significant flows occur. Ebb tides push material out of the inlet where material settles out along the ebb shoal.

In addition to sediment transport induced by flood and ebb tide currents, wave-induced sediment transport moves material up and down the coast, depending on wave direction. Regional net sediment transport is from north to south in this area, however, significant wave-induced sediment transport from the south to north does occur and was evaluated for this project. Wave-induced sediment transport at NRI generally follows the ebb shoal, where waves can be seen breaking (Figure 4-6).

As with most ebb shoals, the NRI ebb shoal (or ebb tidal delta) can vary dramatically in size and shape. Cleary (1996) concluded that during the 1938 to 1995 period of aerial photographic coverage, the mean area of the ebb tidal delta was 1 million square meters (m^2). The data suggest the ebb delta area has increased over time, particularly during the past several decades (Cleary, 1996). Cleary postulated that an increase in the inlet's width and depth would contribute to a larger retention capacity of the offshore shoals and, therefore, in its aerial extent (1996).

4.2.4 HISTORICAL SHORELINE CHANGE

DCM and the U.S. Geological Survey (USGS) have digitized many historical shorelines dating back to the 1900s, based on aerials, surveys, charts and National Oceanic and Atmospheric Administration (NOAA) shoreline surveys (T-sheets). Figure 4-7 presents an overview of these shorelines in the project area.

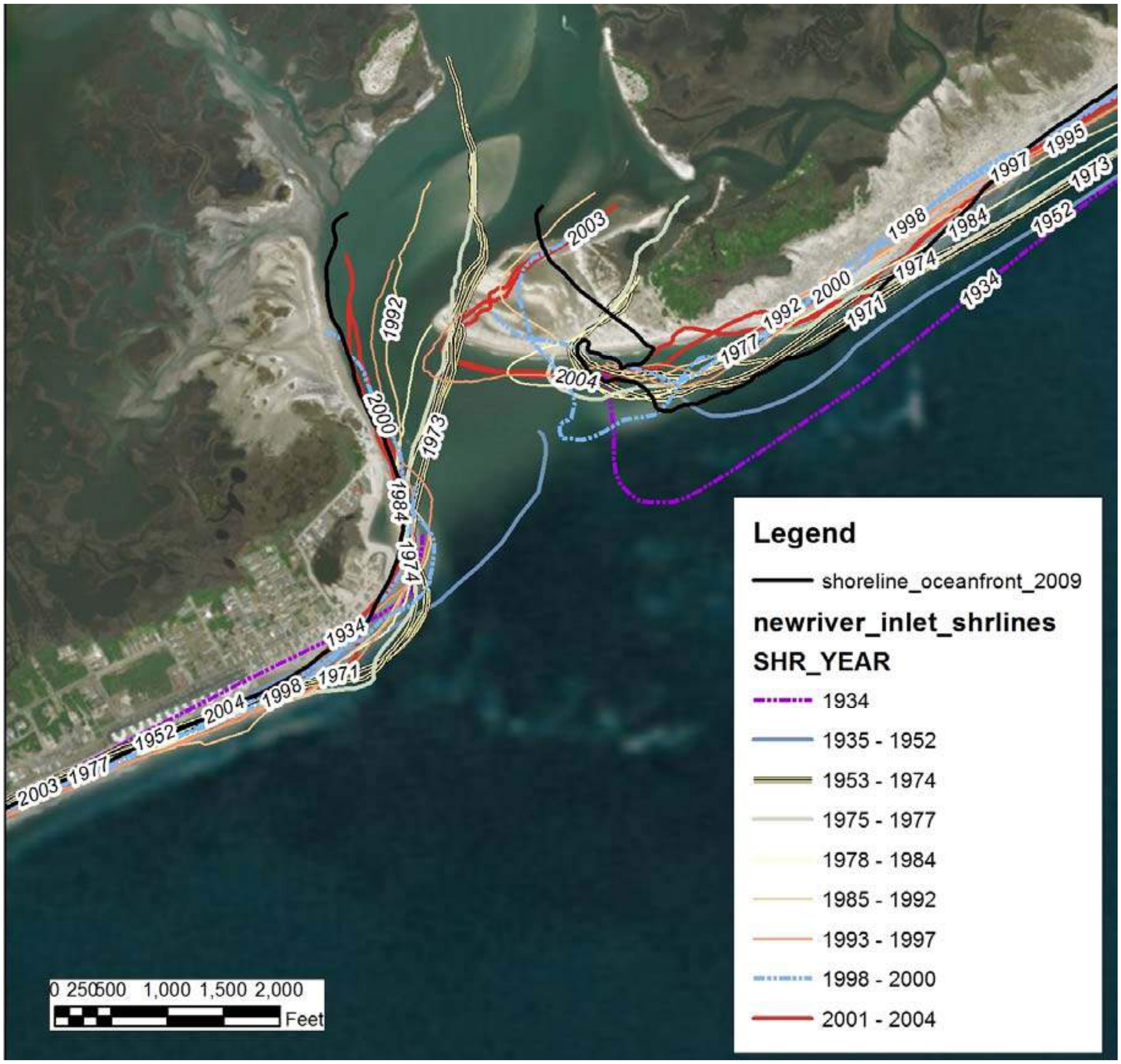


Figure 4-7. Historic Shorelines in the Project Area (2016 ESRI aerial shown)

A much stronger erosion trend is noted on Onslow Beach in comparison to NTB. Northern NTB does show some strong erosional trends in a few "hot spot" areas, as indicated in Figure 4-8.

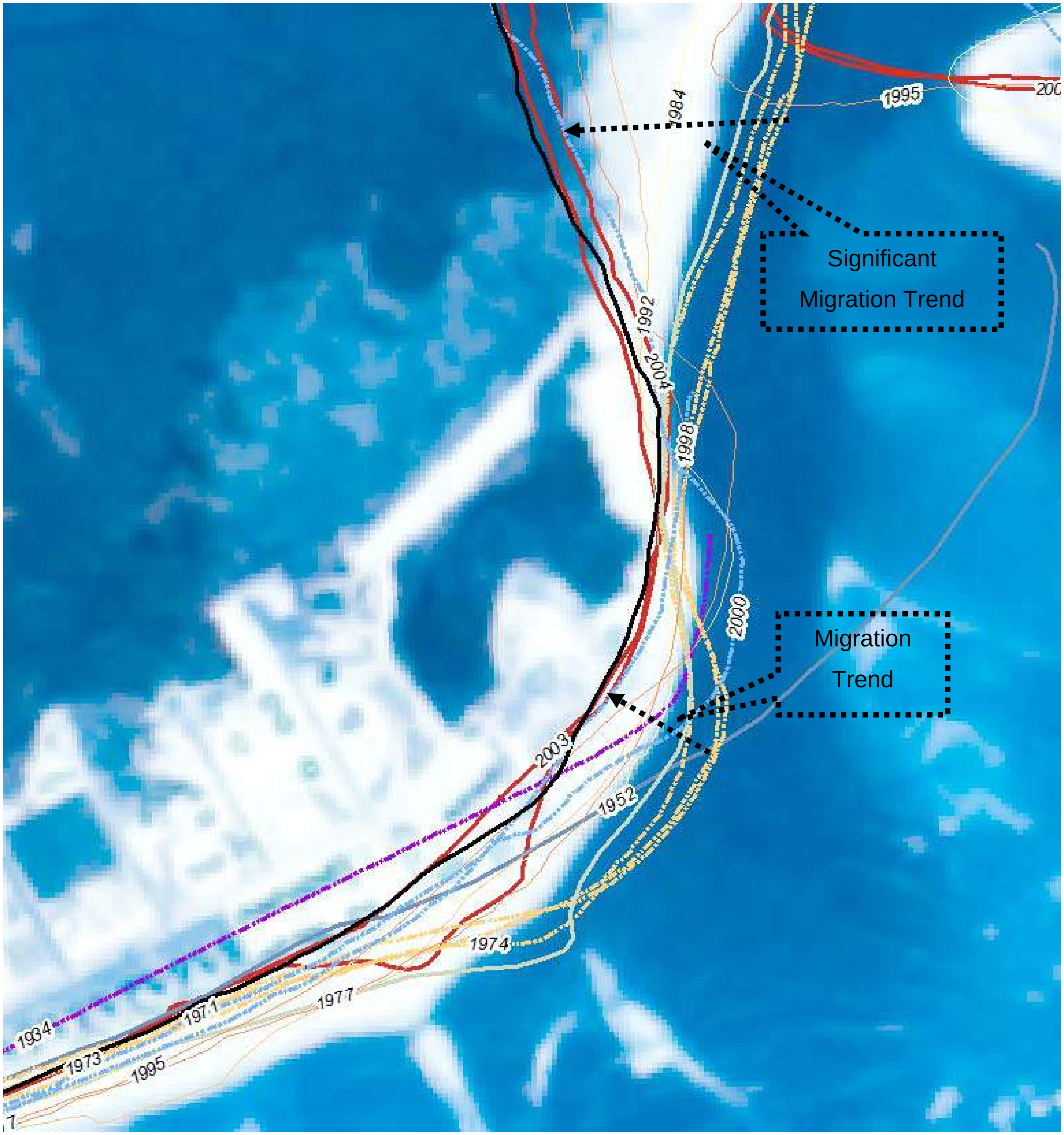


Figure 4-8. Historic Shoreline Erosion along Northern NTB (1983 aerial shown)

4.2.5 OUTER CHANNEL ORIENTATION AND DREDGING

NRI's outer channel (through the ebb shoal) moves naturally in response to varying ebb shoal conditions. The USACE Navigation Branch conducts outer channel sidecast dredging and follows the deep water to achieve a 90-foot-wide channel at 6-foot mean low water (MLW) depth. The dredging extends out to the natural 6-foot MLW contour, which is typically around 1,000 to 2,000 feet offshore (depending on channel orientation). Channel orientations have generally been more northerly, closer to the Onslow Beach shoreline, which can expose vessels to waves on the beam/broadside (which is not optimal for navigation and safety) (Figure 4-9).

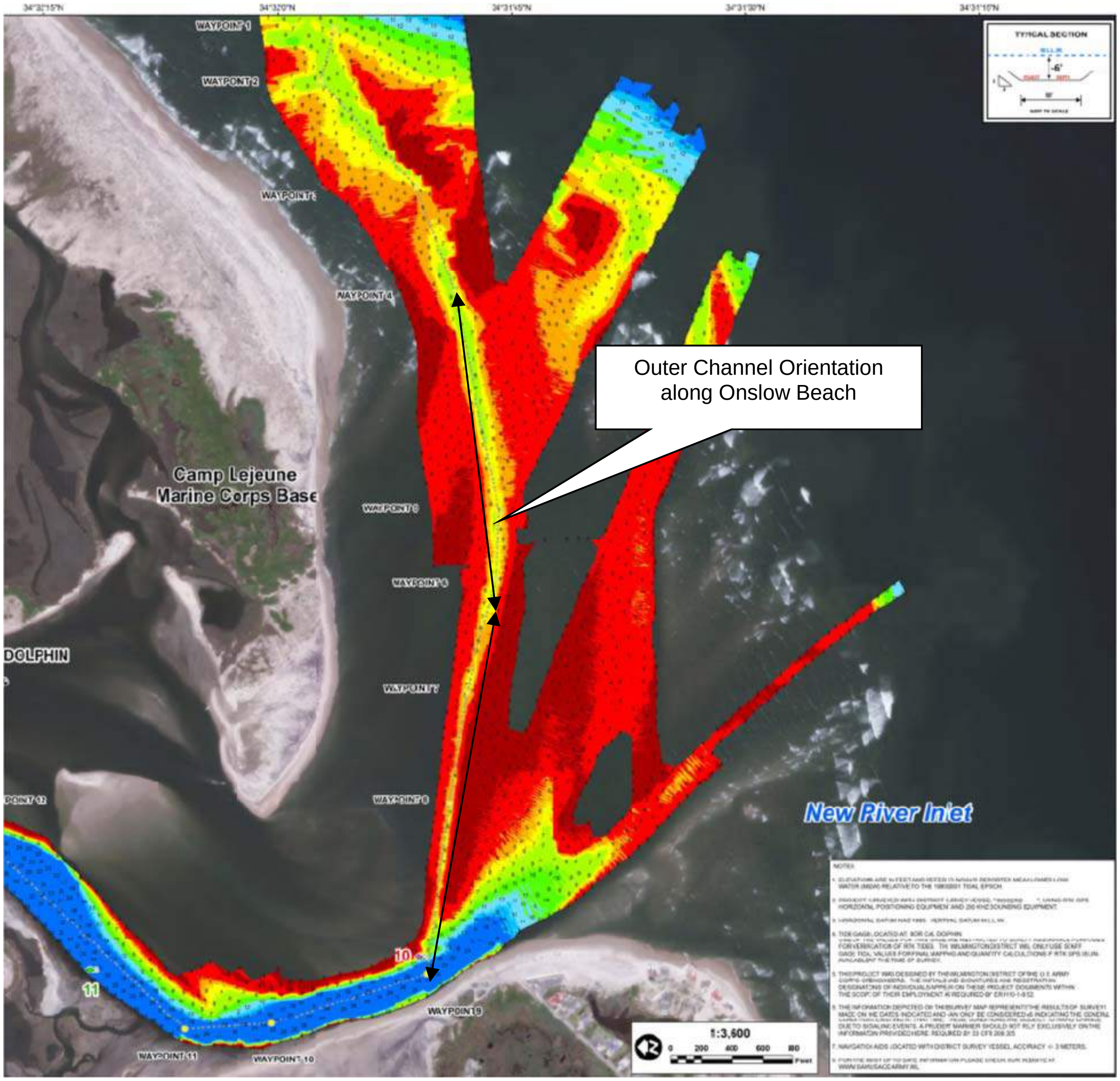


Figure 4-9. June 2017 NRI Outer Channel Orientation

The *Merritt* sidecaster dredge is shown in the outer channel in Figure 4-10. Figure 4-11 presents an oblique aerial of the NRI ebb channel and ebb shoal. NRI's outer channel orientation/alignment has been documented to affect shoreline erosion intensity, which was one of the primary reasons for the 2013 channel realignment project (refer to the 2009 EIS). Additional discussion on channel alignment is presented in Section 4.4.1.

Figure 4-10. Merritt Sidecasting Dredge Working the New River Outer Channel in Spring 2012

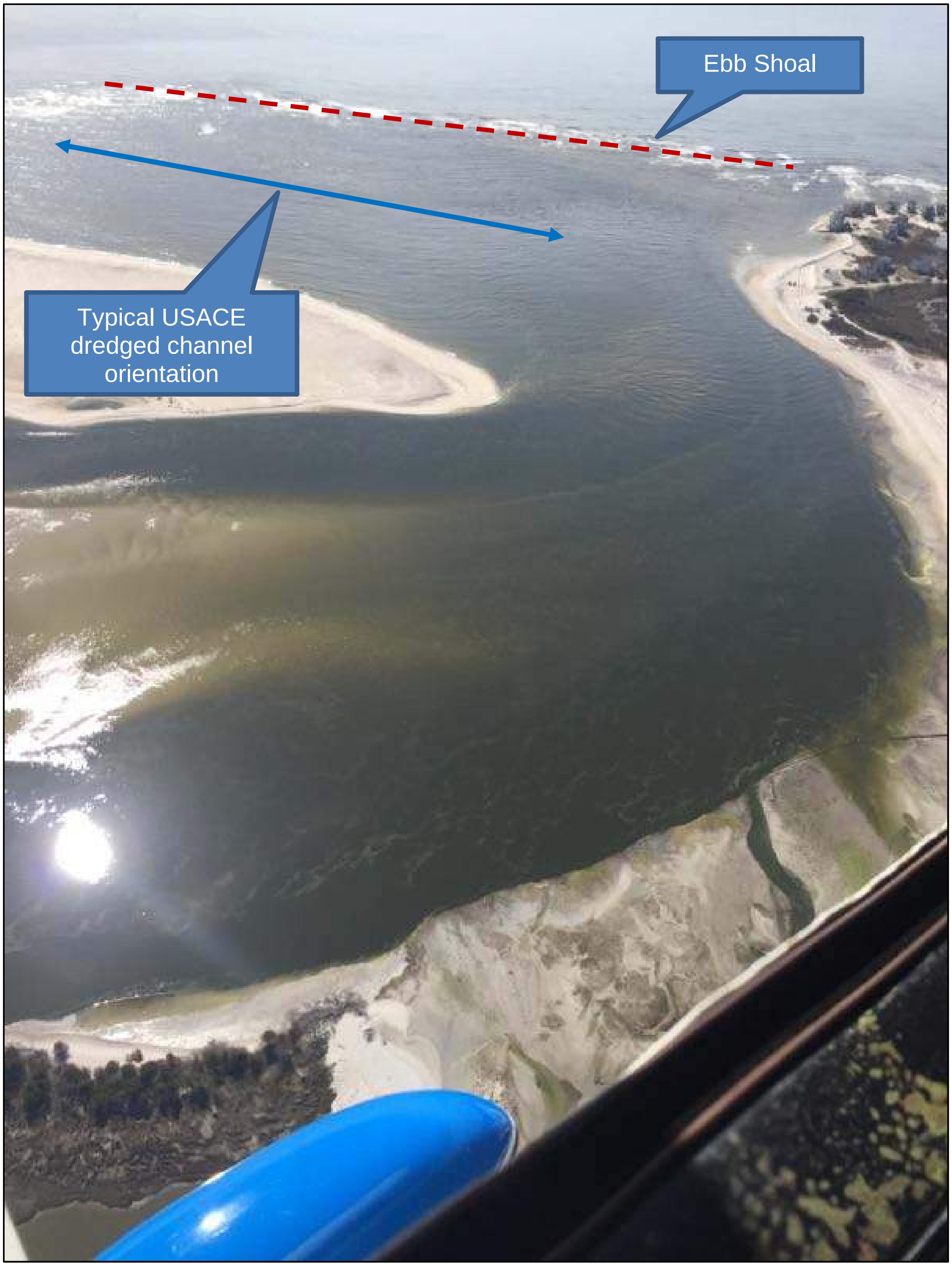


Figure 4-11. Oblique Aerial (March 20, 2016) Showing Typical USACE Dredged Channel Orientation

4.2.6 NRI INFLUENCE ON OCEAN SHORELINES

Warren and Richardson (2010) performed a statistical shoreline analysis (standard deviation of shoreline position and average rate of shoreline change) that identified the point along the oceanfront where NRI processes were no longer dominant (Figure 4-12).

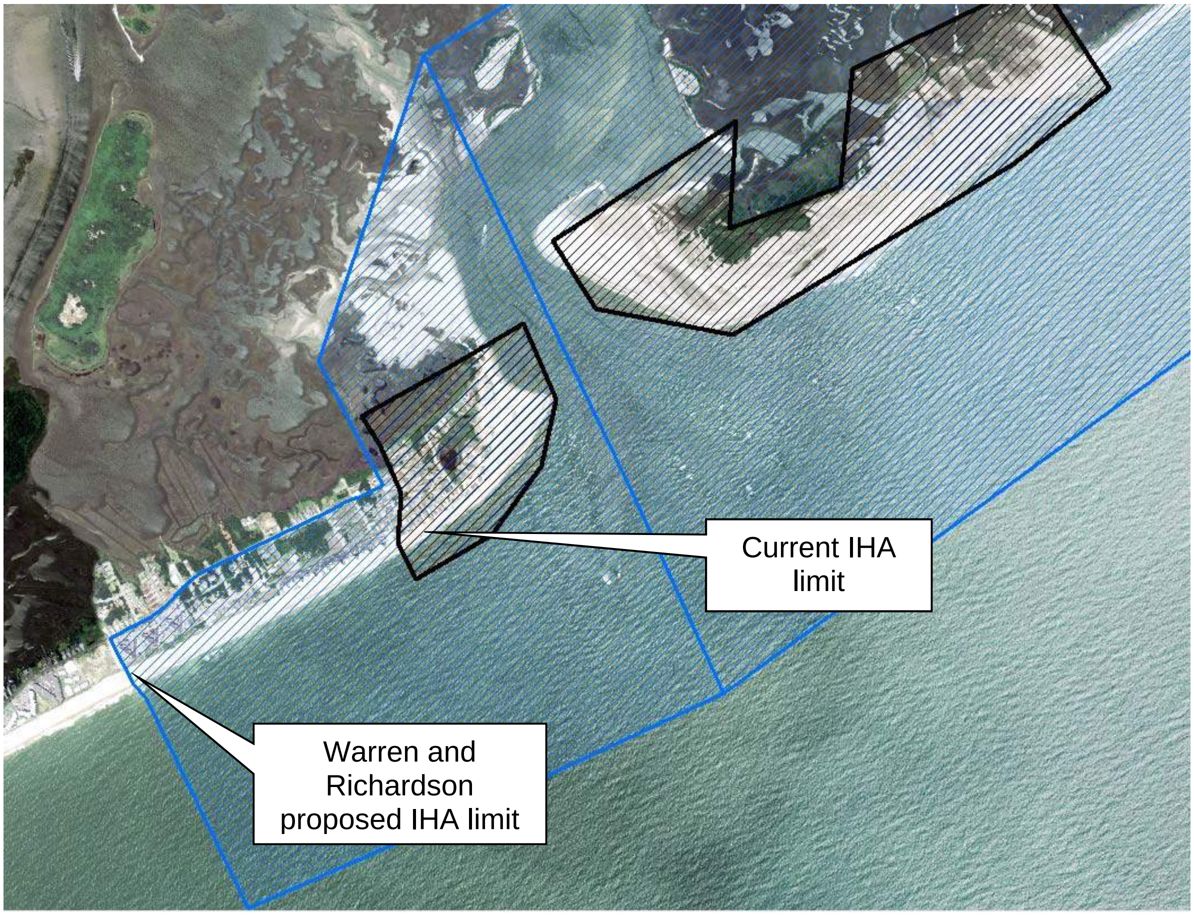


Figure 4-12: Current (black) and Proposed IHA (blue) Boundaries

The proposed Inlet Hazard Area (IHA) boundary followed the line of maximum historical beach width (Warren and Richardson, 2010). Therefore, Warren and Richardson's proposed area of influence of NRI along NTB extends approximately 1 mile. This area generally coincides with the project area, as defined in this study, and represents the reach of shoreline that is the focus of the proposed project.

The influence of NRI on Onslow Beach is much larger, about 2 miles. For the Onslow Beach side of NRI, Warren and Richardson (2010) state, "*The CRC Science Panel determined that beach width data were insufficient and did not illustrate an adequate hazard boundary. The* *proposed IHA boundary follows the back-barrier canal based on the relative position of shorelines, inlet processes, and geomorphology*."

While outer channel location has been correlated with shoreline erosion trends at NRI and at other inlets, shoreline erosion along the NTB Phase 1 shoreline is also affected by other elements, and the erosional signature would likely only be lessened with channel reorientation. Outer channel location is only one component of shoreline erosion in the project area.

4.2.7 COBRA ZONES

Figure 4-13 presents the COBRA zones on the northeastern end of NTB. This topic is discussed at length in the 2009 EIS. The Town has led efforts to remove the project area from COBRA zone designation, and this effort will continue to be monitored.

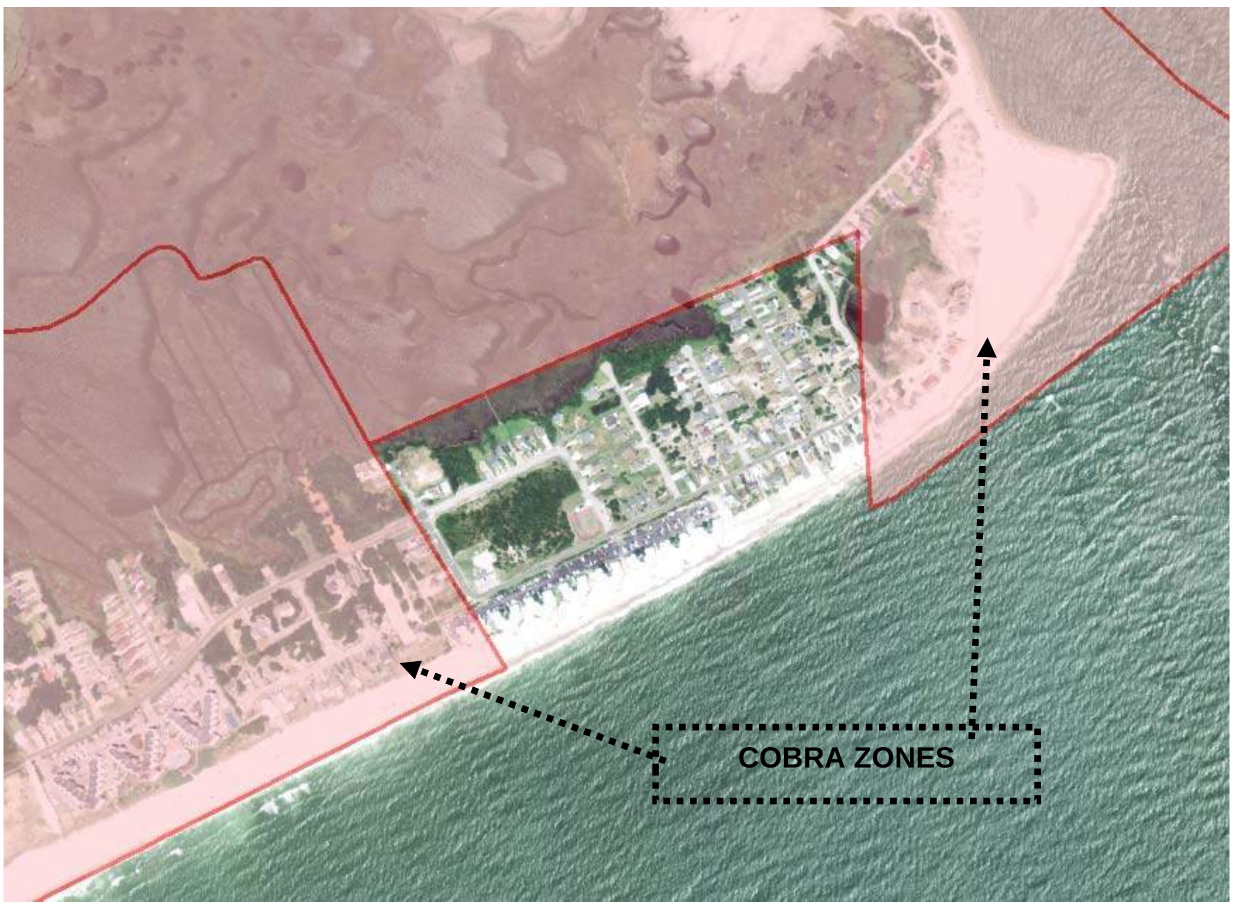


Figure 4-13. COBRA Zones (red) in the Project Area (All of the project area along Onslow Beach is a COBRA zone)

4.2.8 USACE NEW RIVER INLET DREDGING

As described in previous sections, the USACE is responsible for maintaining the federally authorized shallow draft navigation channel at NRI. The USACE performs routine maintenance dredging for navigation using pipeline (i.e., cutterhead), split-hull hopper, and side-cast dredges (when funding is available). Due to different USACE funding sources, two basic routine maintenance activities occur at NRI:

1 Inlet and outer bar side-cast dredging, and
2 Cedar Bush Cut/AIWW/New River cutterhead dredging and beach fill placement.

A large factor that plays into these two regions is the USCG collision regulations (COLREGS) line (Figure 4-14). Working seaward of the COLREGS line requires much stricter standards and generally only large, ocean-going dredges go through this certification process. However, the USACE sidecasters and split-hull hopper dredges are also ocean certified and can work this area. Note that any ocean-certified dredge can also work inland of this line, but there are many smaller, private dredging companies that specialize in inland work (AIWW, harbors, etc.) that cannot work seaward of the COLREGS line.

Significant information related to NRI inlet dredging history was included in the 2009 EIS and several relevant excerpts are included here:

> *In 1964, the Wilmington District Corps of Engineers commissioned the shallow draft sidecast dredge MERRITT to maintain the 90-foot wide, 6-foot deep channel across the inlet's ebb tide delta. The Wilmington District dredge fleet later expanded to include the FRY, a sidecast dredge similar to the MERRITT, and the small hopper dredge known as the CURRITUCK, all of which have worked intermittently in New River Inlet. The majority of the maintenance dredging has been performed by the sidecast dredges.*

Figure 4-14: New River Inlet USCG COLREGS (collision regulations) line

In the case of sidecast dredges, the reported dredge volumes represent the volume of material that passed through the dredge pumps not the in situ volume actually removed from the channel. In this regard, sidecast dredges simply move material from the channel to a point immediately outside the channel by pumping the dredged material through a 90-foot long pipe which projects off to the side of the vessel. Since the authorized channel through New River Inlet is 90 feet wide, the sidecast dredges do not completely remove the material out of the channel with one passage as a certain percentage of the material flows back into the channel and is redredged during a subsequent passage of the dredge. Accordingly, the volume of material reportedly removed by the sidecast dredges is probably greater than the actual amount of in situ material removed. The volume of material removed by the CURRITUCK, which totals about 765,500 cubic yards since 1978, is deposited offshore of the adjacent beaches in water depths of 10 to 15 feet.

Table 4-1 presents historical dredging quantities from 1964 to 2002 (2009 EIS). Additionally Figures 4-15 and 4-16 provide dredging figures overall and for a selected 16-month representative period.

Table 4-1. USACE NRI Dredging History (from Table 5 of the 2009 EIS).

Fisal Year	Name of Dredges	Volume Removed (cy)	5-year Average (cy)
1964	U.S. Merritt	29,620	
1965	U.S. Merritt	46,721	
1966	U.S. Merritt	30,441	
1967	U.S. Merritt	42,056	**1964 to 1968**
1968	U.S. Merritt	3,547	**30,477**
1969	U.S. Merritt	23,992	
1970	U.S. Merritt	42,579	
1971	U.S. Merritt	74,618	
1972	U.S. Merritt	80,956	**1969 to 1973**
1973	U.S. Merritt	65,680	**57,565**
1974	U.S. Merritt	48,370	
1975	U.S. Merritt	59,126	
1976	U.S. Merritt	142,928	
1977	U.S. Merritt	55,683	**1974 to 1978**
1978	U.S. Merritt, Currituck	186,869	**98,595**
1979	U.S. Merritt, Currituck	148,531	
1980	U.S. Merritt	120,977	
1981	U.S. Merritt	152,957	
1982	U.S. Merritt, Currituck	85,386	**1979 to 1983**
1983	U.S. Merritt	97,307	**121,032**
1984	U.S. Currituck	60,255	
1985	U.S. Merritt, Currituck	147,837	
1986	U.S. Merritt, Currituck, Fry	211,353	
1987	U.S. Merritt, Currituck, Fry	224,579	**1984 to 1988**
1988	U.S. Merritt, Fry	152,957	**159,396**
1989	U.S. Merritt, Fry	205,274	
1990	U.S. Merritt, Fry	267,720	
1991	U.S. Merritt, Fry	154,481	
1992	U.S. Merritt, Fry	238,399	**1989 to 1993**
1993	U.S. Merritt, Currituck, Fry	327,491	**238,673**
1994	U.S. Merritt, Fry	297,823	
1995	U.S. Merritt, Fry	236,966	
1996	U.S. Merritt, Currituck, Fry	419,426	
1997	U.S. Merritt, Currituck, Fry	585,093	
1998	U.S. Merritt, Fry	487,646	**1994 to 1998**
1999	U.S. Merritt, Fry	307,724	**405,391**
2000	U.S. Merritt, Fry	361,800	
2001	U.S. Merritt, Fry	331,700	
2002	U.S. Merritt, Fry	45,700	**1999 to 2002**
			261,731

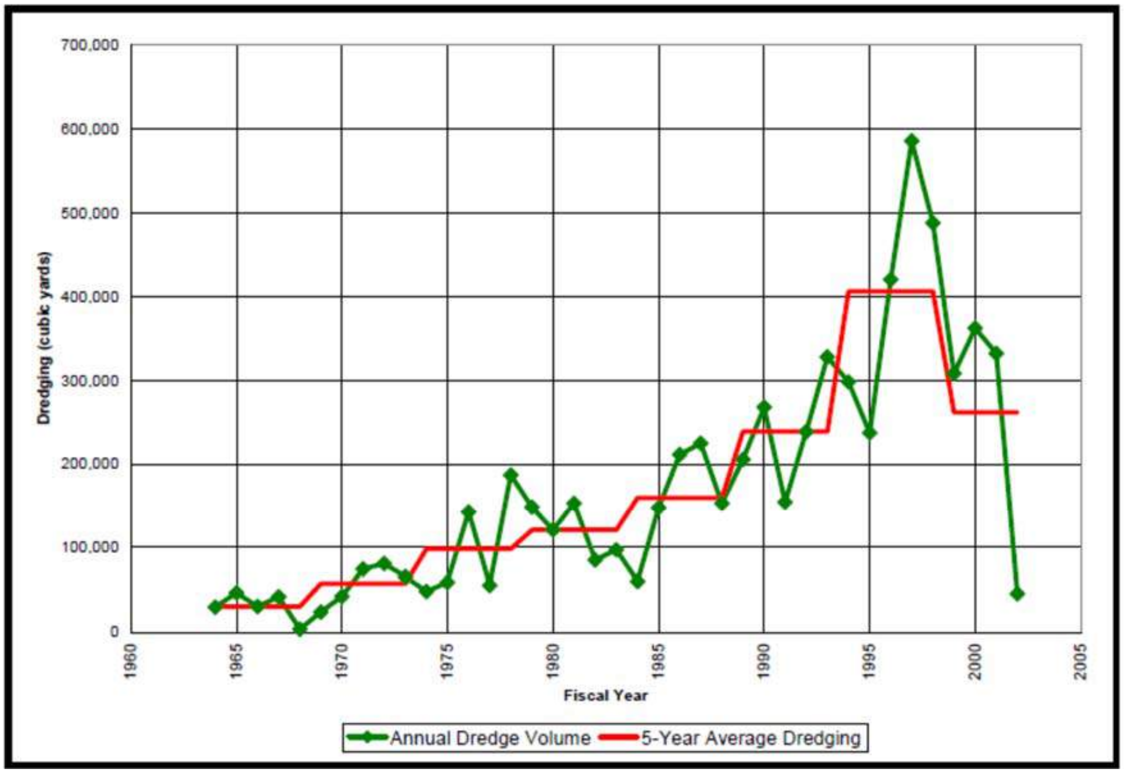


Figure 4-15. NRI Ocean Bar Channel Dredging History (from 2009 EIS)

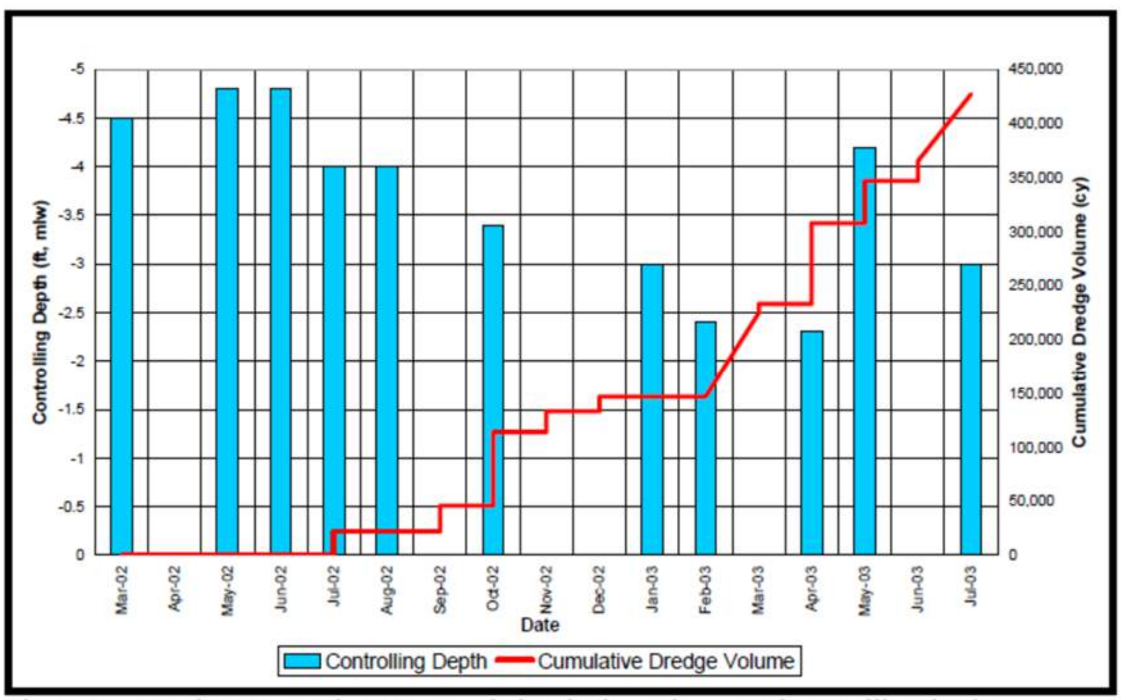


Figure 33. March 2002 to July 2003 cumulative dredge volumes and controlling depths.

Figure 4-16. Example cumulative dredge volume and controlling depths from March 2002 to July 2003 [from 2009 EIS].

NRI dredging activity can be seen steadily increasing from the 1960s to the 2000s. However, USACE funding was generally more available over this period whereas USACE maintenance navigation of the inlet and outer ebb shoal has been generally limited to $500,000 per year in federal funding. According to the USACE navigation district, approximately $1 Million per year (four quarterly dredging events) is the minimum required to keep the inlet navigable (USACE NRI presentation, 2017). Even under ideal conditions, navigation can be hazardous due to shoaling.

The 2009 EIS used the data presented in Table 4-1 to examine a 16-month dredge period to evaluate “controlling depths” over this period. The analysis states that:

> *During the approximate 16-month survey period, the controlling depth in the inlet channel never exceeded 4.8 feet MLW and reached a minimum controlling depth of 2.3 feet MLW in April 2003. This 16-month period, which is generally representative of the efforts by the Corps of Engineers and the results achieved, demonstrates that the present maintenance operations do not provide navigation interests with the channel dimensions needed to safely and efficiently navigate New River Inlet.*

Historical Cedar Bush Cut dredging was also analyzed for the 2009 EIS and a relative excerpt found:

> *Dredging in Cedar Bush Cut, the channel connecting the AIWW with New River Inlet; was initiated in 1976 with the removal of 730,600 cubic yards. Subsequent maintenance of the connecting channel was performed in 1978, 1988, 1997, and every year between 2000 and 2002. The dredging in Cedar Bush Cut is performed by hydraulic pipeline dredges with disposal of the dredged material on the north end of North Topsail Beach. A summary of the dredging in Cedar Bush Cut is provided in Table 6* [4-2 in this document]*. Over the 10-year period 1993 to 2002 inclusive, the costs for maintaining Cedar Bush Cut averaged over $133,000 per year.*

Table 4-2. Cedar Bush Cut Dredging History (from 2009 EIS)

Fiscal Year	Dredge(s)	Volume Removed (cy)
1976	Marion	730,600
1978	Marion, Northwood II, Richmond	747,300
1988	Arlington	124,900
1997	Marion and Richmond	92,000
2000	Richmond	15,800
2001	Blue Ridge	17,500
2002	Marion	154,200

Cedar Bush Cut was dredged in 2016, and the project overview is presented in Figure 4-17. Approximately 130,000 cy of beach-compatible material was dredged from Cedar Bush Cut, the AIWW, and the New River. Note that several sections of Cedar Bush Cut are significantly deep where dredging for navigation is not needed. Non-compatible material (e.g., clay, fines) can also occur, although sometimes these are just thin lenses. Figure 4-17 presents an overall schematic of the 2016 Cedar Bush Cut dredge project.

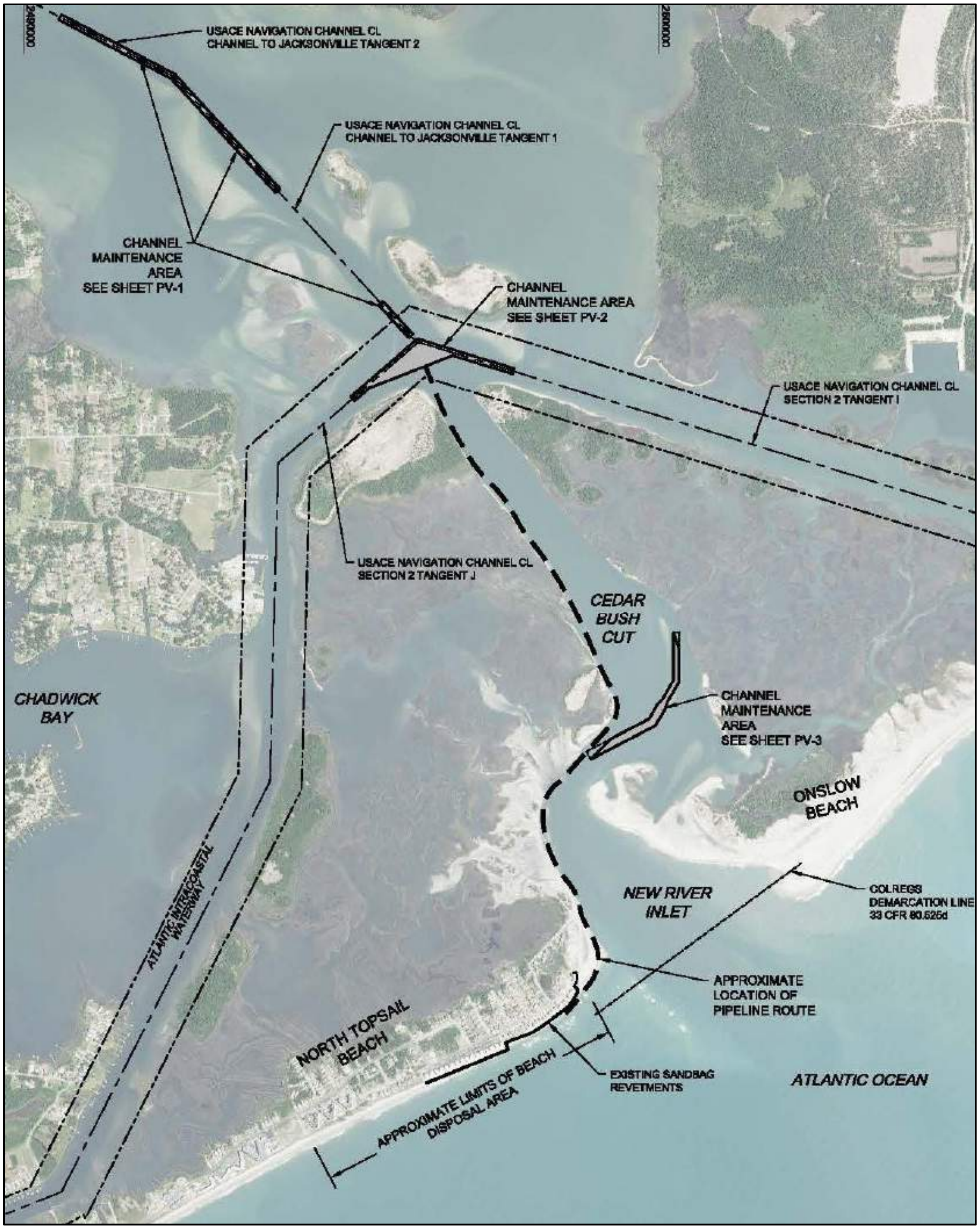


Figure 4-17. 2015 Cedar Bush Cut and New River/AIWW Dredging and Beach Placement Plan (Source: APTIM/CPE 2015 permitting document)

Figure 4-18 provides an example of a 2010 USACE project where only a small portion of the AIWW was dredged.

Figure 4-18: 2010 USACE AIWW Dredging Plans (only AIWW is included)

4.2.9 NORTH CAROLINA BIMP ON DREDGING AND NRI

The Beach and Inlet Management Plan (BIMP) (2006) and the Shallow Draft Inlet reports discuss NRI shoaling and dredging in detail. The following is an excerpt from the Shallow Draft Inlet report [North Carolina Department of Environment and Natural Resources (NCDENR), 2005]:

New River Inlet, NC Open Water and Beach

New River Inlet is located to the north of Topsail Island. The ocean bar channel (6 ft deep and 90 ft wide) has been dredged multiple times annually from 1980 to 2004 using USACE sidecaster dredges or the special purpose hopper dredge

CURRITUCK. The channel that connects the inlet to the AIWW is maintained by pipeline dredge (part of the AIWW inlet crossings contract) and the material is deposited on the beach at the north end of Topsail Beach. The inlet and its connecting channel have been dredged ninety-three times over the period of record, removing 6,592,485 cy of material. This is an average of 70,887 cy per project.

AIWW

Material from Tangents I and J (the New River Inlet crossing) is dredged annually and is placed on the beach at North Topsail Beach, 3,000 ft west of the inlet extending westward to Maritime Way. Additional dredging is placed in upland sites in the right of way adjacent to the AIWW.

Note that while the AIWW is dredged to 12 feet relative to mean low water (ft MLW) (+2 ft overdraft), the outer channel is only dredged to 6 ft MLW (+2 ft overdraft). Table 4-3 presents the dredge types available to USACE for dredging (source: NC Shallow Draft Report. Inlet and outer channel dredging is typically performed four times a year (quarterly) by side-caster, when funds are available.

Table 4-3. Excerpted Table from Shallow Draft Inlet Report (NCDENR/NCDEQ, 2005)

Table VII-2. Dredge Capabilities and Limitations by Type

Factor	Dredge Type		
	Sidecaster	Special Purpose – Small Hopper	Pipeline (cutterhead)
Range of Depth for Dredging	6 to 25 feet	6 to 25 feet	12 to 50 feet or more
Material Placement	Discharges to side of channel	Bottom dump - can transport sediment to nearshore waters, storage sump, or offshore disposal site	Pump to nearby location in-water or onto land
Ease of Deployment	Mobile, flexible	Mobile, flexible	Not self-propelled, pipeline must be installed
Environmental Windows	All Year	All Year	Nov. 16 – Apr. 30**
Wave Conditions	Ocean inlet capable	Ocean inlet capable	Restricted depending on wave size (Typically 3-4 ft.)
Approximate Average Daily Production (cy/day)	4000	1900	3500 (depends on size and downtime)
Approximate Average Cost per cy*	$2.38	$4.31	$6.88

* Costs based on most recent five years (FY 2000-2004)
** Beach and upland placement are restricted due to bird and turtle nesting and habitat considerations (see Regulatory Costs section)

Federal funding has been very limited to non-existent for the last few years; however, the State, Onslow County, and NTB have been able to provide funding to the USACE through a memorandum of agreement (MOA) for the interim to continue outer channel dredging. A future long-term funding plan is difficult to establish due to variations and unknowns with annual Federal and State budgets. Additionally, relatively little advance notice is provided for these annual budgets. The USCG removes the NRI navigation buoys when hazardous shoaling conditions occur. These navigation buoys have been removed for significant time spans (several months at a time) and some even discontinued (refer to USCG Notice-to-Mariner records).

4.3 SEDIMENT TRANSPORT PROCESSES

Gross transport is defined as the sum of sand movement directed both northeastward and southwestward, depending on wind and wave direction, currents, etc. Net transport at the site is defined as the difference between northeastward- and southwestward-directed littoral drift and is typically used when describing sediment transport.

In addition to alongshore sand transport, there is also cross-shore transport and transport in and out of NRI. Cross-shore transport refers to the movement of littoral material onshore (onto the beach) and offshore. Offshore transport is a common response of the beach during storms (i.e., formation of nearshore sand bar), while onshore transport is known to predominate during mild wave activity (i.e., movement of sandbar back onshore).

In terms of sediment transport in and out of NRI, sediment budget estimates indicate a “sink” of sand (material lost from the adjacent beaches and deposited into the inlet flood shoals near Cedar Bush Cut) ranging from 100,000 to 300,000 cy/yr (generated from both NTB and Onslow Beach shorelines). The proposed terminal groin is anticipated to reduce the amount of sand lost to this sink effect and, in turn, reduce annual maintenance dredging costs.

Terminal groins, as with all groins, typically hold sand on the updrift side (forming a fillet), with potentially detrimental effects to downdrift beaches under extremely erosional conditions. It is important to note that sediment transport along the southeastern coast is compartmentalized and does not constitute an integrated “river of sand” (Foyle et al., 2004; Mathews et al., 1980). In a regional net transport sense, NTB is downdrift of the proposed NRI terminal groin. However, locally, where the net transport is toward the northeast, the inlet throat itself is downdrift of any groin placed along the inlet margin (Figure 4-19). Therefore, terminal groin design must consider the potential impacts mainly to NTB itself, including the southwestern shoreline adjacent to NRI.

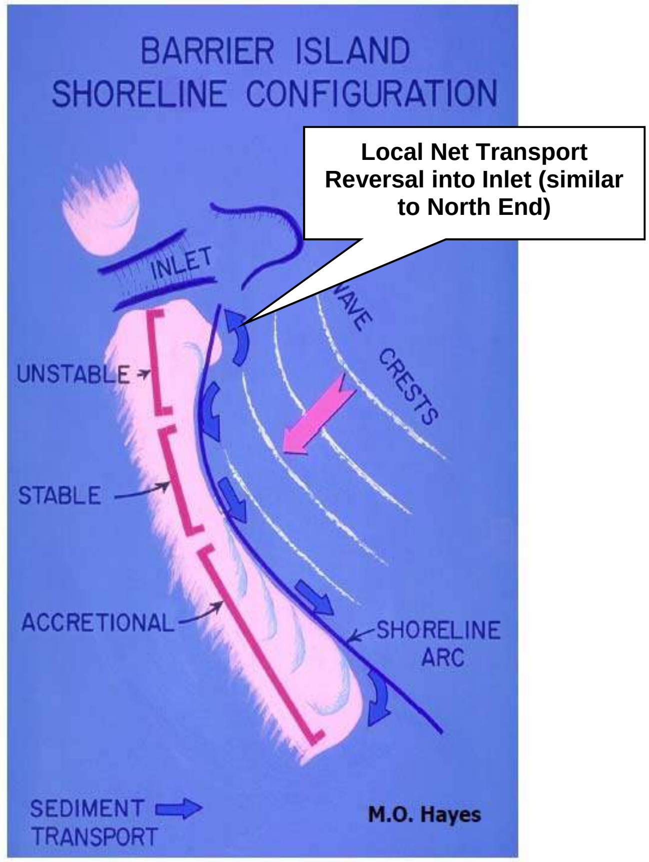


Figure 4-19 Generalized Net Sand Transport near an Inlet (Source: Hayes, 1979). Note that net transport reverses just below the inlet. The above schematic very closely resembles typical net transport trends on North Topsail Beach (i.e., unstable and most erosive on north end)

Nourishment is proposed to be included with any groin installation to minimize potential for negative downdrift impacts. Additionally, combining beach fill and groin structures is typically more effective than nourishment only in areas where longshore processes dominate, and adverse impacts can be minimized or avoided.

Significant work on sediment transport processes was included in the 2009 EIS and a relevant excerpt includes:

> *In general, longshore transport in the study area is predominantly to the southwest; however, reversals in longshore transport predominance, i.e., predominant transport to the northeast, may occur during the months of March through July. Also, two years in the 20-year hindcast record, 1988 and 1994, actually had predominate transport to the northeast.*

As seen in the following excerpted tables from the 2009 EIS, sediment transport can vary significantly both annually and seasonally (Table 4-4). Average net sediment transport was calculated to be 225,000 cy/yr to the south. Under more typical south/southwesterly wind-wave conditions in the spring, net sediment transport is typically to the north during spring-time months (Table 4-5). As expected, sediment transport is directly correlated with wave activity, and larger storms (e.g., hurricanes) move more material. Figure 4-20 presents monthly sediment transport rates and identifies several large storm events. Years with higher than normal wave activity generally result in higher sediment transport rates.

Table 4-4: Summary of computed longshore transport (cy/yr) using Wave Information Study (WIS) Station AU 2044 (source: 2009 EIS)

Year	Northeast	Southwest	Gross	Net[a]
1976	-348,700	438,600	787,200	89,900
1977	-277,900	430,800	708,700	152,800
1978	-357,300	469,600	826,900	112,200
1979	-405,100	608,200	1,013,300	203,100
1980	-275,800	817,900	1,093,600	542,100
1981	-317,700	559,900	877,600	242,100
1982	-184,300	444,000	628,400	259,700
1983	-280,300	591,500	871,800	311,100
1984	-309,700	625,700	935,400	316,000
1985	-237,700	538,000	775,600	300,300
1986	-226,000	547,800	773,800	321,900
1987	-172,100	559,100	731,200	387,000
1988	-320,900	298,000	618,900	-22,900
1989	-346,500	649,800	996,300	303,300
1990	-385,500	534,100	919,600	148,600
1991	-306,800	500,400	807,100	193,600
1992	-194,400	451,700	646,000	257,300
1993	-254,300	612,100	866,400	357,800
1994	-444,000	431,700	875,600	-12,300
1995	-288,700	328,100	616,800	39,400
Average	**-296,700**	**521,900**	**818,500**	**225,200**

[a] + = Net Transport to the Southwest, - = Net Transport to the Northeast.

Table 4-5: Average monthly transport rates (cy/yr) using WIS Station AU2044, 1976 to 1995. (Source: 2009 EIS).

Month	Northeast	Southwest	Gross	Net[(a)]
Jan	-30,600	53,600	84,200	23,000
Feb	-30,300	44,900	75,200	14,600
Mar	-55,500	47,000	102,500	-8,500
Apr	-38,000	34,400	72,400	-3,600
May	-27,000	28,100	55,100	1,100
Jun	-21,100	19,200	40,300	-1,900
Jul	-21,900	14,100	36,000	-7,800
Aug	-10,600	31,100	41,700	20,500
Sep	-8,800	78,500	87,300	69,700
Oct	-6,600	57,600	64,200	51,000
Nov	-19,500	60,000	79,500	40,500
Dec	-26,700	53,300	80,000	26,600
Total	**-296,700**	**521,900**	**818,500**	**225,200**

(a) + = Net Transport to the Southwest, - = Net Transport to the Northeast.

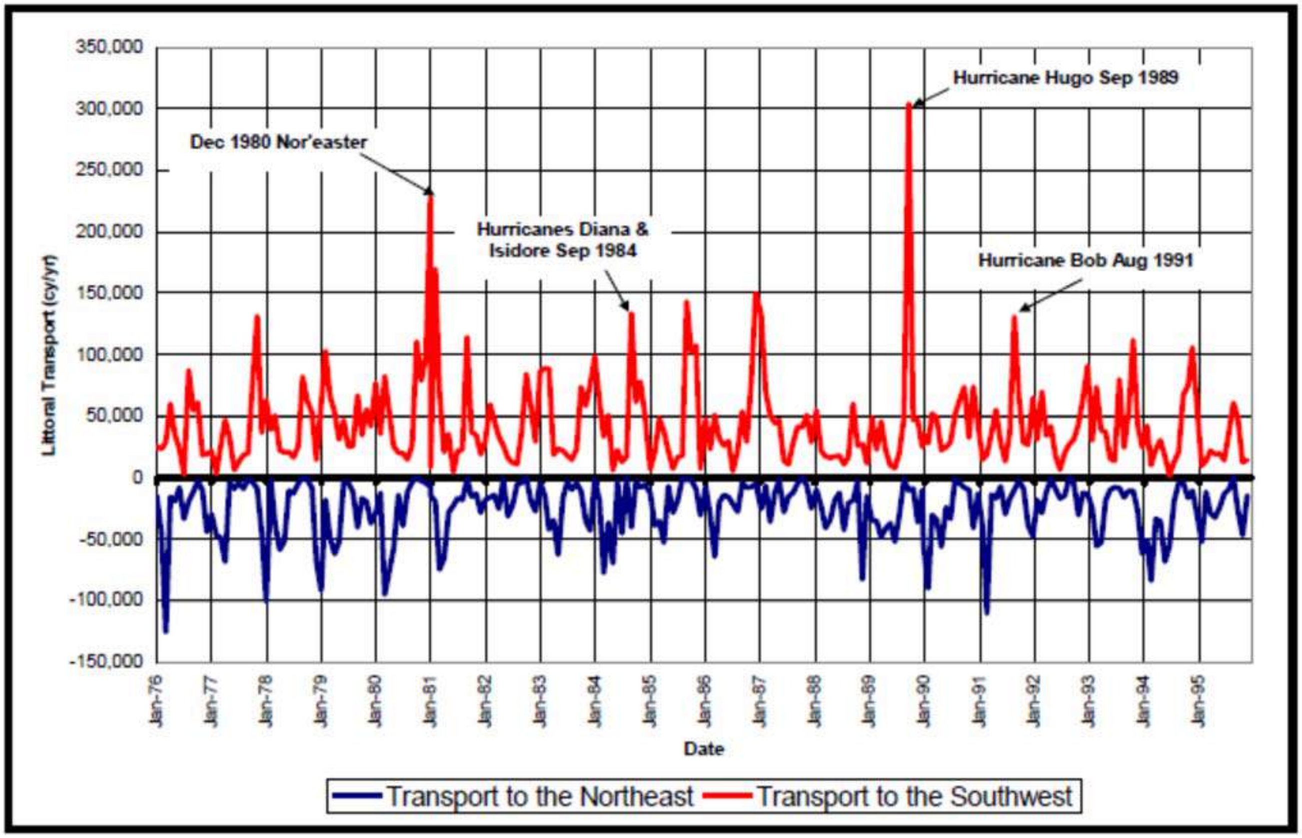


Figure 4-20. Monthly sediment transport, 1976 to 1995. (Source: 2009 EIS).

The 2009 EIS also discusses historical erosion rates along the project site:

> *The Corps recently updated shoreline change rates for North Topsail Beach using the 1963 topographic map and a 2002 topographic map developed for the Federal storm damage reduction feasibility study (USACE, 2004). The Corps 1963 to 1983 and 1963 to 2002 shoreline change rates are also shown in Figure 9* [Figure 4-21a].

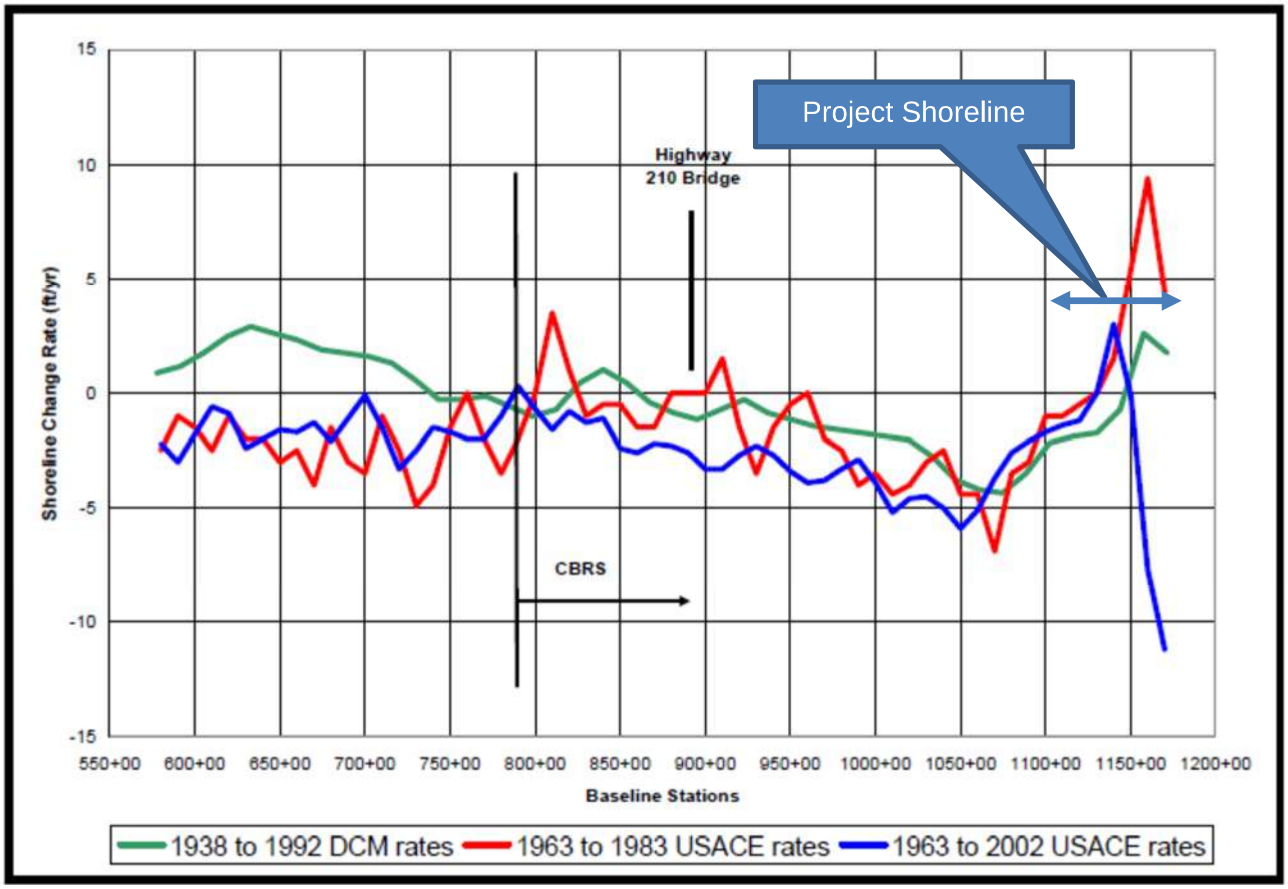


Figure 4-21a. Variability at NRI Stations (1090+00 to 1170+00). Shoreline change rates for three different periods (source: 2009 EIS). NCDCM developed the 1938 to 1992 rate while USACE determined 1963 to 1983 and the 1963 to 2002 rates.

All three sets of shoreline change information indicate an increase in shoreline recession rates from baseline station 810+00 northeast to around baseline station 1060+00 and then a general decrease in the recession rates from baseline station 1070+00 to around baseline station 1140+00. Between baseline station 1150+00 and New River Inlet, the shoreline change rates for the three data sets vary considerably due to the influence of New River Inlet.

Shoreline change rates immediately southwest of New River Inlet (baseline stations 1140+00 to 1160+00) were dramatically different for the two periods with the shoreline advancing at an average rate of 6.4 ft/yr between 1963 and 1983 and retreating at an average rate of 19.7 ft/yr from 1983 to 2002.

4.3.1 SEDIMENT BUDGET

A sediment budget is a conceptual way to visualize the general sediment transport along a shoreline and/or inlet. The 2009 EIS developed the sediment budget presented in Figure 4-21b.

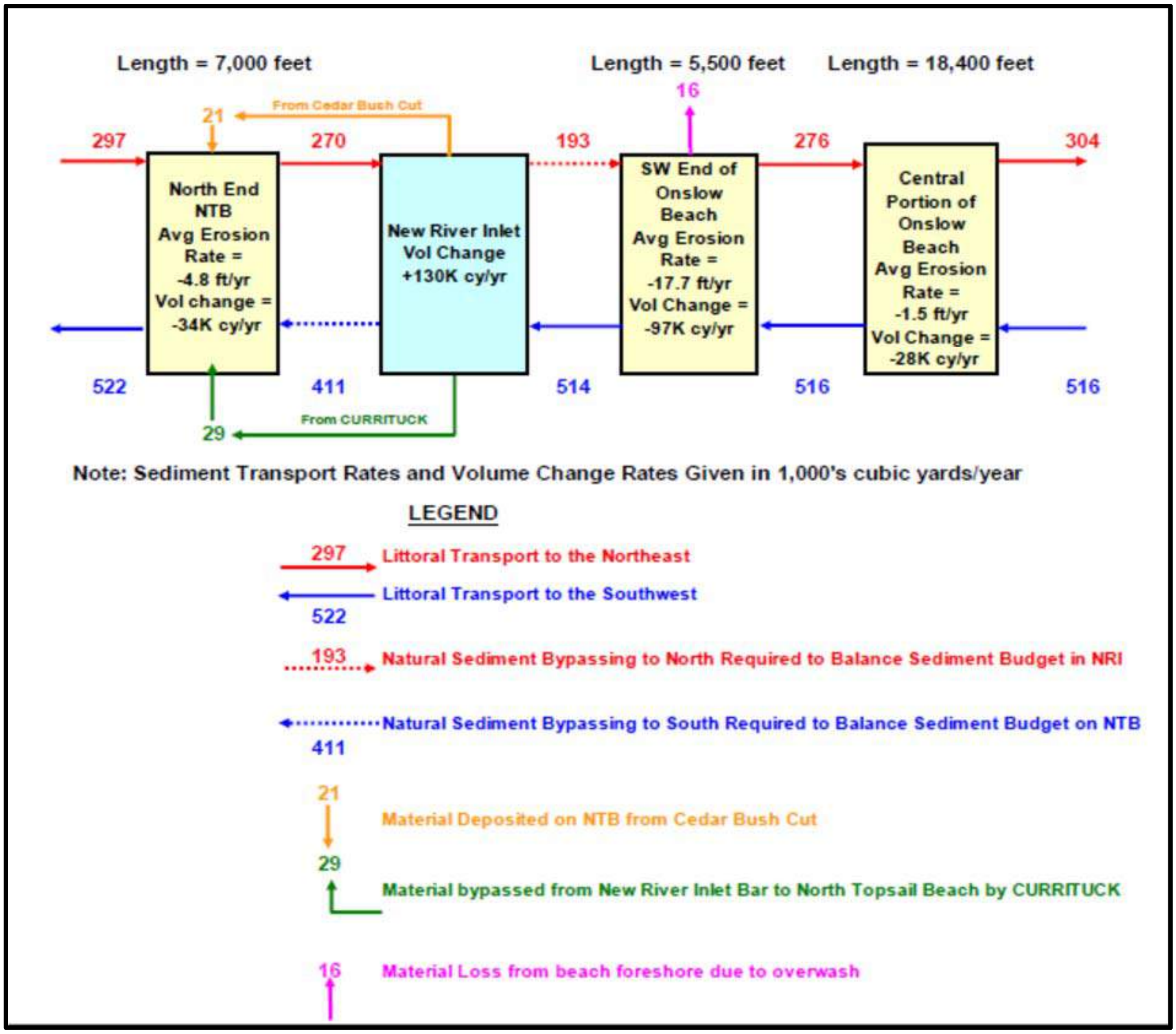


Figure 4-21b. 2009 EIS Sediment Budget for the Project Area

Dredging and erosion rates presented in Figure 4-21b are based on long-term data. As an example, Cedar Bush Cut dredged material deposited on NTB (21,000 cy/yr) was calculated based on the 1984-2003 average amount of material deposited on NTB.

A summary of the Figure 4-21b sediment budget was provided in the 2009 EIS:

Of the total volume of littoral material transported into New River Inlet each year (gross transport) is equal to 784,000 cubic yards. Of this total 52.4% (411,000 cy/yr) is naturally bypassed to the north end of North Topsail Beach with 24.6% (193,000 cy/yr) moving naturally back onto the south end of Onslow Beach. Artificial bypassing from Cedar Bush Cut and nearshore disposal by the CURRITUCK increased the total amount of material bypassed to North Topsail to 461,000 cy/yr or 58.8% of the gross transport.

4.4 PROJECT SITE BEACH MANAGEMENT ACTIVITIES

The primary cause of shoreline retreat along NTB is due to long-term erosion through natural processes of littoral sediment transport, sea level rise, and storm-related recession. In addition, tidal currents, wave focusing, and storage of sediment in the ebb and flood shoals of NRI have considerably affected the shoreline history of the project area. Along the project reach, erosion has been prominent due to the continual shifting and reorientation of the main ebb and flood channel(s) of NRI.

The project site has generally been nourished with material from the following sources:

- Cedar Bush Cut/AIWW
- New River Inlet
- Offshore

Prior to the 2009 management, the history of beach management was summarized in the 2009 DCM application as follows:

According to the North Carolina Division of Coastal Management (NCDCM), 26 permits have been issued for sandbags since 1992. The Atlantic Intracoastal Waterway (AIWW) was constructed in the early 1930's behind New River Inlet. This action connected the sounds behind Onslow Beach and Topsail Island with sounds to the north and south. In 1940 a navigation channel connecting the AIWW, adjacent sounds, New River, and New River Estuary with the Atlantic Ocean was dredged. The US Army Corps of Engineers (USACE) have repeatedly dredged New River Inlet for navigational purposes since 1964. Between 1964 and 1978 dredged material was sidecast from the channel to a point immediately outside the channel (total ~750,000 cy). From 1978 to 2002 a combination of *hopper and sidecast dredges were utilized. The hopper dredge deposited dredged material offshore of the adjacent beach in 10 to 15 ft of water (total ~765,500 cy to 2002), while the sidecast dredge deposited material to a point immediately outside the channel (total ~5,837,000 cy to 2002). In the northern reach of the Permit Area, 1,880,000 cy of dredge material has been deposited on the north end of North Topsail Beach since 1976.*

Since the development of the 2009 EIS, several nourishment and stabilization projects have occurred within the project area. These are discussed further in the following sections. Note that historical fill activities alone have not been able to overcome background erosion on the Phase 1 shoreline (project site).

4.4.1 NEW RIVER INLET CHANNEL REALIGNMENT

One of the primary premises of the 2009 EIS was to create a new alignment to the NRI outer ebb shoal to 1) improve navigation and 2) attempt to move the ebb shoal south, similar to where it was in the 1980s.

As explained by APTIM/CPE in a 2016 sandbag revetment variance request:

According to the FEIS, the erosion of the shoreline south of New River Inlet has been a persistent problem since around 1984 when the bar channel of New River Inlet shifted its alignment toward Onslow Beach. Prior to 1984, the north end of North Topsail Beach was accreting at an average rate of 6.1 feet/year. Following the change in channel position and orientation, the north end began to erode at an average rate of 5.3 feet/year. Most of the accelerated erosion was attributed to the higher degree of exposure of the north end to wave energy. That is, prior to the channel shift, the south side of the ebb tide delta provided a breakwater effect with waves breaking relatively far offshore. With the loss of the south side delta, more wave energy was able to be transmitted directly to the shoreline. This, combined with the development of flood channels running close to and parallel to the north end, greatly increased sediment transport rates to the north.

This project was permitted by the 2009 EIS, and dredging/fill, known as the 1st Event, as it relates to different NTB shoreline reaches and timing, occurred in 2013. This project was estimated to last approximately 4 years, however, the ebb shoal borrow area filled in approximately twice as fast, with project navigation benefits lost after 2 years.

The project was successful in that it provided navigation benefits for 2 years and is a good source of borrow area material for beach placement. Unfortunately, the placed fill was also lost relatively quickly near the NRI inlet around Stations 1040+00 to 1165+00. The shifting of the ebb tidal delta (Figure 4-22a) was not achieved, and erosion rates were higher than anticipated following the nourishment. This necessitated the installation of a sandbag revetment to protect structures along this section of shoreline.

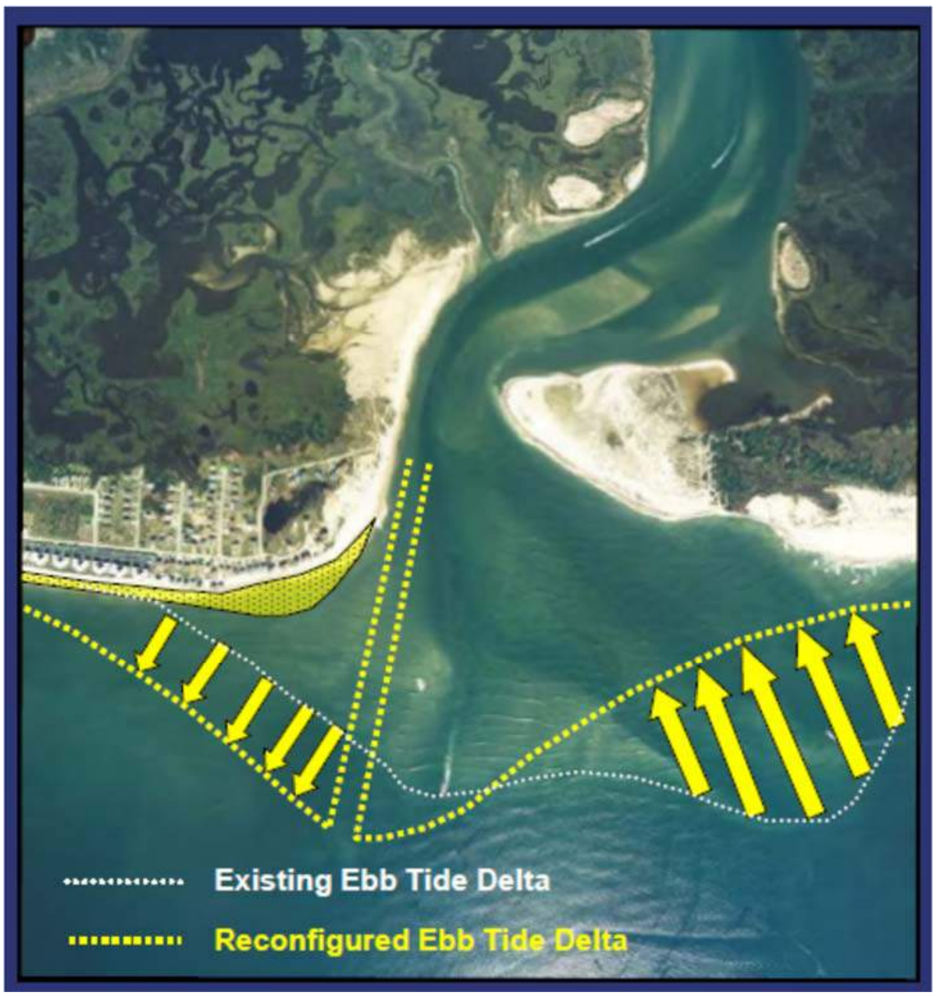


Figure 4-22a. Design Goal of Channel Realignment was Ebb Tidal Delta Shifting South (figure source: APTIM/CPE)

4.4.2 SANDBAG REVETMENT

The sandbag revetment location along the project area is shown in Figure 4-22b. Sandbags (shown in black) are continuous along approximately 3,300 feet from about Station 1163+00 to about 1135+00. Another approximately 300-foot sandbag section is located just south of Station 1170+00. They were constructed in phases and are protecting more than 35 structures, including 8 large condominium units (Topsail Reef Condos, from 1135+00 to 1148+00) and at least 10 duplexes (totaling over 200 residential units).

Figure 4-22b. Sandbags (shown in black) Continuous along ~3,300 feet from ~Station 1163+00 to ~1135+00. Another ~300 ft sandbag section is located just south of Station 1170+00.

The history of the sandbag revetment is provided in this excerpt from an APTIM/CPE 2016 Realignment Study:

> *The erosion of the fill material placed seaward of the homes north of Topsail Reef during the Phase 1 project left these structures in imminent danger comparable to the conditions of the structures prior to the construction of the Phase 1 project. This prompted the Town of North Topsail Beach to construct a sand bag revetment to provide temporary erosion control along this section of shoreline.*

Figures 4-23 through 4-26 present November 2017 photos of the sandbag revetment. Sandbags are shown in these photographs, but a geotube that runs shore-parallel just seaward of the sandbags is buried and not visible (Figure 4-27). Geo-tubes and sandbags are made of similar materials and generally perform the same duty, however, geotubes are generally much longer than sandbags.

Figure 4-23. Sandbags Just South of 1170+00 (November 2017)

Figure 4-24. *Sandbags near ~1163+00 (November 2017)*

Figure 4-25. *Sandbags fronting Topsail Reef Condos, which Extend from ~1135+00 to ~1149+00 (November 2017)*

Figure 4-26. Sandbags along Northern NTB Looking into New River Inlet (2015 photo)

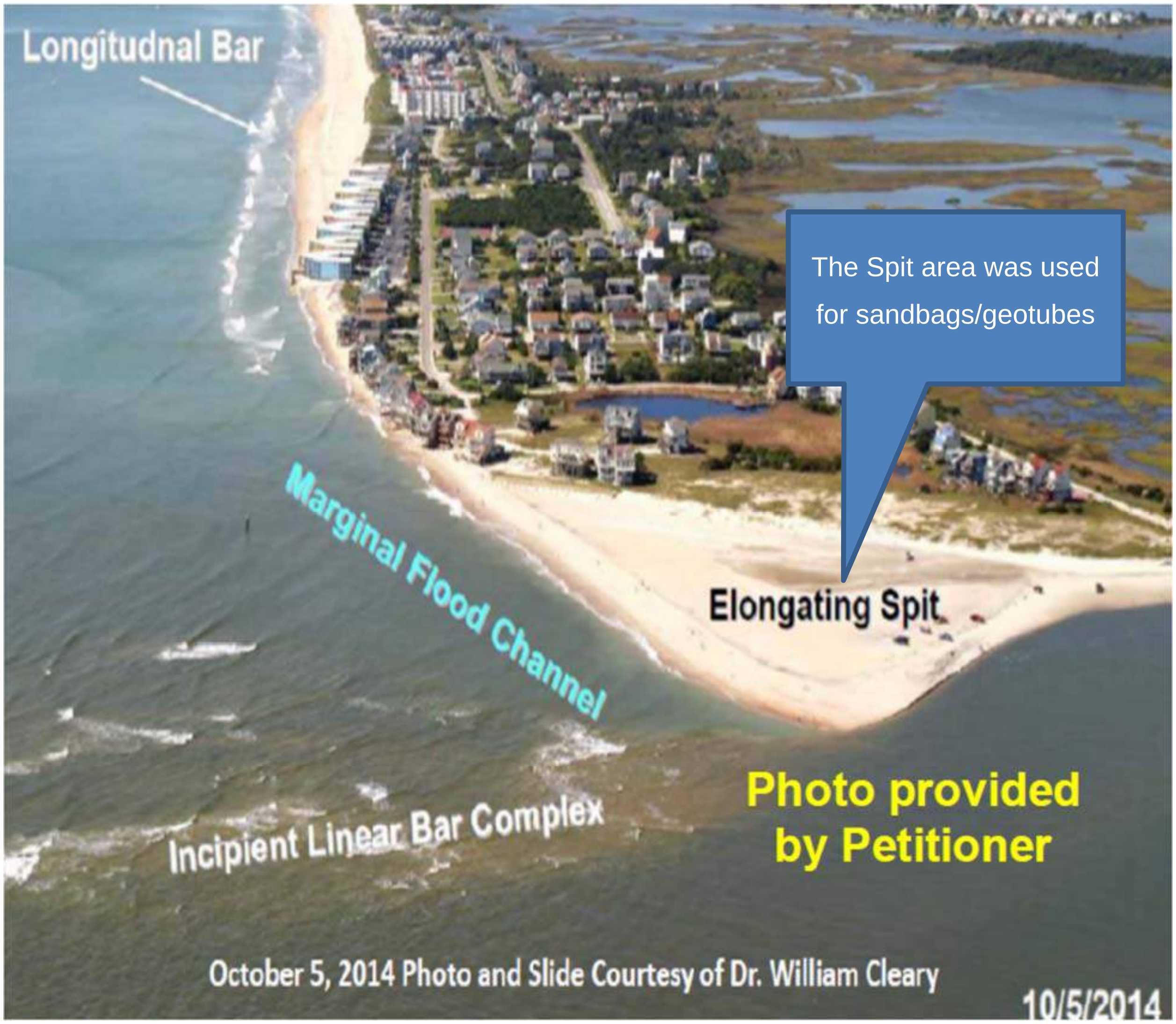


Figure 4-27. Image Showing the Spit Area that was Used for Geotube and Sandbag Filling

The general timeline of the sandbag revetment is provided as follows:

- February 2012 – Topsail Reef condominiums was issued a permit to install about 1,500 feet of sandbag revetment.
- April 2012 – A nor'easter event damaged some of the under-construction sandbag revetment.
- May 2012 – A modification to the sandbag revetment was granted to allow for a larger and taller structure along Topsail Reef condominiums.
- March 2013 – 1st Event Channel Realignment Nourishment completed.
- August 2014 – Permit application for 1,450 feet of shoreline sandbag revetment (7.5-foot-tall and 45-foot-circumference tubes) north of Topsail Reef condominiums.

- September 2014 – Modification request proposing to place 35,000 to 50,000 cy in a long geotube fronting the sandbags north of Topsail Reef condominiums. The proposed borrow site was a 5-acre area along the inlet shoreline known as the spit.
- February 2015 – DCM sent a letter to NTB about beginning removal of the structure.
- August 2015 – NTB filed another variance request related to permitting the structure.
- March-April 2016 - The Cedar Bush Cut places 130,000 cy of material in front of the sandbag revetment.
- June 2016 – DCM issues another letter about removing the sandbags.

Recently, the Town was seeking a new engineering firm (April 2018) to maintain the sandbag revetment system because there are currently some weak spots where wave run-up and overtopping occur during spring tides and elevated wave conditions.

Figure 4-28 presents a schematic of one cross-section of the sandbag revetment. It is estimated that sandbag revetment construction and maintenance has cost more than $3.5 million, not including lawsuit fees. The lawsuit was largely based on the performance of the sandbag revetment and has been settled.

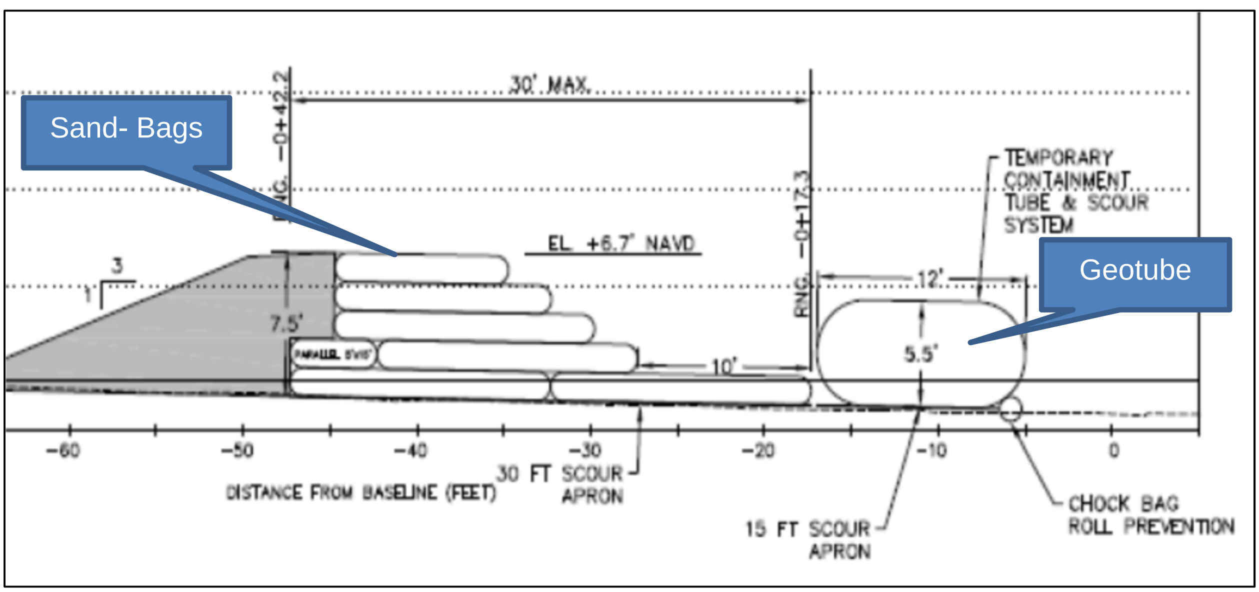


Figure 4-28. Sandbag Revetment with Geotube Included (source 2016 APTIM/CPE variance request). Note that the revetment cross-section formation (e.g., height, footprint) can vary based on shoreline location.

4.4.3 INLET MANAGEMENT PLAN

An inlet management plan (IMP) developed for the 2009 EIS project included all reaches of NTB shoreline and NRI monitoring. This IMP has served as the basis for project planning and permitting since its development. It will continue to serve as the basis for future monitoring and management activities, however, some additional elements may be needed based on the project described herein.

A few relevant conditions related to Inlet Channel Maintenance Events (i.e., channel realignment dredging of the outer ebb shoal) include two threshold triggers:

3.a) No maintenance event can be initiated unless one or both of the following thresholds, as stipulated in the Final EIS, have occurred: 1) only if shoaling of the new channel totals 85 percent of the actual dredged volume of the initial construction and/or 2) only if the channel thalweg migrates outside of the 500-foot-wide corridor established during the initial construction.

3.b) Channel maintenance will be limited to a maximum of once every 4 years during the 30-year period, resulting in no more than seven (7) maintenance events.

The standard November 16 to March 31 window for dredging events also applies, although modifications can be requested for any existing permit conditions.

4.5 ONSLOW BEACH PROJECT AREA MONITORING

The portion of Onslow Beach within the project study area has not been nourished. This is an undeveloped section of the beach that is also in a COBRA zone. The COBRA zone extends approximately 4 miles to the northeast (Figure 4-29).

Figure 4-29. The Majority Onslow Beach is COBRA Zone, including the Project Area

While no active beach management activities have occurred along the southwest portion of Onslow Beach, the Town of North Topsail Beach has been monitoring the southwestern portion of Onslow Beach shoreline as a component of its 2009 IMP. This portion of the shoreline is within the influence of NRI and monitoring its shoreline erosion/accretion helps in providing a comprehensive view of NRI inlet sediment processes.

5.0 AVAILABLE ALTERNATIVES

Several alternatives are considered in this section for the proposed project, including:

1. Threatened structure relocation and/or abandonment (buyout),
2. Beach nourishment without inlet channel realignment,
3. Beach nourishment with inlet channel realignment,
4. Terminal groin with beach nourishment (without channel realignment),
5. Terminal groin with beach nourishment (with channel realignment), and
6. Jetty with beach nourishment.

The alternatives are described briefly in the following subsections. The more feasible alternatives listed will be further analyzed in Sections 6 through 9 of this report.

5.1 NO-ACTION ALTERNATIVE

The no-action alternative would allow erosion to continue and would result in the loss of additional property. Under this alternative, the Town of North Topsail Beach is assuming that USACE dredging projects (Cedar Bush Cut, AIWW, New River, NRI) will continue and that any beach-compatible material will be placed on the Phase 1 project reach. While this alternative can offset some background erosion, properties would likely be condemned and require removal where homes and infrastructure are impacted in the long term.

The no-action alternative would also likely limit beach recreation and tourism due to reduced access and minimal available dry beach at higher tides. The existing 3,600-foot sandbag revetment will require continual maintenance to prevent failure and may even need to be enlarged over time to protect the 35 imminently threatened structures. The sandbag revetment will fail over time without maintenance.

The No-Action Alternative from the 2009 EIS states:

> During the past year, 17 duplex structures located at the extreme north end of Town, which have a total tax value of over $17 million, have become imminently threatened. Two (2) of the imminently threatened duplexes were relocated to other parts of North Topsail Beach at the expense of the property owners. Six (6) of the remaining duplexes have been declared uninhabitable due to the loss of water,

sewer, and electrical connections and were demolished in February 2009 at a cost to the Town of $2 million.

Table 5-1 presents an average economic impact of the no-action alternative from the 2009 EIS. Total damages and losses exceed $26 million for the North Section alternative (Station 950+00 to NRI).

Table 5-1. Average annual economic impact of the 2009 EIS No-Action Alternative (source: 2009 EIS)

Economic Impact	No Action Alternative		
	Central Section	North Section	Total
Damages & Losses			
Erosion & Storm Damages	$5,738,200	$17,688,400	$23,426,600
Rental Income Loss	$529,500	$3,709,800	$4,239,300
Reduction in Household Spending	$207,000	$5,437,600	$5,644,600
Total Damages & Losses	$6,474,700	$26,835,800	$33,310,500
Reduction in Tax Revenues			
Town Ad Valorem	$31,700	$115,500	$147,200
County Ad Valorem	$46,900	$172,000	$218,900
Sales Tax (Local & State)	$14,600	$380,600	$395,200
Accommodation Tax	$31,800	$222,800	$254,600
Total All Tax Revenues	$125,000	$890,900	$1,015,900

Figure 5-1 presents a 1996 aerial of the NTB shoreline just northeast of the Topsail Reef condominiums, where 11 homes/structures are no longer present. The no-action alternative does not address the Town's purpose and need to restore eroded beaches and to provide a widened dry-sand beach for storm buffer as well as recreational and habitat reasons.

Figure 5-1. Comparison of 1996 and 2016 Aerials Showing 11 Lost Homes

Under the no-action alternative, buildings will eventually become undermined. Public and private use of the beachfront would be adversely affected by the presence of failed structure(s) along the shoreline. Once a structure is on active public trust beach, it can either be left to deteriorate or removed. Derelict structures would hinder the public's recreational use of the shorefront and represents a hazard to the public and wildlife (e.g., nesting sea turtles). Additionally, an erosional shoreline can become a stigma for the community as a whole, where all real estate values are negatively impacted.

Addressing abandoned structures on an active beach has many legal ramifications. Theoretically, removal of the structure would be the responsibility of the landowner. However, several recent legal cases involving the other municipalities (e.g., Town of Nags Head) versus owners of condemned houses puts this assumption in question (K&L Gates, 2012). In any event, potentially dozens of adversely impacted properties would require removal in the long-term, while others are in short-term jeopardy (i.e., imminently threatened) due to sandbag revetment failures from episodic storm events. This is not a long-term practicable alternative considering the possible damage to the oceanfront environment due to derelict structures and the potential cost to the Town for removal of condemned structures and legal fees.

From a short-term perspective, the no-action alternative results in little to no recreational beach at high tide, which affects tourism and rental properties (with associated indirect impacts). From a natural resources perspective, sea turtle nesting habitat would likely decrease and require more nest relocations.

5.1.1 THREATENED STRUCTURE RELOCATION

Relocation of buildings within the Town of North Topsail Beach, away from the path of the eroding beach, is an expensive alternative and does not meet the Town's purpose and need. Structure relocation can also be exceedingly expensive. For example, a 2006 Nags Head EIS estimated a cost of $1,579,000 to relocate a house to a non-oceanfront lot (including condemned property losses and new property acquisition). This was based on a 1,350 square foot (ft^2) footprint for a two-story 2,700 ft^2 home (CSE, 2006).

Similar to the 2009 EIS, the economic impact of the no-action alternative in Section 9 includes an assessment of the following:

1 Costs for installing temporary sandbags to protect threatened structures,

2 Cost for relocating and/or demolishing threatened structures, and
3 Potential storm damages to existing structures and infrastructure.

Relocation of these structures to non-waterfront locations would diminish their value as vacation rental, primary residence, and/or investment properties. The Town and Onslow County would lose revenue from the loss of the eroded property as well as the tourism-driven economic benefits derived from these properties. Finally, relocation of structures does not address the loss of the beach itself.

5.1.2 PROPERTY BUYOUTS

FEMA has buyout assistance programs for properties that are in jeopardy of being destroyed, however, most of the project area is in a COBRA zone, therefore no FEMA assistance can occur.

Even for the few residences that are not located in the COBRA zone, FEMA buyout programs are geared generally toward lower income owners and properties that are categorized as a primary residence. Qualification for such funds is prioritized for those primary residences that have experienced a repetitive loss or that have owners who are currently displaced in temporary housing.

Due to the resort nature along NTB, high property values, and current status of most of the properties, it is highly unlikely that FEMA would qualify these properties for buyout funding at this time. The voluntary buyout program for Superstorm Sandy also exhibited a similarly unfavorable response from most resort-destination communities (Schuerman, 2013).

The Heinz (2000) report found that:

> A previous attempt to encourage removal and relocation of threatened structures—the Upton-Jones Program, which existed from 1987 to 1994–was suspended because of limited usage and unintended outcomes. A relocation program, if pursued, would have to be carefully designed to avoid the shortcomings of the Upton-Jones Program.

Additionally, a recent study of the beaches in the state of Delaware by Parsons and Powell weighs the cost of beach retreat against the cost of beach nourishment over the next 50 years. The study concluded that the cost of retreating from eroding coasts will be approximately four times the cost of renourishing the state's beaches (Parsons and Powell, 2001).

Salvesen (2004) also noted that buyout programs can have disadvantages, including:

- High up-front cost
- Reduced local tax base
- Disrupted neighborhood
- Potential increased housing costs (in the short term)
- Incomplete participation limits effectiveness
- Higher costs of replacement housing

5.2 BEACH NOURISHMENT WITHOUT INLET CHANNEL REALIGNMENT

This alternative avoids using the Inlet channel realignment (e.g., 1st Event in 2013) as a borrow area. Phase 1 project reach beach nourishments without inlet channel realignment have been implemented using material from Cedar Bush Cut and the AIWW/New River. However, these project volumes have not been able to keep up with background erosion of the project area shoreline. These projects have improved navigation in inland waterway reaches, however, inlet navigation remains the most problematic issue, and an outer ebb shoal project is needed to optimize inlet navigation.

5.3 BEACH NOURISHMENT WITH INLET CHANNEL REALIGNMENT

This alternative features the inlet ebb channel realignment as the preferred borrow area. The inlet main ebb channel orientation has been cited as having a direct effect on erosion/accretion trends on the adjacent NTB shoreline.

The USACE policy of "dredge following deep water" for maintaining its established channel (90 feet wide, 6 feet deep) will not provide the realignment necessary and, therefore, the Town will be the primary sponsor of this realignment project. This project is a good source of beach compatible sand and is a critical component to making NRI navigation safer.

5.4 TERMINAL GROIN WITH NOURISHMENT (WITHOUT CHANNEL REALIGNMENT)

An additional alternative proposed herein is the construction of a terminal groin with beach nourishment not from the ebb channel. This alternative is discussed in detail in subsequent sections. For this terminal groin alternative, it is assumed that nourishment material will *not* come from a channel realignment project. Rather, the nourishment material may come from several sources:

- Offshore
- Upland
- Cedar Bush Cut/AIWW/New River
- USACE Confined Disposal Facility (CDF) DA43

Borrow site options are described in more detail in Section 6. The upland and inland waterways generally do not have enough sand required for the project in the long term. The offshore borrow areas do have enough material for long-term use, however incompatible (e.g., rocky) material has been a problem.

5.5 TERMINAL GROIN WITH CHANNEL REALIGNMENT

A terminal groin with the channel realignment project is included in the analysis. Nourishment material for the groin will come from channel realignment projects. In addition to simply acting as a sand source, the channel realignment strives to relocate the channel in such a way as to lessen shoreline erosion along the Phase 1 reach. This project alternative features ebb channel dredging and, therefore, is generally favorable for NRI navigation.

5.6 JETTY WITH BEACH NOURISHMENT

A jetty system has been discussed by NTB and Onslow County staff as well as commercial and recreational fishermen that require access to the ocean. Figure 5-2 presents an example schematic of a two-jetty system. The jetties extend out to approximately the -10 ft referenced to the North American Vertical Datum of 1988 (NAVD88) depth contour and are approximately 3,000 feet in length. These lengths are similar to the Masonboro Inlet Jetties (Figure 5-3) approximately 35 miles southwest of the project area.

These jetty structures will block the majority of natural sediment bypassing NRI, which will cause areas of erosion and accretion. From an accretion perspective, sand fillets will form on the updrift/downdrift sides of the jetty system. A weir could conceivably be constructed, similar to the Masonboro Inlet jetties (Figure 5-3), which is the closest jetty system to the project site and has similar coastal processes/conditions. As with all hardened structures (e.g., groins, bulkheads, seawalls, revetments), jetties require an ongoing nourishment program and monitoring.

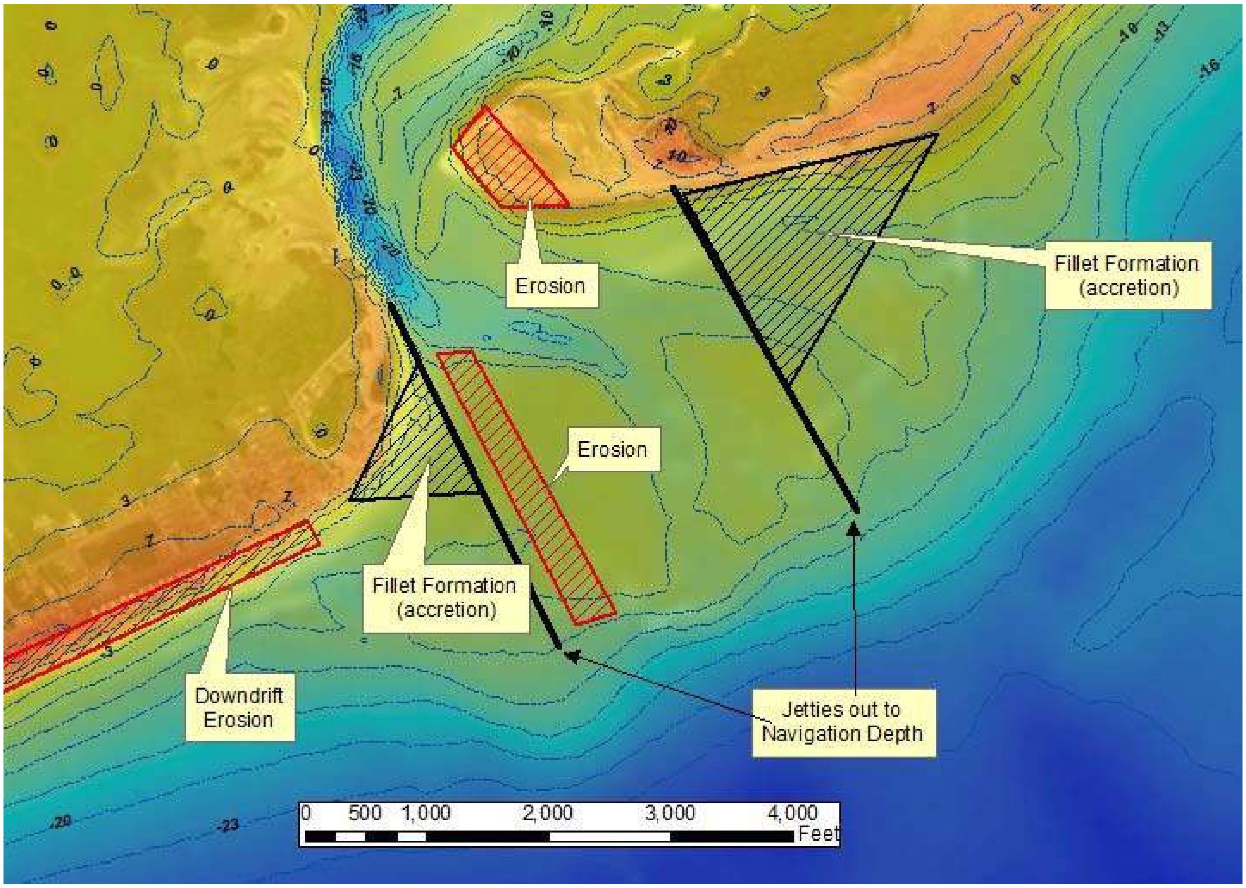


Figure 5-2. Example Jetty Schematic. Two jetties shown extending out to ~-10 ft NAVD88 depth contour.

The Masonboro Inlet jetties are considered to be working as designed (GAO, 2002). Jetties were also studied extensively for Oregon Inlet, however, these structures have not been constructed.

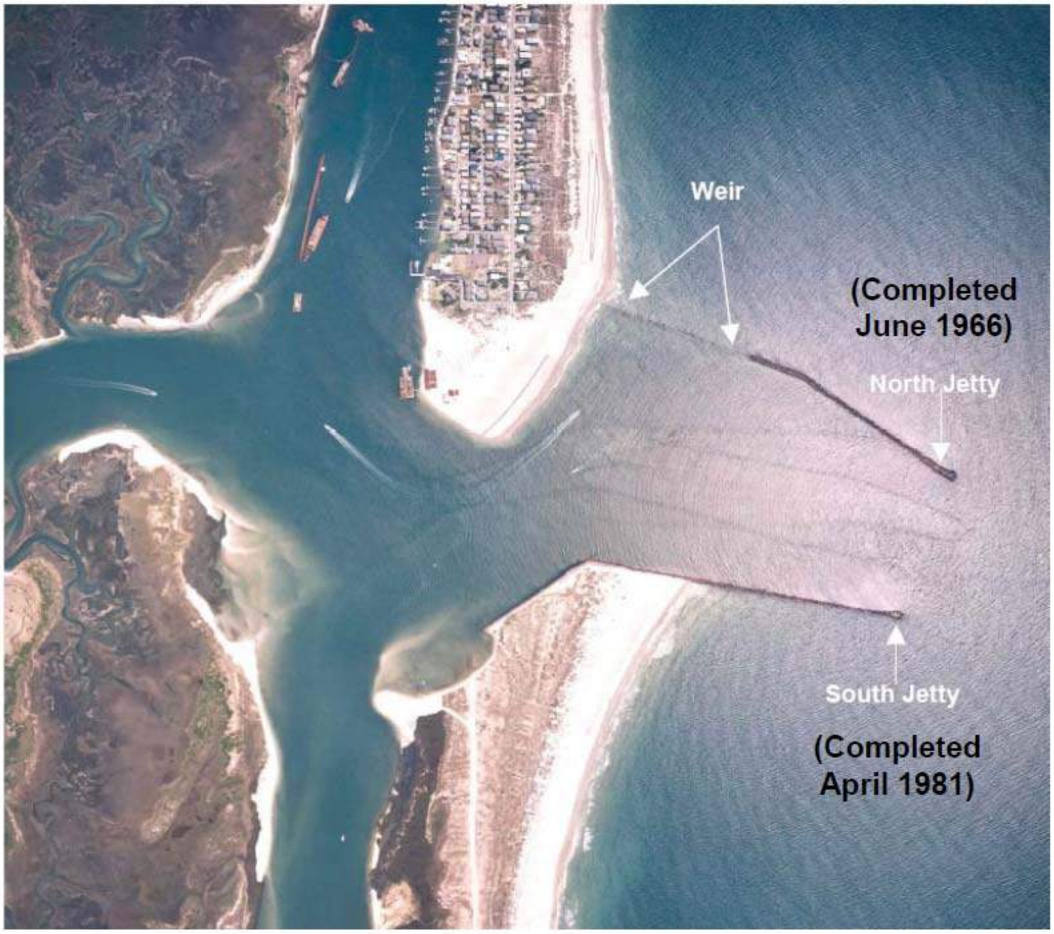


Figure 5-3. Masonboro Inlet Jetties (source: USACE, 2002)

As with NRI, Oregon Inlet can also be treacherous for navigation. The GAO Oregon Inlet study (2002) stated:

> This high-energy environment often creates sand bars and large breaking waves at the inlet's entrance to the ocean, commonly known as the ocean bar. These conditions, especially when combined with the severe storms that frequent the area, can swamp a boat or run it aground, imperiling both life and property.
>
> According to the U.S. Army Corps of Engineers and U.S. Coast Guard data, over the 40-year period 1961 through 2001, hazardous conditions in the inlet were a factor in 25 deaths and the loss of 22 vessels.

Figure 5-4 shows the USACE design for the proposed Oregon Inlet jetty project. The project included construction of dual rock jetties about 3,000 feet apart. The north jetty was to be approximately 10,000 feet long. The south jetty was to extend about 3,500 feet beyond the Pea Island terminal groin, for a total length of about 6,600 feet.

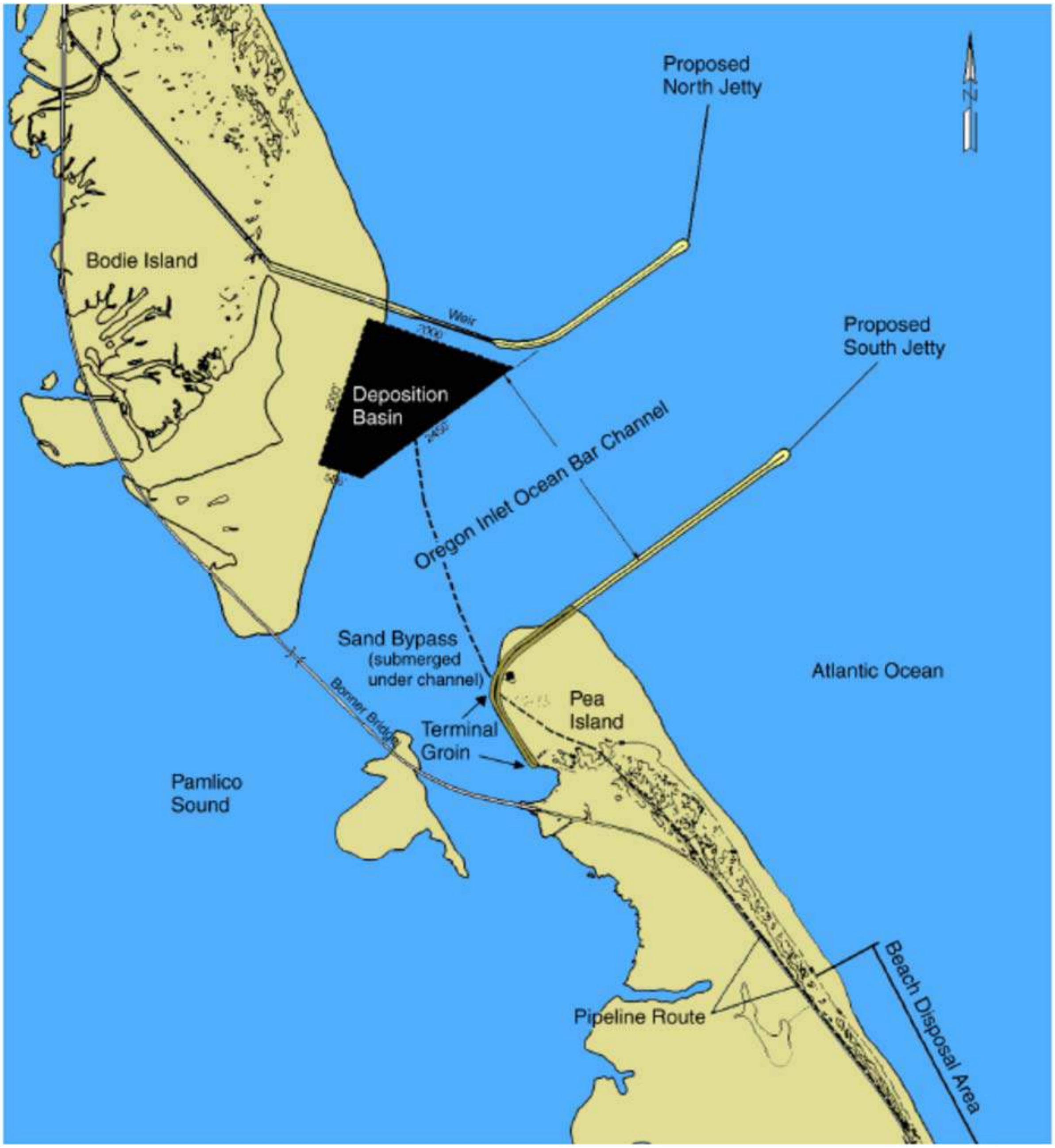


Figure 5-4. USACE Proposed Jetty Design for Oregon Inlet (GAO, 2002).

In addition to a dual jetty system, a single jetty is also a potential alternative to aid in navigation and ease erosion along NTB. Figure 5-5 presents a conceptual image of a single jetty system at NRI. In general, jetties extend out to beyond the ebb shoal and generally block sediment transport almost completely. Therefore, areas of accretion and erosion can be anticipated based on sediment transport processes.

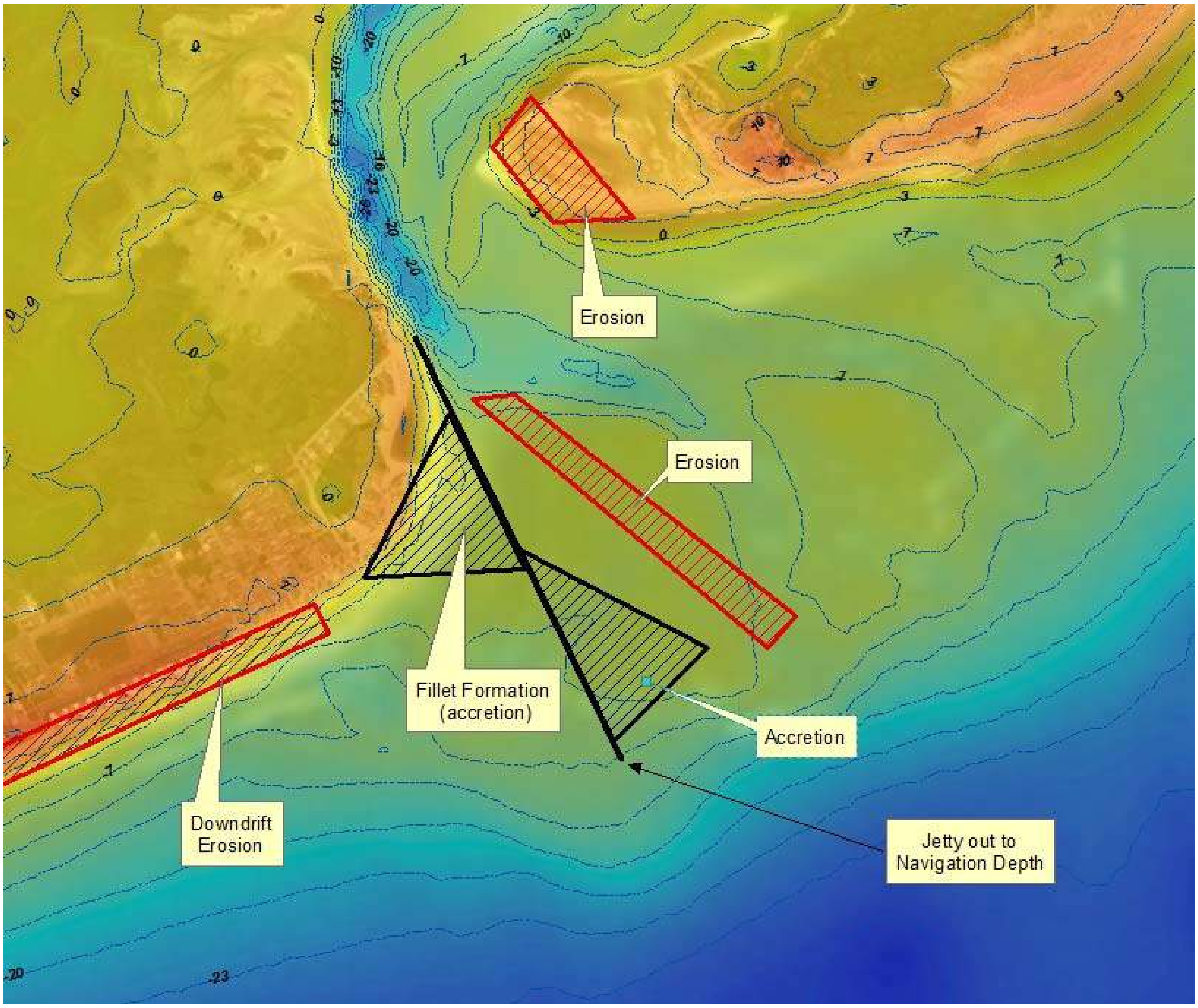


Figure 5-5. Single jetty alternative schematic.

At Masonboro Inlet, the single-jetty system was not deemed successful in maintaining navigation. Migration of the navigation channel toward the north jetty was cited (which did not align well with the inlet channel). This migration was not caused by the weir, but was a typical channel response to a single-jettied system (Kieslich, 1981)

The dual jetty system is conceptually estimated to cost $20 million to $25 million, while a single jetty system is conceptually estimated at $12 million to $15 million. A jetty system would require a lengthy design, analysis and permitting process and is currently not recommended for NRI and will not be included in Sections 6-9.

5.7 CHANNEL RELOCATION

Another option potentially available at the project site would be channel relocation, not just realignment. One nearby inlet that was successfully relocated was Mason Inlet. Figure 5-6 presents a schematic of this at NRI, where a new inlet approximately 500 feet wide would be excavated through Onslow Beach. Excavated material would be placed into the existing NRI channel to close it. The Mason Inlet relocation project included temporary sheet pile and geotubes during construction (see Figure 5-7), and an NRI channel relocation would likely use similar methodologies. The channel relocation is conceptually estimated to cost $10 million to $15 million and would require a lengthy design, analysis and permitting process. This is currently not recommended for NRI and will not be included in Sections 6 through 9.

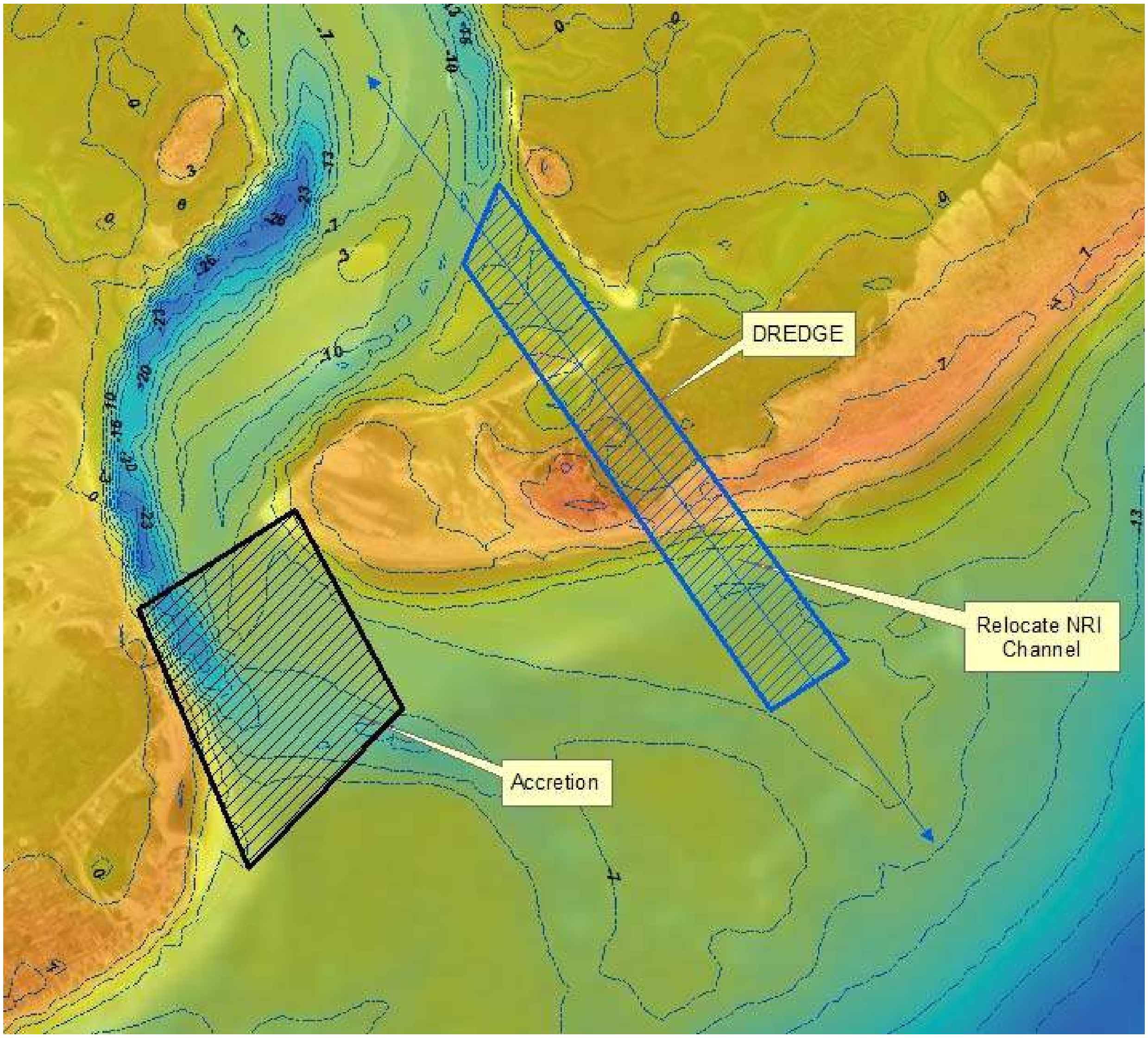


Figure 5-6. Channel Relocation Schematic at New River Inlet.

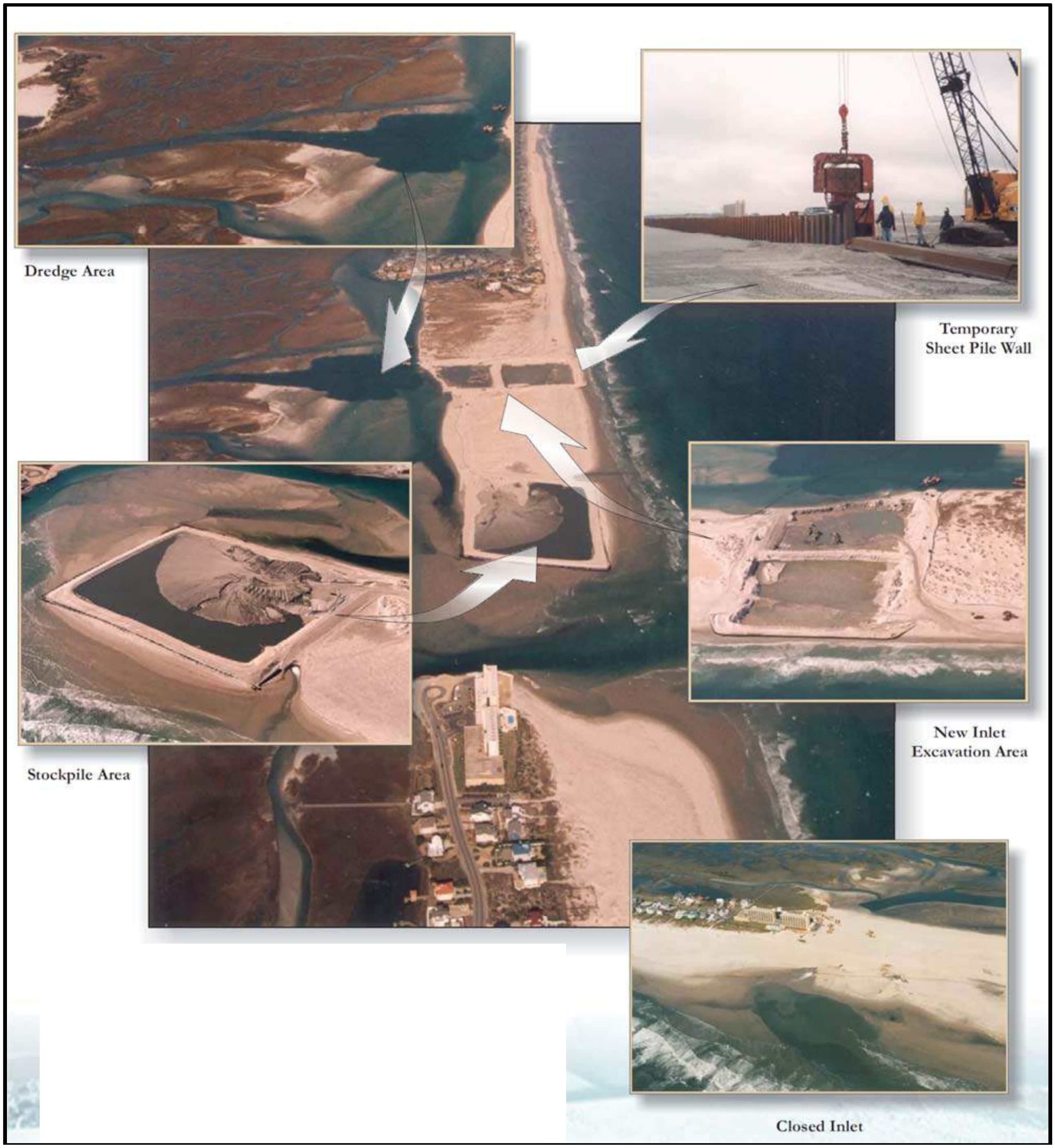


Figure 5-7. Mason Inlet Relocation Images (source: ATM, 2002).

6.0 BORROW SITE ANALYSIS/SELECTION

The Town of North Topsail Beach, as a part of its ongoing beach management program, has developed a comprehensive list of potential borrow areas over the last decade. Previously used borrow sources generally include:

- Upland
- Cedar Bush Cut/AIWW/New River
- AIWW dredge spoil islands (e.g., DA-143)
- Offshore
- New River Inlet.

General positive and negative aspects associated with each alternative are summarized as follows.

Upland Sources

- Suitable for small projects (less than 200,000 cy) and to supplement other larger fill projects
- Good for dune rebuilding and creation
- Sand color and grain size typically not as good as in-water sources
- Slow production rates and shorter lifecycles (every 1 to 3 years)
- Truck traffic and NC Department of Transportation (NCDOT)/road maintenance issues
- Unit prices ($/cy) can be relatively high, depending on trucking distance
- Can be good for FEMA mitigation projects (where unit volumes are about 10 cy/ft)

Cedar Bush Cut/AIWW/New River Maintenance Dredging

- Suitable for small projects (less than 200,000 cy) related to navigation maintenance authorized depths/widths
- Reusable resource
- Timing typically dependent on shoaling conditions
- USACE-related navigation dredging landward of the COLREGS line has historically placed less than 50,000 cy/yr.
- State dredging fund eligible

Dredge Spoil Islands along the AIWW (i.e., CDFs)

- Consist of layered material that would require separation of beach compatible and non-beach compatible material
- Reuse of this material increases CDF disposal capacity to allow continued disposal operations
- Islands have become valuable for natural resources, recreation, and in some cases, development

New River Inlet

- Currently not fully utilized/optimized because of side-casting operation and only following deep-water USACE permit criteria
- Critical to long-term beach and inlet management
- Sand color and grain size typically very compatible
- Channel alignment/orientation and shoaling patterns have been cited to exacerbate problems to adjacent shorelines
- State dredging fund eligible

Offshore Borrow Areas

- Suitable for large projects (greater than 500,000 cy)
- Sand color and grain size typically very compatible
- Rocks have been encountered in previous nourishment projects
- Fast production rates and longer lifecycles (every 4 to 5 years)
- Large ocean-certified hopper dredge mobilization/demobilization costs ($1 to $4 million)
- Can also be good for FEMA projects due cost-sharing of mobilization/demobilizations as well as beach fill placement

Additional discussion on borrow area sources is provided in the following sections.

6.1 AVOIDANCE AND MINIMIZATION

The Town has been actively assessing available borrow areas for decades, and one overarching goal during this process is avoidance and minimization of potential impacts. Reducing potential impacts includes, but is not limited to, the following:

- Borrow area location that is reasonably accessible to NTB and a sufficient distance from significant natural resources
- Documented strata of high-quality beach-compatible sediment suitable for meeting both State standards and post-placement performance criteria acceptable to the Engineer
- Lack of significant benthic or other resources to be temporarily impacted by borrow area excavation
- Exposed hardbottom resource avoidance (including 500-meter borrow area buffer)
- Cultural resource avoidance
- Critical habitat avoidance
- Essential Fish Habitat (EFH) Habitat Areas of Particular Concern (HAPCs) avoidance/minimization
- Proposed work to occur in established environmental winter window (to minimize natural resource impacts)
- Implementation of beach nourishment construction best management practices (BMPs) and following of all established protocols related to dredging

6.2 NATIVE BEACH CHARACTERIZATION

Native beach composite data are provided in Table 6-1 and were developed for the 2009 EIS.

Table 6-1. Composited Native Beach Sediment Characteristics from Finkel et al., 2009

	Mean Grain Size (mm)	Sorting (Phi)	Percent Fines (%)	Percent Carbonate (%)
Composited Average	0.23	1.03	1.7	28

6.3 UPLAND BORROW AREAS

The Town has recently selected an upland borrow area for the Town's Phase 5 FEMA mitigation due to Hurricane Matthew. Fill projects utilizing upland borrow areas can be extremely valuable for unplanned/emergency mitigation efforts. Additionally, truck haul projects do not involve the expensive mobilization/demobilization costs associated with offshore dredges and can occur much more quickly.

Potential negative aspects of upland borrow areas in the region include variations in sand color, practical volume limitations, and placement methods (i.e., trucking). Additionally, the NCDOT requires permitting and has the ability to shut down operations or require roadway mitigation.

Several sites were located as potential providers for the FEMA Phase 5 project and these include:

- Wellman Site
- Haulsville Site
- ST Wooten Site (active mine)

The ST Wooten site was recommended for the Phase 5 project and it is an active sand mine approximately a 50-mile 1-way haul distance to NTB (see Figure 6-1). The Wellman and Haulsville sites are not active mines and would require vegetation removal as well as about 1 foot of removal of the organic topsoil layer, known as overburden. Upland truck haul sites can be valuable and economical, especially for smaller projects and when the borrow site is relatively close to NTB (less than 10 to 15 miles is ideal).

Figure 6-1. ST Wooten Sand Mine (outlined in red) (image source: 2017 TI Coastal presentation).

6.4 AIWW BORROW AREAS

AIWW borrow areas include upland disposal areas (e.g., DA 143) as well as Cedar Bush Cut and reaches of New River and the AIWW that have beach-compatible material and require dredging. Figure 6-2 presents an overall image of historical AIWW disposal areas near the project area.

Figure 6-2: USACE Historical AIWW Borrow Areas near the Project Area (image source, USACE Wilmington District).

6.4.1 DA-143

DA-143 is a dredge spoil site located adjacent to the AIWW and New River. It is estimated that the disposal island contains approximately 1.9 million cubic yards of material (MCY) (APTIM/CPE, 2013 Supplemental EIS). Cores were collected at DA-143 in 2011, and geotechnical analysis showed a considerable amount of beach-compatible material available, although there are small lenses of silty material.

Figure 6-3. DA-143 Confined Disposal Facility (CDF)

6.4.2 CEDAR BUSH CUT

Cedar Bush Cut has acted as a beneficial use of dredged material (i.e., a borrow area for beach nourishment) for decades. The primary reason for the Cedar Bush Cut dredging projects is navigation; however, the dredged material is beach compatible and the beginning of the beach fill placement on NTB is less than 4,000 feet away.

Several reaches of Cedar Bush Cut are naturally deep and, therefore, do not require dredging for navigation. However, other reaches do shoal regularly and require dredging for navigation. Cedar Bush Cut is currently permitted to dimensions of 8 feet by 90 feet for the navigation channel inland of COLREGS line. However, enlarging the channel dimension to 16 feet by 300 feet has been evaluated, and this would provide up to 780,000 cy (APTIM/CPE, 2017).

Figure 6-7 presents typical Cedar Bush Cut borrow areas and north end placement footprint.

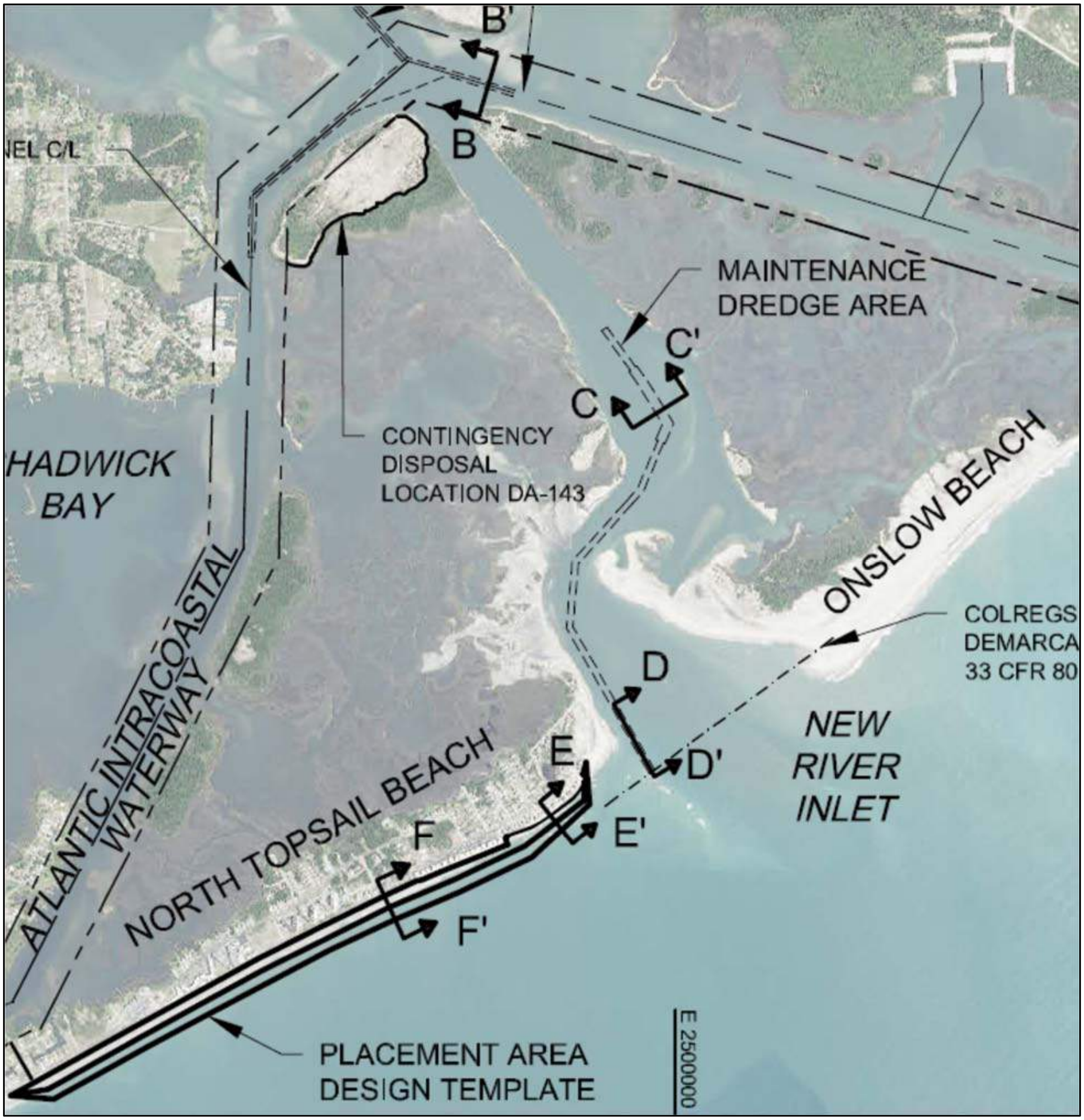


Figure 6-4. 2014 Cedar Bush Cut Dredging and Beach Placement Plan (Source: APTIM/CPE 2014 permitting document).

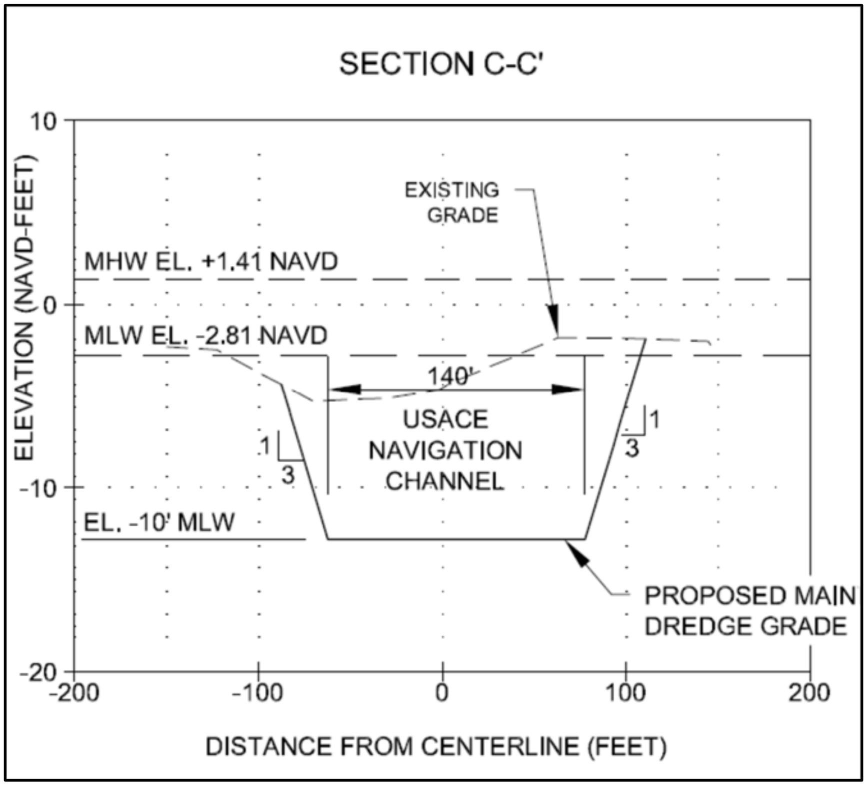


Figure 6-5. Cedar Bush Cut Permitted Width (140-feet) for 1,200 Feet of Channel Inland of COLREGs Line (Source: APTIM/CPE 2014 permitting document).

Average sedimentation rates for the NRI are estimated at approximately 100,000 to 200,000 cy/yr (refer to Section 4), however, this material is not focused in one area and can mix with incompatible material. Nonetheless, Cedar Bush Cut can serve as a valuable source of renewable beach compatible material (just not as regularly as desired).

6.4.3 AIWW AND NEW RIVER DREDGING

Reaches of the AIWW and New River that are proximal to the project area shoreline and have beach-compatible material have been used for nourishment projects in the past and will likely continue to be. As with Cedar Bush Cut, there are several reaches that are naturally deep and do not require dredging. Figure 6-6 presents a January 2018 USACE survey identifying shoaling and deep areas.

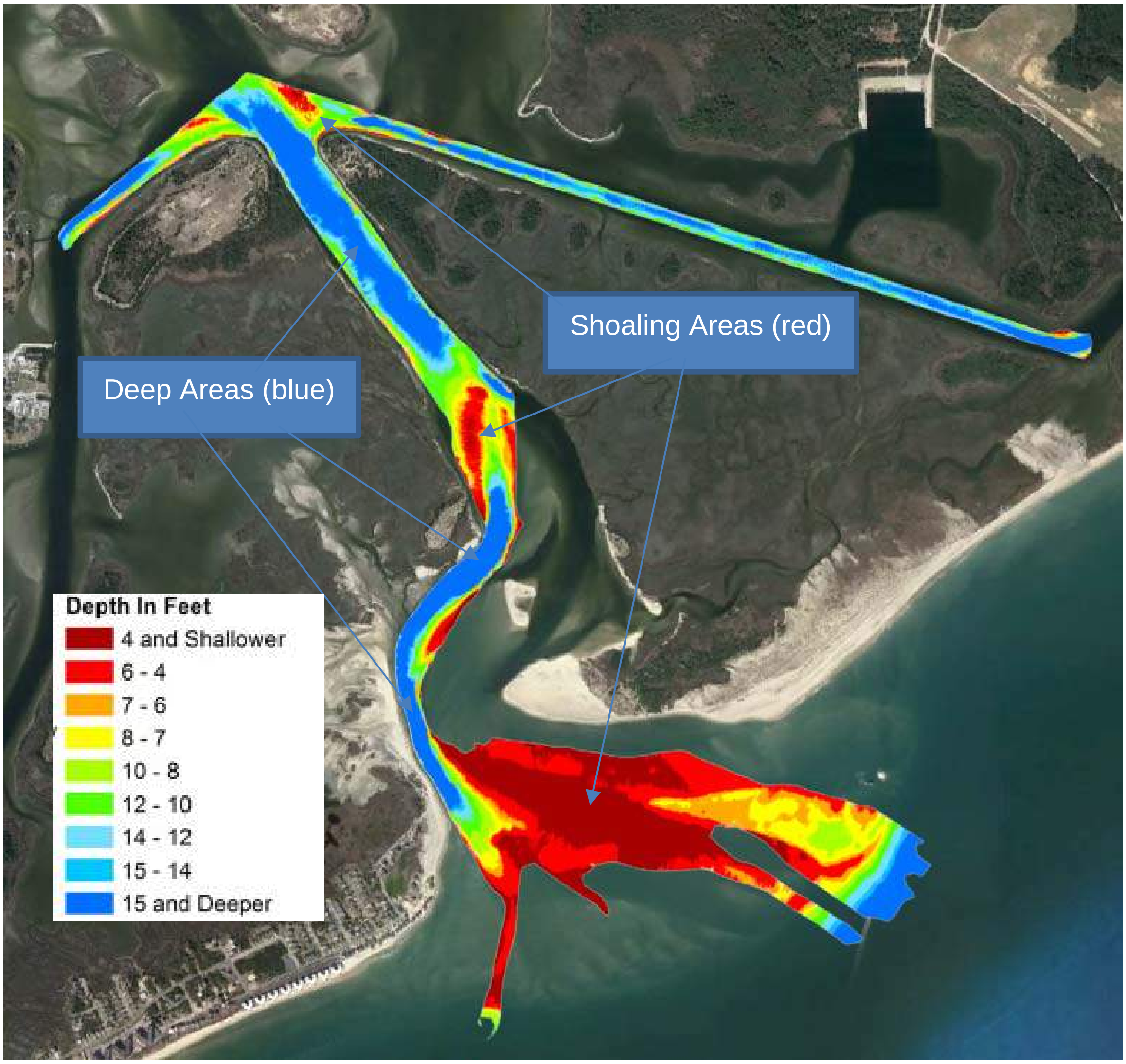


Figure 6-6. January 2018 Survey of New River Inlet, Cedar Bush Cut, and AIWW. Note several areas are naturally deep and do not require dredging.

6.5 OUTER EBB CHANNEL DREDGING

As previously noted, side-caster dredges are primarily used by the USACE to maintain the outer navigation channel at NRI. However, the new USACE shallow draft split-hull hopper dredges (the *Murden* and the *Currituck*) have been more active recently due to the extensive side-caster dry-dock work (fall 2017 through mid-2018). The shallow draft hoppers can place material closer to the nearshore when comparted to the sidecasting method. However, USACE Navigation staff have noted that the sidecaster has a higher production rate than the shallow draft hoppers and is preferred when available. Nearshore disposal by the split-hull hoppers is preferred over side-caster operations, however, placement of fill on the beach is required for the proposed project.

6.5.1 CHANNEL REORIENTATION

While the USACE has taken a minimal approach to outer ebb channel dredging and navigation, the Town has recently used the NRI outer ebb channel as a sand source and has plans to continue the use of this dredge footprint in the future. The 2013 borrow area yielded approximately 600,000 cy of material, however, it shoaled in within about 2 years, when 4 years was anticipated. Additionally, the NTB shoreline between Topsail Reef condominiums (about 1148+00) to 1163+00 eroded very quickly to where the sandbag revetment was installed.

It has been posited that the shoreline erosion was not caused by the dredged channel; however, it was decided to move the dredge footprint farther from the shoreline as a cautionary measure. The pivot channel will yield a similar amount of material (about 600,000 cy), while moving farther east, away from the beach and nourished shoreline. This borrow area can be used every 4 years, according to the inlet management plan, and is a valuable renewable resource for sand. Channel dredging is also very beneficial to navigation. Figure 6-7 presents an image of the 2013 channel realignment compared to the proposed pivot channel alignment.

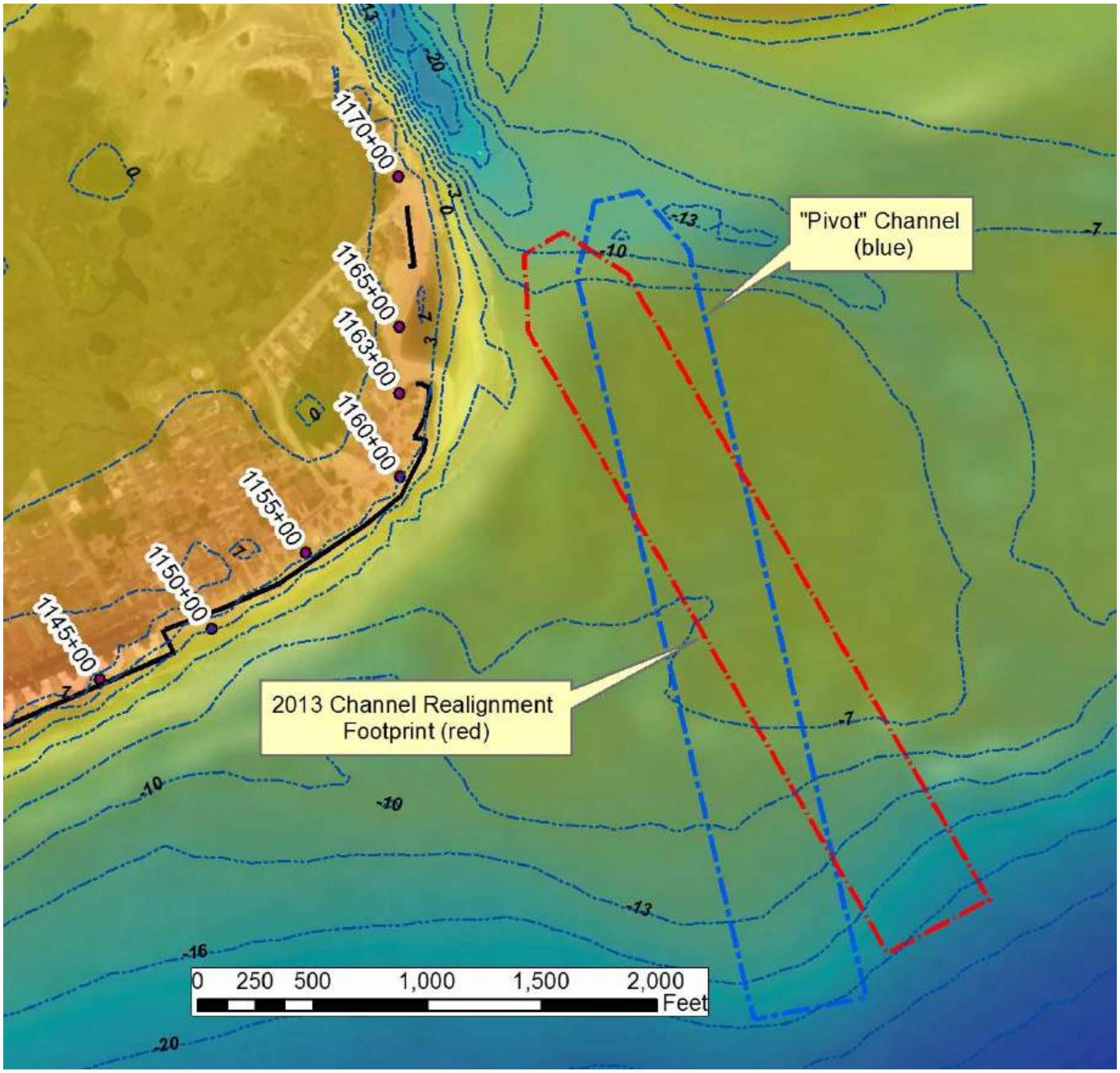


Figure 6-7. 2013 and Future Pivot Channel Alignments (depth contours in feet-NAVD88).

6.6 OFFSHORE BORROW AREAS

The Town of North Topsail Beach as well as the towns of Surf City and Topsail Beach have all participated in offshore borrow area investigations that are relevant to the project area. Figure 6-8 presents an overview of offshore borrow areas for the Surf City CSDR project. Sixteen borrow areas are shown in Figure 6-8.

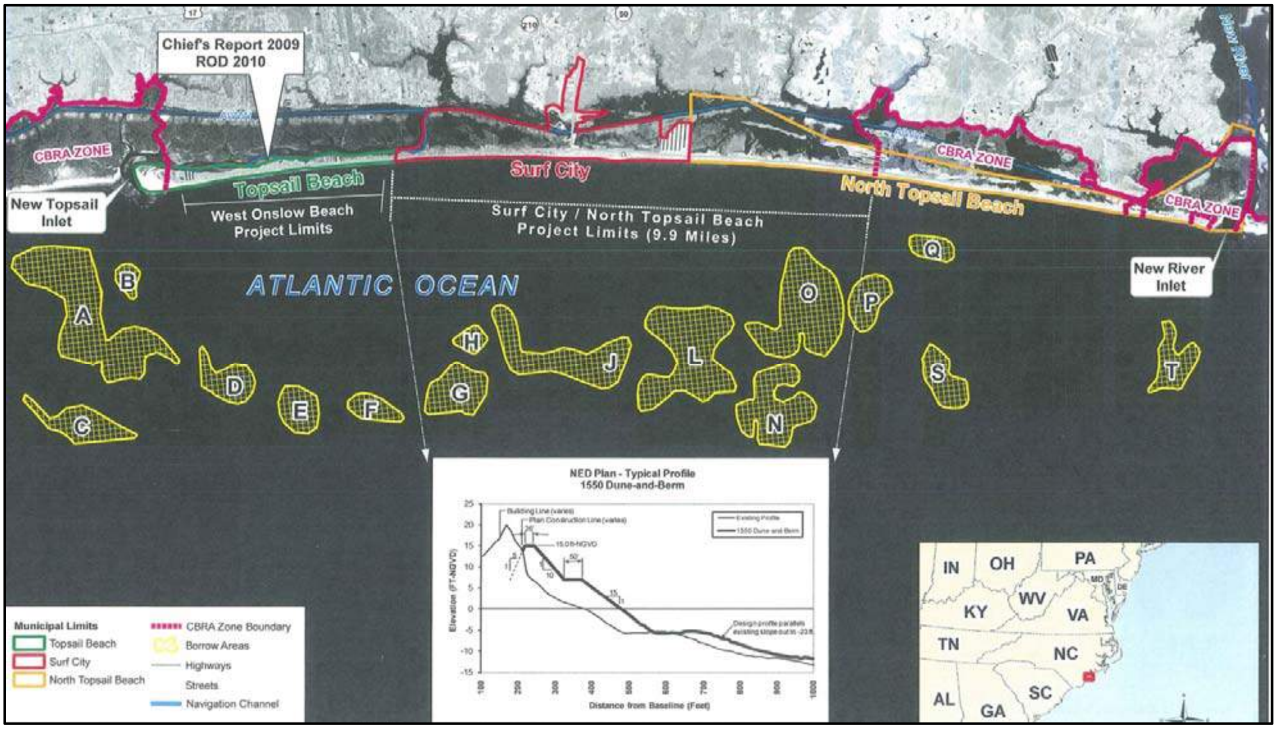


Figure 6-8. *Offshore borrow areas identified for the Surf City/NTB USACE project (source: USACE Surf City CSDR).*

For the 2009 NTB EIS, the identified offshore borrow area was located between USACE baseline stations 780+00 and 870+00 and approximately 0.4 and 1.6 mi offshore (see Figure 6-9).

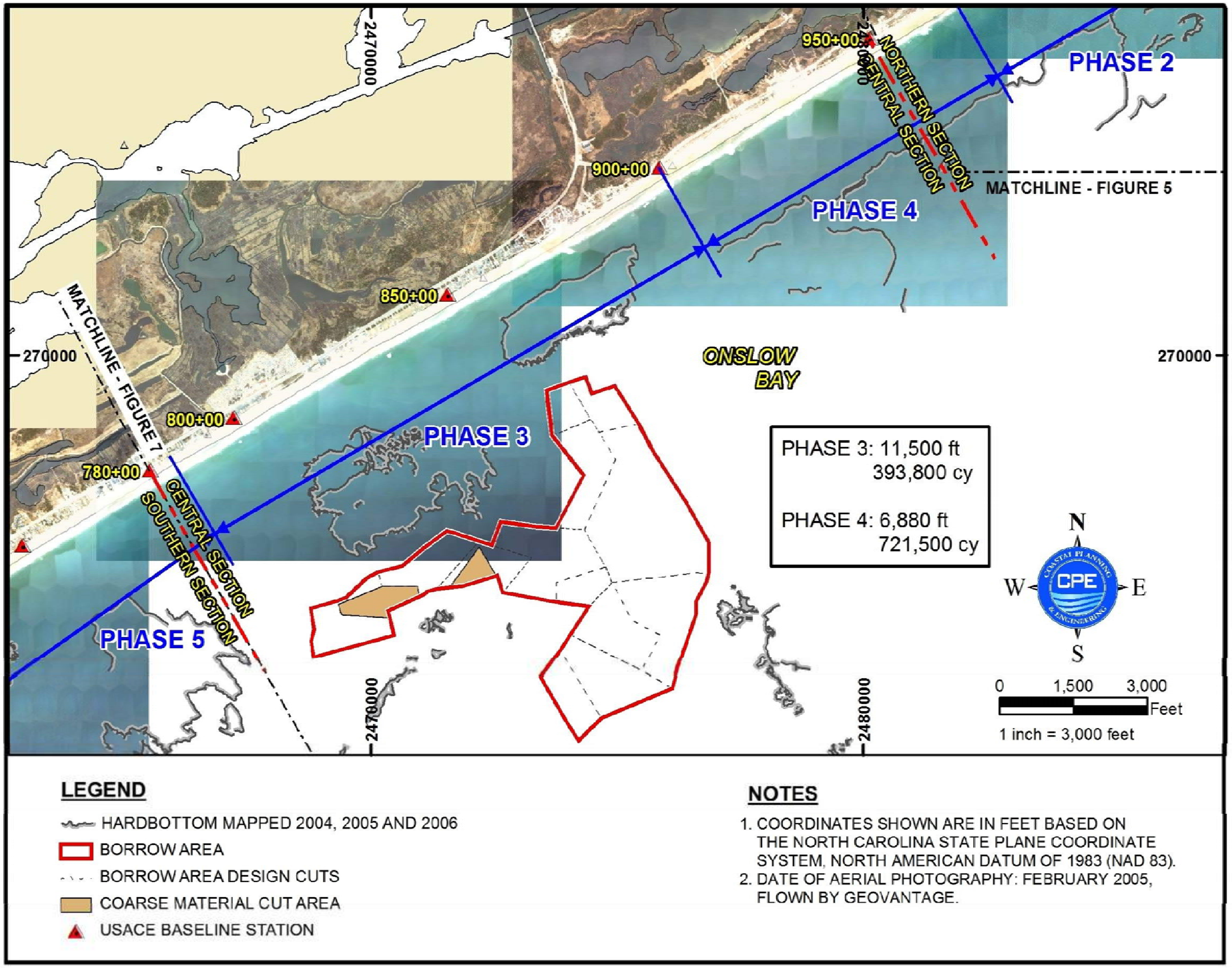


Figure 6-9. Offshore Borrow Area Identified in the 2009 EIS.

Geotechnical and biological research investigations confirmed the location of exposed hardbottoms in the nearshore and offshore of NTB to avoid these areas. A detailed evaluation of the offshore borrow area is available in Appendix C (Final Geotech Report) of the 2009 EIS.

The offshore borrow area is divided into 16 different cuts (areas with different cut- depths), shown in Figure 6-10.

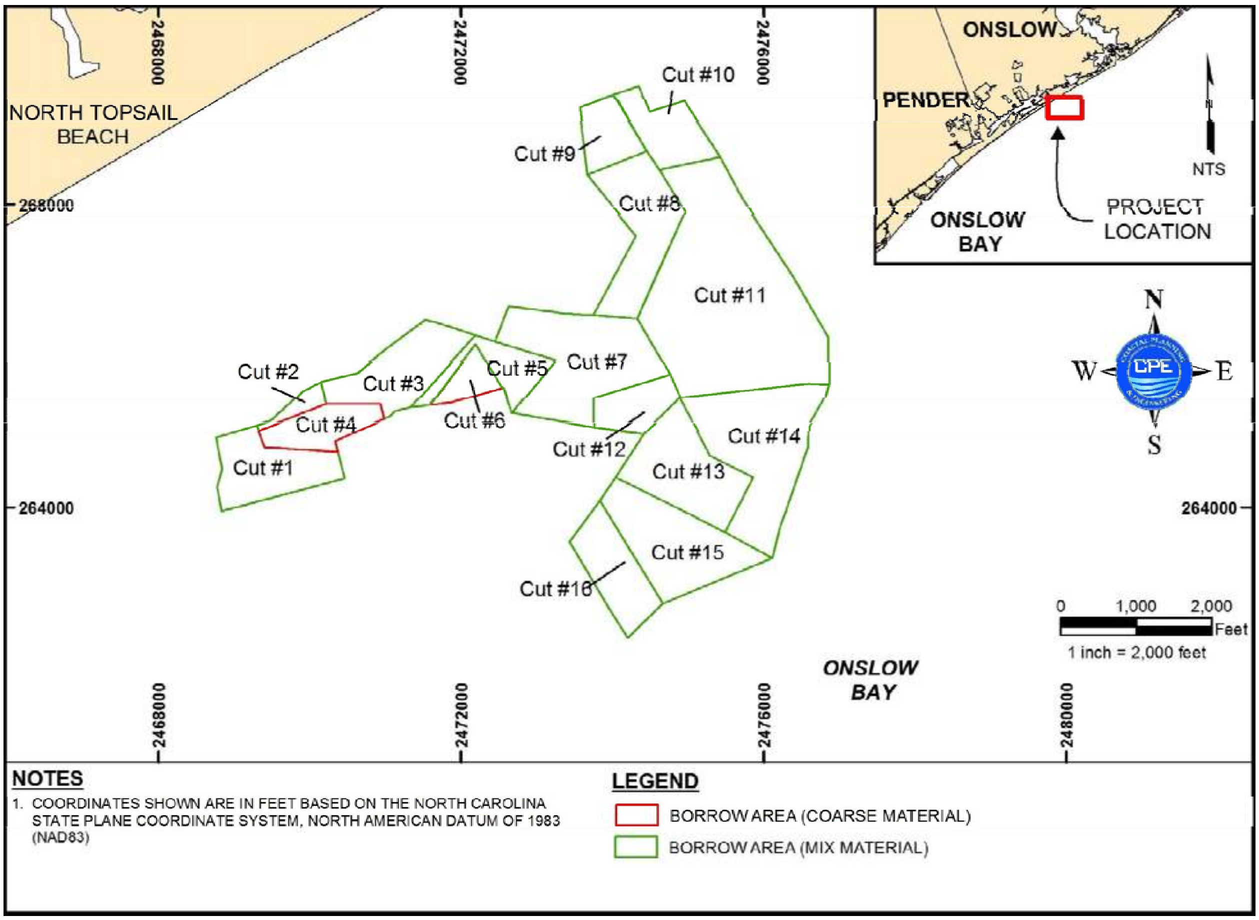


Figure 6-10. Map Depicting the 16 Different Cuts that Comprise the Offshore Borrow Area (source: 2009 DCM application).

The 2009 EIS calculated the total volume of material available in the offshore borrow area as 6,551,000 cy, including both fine and coarse fill. This borrow area was used for the Phase 5 nourishment (south of the project area) in 2016 and, unfortunately, unacceptable levels of larger rock were encountered, necessitating mitigation and removal of rock. Regulatory and natural resource agencies recently requested additional rock removal in spring 2018 (prior to turtle nesting season).

6.7 BORROW AREA SEDIMENT COMPATIBILITY

All borrow areas cited in this report have been compared with State compatibility criteria in other reports dating from the 2009 EIS and in subsequent projects, including the following:

- 2013 NRI channel realignment (borrow area: NRI ebb shoal)
- 2015 Phase 5 (borrow area: offshore)
- 2016 Cedar Bush Cut (borrow area: Cedar Bush Cut)
- 2018 planned channel realignment (borrow area: NRI ebb shoal)
- 2018 planned FEMA Phase 5 project (borrow area: upland)

While there is some variability in the sediment, the material is acceptable for use as beach fill in the project area. Table 6-2 presents summary sediment compatibility from the 2009 EIS.

Table 6-2. 2009 EIS Summary Sediment Characteristics of Native Beach, Offshore Borrow Area and Channel Borrow Area.

Source	% Silt	% Carbonate	% Granular	% Gravel	Volume (cy)	Acreage
Native Beach	1.5	26	1.07	0.43	NA	NA
State Standard Allowance	5	15	5	5	NA	NA
State Standard Cutoff	6.5	41	6.07	5.43	NA	NA
Offshore BA (Fine)	6.4	16	1.13	1.43	6,194,454	459
Offshore BA (Coarse)	1.75	20	0.63	0	356,839	23
Inlet Channel (with side slopes)	1.53	22	5.38	3.64	544,400	45
Total Volume of Borrow Areas					**7,187,093**	**527**

Table 6-3 presents summary sediment criteria from the 2013 supplemental EIS.

Table 6-3: Summary Ebb Channel Borrow Area Sediments from the 2013 Supplemental EIS.

	MEAN (mm)	PHI MEAN	PHI SORTING	SILT (%)	CARBONATE (%)
Pre-Dredge Vibracore Data*	0.39	1.35	1.49	1.53	27
Post-Dredge Vibracore Data**	0.51	0.96	1.77	1.02	22
North Topsail Existing Beach***	0.23	2.15	1.03	1.72	28

*Pre-Dredge composite data can be found in Finkl et al., 2009
**Post-Dredge composite data can be found in Appendix 9
***Existing beach data were generated using sampling profiles TB20, TB21, TB22, TB23, TB24 (Finkl *et al*., 2009)

6.8 BORROW AREA VOLUMES

Conceptual volumes available for each borrow area were estimated based on available geotechnical data and corresponding cut depths. Table 6-4 presents these conceptual available volumes.

Table 6-4. Volumes Available for Borrow Area Alternatives

Location	Volume Available	Reusable
Offshore Borrow Area	6,000,000	Possible
DA-143	1,900,000	No
Cedar Bush Cut	100,000 to 700,000	Yes
AIWW/New River Dredging	50,000	Yes
Outer Ebb Channel	600,000	Yes
Upland Borrow Areas	>200,000	Yes

The NRI borrow area is the preferred borrow area due to its reusable qualities, the consistent volume yield, and the navigation benefits. Cedar Bush Cut and the New River/AIWW borrow areas are also valuable and useful sources of material that provide navigation benefits. With an expanded channel through Cedar Bush Cut, up to 700,000 cy of material may be available, however, in the long-term shoaling in this area will likely result in smaller volumes, more like 100,000 to 200,000 cy.

DA-143 is also a good source of material at the moment. However, it is anticipated that only non-compatible material will be placed in DA-143 from now on, therefore, its future use for beach-compatible material is doubtful. The upland borrow areas are not reusable in the sense that the borrow areas will not naturally refill with beach-compatible sediment. However, future upland borrow areas may also become available.

The offshore borrow area has a significant amount of sediment; however, the costs of mobilizing an ocean-certified dredge can range from $2 million to $4 million. Therefore, only very large beach nourishment projects (greater than 500,000 cy) would justify its use. Additional offshore borrow area research is also likely required to minimize encounters with rock-substrate layers.

6.9 FUNDING SOURCES

While many borrow areas are available to the Town, it is recommended that borrow sources with a navigation component be considered highest priority since the State has a funding mechanism dedicated to these projects.

Dredging funds can be obtained directly from the State via the Water Resources Development Grant process. The Town has used this mechanism for the 2016 Cedar Bush Cut project. The State cost-sharing is currently 66.7 percent, and the dredging fund has expanded in scope since its inception in 2013 and funding has also increased. More than 12 federally authorized inlets and associated channels are included, and some non-federal channels are also included (mostly related to state ferry routes). There is also a lake/freshwater component of the fund that can compete for funding. The fund has shown robust growth and availability since its inception.

A State beach nourishment fund was established in 2017, however, no funding mechanism is currently in place. The beach nourishment fund is currently designed to finance 50 percent of the non-federal share of beach nourishment projects that do not have a dredging component.

7.0 MODELING STUDIES

A rigorous modeling analysis of the proposed project alternatives has been conducted since the 2009 EIS project. The analysis includes the Delft3D modeling suite and was performed by APTIM/CPE. Modeling reports developed related to NRI primarily include the following:

- Channel Realignment Analysis - 2016
- Terminal Groin Analysis - 2016
- Updated Channel Realignment - 2017

These three reports are included as attachments to this report. The 2016 Channel Realignment study includes Delft3D model setup and calibration. This report also models channel realignment alternatives related to the NRI ebb shoal channel, focusing on the 2013 dredging project. The 2016 Terminal Groin Analysis modeled six different terminal groin alternatives using the model application as described in the channel realignment study. The 2017 analysis features an updated pivot channel modeling, as well as modeling different beach nourishment volumes/sizes.

7.1 DELFT3D MODEL APPLICATION

Delft3D is a three-dimensional modeling suite that investigates hydrodynamics, sediment transport, and morphology for estuarine and coastal environments. The Delft3D morphological model was used to simulate several alternatives to provide insight into the relative and absolute effects each option would have within the immediate project vicinity, as well as adjacent areas. Delft3D has been used worldwide for hundreds of coastal applications, including the Bald Head Island and Ocean Isle terminal groin modeling projects.

7.1.1 MODEL CONFIGURATION

The 2016 APTIM channel modeling describes in detail the Delft3D model application setup and calibration. Bathymetric and topographic data used in the model study were compiled from the following sources:

- NTB annual beach and bathymetric surveys
- USACE AIWW, Cedar Bush Cut, New River and NRI bathymetric surveys
- Light detection and ranging (LiDAR) topographic data (NOAA Digital Coast, 2004 North Carolina Flood Mapping)
- NOAA Charts (inshore and offshore)

All elevation data was either downloaded in NAVD88 or converted to NAVD88. This is the most recent and standard vertical datum for the United States and it is similar to mean sea level. Table 7-1 presents tidal datums for the project site based on a nearby USACE station in NRI. Note that previous studies have also used Wrightsville or Wilmington tidal datum offsets that can differ slightly.

Table 7-1. Project Site Tidal Datums (New River Inlet, North Cal Dolphin)

Tidal Datum	Feet	Meters
MHHW	1.4	0.42
MHW	1.1	0.35
NAVD88	0.0	0.00
MLW	-1.7	-0.51
MLLW	-1.8	-0.55
Range (MHHW-MLLW)	3.2	0.97

Comprehensive bathymetry data sets include the entire model domain and were developed for 2013 and 2015. The modeling consists of two basic model grids:

- Regional
- Local

The regional and local flow grids are provided in Figure 7-1. The regional flow grid includes all the New River Estuary and extends to offshore. The regional grid is coarser than the local grid and feeds the local grid boundary information.

The local grid has both a flow component and a wave component. Flow includes the river and ocean hydrodynamics, as well as tidal effects. The wave model grid feeds information to the flow grid where sediment transport processes are modeled.

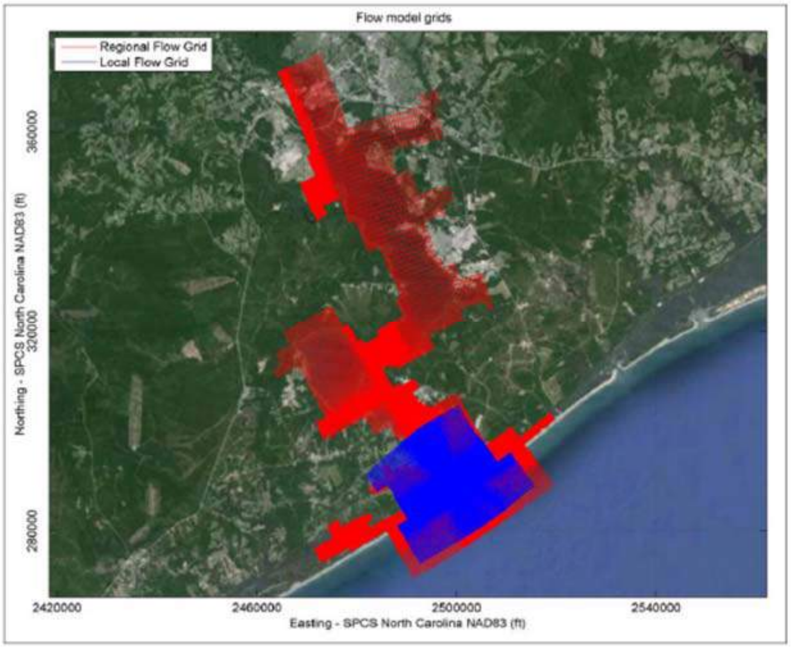


Figure 7-1. Regional and Local Flow Model Grids (image source: APTIM 2016 inlet realignment study)

Figure 7-2. Wave and Flow Local Model Grids (image source: APTIM 2016 inlet realignment study)

7.1.2 MODEL CALIBRATION

The model was calibrated for both the flow and wave model applications. As summarized in the 2016 APTIM Realignment study wave calibration was performed using three different sources:

The wave model was calibrated by comparing model results with measured wave data from USACE 190, located approximately 3.8 miles offshore in approximately 40 feet of water and WHOI 09, located approximately 1.0 mile offshore in approximately 30 ft of water. [see Figure 7-3]

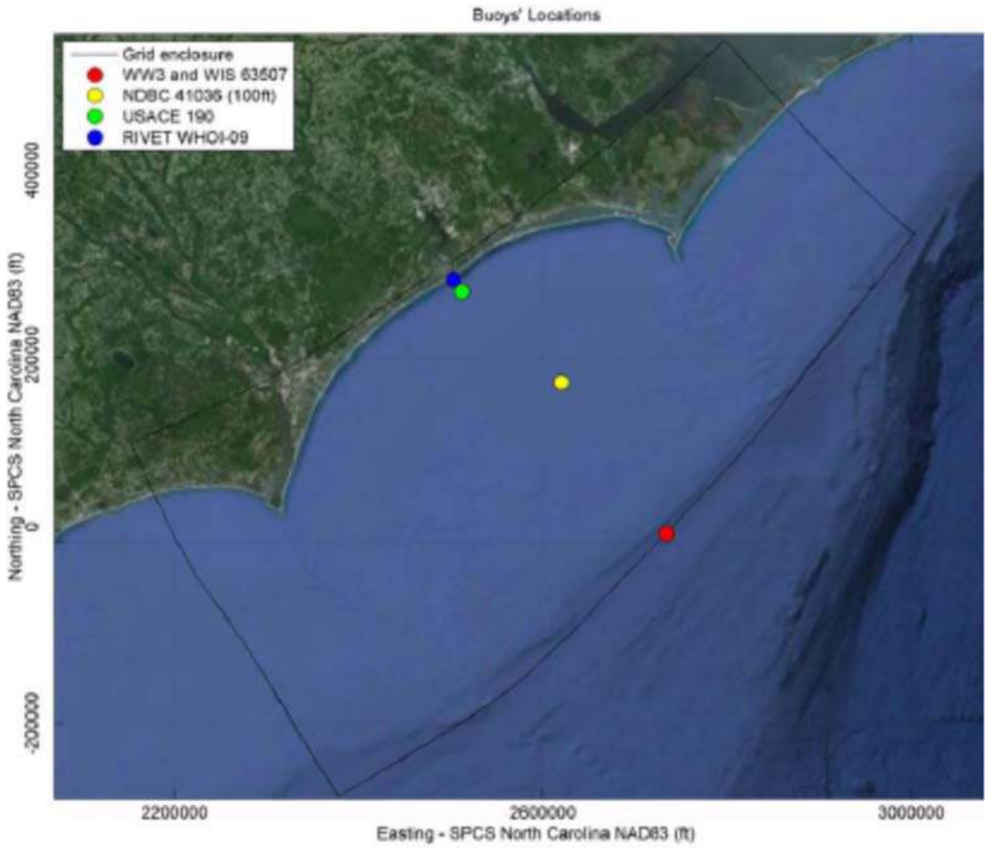


Figure 7-3. Regional Wave Grid Boundary and Wave Data Locations (source: APTIM 2016 inlet realignment study)

Good agreement between the modeled and measure wave data is observed. In addition to comparisons with measured data, Delft3D modeled output was also compared with NOAA and USACE wave modeling products (WW3 and WIS, respectively).

7.1.3 DELFT3D ALTERNATIVES MODELING

Delft3D alternatives modeling generally includes:

- Beach nourishments of varying size/length
- Terminal groins of varying size and location
- Ebb channel dredging/realignment alternatives

The modeling necessarily included existing condition modeling, which was used for calibration purposes as well as to make relative comparisons between modeled alternatives.

Important Note: For the modeling, no-action refers to simulations that include no beach management activity (nourishment, groins, dredging, etc.) and essentially represents background erosion. For the other sections of this report (alternatives, costs, etc.), no-action refers to currently occurring activities, which include some nourishment activity, depending on funding and other factors.

7.1.3.1 Groin Alternatives

The six terminal groin alternative layouts are shown in Figure 7-4. The 2013 and pivot channel dredge footprints are provided for reference. All groin alternatives were run with a 46 cy/ft nourishment along the project shoreline (about 300,000 cy for the Phase 1 shoreline).

Figure 7-4. Terminal Groin Alternatives (six shown).

The groin alternatives include a north option ("NGroin") and a south option ("SGroin"). These north and south orientations were increased in length by 250 feet and 500 feet (e.g., NGroin250, NGroin500). Table 7-2 presents these alternative lengths.

Table 7-2: Terminal Groin Lengths of the Modeled Alternatives.

Alternative	Anchor Section (ft)	Upland Section (ft)	Water Section (ft)	Total Length (ft)
NGROIN	345	894	782	2,021
NGROIN250	345	894	1,032	2,271
NGROIN500	345	894	1,282	2,521
SGROIN	345	1,210	490	2,045
SGROIN250	345	1,211	739	2,295
SGROIN500	345	1,212	988	2,545

The groins generally have three sections: anchor, upland, and in-water. The different sections are exposed to differing levels of wave and hydrodynamic forces, and construction/design can vary significantly based on each section. The anchor section is the most protected, whereas the in-water section is the most exposed. More discussion on groin design is provided in Section 8.

The general idea behind the need for a terminal groin at the project site is related to the 2013 channel realignment project (Figure 7-5), which placed nourishment material along the oceanfront. This material eroded very quickly (less than 2 years in the project area) and necessitated the expansion of the sandbag revetment. A significant amount of beach fill material ended up in the inlet and a spit feature also formed along the inlet (see Figure 7-5).

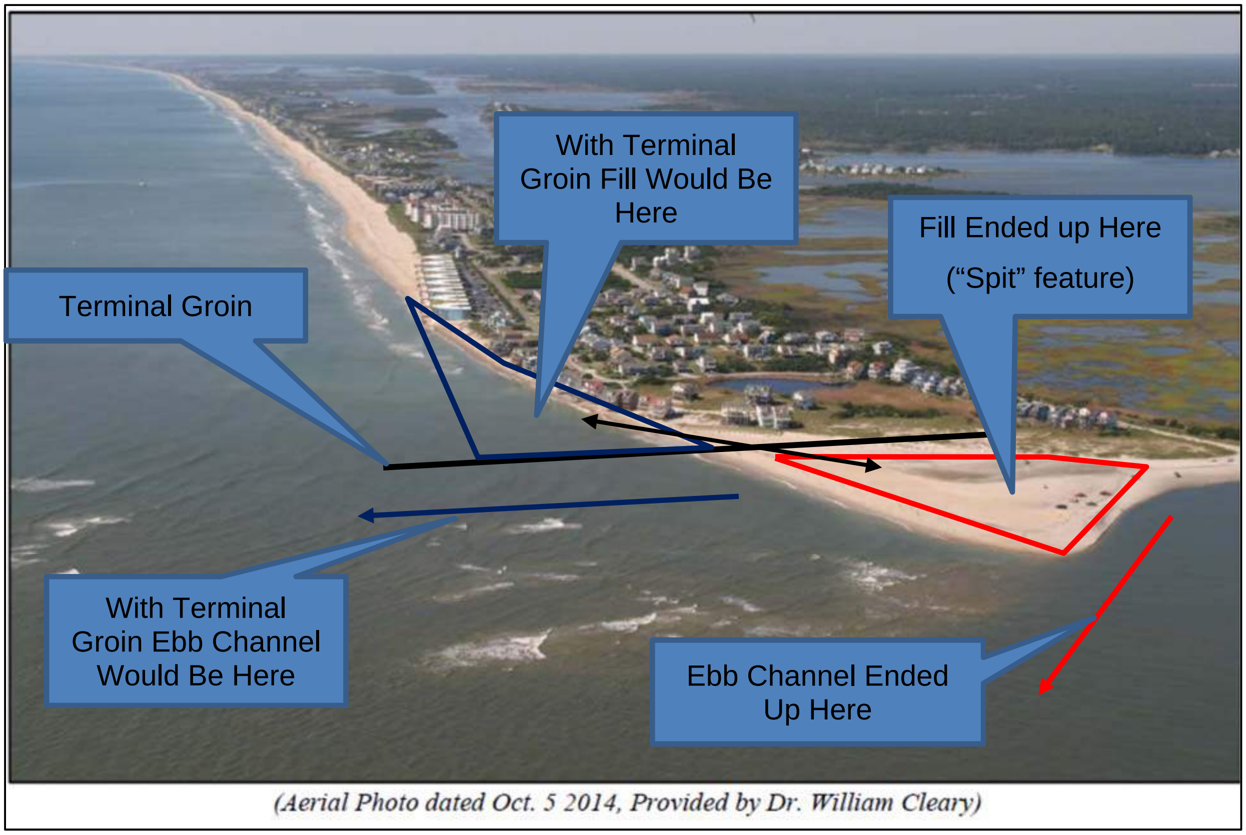


Figure 7-5: General terminal groin concept in relation to the 2013 nourishment project on a 2015 oblique aerial (image from Cleary)

The spit feature also helped to realign the ebb channel to the north, away from the desired southerly orientation. The 2016 APTIM study compared all six groin alternatives and, while they behaved similarly, the SGroin250 alternative was chosen as the most favorable. More recently, Onslow County also requested that navigation should be considered a higher priority in groin design. In reviewing both the response of the shoreline and the potential navigation benefits of each alternative structure, the north groin (“NGroin”) was determined as the preferred alternative.

Figure 7-6 presents final bathymetry for the NGroin alternative and does not include the borrow area dredging. To isolate project component effects, some model runs were conducted running nourishment/groin components separate from inlet borrow area components. Initial bathymetry conditions were the 2015 digital elevation model (DEM).

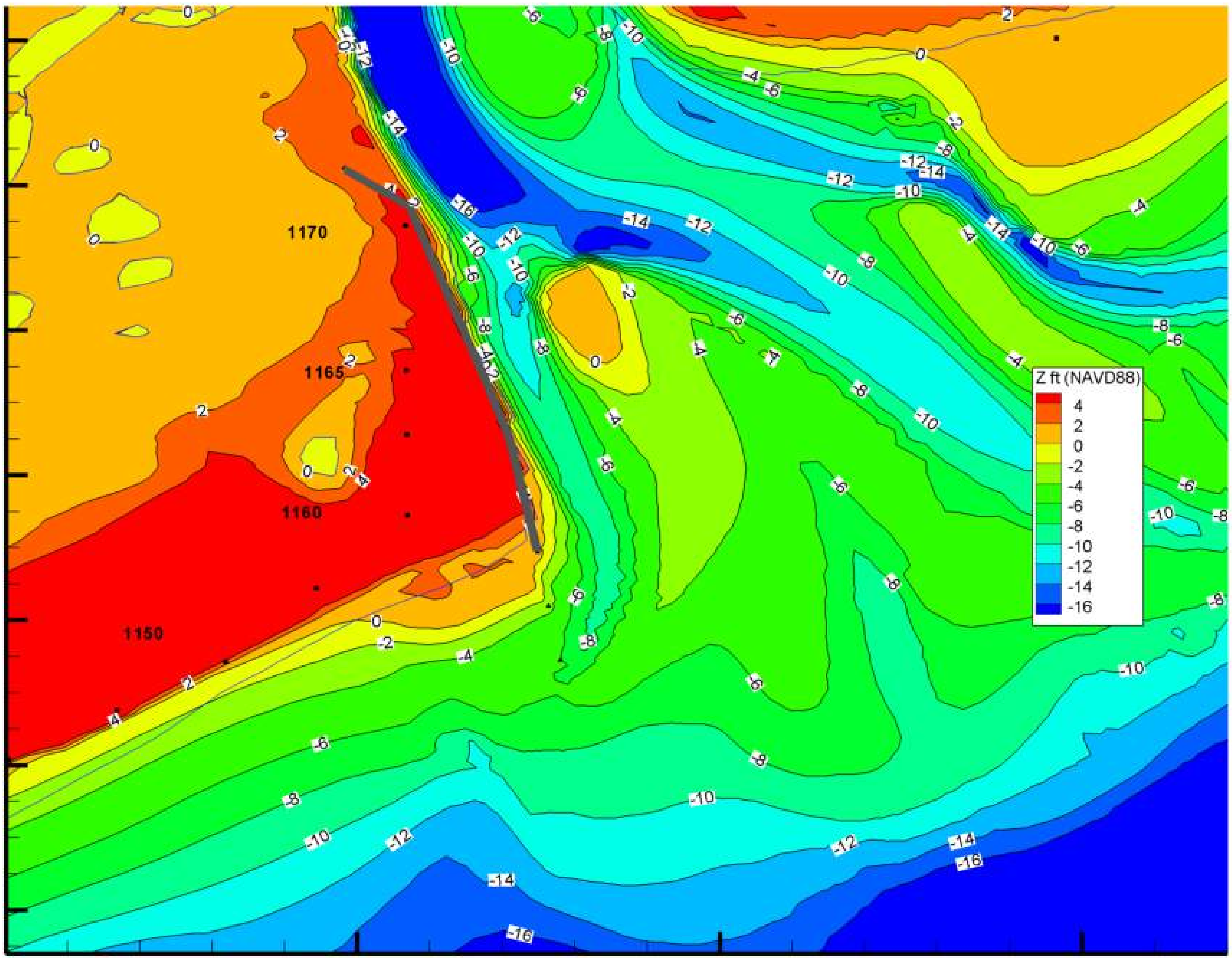


Figure 7-6. NGroin Final Bathymetry (post 2-yr run).

Figure 7-7 presents the relative change between the no-action (baseline) and NGroin alternatives 2-year final bathymetries. Yellow contours indicate accretion (i.e., sand), while blue contours indicate erosion (deeper areas) relative to no-action. In this instance, no action refers to existing condition 2-year runs with no dredging or beach fill placement in the model domain. Accretion to the southwest of the groin is clear, where over 6 feet of sand is shown relative to baseline conditions after 2 years.

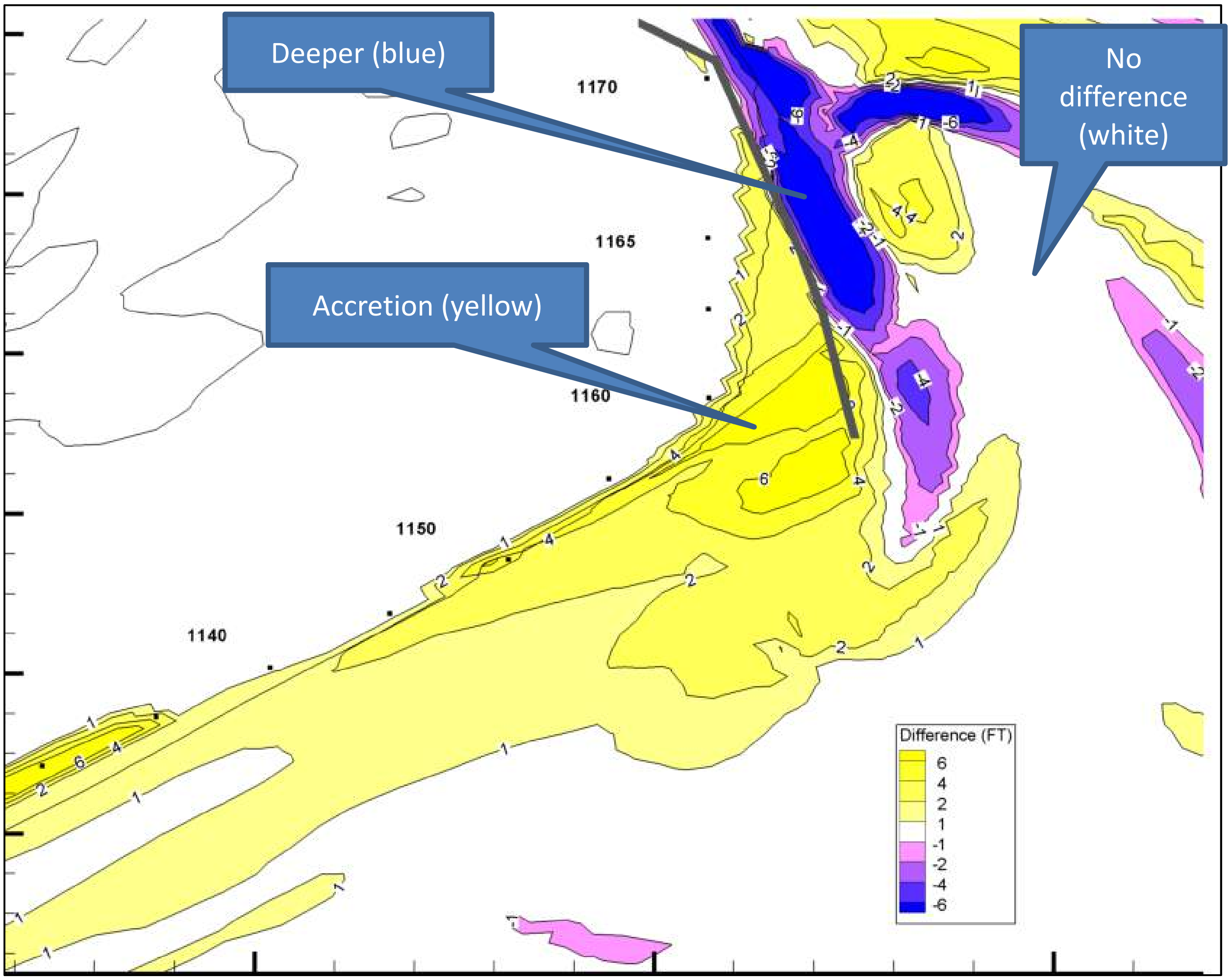


Figure 7-7. Relative Erosion/Accretion Comparison (NGroin versus No-Action) at the End of the 2-Year Run

Figure 7-8a presents volume comparisons along the project shoreline of

- NGroin with 46 cy/ft of fill
- Nourishment-Only (46 cy/ft)
- No-Action (no nourishment)

Volumes were measured out to -12 feet but were limited to 1,200 feet from the station origin at Stations 1155+00 and 1160+00 where the ebb shoal occurs.

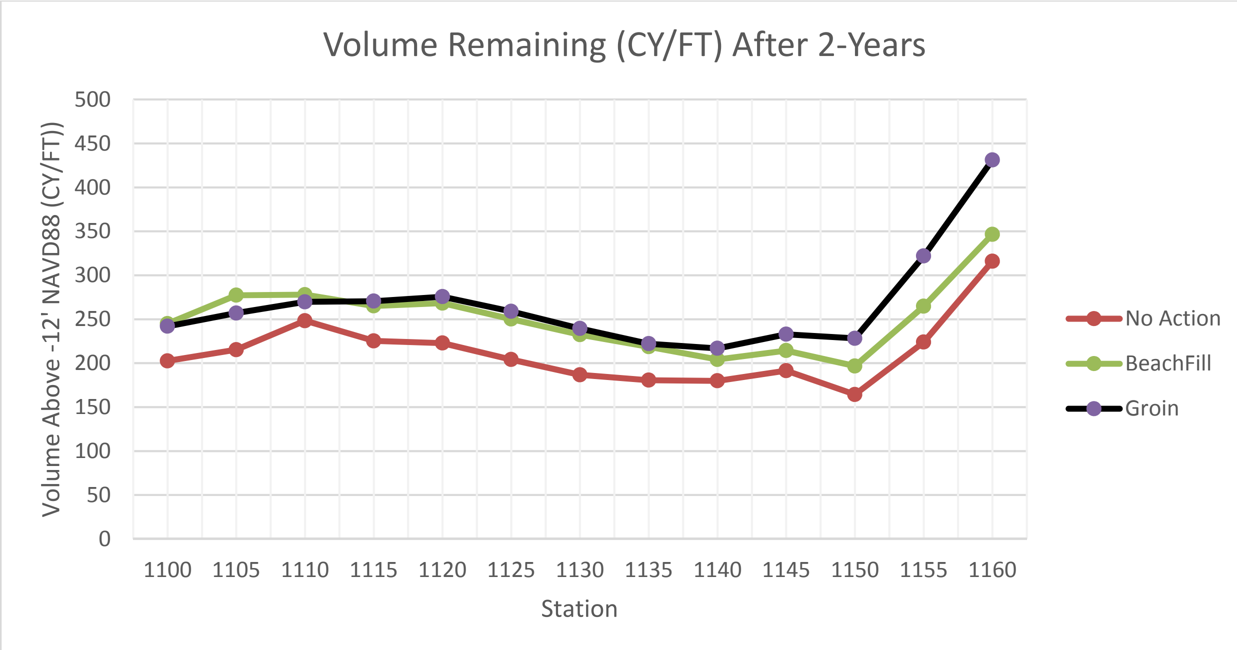


Figure 7-8a. Project Volumes after 2-Year Modeling Run for NGroin, Nourishment-Only, and No-Action Alternatives

Only shoreline perpendicular stations are presented in Figure 7-8a, even though the groin alternative has significant benefits along the inlet shoreline to Station 1170+00. As seen in the figure, the NGroin alternative has significant volumetric benefits for Stations 1140+00 to 1160+00 (about 50 to 100 cy/ft).

Figure 7-8b presents MLW change at the end of the 2-year modeling run for NGroin, nourishment-only, and no-action alternatives. MLW was chosen to reflect lower tide conditions along the beach and to minimize any effects of the sandbag revetment, which was included in the modeling.

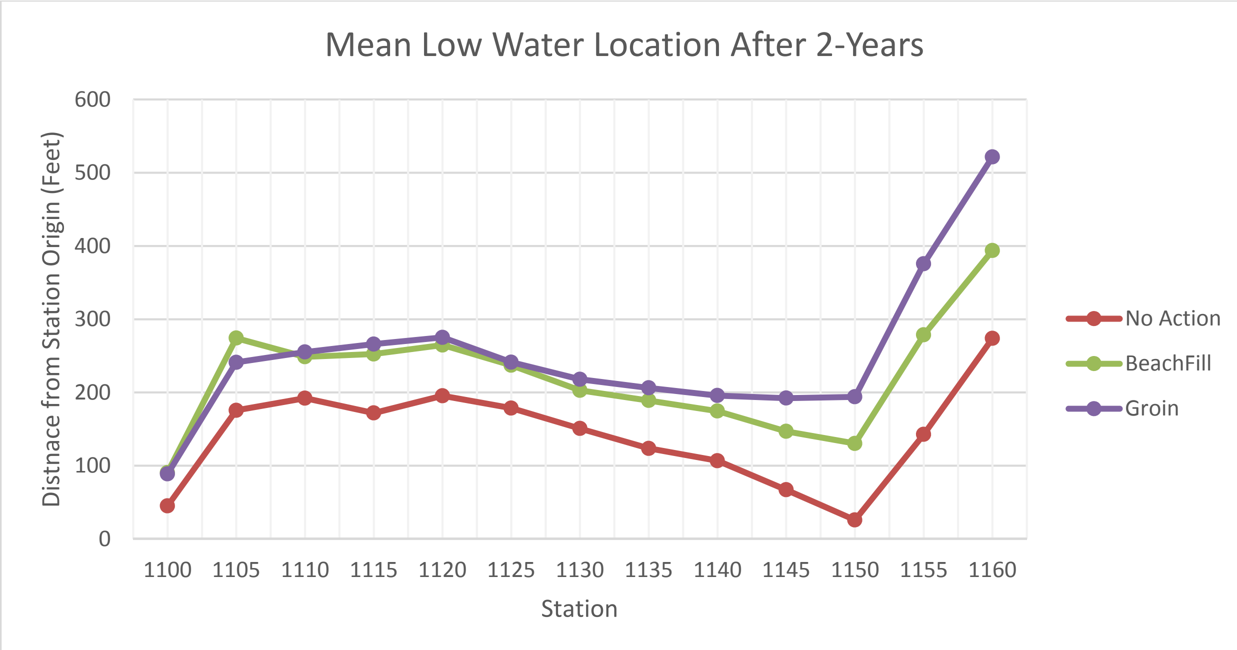


Figure 7-8b. Mean Low Water Shoreline Position after 2-Year Modeling Run for NGroin, Nourishment-Only, and No-Action Alternatives.

7.1.3.2 Channel Realignment Alternatives

Channel realignment modeling was performed separately from the groin modeling, and the 2016 APTIM report modeled several different alternatives including:

- No-Action (baseline)
- 2013 Realignment Orientation
- Parallel Channel
- Curved Channel
- Pivot Channel
- Pivot Channel shifted 250 feet

For these modeling alternatives, no beach fill (or groin) was included. All simulations were run using the April 2015 beach and inlet bathymetry/topography as the initial condition (see Figure 7-9).

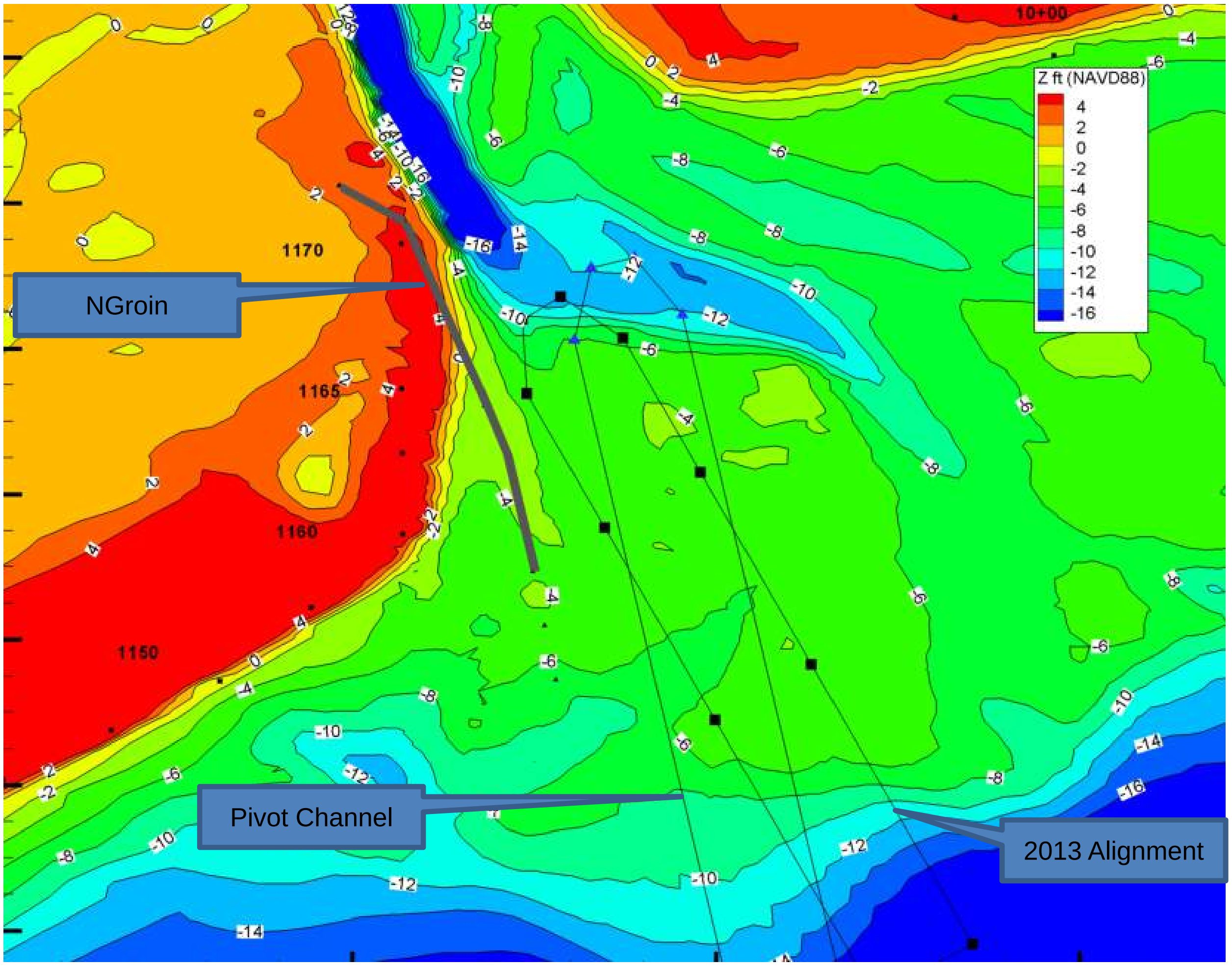


Figure 7-9. *No-Action Initial Bathymetry. NGroin and Channel Alternatives (2013 alignment and pivot channel) are only shown for reference. This digital elevation model (DEM) is representative of April 2015.*

The 2013 realignment alternative modeling was conducted to essentially confirm that the model reasonably simulated the 2013 channel realignment, where fill material north of Topsail Reef condominiums eroded extremely quickly. As stated by APTIM (2016):

> *The erosion of the fill material placed seaward of the homes north of Topsail Reef during the Phase 1 project left these structures in imminent danger comparable to the conditions of the structures prior to the construction of the Phase 1 project.*

The sandbag revetment variance requests cited erosion rates on the order of 8 to 10 feet *per month* and more than 100 ft/yr (source: Topsail Reef sandbag revetment variance request, 2014). The 2016 APTIM channel realignment study identifies the pivot channel as the recommended alternative. This is presented as the preferred alignment for this analysis as well. Figures 7-10 and 7-11 present initial and final (2-year) bathymetries for the pivot channel alternative.

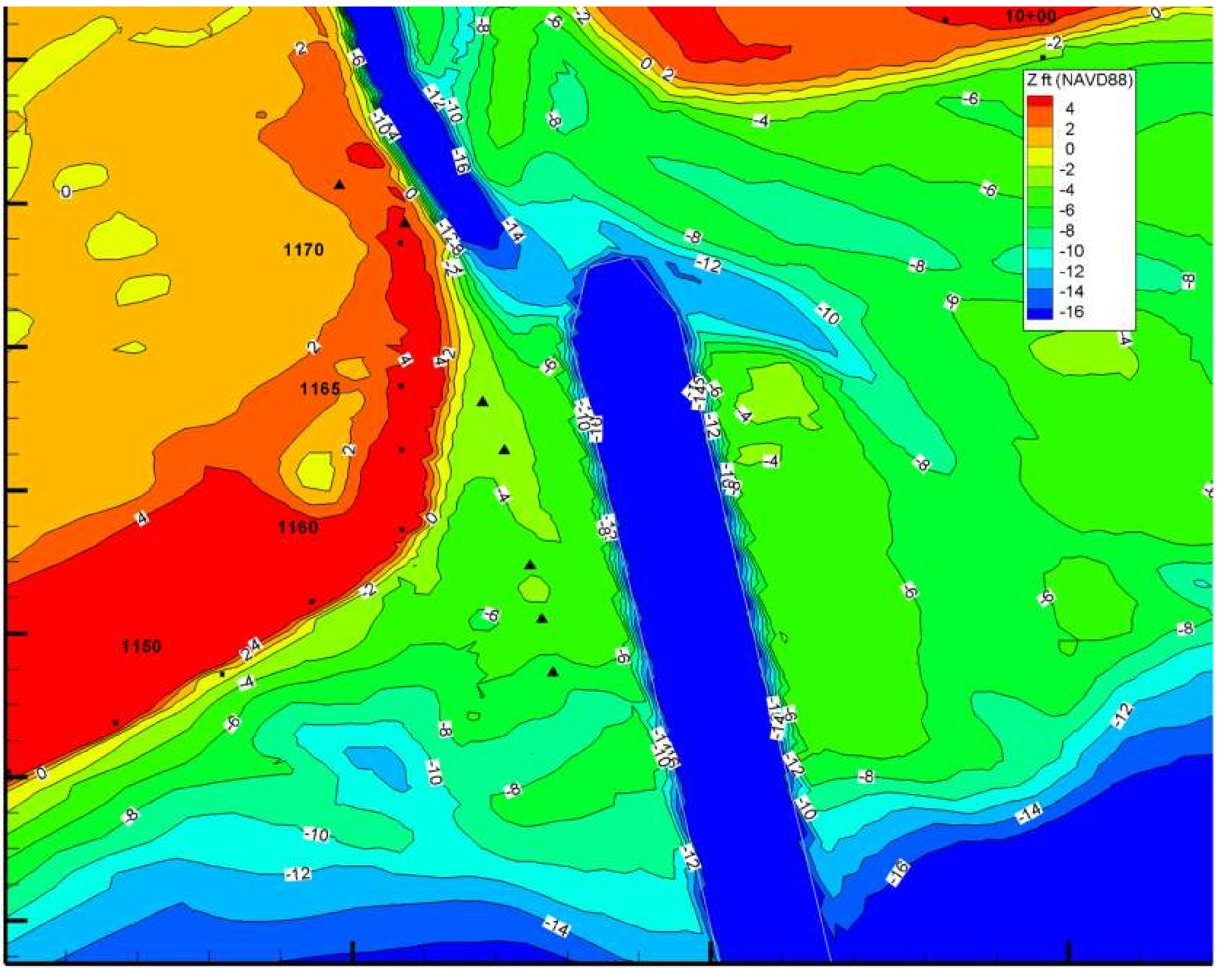


Figure 7-10. Pivot Channel Initial Bathymetry.

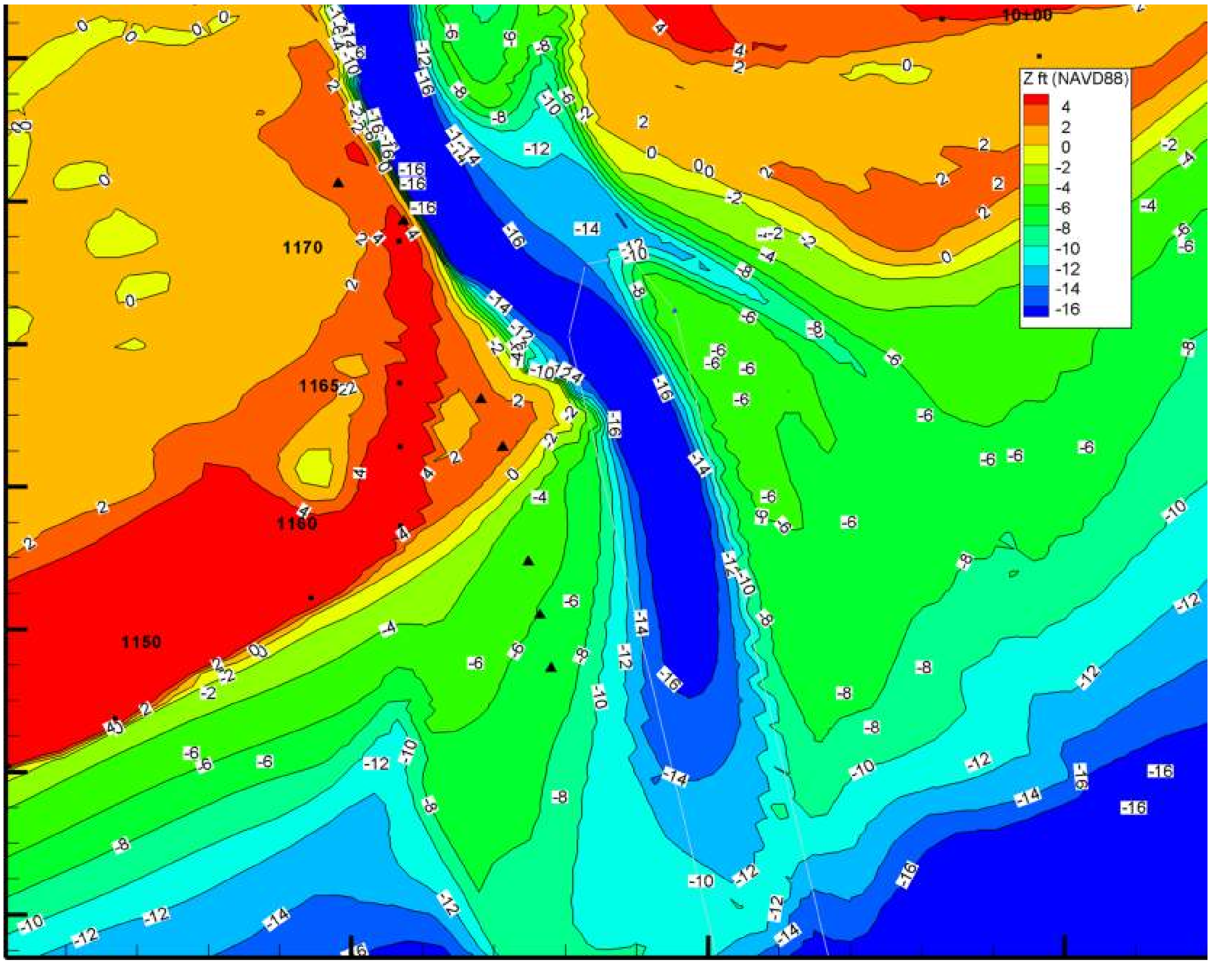


Figure 7-11. Pivot Channel final bathymetry

Figure 7-12 presents relative change between the no-action (baseline) final bathymetry and the pivot channel final bathymetry. Relative changes are valuable at isolating significant differences in the model solutions. As with most coastal process models, the Delft3D model is based on physics. The calibration effort strives to simulate existing conditions in absolute terms, however, there are always areas of slight under or overprediction related to currents, wave dissipation, bottom friction responses, etc. By comparing model runs on a relative basis, these areas of slight under- or over-prediction get "zeroed out", which allows the reviewer to focus on the changes in physics between model runs. Intuitively, hydrodynamic and sedimentation processes farther away from the project should show relatively little or no differences (shown as white in the figures) between alternatives, which is generally the case for these alternatives.

As anticipated, changes in Figure 7-12 are focused on the pivot channel dredged footprint and are lessened the farther away from the footprint.

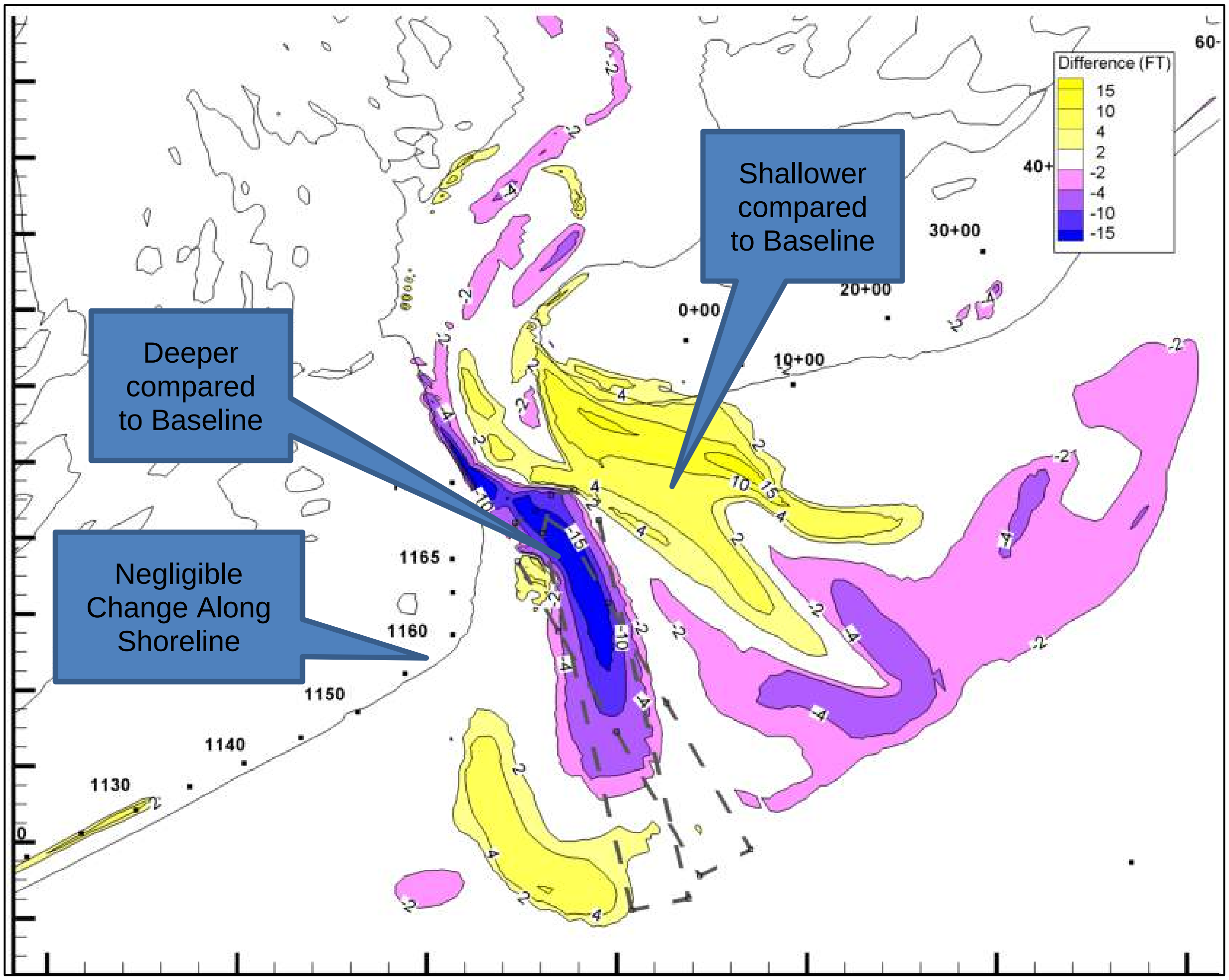


Figure 7-12. Pivot Channel versus No-Action Final Bathymetry Comparison (Baseline=No Action). Yellow contours indicate Pivot Channel is shallower than baseline, blue contours=Pivot Channel is deeper than baseline.

In comparing relative change figures between groin alternatives and channel dredging alternatives, channel dredging alternatives have a larger impact area/influence within the project area. This is not unexpected, as the inlet is more dynamic and complex than the nearby shoreline regions. Figure 7-12 shows a negligible change along the project area ocean-facing shorelines, indicating that channel dredging does not have much influence on these areas (positive or negative). The dredged channel is still deeper relative to the baseline condition at the end of the 2-year run. In absolute terms, the 2013 channel dredging filled in in about 2 years. As discussed in the 2016 APTIM channel realignment study, the model simulated dredged channel infilling at a slower rate than occurred following the 2013 project. This could be related to more energetic wave conditions than modeled or several other naturally varying elements related to sediment transport processes. Again, relative comparisons between runs versus absolute comparisons is a more useful tool in evaluating project alternatives.

7.1.4 SEDIMENT TRANSPORT VECTORS AND ANALYSIS

A sediment transport analysis was conducted using model results from several cases to compare the preferred terminal groin to baseline (no-action) conditions. Mean sediment transport data over the 2-year model runs is the primary variable considered for this analysis. Figure 7-13 presents sediment transport vectors for the nourishment-only alternative on the final (2-year) bathymetry of the nourishment-only alternative. Vectors are shown in every other cell, and vector lengths are based on magnitude (i.e., the longer the vector, the higher the transport).

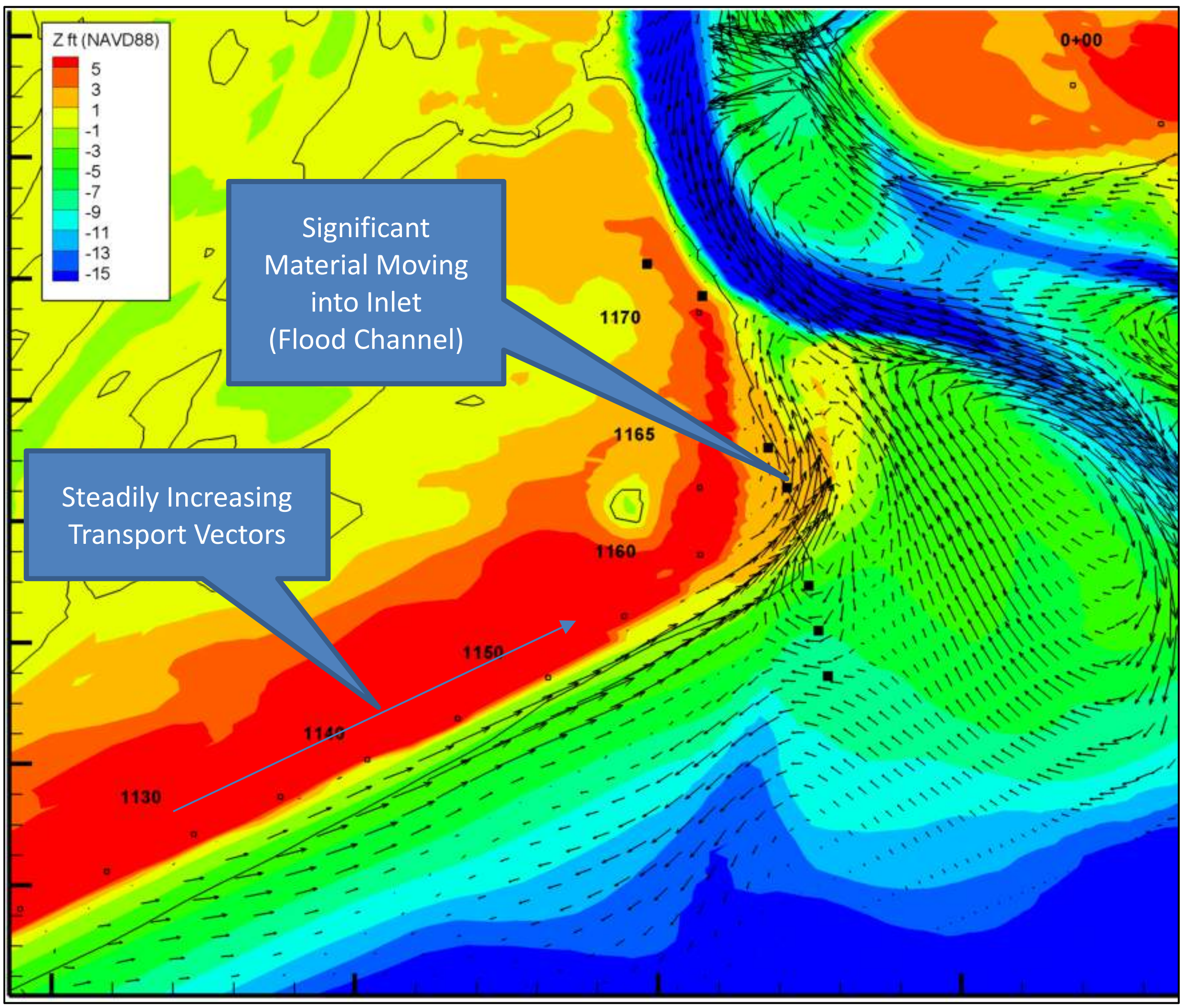


Figure 7-13. Nourishment-Only Transport Vectors Overlain on Nourishment-Only Final Bathymetry

As experienced following the 2013 nourishment project, significant sediment transport can be seen moving material into the inlet along the northern NTB shoreline (Figure 7-13). This is the location of the "flood channel" along the nearshore that has been cited as occurring following the 2013 nourishment. The transport vectors are steadily increasing along the NTB shoreline, indicating erosion along this entire shoreline (steadily decreasing vectors indicate accretion).

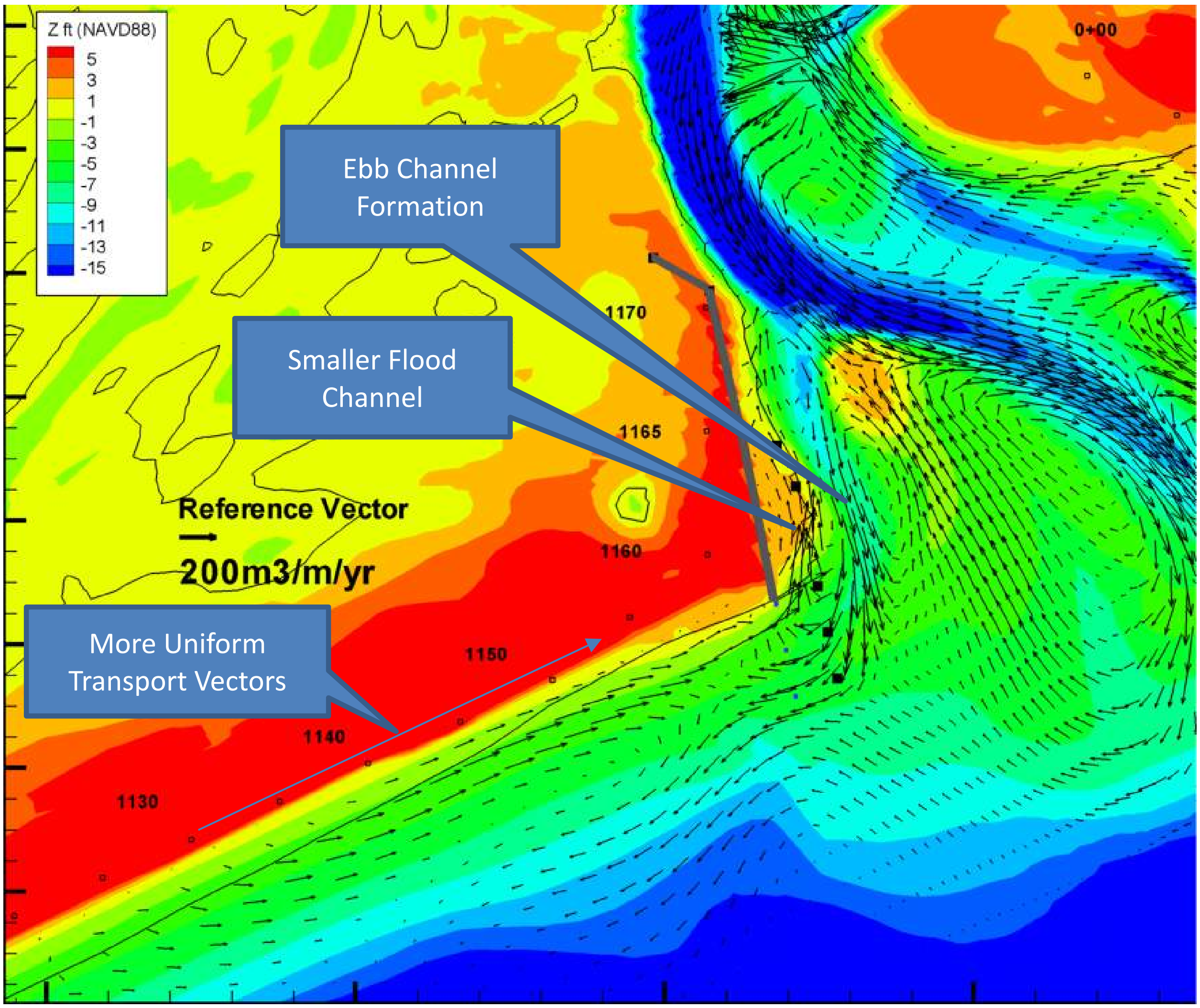


Figure 7-14. NGroin mean sediment transport vectors overlain on final NGroin bathymetry

Figure 7-14 presents the NGroin alternative in the same format as Figure 7-13. Significant changes occur at the groin structure and also along the NTB shoreline. An increase in erosion (increasing transport vectors) is still seen at Station 1145+00, however, the transport is significantly decreased in general. A flood channel along the groin still occurs as well, just at a significantly smaller magnitude as the nourishment-only alternative.

As previously noted, net transport is to the southwest from a regional perspective, although localized sediment transport patterns do occur closer to NRI. Figure 7-15 presents sediment transport vectors, with sediment transport color contours (only showing every 4th vector cell for legibility) for the no-action (baseline) case.

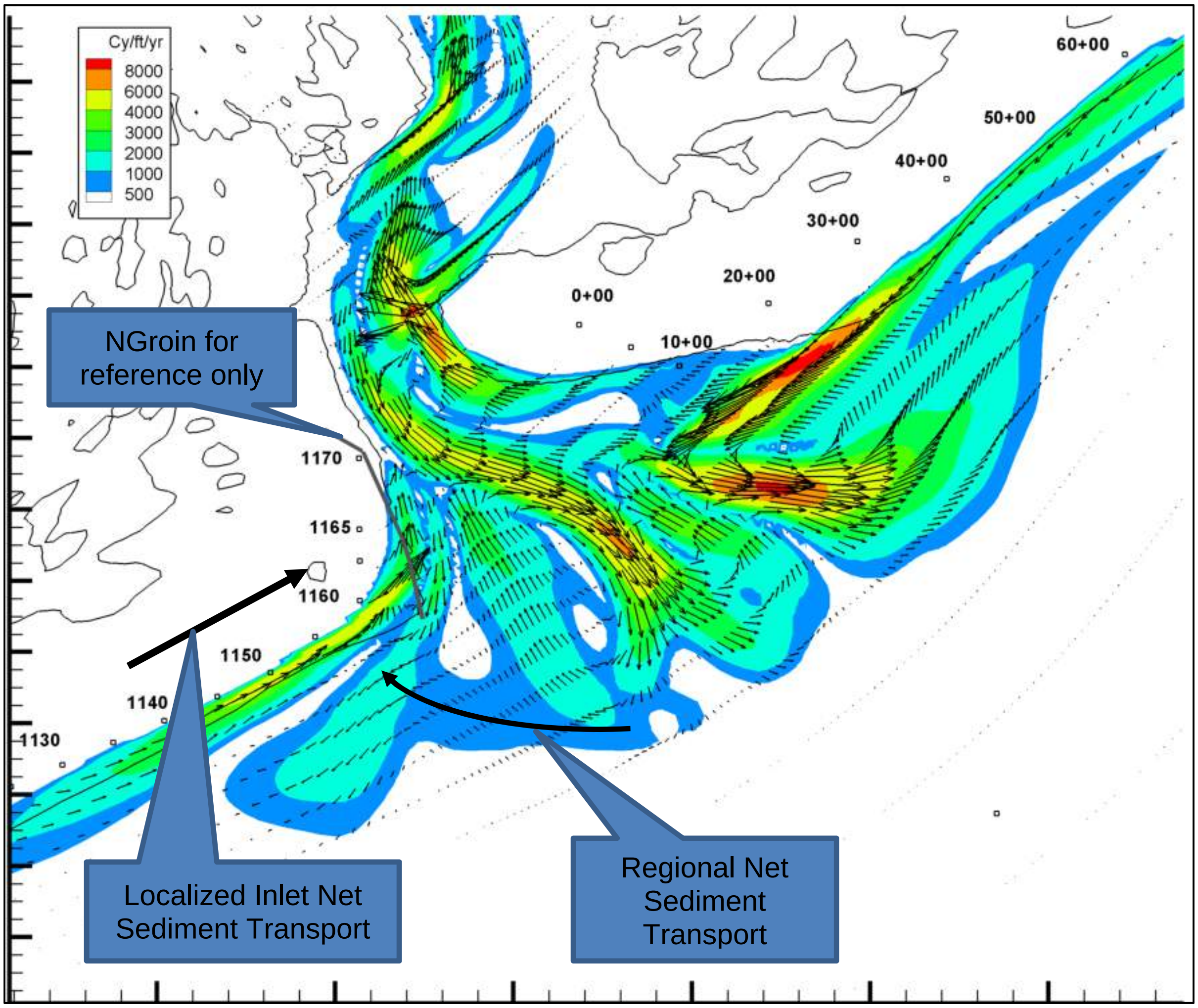


Figure 7-15. No-Action (Baseline) Total Mean Sediment Transport Color Contours and Vectors. NGroin shown only for reference.

Sediment transport vectors are the strongest in the inlet, and the model simulations capture both regional sediment transport as well as localized sediment transport. The NGroin alternative design is short enough to interrupt typical localized sediment transport only and not regional sediment transport. Jetties typically interrupt both localized and regional sediment transport processes. Figure 7-16 presents the no-action (baseline) alternative contours more zoomed in to the northern NTB shoreline. The flood channel and steadily increasing transport is exhibited along the shoreline.

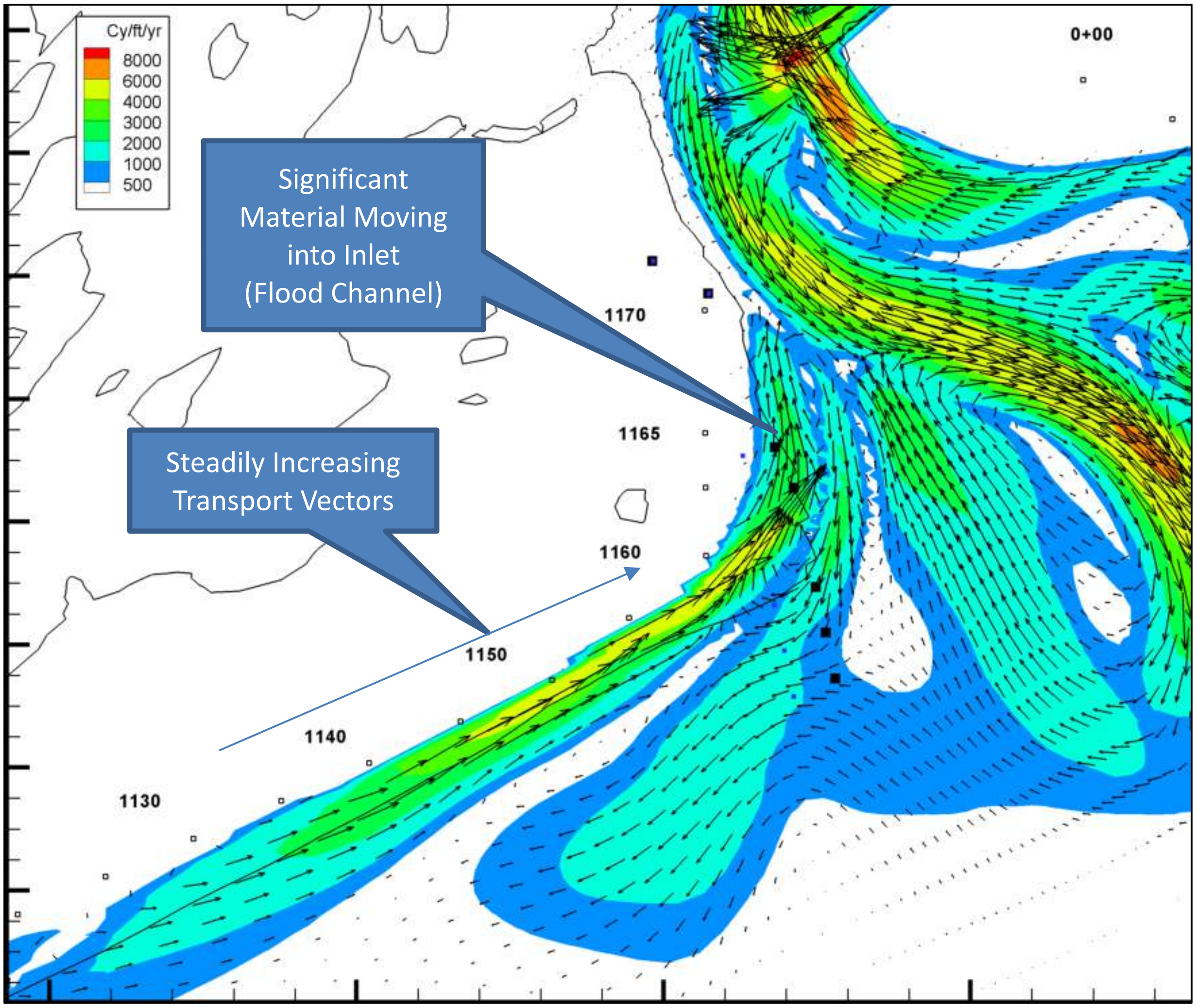


Figure 7-16. No-Action (Baseline) Total Mean Sediment Transport Color Contours and Vectors Zoomed in to Northern NTB Shoreline

Figure 7-17 presents the NGroin sediment transport contours for the project area. The same features are provided as those in Figure 7-13. From a navigational perspective, a more stable ebb channel has formed along the NGroin. Note that these model runs do not include the channel dredging. This channel location is anticipated to be a permanent feature along the NGroin, which should significantly aid navigation at NRI. With channel dredging, a more stable channel will occur along the NGroin terminal groin. Similar regional effects are shown when comparing the NGroin alternative to baseline conditions.

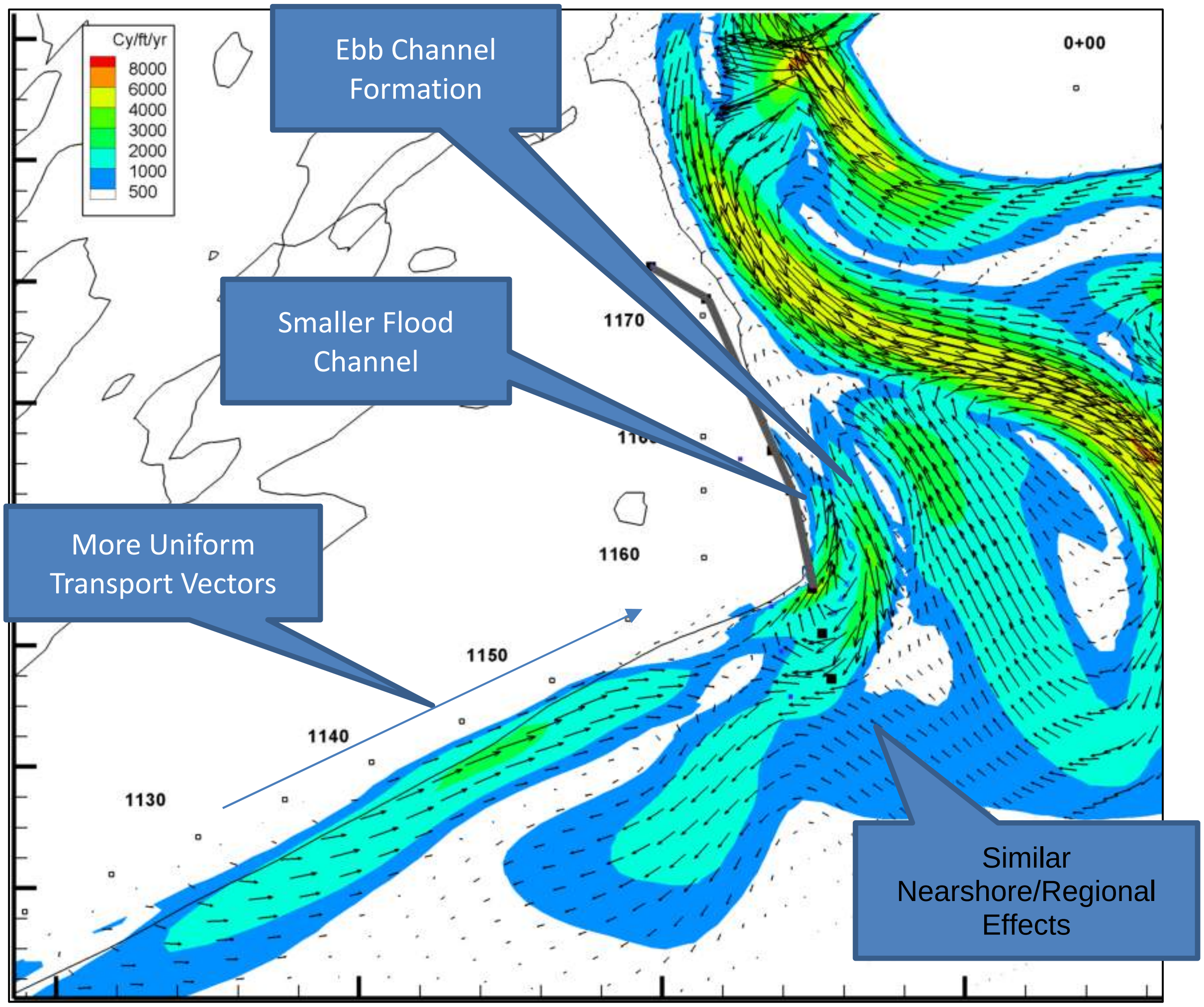


Figure 7-17. NGroin Vectors Total Mean Sediment Transport Color Contours and Vectors

Figure 7-17 presents a relative comparison between the NGroin and nourishment-only alternatives. The NGroin alternative has a significant effect in decreasing sediment transport between Station 1125+00 and the terminal groin along the inlet. Some increased sediment transport is located at the seaward end of the terminal groin. Additionally, some changes to sediment transport are seen in the inlet area, however, these changes are relatively small when compared with the significant transport processes simulated for baseline conditions. The channel dredging was not modeled for this alternative to isolate effects of the groin and nourishment from the channel dredging.

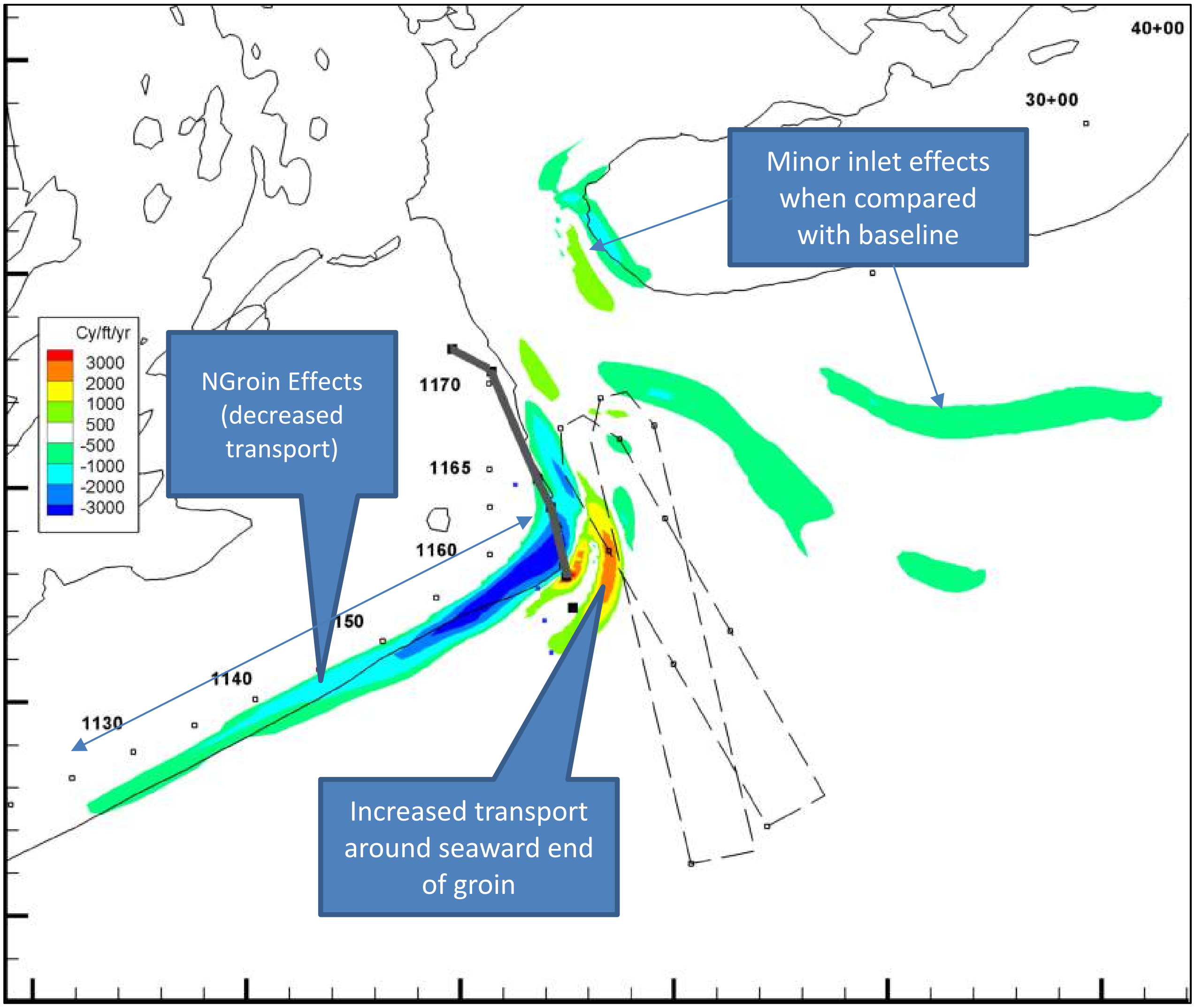


Figure 7-18. NGroin versus Nourishment-Only Mean Annual Transport Relative Comparison. Channel alignments only shown for reference (not modeled for this case).

Figure 7-19 presents the change in sediment transport related to the pivot channel versus nourishment-only conditions. No groin was simulated in this figure, just the pivot channel dredging. Much larger changes are seen in the inlet when comparing Figures 7-18 and 7-19.

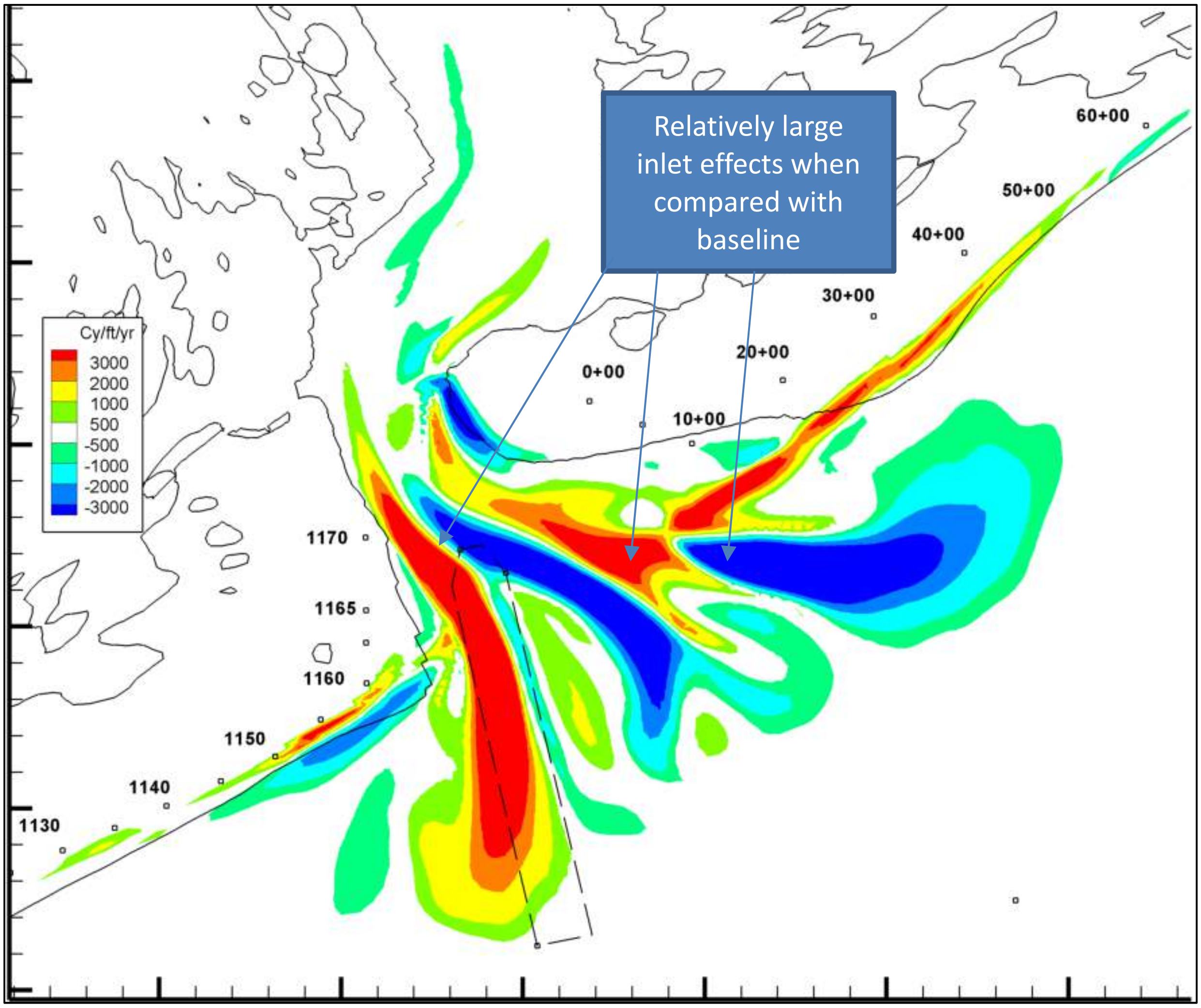


Figure 7-19. Pivot-Channel versus Nourishment-Only Mean Annual Transport Relative Comparison. Channel alignment shown was modeled for this case.

7.1.5 NODAL AREA

The model alternatives are run under identical forcings (waves, currents, etc.) for the 2-year timespans. As previously discussed, comparison between model runs can provide valuable insight into relative changes. The nodal area discussed in Section 4 has generally been identified based on measured data near Station 1135+00 or 1140+00. The nodal area is where sediment transport direction diverges and is related to the regional and localized sediment transport influences.

Figure 7-20 presents sediment transport contours for the nourishment-only alternative. Net northerly transport is shown all the way to about Station 1120+00, with significant increases in transport near Station 1140+00 (Topsail Reef condominiums), where erosion is generally very high. This increase is sediment transport agrees with the erosion rates in this area.

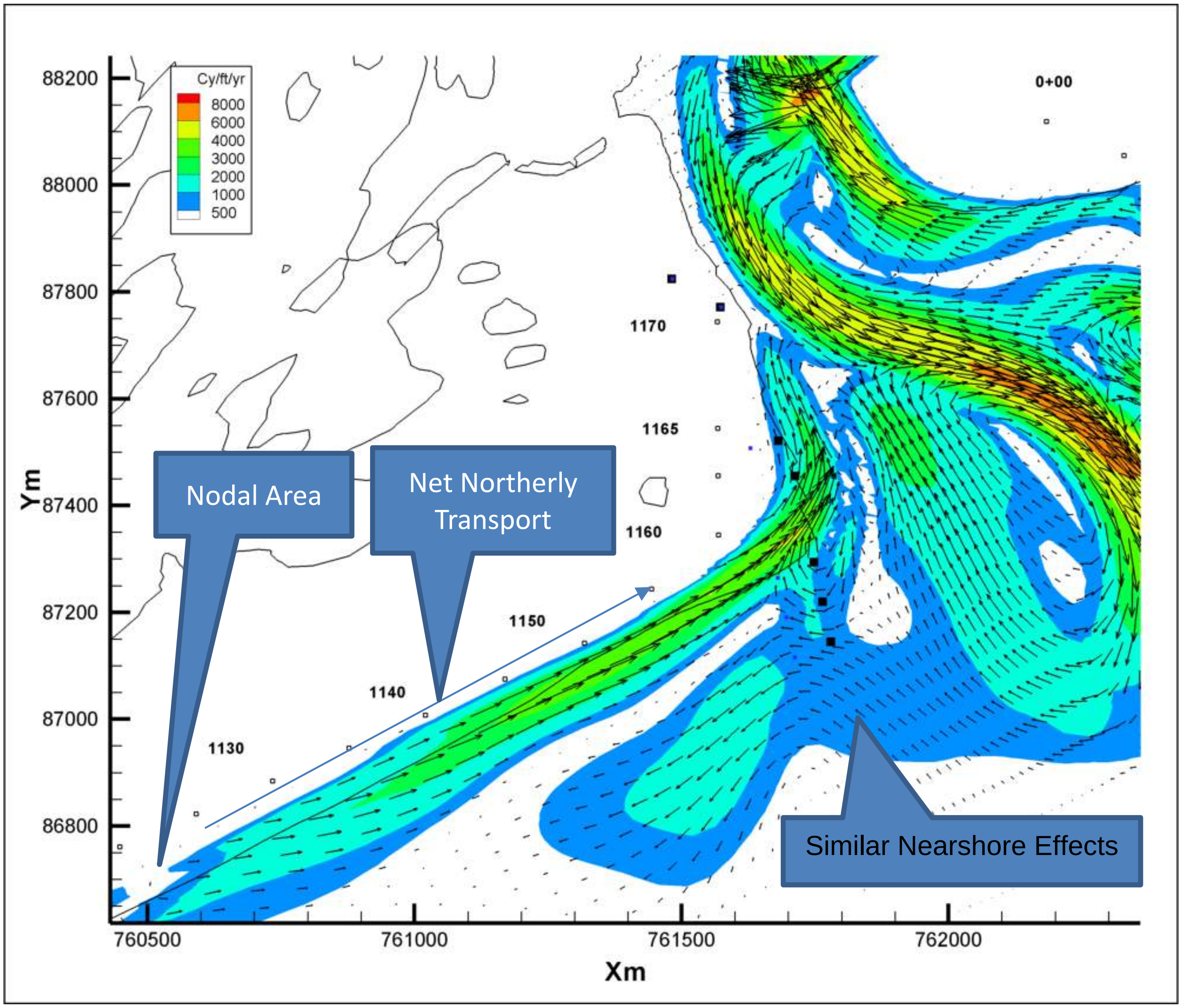


Figure 7-2. Nourishment-Only Sediment Transport Contours. The Nodal area is shown near Station 1120+00.

Figures 7-21 and 7-22 locate the nodal area for the no-action (baseline) and NGroin alternatives, respectively.

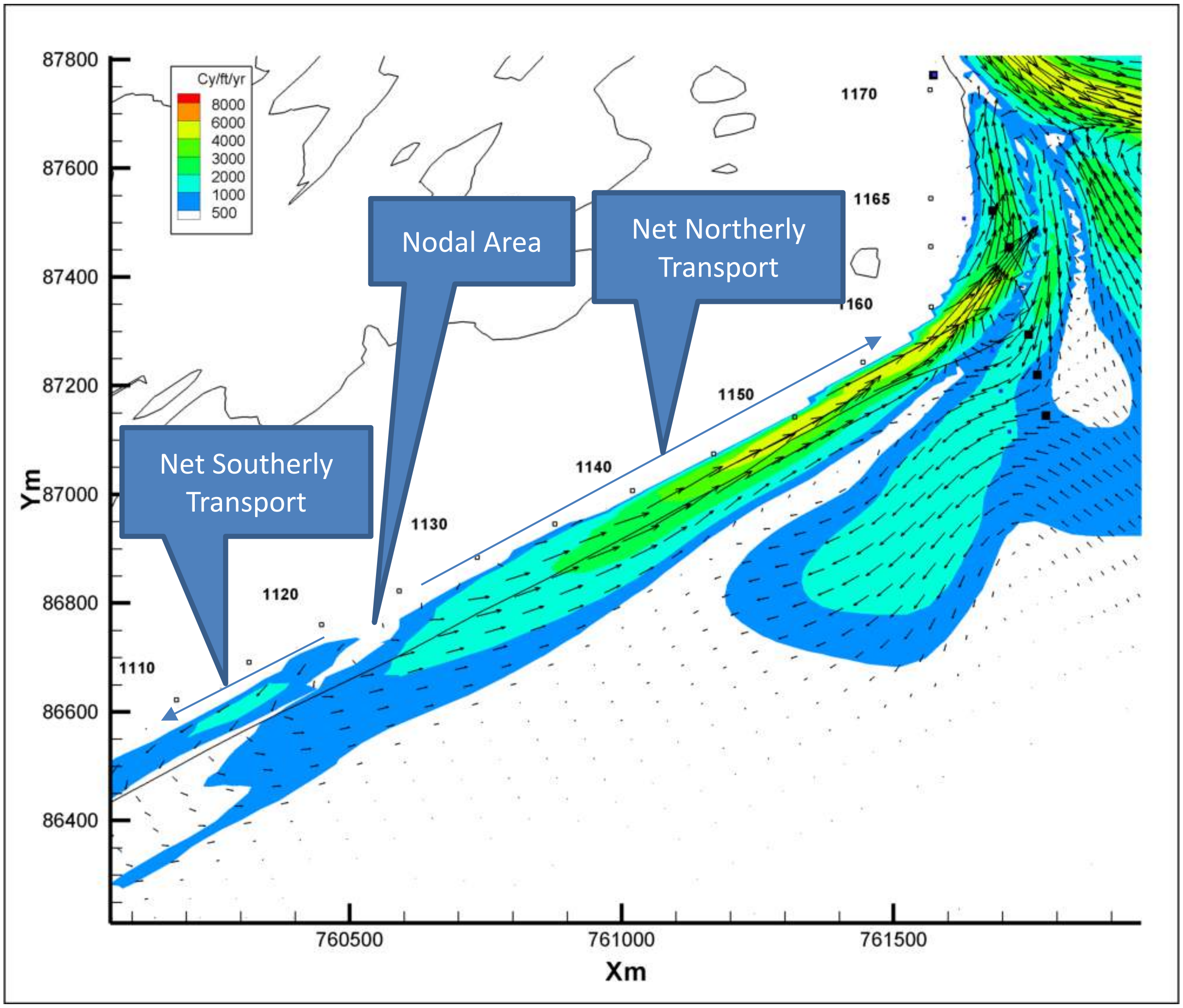


Figure 7-21. No-Action (Baseline) Conditions Show Nodal Area near Station 1125+00

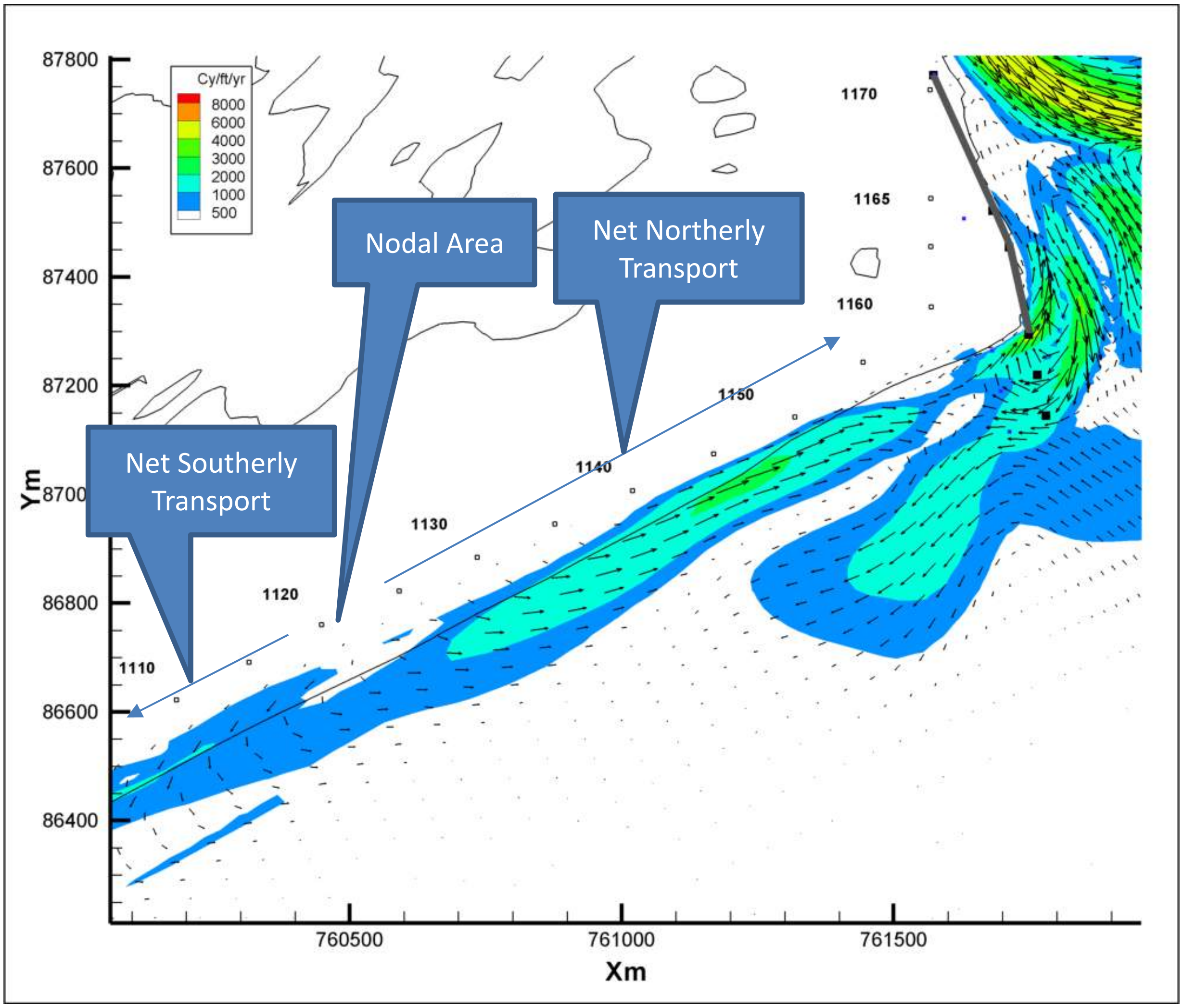


Figure 7-22. NGroin and Nourishment Alternative Shows Nodal Area near 1120+00

The nodal area location can vary based on wave climate and directionality, however, the modeling does indicate that nourishments, channel realignments, and groins can also affect the nodal location.

The results of the 2-year simulations and sediment analyses illustrate several key processes:

1. Longshore transport generally occurs in the nearshore (correlating with sandbars and surf zone);
2. More sediment movement associated with inlet processes;
3. Increased sediment transport along northeastern end of NTB relative to the surrounding oceanfront shoreline; and

4. Groin alternatives decreased local sediment transport in the nearshore on northeastern NTB, while regional transport remains unaffected.

7.1.6 LONG TERM BEACH RENOURISHMENT INTERVAL ANALYSIS

Figure 7-23 illustrates a possible nourishment schedule scenario comparing the nourishment-only and groin-and-nourishment alternatives. These nourishment schedules are based upon the erosion rates simulated for each modeled alternative, as well as the performance of the 2013 nourishment, which was similar in size/scope. The analysis assumes 300,000 cy renourishments after the initial project (also of 300,000 cy). Since the nourishment-only alternative erodes faster than the groin alternative, Figure 7-23 shows that the nourishment-only alternative is anticipated to occur every 2 years whereas the groin-and-nourishment alternative is anticipated to occur every 4 years. The project schedule of the groin/nourishment alternative results in substantial savings over the shown 30-year timespan by reducing 15 nourishment-only events to 8 nourishment events with the groin constructed. The benefits of decreased nourishment frequencies provided by the groin are discussed further in subsequent sections. An additional benefit provided by the groin alternatives is reduced infilling (and subsequently reduced dredging costs) of the New River ebb shoal navigation channel.

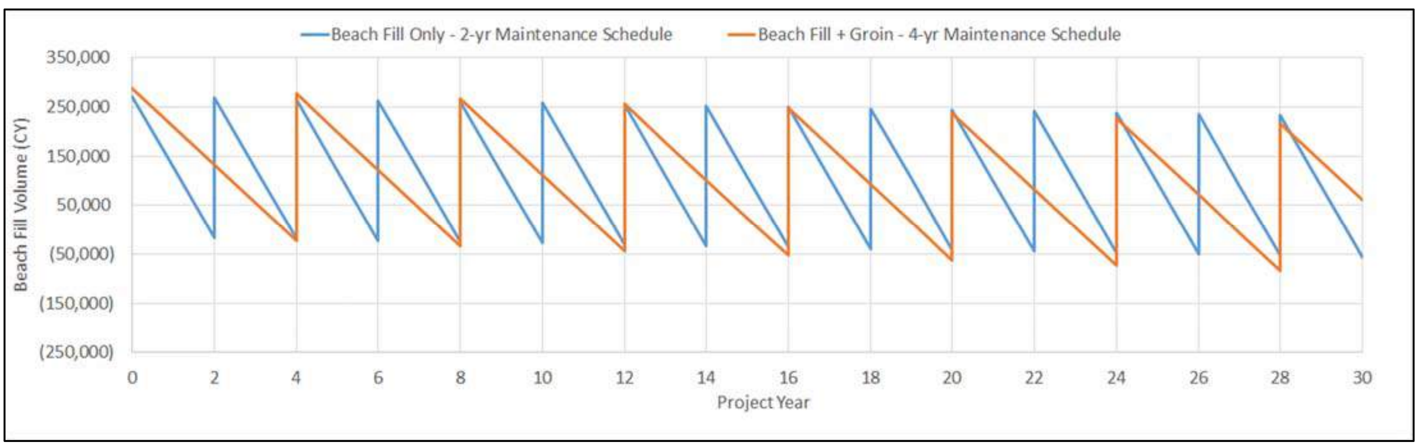


Figure 7-23. Potential Beach Volumes and Nourishment Schedules Based on Decreased Groin Alternative Nourishment Frequency.

7.1.7 HYDRODYNAMICS ANALYSIS

In addition to sediment transport and erosion/accretion analysis, comparisons of hydrodynamics (flows, currents, etc.) were also conducted between alternatives. APTIM performed some hydrodynamic analyses for its 2016 channel realignment modeling. Included in APTIM's analysis was tidal prism evaluation for the different dredged channels. Figure 7-24 presents summary results from that analysis.

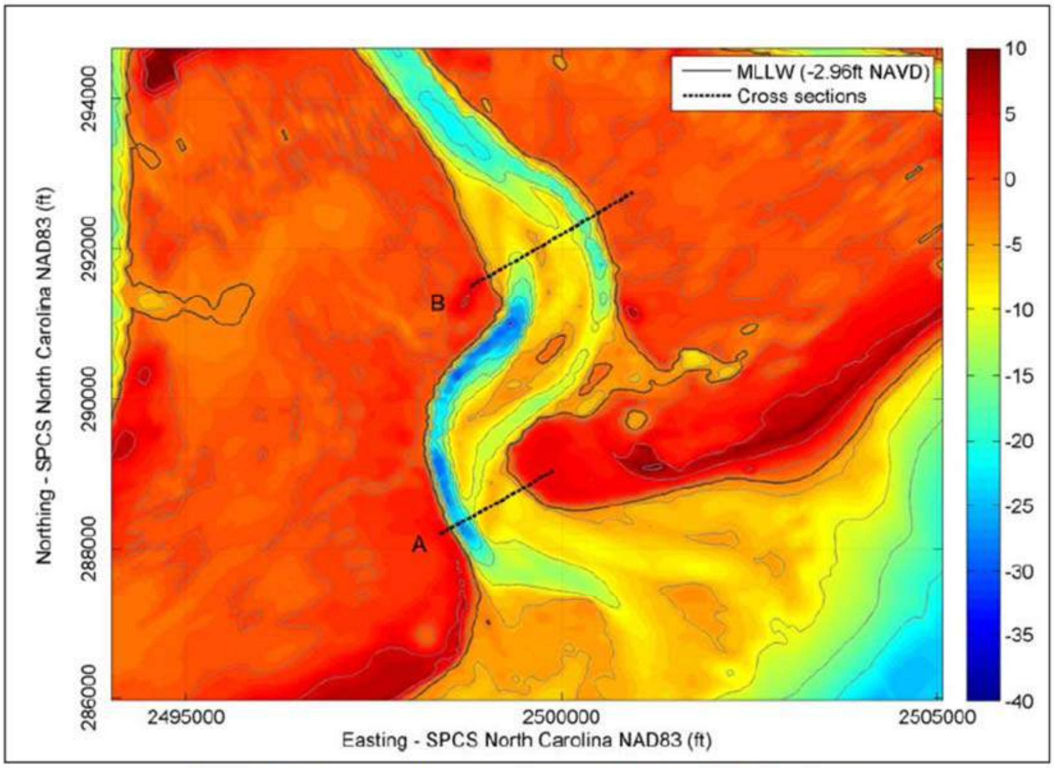


Figure 62. Cross-Sections used to compute model tidal prisms.

Table 11. Average Ebb Tidal Prism for Alternatives.

Alternative	Mean Tidal Prism (10^6 ft^3)		Percent Difference Relative to No Action	
	A	B	A	B
No Action	509.8	503.0	--	--
Alternative 1	526.0	519.2	3.2%	3.2%
Alternative 2	521.1	514.4	2.2%	2.3%
Alternative 3	521.1	514.4	2.2%	2.3%
Alternative 3-250 ft. Shift	524.9	518.1	3.0%	3.0%
2013 Channel	526.8	519.8	3.3%	3.3%

Figure 7-24. Channel Dredging Alternatives Tidal Prism Analysis (source: APTIM, 2016)

A small increase in tidal prism is noted due to the channel dredging, which allows slightly more volume flow to enter the New River estuarine system. The Delft3D sediment transport module uses a morphological factor (morfac) coefficient to allow for longer-term (e.g., 2-year-long model runs) to run in a reasonable amount of time (on the order of 7 to 10 days). For the time series analysis presented in this section, shorter model runs of 40 days were run to evaluate water level, currents, and waves at the time series points/stations shown in Figure 7-25.

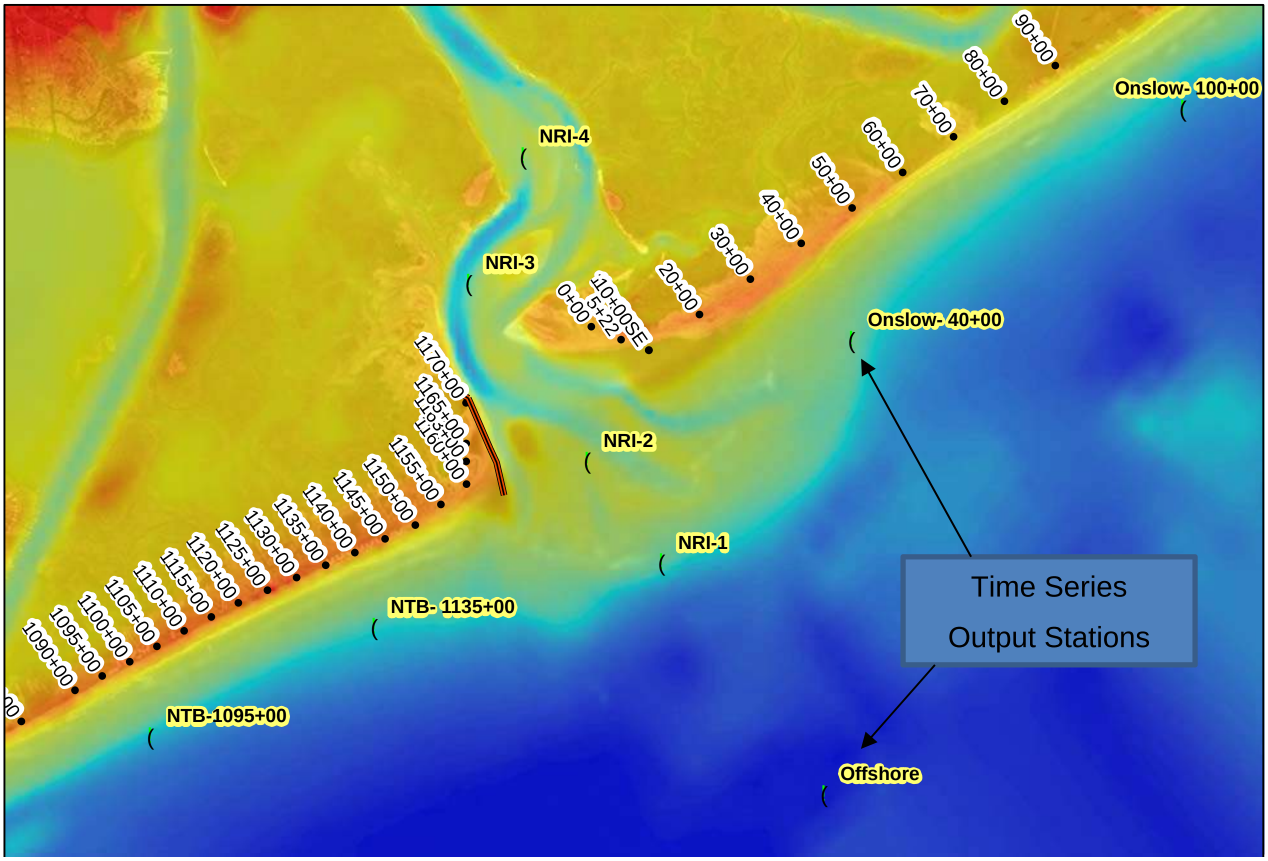


Figure 7-25. Model Time Series Output Stations

An example time series of water level and currents is provided in Figure 7-26 for the Offshore station. Currents are shown as U and V components in meters per second (m/s). Currents are generally wave dominant for the ocean stations, whereas inlet stations are tidally dominated.

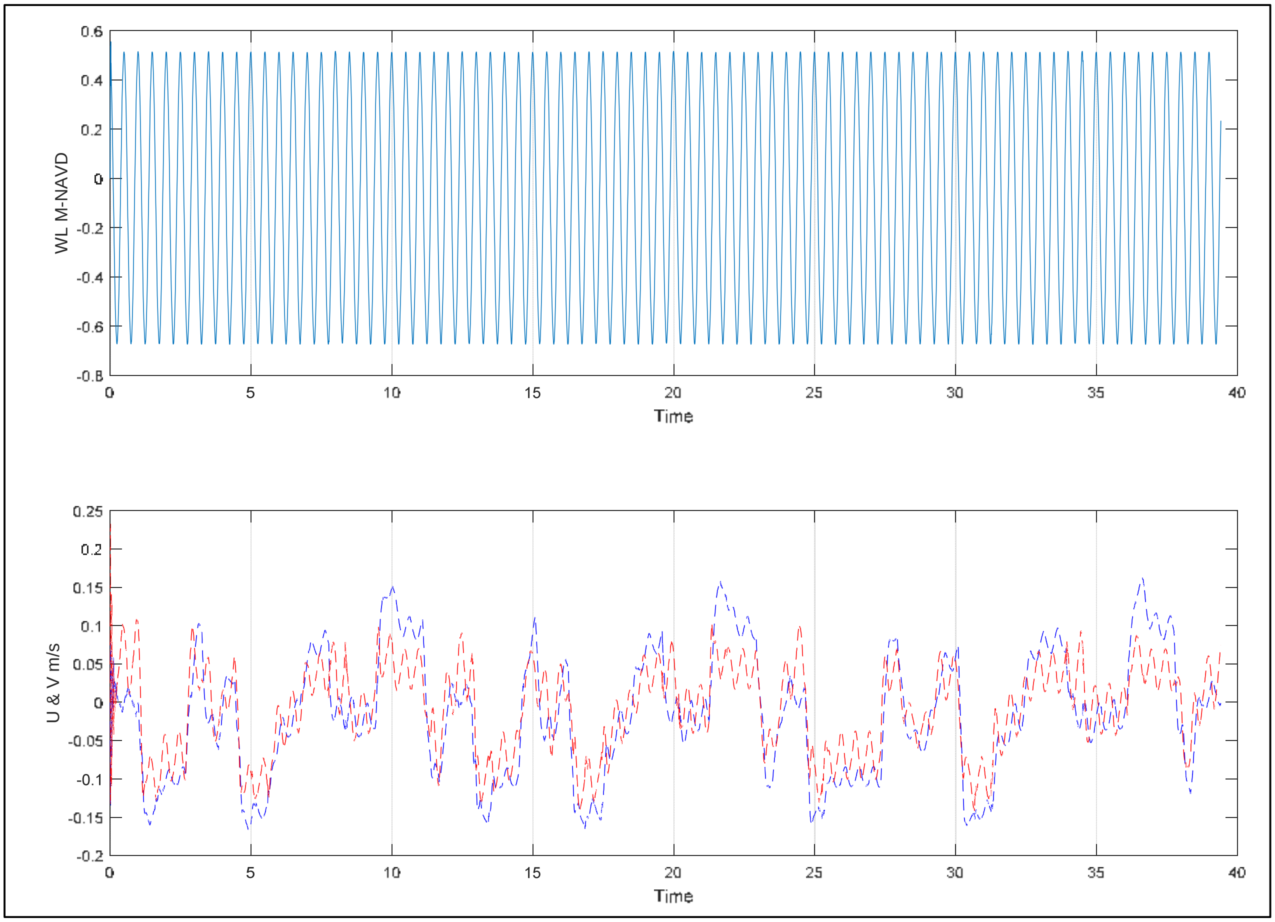


Figure 7-26. "Offshore" Station (NGroin Alternative) Water Level and Current Time Series

Water level statistics for 10 percent, 50 percent and 90 percent are provided in Table 7-4 for the NGroin and nourishment-only alternatives. As anticipated, only negligible changes (about 0.01 m) are found. No borrow area dredging is included in these two alternatives.

Table 7-4: Water Level Time Series Statistics Comparison for NGroin and Nourishment-Only Alternatives

	NGROIN				NOURISHMENT ONLY			
Water Level (Meters-NAVD88)	10%	50%	90%	Range (90-10)	10%	50%	90%	Range (90-10)
NRI1	-0.65	-0.10	0.48	1.13	-0.65	-0.10	0.48	1.13
NRI2	-0.56	-0.05	0.47	1.03	-0.55	-0.05	0.47	1.03
NRI3	-0.41	-0.02	0.39	0.80	-0.41	-0.02	0.39	0.79
NRI4	-0.35	-0.01	0.34	0.69	-0.35	-0.01	0.33	0.68
Onslow 40+00	-0.64	-0.09	0.49	1.13	-0.64	-0.09	0.49	1.13
Onslow 100+00	-0.64	-0.08	0.49	1.13	-0.64	-0.08	0.49	1.13
Offshore	-0.64	-0.10	0.48	1.13	-0.64	-0.10	0.48	1.13
NTB 1135+00	-0.65	-0.10	0.48	1.13	-0.65	-0.10	0.48	1.13
NTB 1095+00	-0.65	-0.10	0.49	1.13	-0.65	-0.10	0.49	1.13

Table 7-5 presents a similar table comparing the pivot channel dredging alternative and the nourishment-only alternative. Some larger changes in water level (up to 0.07 m) at NRI-2, NRI-3 and NRI-4 are evidenced, although these changes are small relative to the water level ranges at each station.

Table 7-5. Water Level Time Series Statistics Comparison for Pivot Channel and Nourishment-Only Alternatives

	PIVOT CHANNEL				NOURISHMENT ONLY			
Water Level (Meters-NAVD88)	10%	50%	90%	Range (90-10)	10%	50%	90%	Range (90-10)
NRI1	-0.65	-0.10	0.48	1.13	-0.65	-0.10	0.48	1.13
NRI2	-0.59	-0.06	0.48	1.07	-0.55	-0.05	0.47	1.03
NRI3	-0.48	-0.03	0.39	0.87	-0.41	-0.02	0.39	0.79
NRI4	-0.41	-0.02	0.34	0.75	-0.35	-0.01	0.33	0.68
Onslow 40+00	-0.64	-0.09	0.49	1.13	-0.64	-0.09	0.49	1.13
Onslow 100+00	-0.64	-0.09	0.49	1.13	-0.64	-0.08	0.49	1.13
Offshore	-0.64	-0.09	0.48	1.13	-0.64	-0.10	0.48	1.13
NTB 1135+00	-0.64	-0.09	0.48	1.13	-0.65	-0.10	0.48	1.13
NTB 1095+00	-0.64	-0.09	0.48	1.13	-0.65	-0.10	0.49	1.13

Current statistics for 10 percent, 50 percent and 90 percent are provided in Table 7-6. As anticipated, only negligible changes are seen (similar to water level changes). More significant changes do occur in New River due to channel dredging. Table 7-7 presents current data (UV, m/s) for the pivot channel and nourishment-only alts.

Table 7-6. Current Time Series Statistics Comparison for NGroin and Nourishment-Only Alternatives

Currents (m/s)		NGROIN 10%	NGROIN 50%	NGROIN 90%	NGROIN Range (90-10)	Nourishment Only 10%	Nourishment Only 50%	Nourishment Only 90%	Nourishment Only Range (90-10)
NRI1	U	-0.13	0.00	0.13	0.25	-0.13	0.01	0.13	0.25
	V	-0.15	-0.03	0.12	0.27	-0.15	-0.02	0.12	0.27
NRI2	U	-0.21	0.07	0.26	0.47	-0.20	0.07	0.27	0.48
	V	-0.36	-0.04	0.36	0.72	-0.40	-0.05	0.36	0.76
NRI3	U	-0.17	-0.03	0.12	0.29	-0.17	-0.03	0.11	0.28
	V	-0.75	-0.17	0.79	1.54	-0.75	-0.17	0.79	1.54
NRI4	U	-0.07	0.03	0.10	0.17	-0.07	0.03	0.10	0.17
	V	-0.75	-0.19	0.90	1.65	-0.75	-0.19	0.90	1.65
Onslow 40		-0.06	0.05	0.16	0.22	-0.06	0.05	0.17	0.23
	V	-0.11	0.01	0.13	0.24	-0.11	0.01	0.13	0.24
Onslow 100		-0.13	0.01	0.09	0.22	-0.13	0.01	0.09	0.22
	V	-0.13	-0.01	0.05	0.18	-0.13	-0.01	0.05	0.18
Offshore	U	-0.13	-0.02	0.09	0.23	-0.13	-0.02	0.09	0.23
	V	-0.09	0.00	0.06	0.16	-0.09	0.00	0.06	0.15
NTB 1135		-0.14	-0.02	0.11	0.24	-0.14	-0.02	0.11	0.25
	V	-0.11	-0.02	0.04	0.15	-0.11	-0.02	0.04	0.15
NTB 1095		-0.09	0.01	0.11	0.21	-0.09	0.01	0.11	0.21
	V	-0.07	-0.01	0.05	0.12	-0.07	-0.01	0.05	0.12

Table 7-7. Current Time Series Statistics Comparison for Pivot Channel and Nourishment-Only Alternatives

Currents (m/s)	Pivot Channel				Nourishment Only			
	10%	50%	90%	Range (90-10)	10%	50%	90%	Range (90-10)
NRI1 U	-0.13	-0.03	0.08	0.21	-0.13	0.01	0.13	0.25
V	-0.11	-0.01	0.10	0.21	-0.15	-0.02	0.12	0.27
NRI2 U	-0.24	-0.01	0.18	0.42	-0.20	0.07	0.27	0.48
V	-0.28	-0.01	0.29	0.57	-0.40	-0.05	0.36	0.76
NRI3 U	-0.17	-0.03	0.11	0.28	-0.17	-0.03	0.11	0.28
V	-0.80	-0.19	0.82	1.62	-0.75	-0.17	0.79	1.54
NRI4 U	-0.08	0.03	0.11	0.19	-0.07	0.03	0.10	0.17
V	-0.79	-0.19	0.91	1.70	-0.75	-0.19	0.90	1.65
Onslow 40	-0.07	0.03	0.14	0.21	-0.06	0.05	0.17	0.23
V	-0.12	0.00	0.10	0.22	-0.11	0.01	0.13	0.24
Onslow 100	-0.14	0.00	0.09	0.22	-0.13	0.01	0.09	0.22
V	-0.13	-0.02	0.05	0.18	-0.13	-0.01	0.05	0.18
OffshoreU	-0.13	-0.02	0.09	0.22	-0.13	-0.02	0.09	0.23
V	-0.09	0.02	0.10	0.18	-0.09	0.00	0.06	0.15
NTB 1135	-0.13	0.00	0.11	0.24	-0.14	-0.02	0.11	0.25
V	-0.11	-0.02	0.04	0.15	-0.11	-0.02	0.04	0.15
NTB 1095	-0.09	0.01	0.11	0.20	-0.09	0.01	0.11	0.21
V	-0.07	0.00	0.05	0.12	-0.07	-0.01	0.05	0.12

Biological Studies Related to Groins

The modeling study was used to assess potential changes to biological resources in the project study area. Blanton et al. (1999) performed the South Atlantic Bight Recruitment Experiment (SABRE) to study the transport of winter-spawned fish larvae into estuaries. Blanton et al. (1999) found larvae concentrated on the shelf in a narrow “withdrawal zone” upwind of an inlet within the 23-ft (7-m) depth contour. When the ocean currents were appropriate, the larvae passed through the inlets (Blanton et al., 1999). Even with the best wind and tidal conditions, only about 10 percent of the available larvae are successfully drawn into the inlet (Blanton et al., 1999).

The Blanton study found that the 7-m contour was of particular relevance to larval recruitment. Figure 7-27 identifies the 7-m contour relative to the final NGroin bathymetry. The 7-m contour is approximately 2,700 feet seaward of the NGroin structure, therefore, based on this metric, larval recruitment should be negligibly affected.

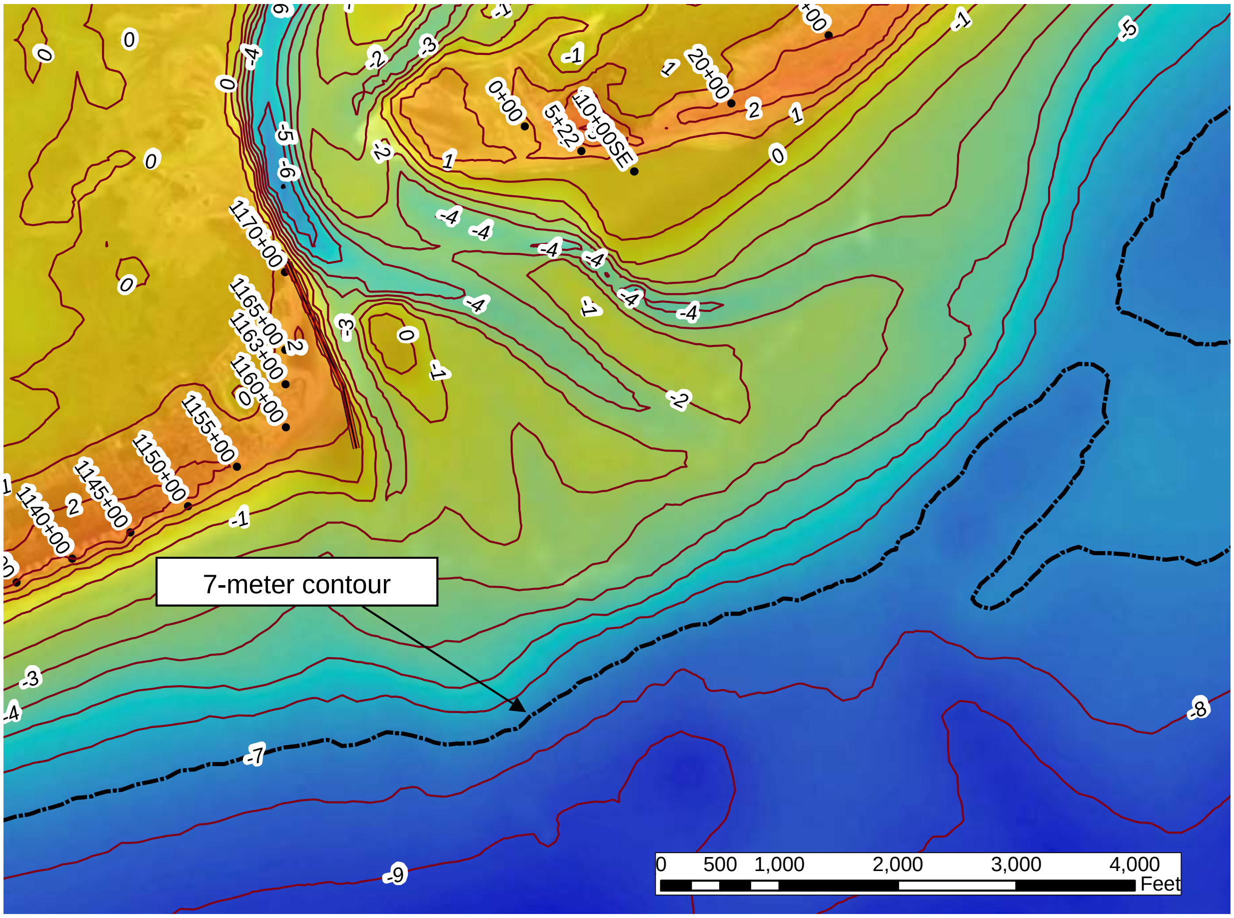


Figure 7-27: NGroin Relative to Depth Contours (in meters). The 7-m depth contour is a significant distance from the NGroin (~ 2,700 feet seaward of the NGroin) (final 2-yr bathymetry shown).

The 2010 NC Coastal Habitat Protection Plan – Appendix I provides several factors that appear to minimize biological impacts of nourishment projects to the intertidal beach community. These include, but are not limited to, the following:

- Use of sand similar in grain size and composition to original beach sands (specific minimum and maximum standard needed)
- Restrict beach nourishment to winter months to minimize mortality of infauna and enhance recovery rates of intertidal benthic organisms, an important prey source for many surf fish (Donoghue, 1999)
- Limit time interval between projects to allow full recovery of benthic communities (1 to 2 years, depending on timing of project and compatibility of sediment)

- Limit linear length of nourishment projects to provide undisturbed area as a source of invertebrate colonists for the altered beach and a food source for fish

All these avoidance and minimization guidelines were used in evaluating the proposed project. A major goal of the groin and nourishment project is to increase the interval between projects.

Potential impacts to natural resources were evaluated in the State Terminal Groin Report (Moffatt & Nichol, 2010). Excerpts of some potential benefits include the following:

- As supported by the NOAA National Marine Fisheries Service (NMFS), a rock rubble structure extending below the intertidal zone in a sandy bottom location would likely induce and support the development of a diverse benthic community supporting higher trophic levels of both fish and birds within the vicinity and footprint of a terminal groin.
- In the case of rubble-mound structures (e.g., jetties, groins, breakwaters, etc.), one beneficial aspect of construction is the creation of artificial reef habitat. This is evidenced by the popularity of coastal rubble-mound structures as recreational fishing spots.
- Groin habitat may provide a foraging site and shelter for fishes in the surf zone and is associated with higher fish abundances and species richness than in other surf zone communities (Peters and Nelson, 1987; Clark et al., 1996).
- Birds in a few ecological categories feed on or near groins and can be considered part of the rubble structure community. These include surface-searching shorebirds, aerial searching birds, floating and diving waterbirds, and wading birds.
- The ruddy turnstone is often found feeding on groins in groups of 100 or more in the Fort Macon State Park area, and purple sandpipers are occasionally abundant in flocks of 40 to 50 on the jetties at Masonboro Inlet (Personal communication, R. Newman, Fort Macon State Park, October 2009; Personal communication, J. Fussell, Birder and Author, February 2010). Both species use rocks and groins as their primary feeding habitats. Other shorebirds use them only on occasion, feeding on surrounding habitats as well (Peterson and Peterson, 1979; Thayer et al., 1984).

A USACE (1996) study also found that:

> *Groins are very effective fish attractors and provide excellent sport fishing sites. These structures, particularly those of rubble-mound construction, may provide*

beneficial protective cover, as well as feeding and resting areas for both juvenile and adult fishes and shellfishes during coastal migrations.

8.0 GROIN DESIGN

Groins are an old and intuitive means of reducing beach erosion and are found along the coast worldwide as both engineered and non-engineered, ad-hoc structures (Kraus and Rankin, 2004). Additionally, groins can and have functioned effectively and economically when properly employed (Meadows et al., 1998). Without the use of groins in conjunction with beach nourishment, two rows of houses along Folly Beach and Edisto Beach, SC would now be in the surf, and most of the high ground on the northern end of Pawleys Island, SC would have been destroyed (Kana et al., 2000).

Groin design considerations for NTB are included in the modeling analysis and alternatives analysis and are described in more detail in this section. The general design goals include: protection of public access, improvement of recreational beach area, enhancement of upper beach/dune habitat, stabilization of the northeast end of the beach (which represents the highest erosion rates on the island) from short-term and long-term fluctuations, reduction of beach nourishment and inlet dredging maintenance costs as well as safer navigation. Groin design parameters have been selected based on these goals, while also minimizing downdrift impacts.

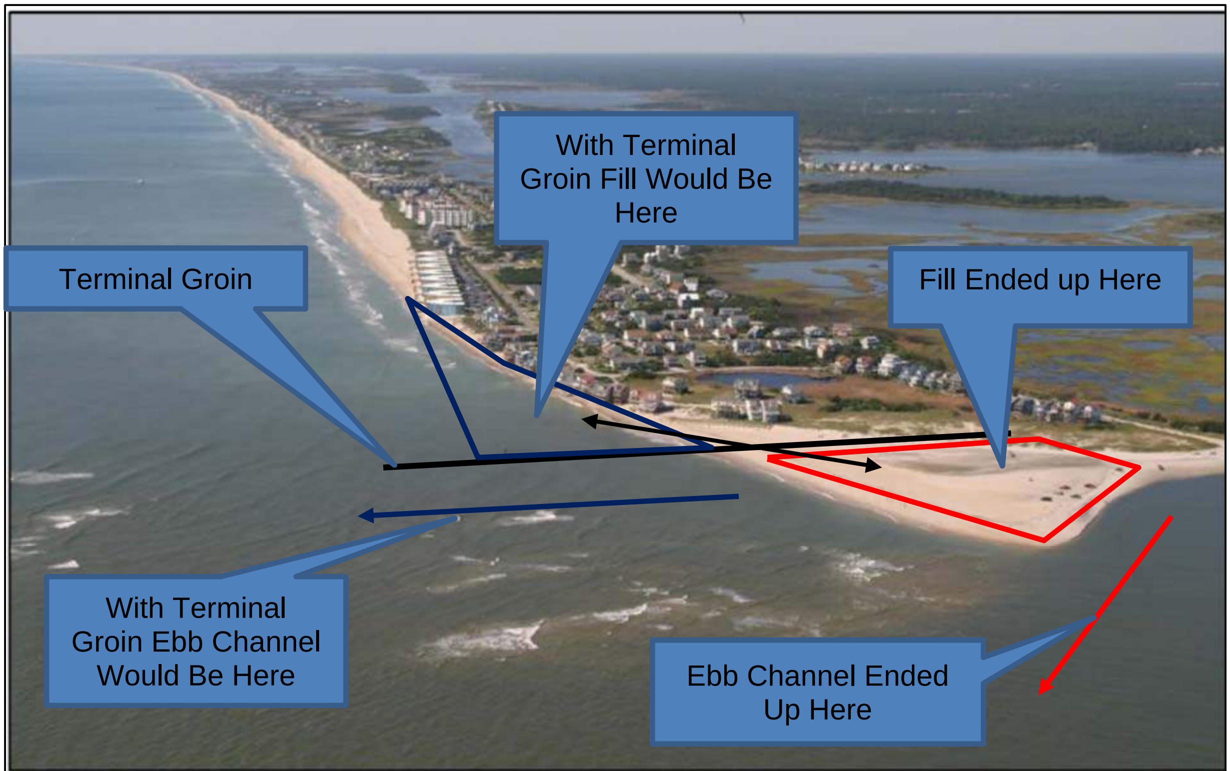
Figure 8-1 again presents a 2015 oblique area showing a general schematic of the proposed groin and how it will interact with the inlet.

8.1 DESIGN CONSIDERATIONS

8.1.1 LENGTH

In general, the length of the terminal groin is dictated by the size of the inlet, the configuration of the end of the island, and the length of shoreline the groin is designed to stabilize. The design groin length is based on modeling as well as on existing structures within Onslow Bay and other nearby areas. Onslow Bay extends approximately 100 miles from Cape Lookout down to Cape Fear and generally displays a similar geology as well as similar tides and waves.

Existing groin structures in Onslow Bay include the Fort Macon terminal groin. The Masonboro Inlet jetties also provide a good nearby reference for design even though they are longer than terminal groins. Additional analysis on existing groins in other areas of the state (e.g., Oregon Inlet, Hatteras, and Bald Head Island) and the region were also assessed. The North Carolina Terminal Groin Report contains significant information on this topic.



(Aerial Photo dated Oct. 5 2014, Provided by Dr. William Cleary)

Figure 8-1: General Terminal Groin Concept in Relation to the 2013 Nourishment Project on a 2015 Oblique Aerial (image from Cleary)

To prevent flanking, a terminal groin should be extended landward of the primary dune and account for historical shoreline positions as well as potential future positions. This "anchor" distance is estimated to be approximately 345 ft for the NGroin. Figure 8-2 presents the NGroin relative to historical shorelines. The landward anchor section will be buried and allow for vehicular beach access at the end of River Drive. The landward anchor section is between 100 and 300 feet away from historical and current shoreline positions and to save costs, this anchor section can likely be constructed a few years after initial construction. The erosional long-term trend (5 to 10 ft/yr at the anchor section shoreline) indicates that the river meander will continue to migrate west and, therefore, the anchor section will likely be needed at some point.

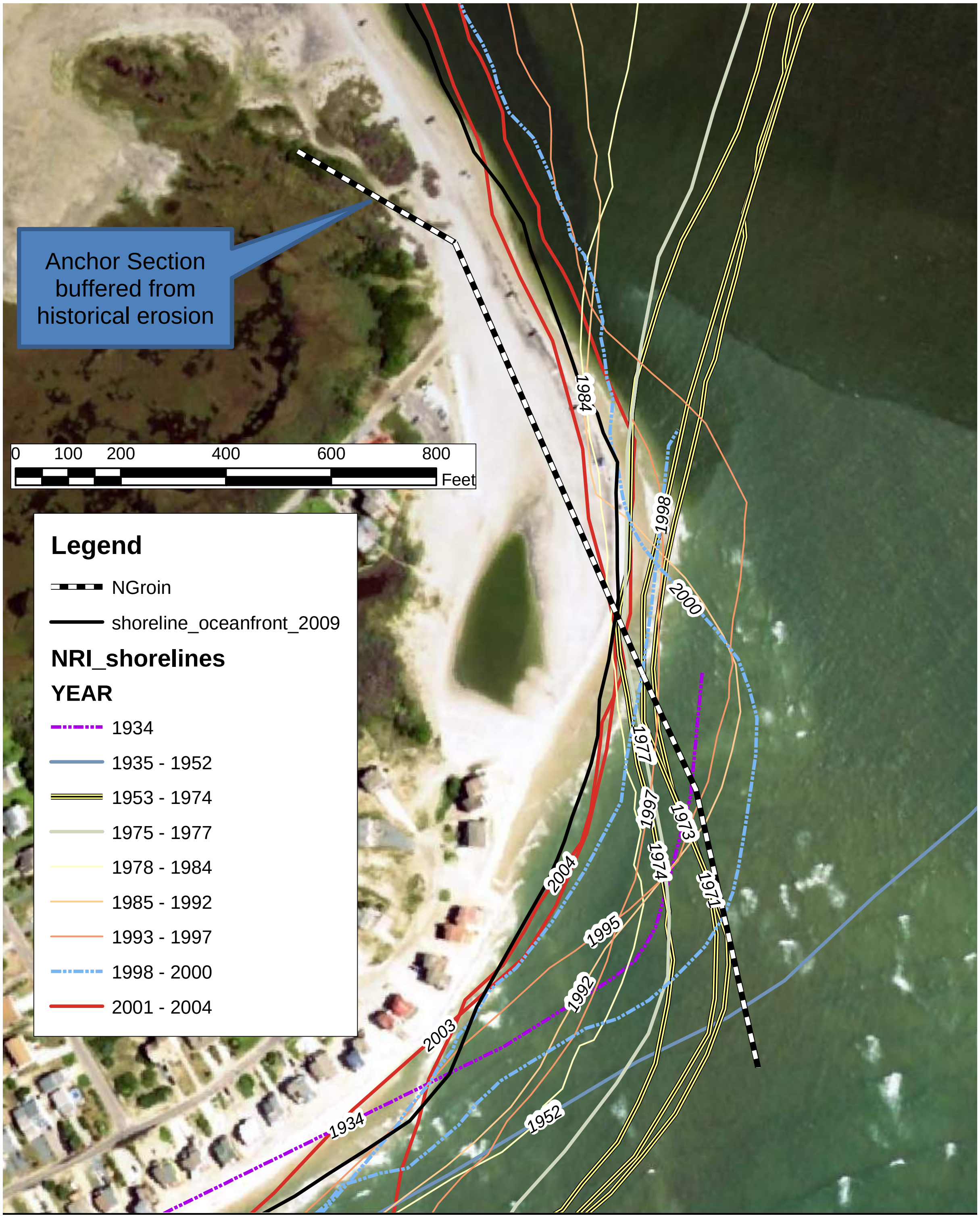


Figure 8-2. NGroin Layout Relative to Historical Shorelines.

8.1.2 MATERIALS AND DURABILITY

Terminal groin structures are typically composed of rock (i.e., rubble mound), sheetpile (steel or aluminum), concrete pre-fabricated units, or some combination of these materials. A rubble mound structure is the preferred material for the in-water sections due to durability and permeability considerations. Durability is affected primarily by stone size and placement-slope of groin. The stone size is preliminarily set at 4 to 6 ft in diameter. This is in line with or slightly larger than existing structures in the Onslow Bay region. More complete analyses will determine final stone gradation, but the current assumptions indicate that this size range is valid. It is anticipated that granite rock (as opposed to limestone, etc.) will be utilized.

The design incorporates the use of geo-textile mattresses (or similar) as a bedding layer (Figure 8-3). The primary function of the mattresses is to provide a base for the rock and prevent settlement. These mattresses can also aid in structure removal, if deemed necessary in the future.

Figure 8-3. Groin Construction Showing Mattress Placement

In terms of design life, if groins are not maintained, they will eventually fail, and the design assumes this will begin to occur in 25 years. However, if the structure is routinely inspected and repaired as necessary, the structures should last more than 25 years. As an example, the original Fort Macon terminal groin structure was built in the 1840s. Over the decades, occasional restacking of stones and some modifications have occurred to the Fort Macon groin and it remains effective today. An additional study from Delaware found that the combined effects of the groins and beach fill essentially stabilized the shoreline for nearly 50 years with minimal groin maintenance (Galgano, 2004).

8.1.3 PROFILE

Groin profile is a key element in effectively trapping sand. The groin profile refers to its cross-shore slope and how well it mimics the natural shoreline slope from the dune out to the surf zone. There is also a navigation element to this groin, where a more consistent slope is preferable so that it is visible and/or marked even during highwater events. Figure 8-4 presents a recent terminal groin structure with similar profile to the one preferred for the NTB terminal groin.

Figure 8-4. Terminal Groin on Hilton Head, SC during construction. Groin was constructed in 2012 (source: Olsen Associates)

All the groin alternatives in this report have been developed as relatively low-profile structures along the beach and nearshore for both sand bypassing and recreational reasons.

The landward section of the groin will be constructed to allow for sand cover and facilitating foot and vehicle traffic along the inlet beach. This elevation will limit sand trapping and allow some sand over-passing, even at the end of a nourishment cycle (i.e., eroded conditions). Figure 8-5 presents the cross-shore profile of the preferred NGroin. The final design may change the groin profile and/or crest width.

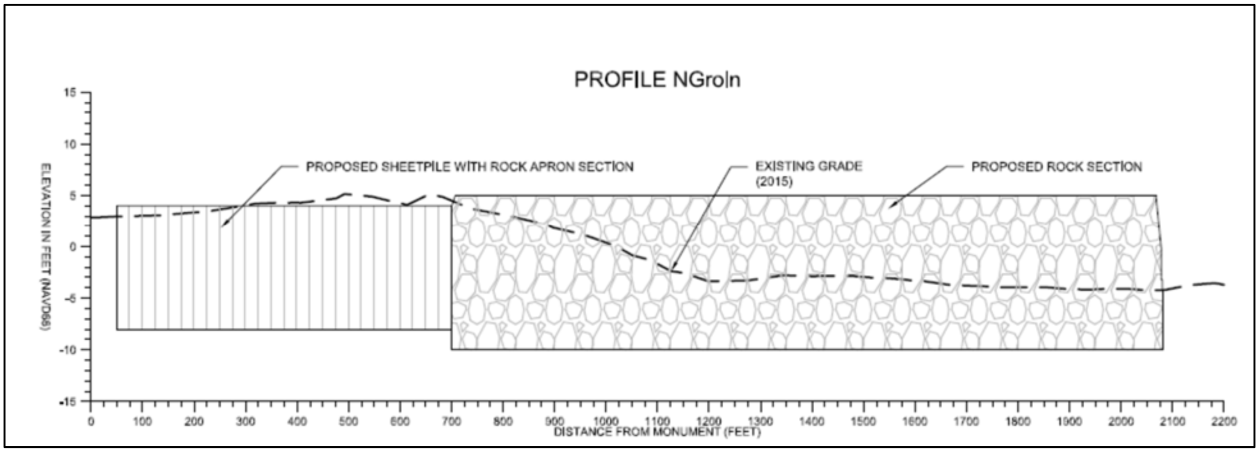


Figure 8-5. Groin Cross-Shore Profile in relation to 2015 Bathymetric Profile (Vertical/horizontal scale is exaggerated).

8.1.4 RIP CURRENTS

Rip currents are often cited as a detrimental side effect to groin construction. Along all coastlines, nearshore circulation cells may develop when waves break strongly in some locations and weakly in others. These weaker and stronger wave-breaking patterns are most often seen on beaches with a sand bar and channel system in the nearshore zone. They have also been noted at groins. Figure 8-7 shows the rip current effect between sandbars and at a groin. Rip currents are strongest under heavy wave conditions.

A Florida study of rip currents by Engle et al. (2002) determined that the frequency of rip current rescues increased during the following conditions:

1. Shore-normal wave incidence,
2. Mid-low tidal stages,
3. Deep water wave heights of 0.5 to 1.0 m, and

4. Wave periods from 8 to 10 seconds.

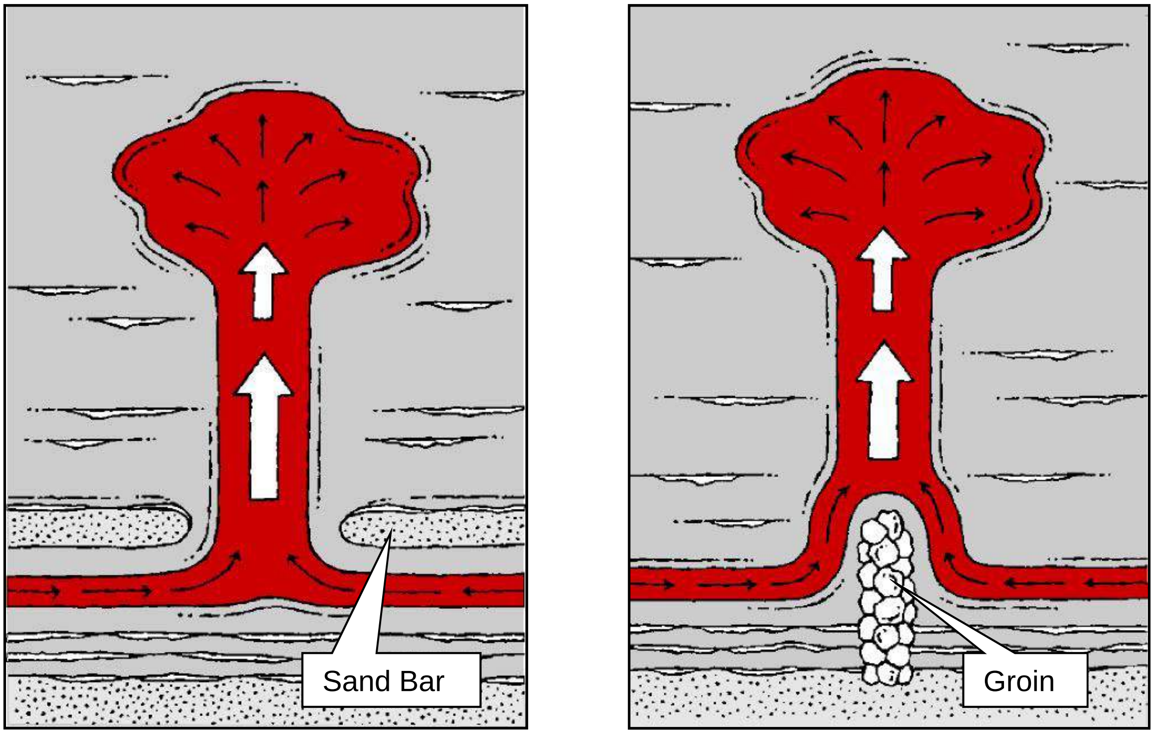


Figure 8-7. Rip Current Schematic between Sand Bars (left) and Groin (right) (from www.ocean.udel.edu)

As with all beaches in North Carolina, rip currents can form and pose problems to swimmers, but these are not due to groins. In addition to the wave conditions cited for creating rip currents, NRI ebb tide outflow (Figure 8-8) and the expansive sandbar/shoal system associated with inlets can also pose an additional risk. The proposed groin is designed to minimize rip currents; however, the NRI currents (greater than 5 ft/sec) will still be a hazard to swimmers, regardless of whether a terminal groin is constructed.

In a groin notching field study in New Jersey, Rankin et al. (2003) found that their study groin did not appear to exert an influence on the cross-shore flows (i.e., rip currents).

Figure 8-8. Existing Potential for Rip Current Effects at New River Inlet (8/2005 photo) Source: USACE Wilmington District.

8.1.5 CONSTRUCTABILITY

The majority of the preferred groin can be constructed from the shore/upland, which is the most cost-effective method. Construction access and staging areas for materials are also available via the public access parking lot. Additionally, road and bridge access to and from this site can handle relatively large payload trucks. A barge or trestle system may be required for the seaward groin construction.

8.1.6 ADJUSTMENT/REMOVABILITY

The ability to adjust or remove the groin at a future date is a design consideration because of the regulatory stipulation that requires groin modification or removal if adverse downdrift impacts occur. Adjustments to the structure include increasing or decreasing crest width, notching, adding a weir, or grouting to make it less leaky (if future needs dictate). In terms of removal, this design incorporates the use of mattresses as a bedding layer. Some subsidence or covering by sand can be expected, but the mattresses can be uncovered by common construction methods (e.g., excavation, jetting). The rock should be readily available for removal because it will lie on top of the mattresses. More information on groin mitigation is provided in Section 8.4.

8.2 SEA LEVEL RISE

Long-term sea-level rise (SLR) can have potential impacts along the coastline. While there is much debate about the magnitude and acceleration of future SLR, the USACE (2011) suggests an analysis that includes predictions in SLR for projects related to water resources. Table 8-1 shows the SLR for the project location under scenarios of low, intermediate, and high conservatism (for 50 years of project life), based on an updated version of the recommended analysis (National Research Council, 1987; USACE 2011).

Table 8-1. Sea-Level Rise Predictions and Subsequent Beach Losses

	Sea-level rise		Shoreline Erosion, width (ft)	
Project Life	Scenario	SLR (ft)	SLR Shoreline Erosion (Bruun, 1988)	Existing Background Erosion
50 years	Low	0.34	11.9	250 (min.*)
	Intermediate	0.74	25.9	-
	High	2.01	70.3	350 (max.*)

Note: * min. uses 5 ft/yr erosion rate, max. uses 7 ft/yr

A possible cumulative effect of SLR related to beach nourishment is the accelerated loss of beach and subsequent alteration of nourishment scheduling and volumes. Using a typical beach slope of 1V:35H (vertical to horizontal ratio), the predicted SLR under all scenarios is converted to shoreline erosion in Table 8-1. Table 8-1 also compares losses of beach width resulting from SLR projections and existing background erosion rates. While short-term erosion rates have averaged over 100 ft/yr in the project area (following the 2013 nourishment), long-term shoreline erosion rates ranging from 5 ft/yr to 7 ft/yr are used in the analysis presented in Table 8-1.

As seen in Table 8-1, shoreline erosion due to SLR is significantly less than existing background erosion. Existing background erosion does factor in historical SLR by default. Effects of long-term SLR (such as loss of usable beach width) are minor when compared to existing background erosion.

Over the next 50 to 100 years, incremental changes to SLR may become more significant to beach management. There are two primary ways to deal with increased erosion: 1) nourish more frequently with the same volume or 2) place more volume with the same frequency. An additional option to address increased erosion and sea level rise is to modify or enlarge the terminal groin structure. Repairs and modifications have occurred to the Fort Macon terminal groin since initial construction in the 1840s.

8.3 GROIN FILL REQUIREMENTS

For modern coastal engineering practice that adopts a regional perspective, provision exists in the groin functional design process to allow a certain amount of sediment to bypass a groin or groin field (Kraus and Rankin, 2004). When a well-designed groin fills to capacity with sand, longshore transport resumes at about the same rate as before the groins were built, and a stable beach is maintained.

The sand fillet volume of the proposed groin was calculated based on an area of sand accreting along the shoreline west of the proposed terminal groin. Basically, the terminal groin will reorient the shoreline and a minimum volume of sand is needed for this initial reorientation. Nourishment volumes can be computed by determining the cross-sectional area differences between the anticipated shoreline reorientation and the latest surveyed beach profile, and then multiplying by the alongshore reach length. To arrive at a volume, total minimum beach nourishment equates to the minimum cy/ft multiplied by the alongshore reach length divided by 2 (for a triangular fillet).

In this way, a nourishment volume can be established for an individual groin. Fillet volume will change based on the latest shoreline position, with more volume needed for a more eroded condition.

Recent project area beach fills have placed unit volumes from about 40 cy/ft to 100 cy/ft. Fill templates for recent projects typically feature an upper beach berm with crest elevation of +6 NAVD. Table 8-2 presents the proposed beach fill characteristics. Minimum groin fill requirements based on 2015 and 2016 survey data and the NGroin alternative are approximately 75,000 to 100,000 cy. This volume assumes a fillet 2,000-ft alongshore. The proposed fill template is about 300,000 cy; therefore, significantly more volume is proposed to be placed than required. This additional fill will ensure immediate downdrift bypassing of sediment.

Table 8-2. Beach Fill Design Characteristics

Nourishment Feature	Dimension	
Dune Height	9	ft NAVD
Dune Width	50	ft
Dune/Berm Slope	5	
Berm Height	6	ft NAVD
Berm Width	varies	ft
Berm/Toe Slope	15	
Unit Fill Volume Range	20 - 100	cy/ft

In terms of sand bypassing sediment characteristics, Aminti et al. (2003) found that the sedimentological impact (mean grain size, percent fines, sorting) of a submerged groin on a beach is negligible (i.e., there was no significant difference between updrift and downdrift sand samples).

Groins can also have a beneficial effect on dune growth. A Westhampton, NY groin field study found that the largest rate of dune growth west (downdrift) of the groin field from initial construction in 1996 to February 2009 was approximately 2.0 cy/ft-yr, whereas the beachwide average rate of growth was 1.25 cy/ft-yr (Bocamazo et al., 2011). Dune growth via Aeolian transport[1] due to the groin field has added to the stability of the beach-dune cross-section, contributed habitat to some creatures, and most significantly, has increased the width of the dunes for additional storm protection (Bocamazo et al., 2011). Predominant south/southwest winds during spring and summer at NTB can promote dune growth (through Aeolian transport) along the proposed terminal groin.

8.4 GROIN MITIGATION

It is acknowledged that some groin projects (in most cases, without concurrent beach nourishment components) have been cited as adversely impacting downdrift shorelines. The Town has developed a beach nourishment and groin project to minimize downdrift impacts. A 2004 paper by Galgano found that “in many circumstances, groins have functioned effectively and stabilized an eroding beach without seriously harming adjacent areas….the groins, in conjunction with beach fill, arrested beach erosion at the site and effectively stabilized the beach for nearly 50 years notwithstanding their structural deficiencies.”

[1] Aeolian transport refers to the movement of sediment by wind

Pawleys Island, SC (in southern Long Bay) has 23 groins that were sand tightened and nourished in 1999. The downdrift neighbor, northern Debidue Island, has remained accretional or stable since this time (Kana et al., 2004). Kana found that *Pawleys Island groins indicate that groins can stabilize an entire littoral cell without adversely impacting the adjacent cell (northern Debidue Beach).*

Another example of a successful groin project is provided by the NOAA Coastal Services Center (CSC) regarding the Folly beach groins:

> The beach compartments between groins can be filled with beach quality sand to prevent the longshore material from being blocked until the groin field is filled by natural processes, as was the case, for example, in Folly Beach, South Carolina (Ebersole, Nielans, and Dowd 1996). There, the groins extended along about one-half mile of the nearly five miles of nourishment. The area where they were installed was more rapidly eroding than the adjacent beaches. After the nourishment, it was apparent that this "hot spot" had been largely controlled by the presence of the groins added at the time of the beach fill. (www.csc.noaa.gov/beachnourishment/html/geo/shorelin.htm)

Dr. Orrin Pilkey has also co-authored a paper stating that groins can increase beach nourishment longevity (Leonard, Dixon and Pilkey, 1990):

> On the Atlantic coast, groins appear to increase the longevity of replenished beaches. Examples of this include Edisto Beach, SC, where groins have been used in conjunction with replenishment, and Virginia Key, FL, where groins were added in 1977. In both cases, the presence of the groins is believed to have increased the stability of the emplaced fill, so that some of the fill was apparently still in place more than five years after emplacement.
>
> Similarly, the Pacific coast has repeatedly experienced general success at least partly attributable to the presence of groins. Capitola, Cabullo Beach, Redondo Beach, and Newport Beach are examples of beaches where a terminal groin has assisted in stabilizing the beach.

Many other studies or publications have supported the use of groins in conjunction with beach nourishment. As an example, the Select Committee on Beach Nourishment and Protection issued its final report in January 1995 (NRC, 1995). The Committee was under the auspices of the Marine Board of the National Research Council (NRC) and asked to conduct a multidisciplinary assessment of the engineering, environmental, economic and public policy aspects of beach nourishment. Committee members were:

Orrin H. Pilkey, Duke University, Durham, North Carolina

Richard J. Seymour (chair) Texas A&M University and Scripps Institution of Oceanography

Nancy E. Bockstael, University of Maryland, College Park

Thomas J. Campbell, Coastal Planning and Engineering, Inc., Boca Raton, Florida

Robert G. Dean, NAE. University of Florida, Gainesville

Paul D. Komar, Oregon State University, Corvallis

Anthony. P. Pratt, Delaware State, Dept. of Natural Resources

Martin P. Snow, Great Lakes Dredge & Dock Co., Chicago, IL

Robert F. van Dolah, South Carolina, Dept of Natural Resources

J. Richard Weggel, Drexel University, Philadelphia, PA

Robert L. Wiegel, NAE, University of California, Berkeley

The following recommendations were made by this committee as applicable to the proposed project.

> RECOMMENDATION: Agencies should modify their prescriptive laws, regulations, and management plans for the coast to allow the use of fixed structures in conjunction with beach nourishment projects where project performance can be significantly improved, out-of-project negative effects are acceptably small or are mitigated as necessary, and beach access or use is not impaired. The costs of the structures should not exceed the savings achieved by increasing the level of protection or the times between successive renourishments. Environmental impacts should also be considered. (p. 143-144)
>
> and
>
> RECOMMENDATION: Each fixed structure that is used in conjunction with a beach nourishment project should be filled to the upper limit of its holding capacity if it would

> otherwise accumulate sand. Where uncertainties exist, fill should exceed the calculated upper limit of the holding capacity of the structure. If a beach, nourishment project is not maintained, adverse effects of any structure should be mitigated or the structure should be removed. (p. 144)

The groin and nourishment project is designed to continue allowing nourishment sand to benefit downdrift shorelines when compared to the naturally occurring background erosion. Therefore, negligible impacts are anticipated due to downdrift erosion. Downdrift monitoring will be conducted to document impacts. If negative impacts due to the presence of the groin are documented, mitigation, including additional sand placement, groin modification, and/or groin removal, may occur.

There have been several cases of successful groin notching modifications, including northern New Jersey (Donahue et al., 2003) and Tybee Island, Georgia (USACE, 1997) (Figure 8-10). In a 1997 Tybee Beach groin tuning paper by the USACE, the estimated groin modification cost for removing (i.e., notching) six modules and placing these modules adjacent to a nearby seawall was a total of $5,800 for use of a small crane and labor. The groin modules are 14-ton concrete structures 8 ft long, 5 ft high, and 10 ft wide. Construction materials for the proposed project will differ and construction costs have increased, however, $5,800 for removing six groin modules to lower the groin profile (i.e., modify it) provides an example of the relatively inexpensive costs of groin modification/removal.

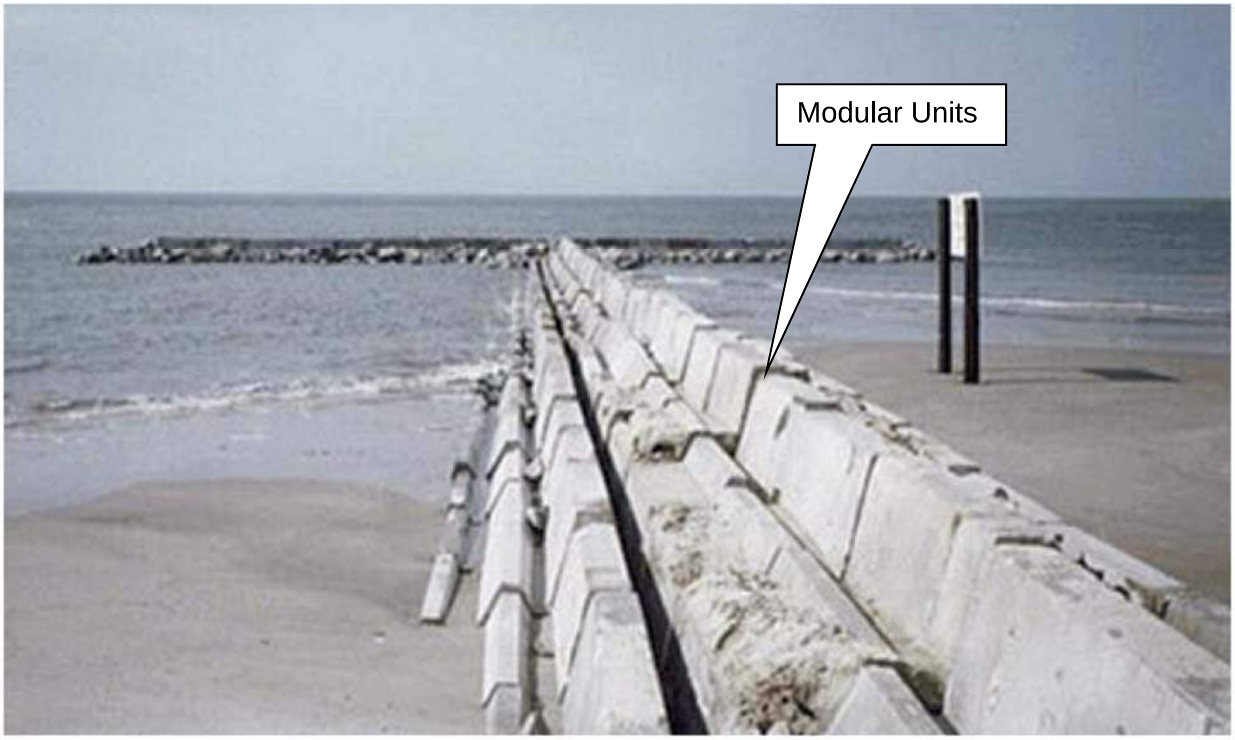


Figure 8-10. Tybee Island, Georgia Terminal Groin Structure that was Successfully Modified by Removing Six Modular Units on the Seaward End (source: USACE, 1997). Also note T-Head feature.

9.0 BENEFITS AND COSTS

Consideration of benefits and costs are critical when evaluating beach management alternatives. The key to a well-designed groin structure is ensuring that it will increase the nourishment interval while minimizing downdrift impacts once constructed. While increasing the nourishment interval represents the most significant construction-related cost savings for the proposed project, other benefits and cost savings are also anticipated (including removal of the sandbag revetment shown in Figure 9-1).

Figure 9-1. Sandbags (shown in black) Continuous along ~3,300 feet from ~Station 1163+00 to ~1135+00. Another ~300 ft sandbag section is located just south of Station 1170+00.

Approximately 3,600 feet of shoreline is currently armored by a large sandbag revetment. This revetment protects 35 imminently threatened structures (including the 8 Topsail Reef condominium units) and at least 10 duplexes (more than 200 residential units total).

A general overview of benefits and costs associated with maintenance of the project area shorelines (e.g., nourishment, terminal groin, no-action) as well as benefits and costs associated with maintenance of NRI (e.g., pivot channel, side-caster dredging) are summarized in this section.

The preferred alternative includes several components:

- Terminal groin
- Beach nourishment
- Pivot Channel borrow area

In addition to these components, benefits and costs associated with other alternatives are discussed and include:

- Retreat
- Beach nourishment only (including no-action)

In general, major expenditure items (i.e., hard costs) such as dredge mobilization/ demobilization, beach nourishment, and structure relocation are identified, whereas additional costs such as permitting, design and surveying (i.e., soft costs) are included when quantifiable. In other cases, assumptions are made (for example, permitting, design and surveying typically represent about 10 percent of the total construction costs).

9.1 GROIN CONSTRUCTION COSTS

Groin costs primarily include equipment and materials mobilization/demobilization, materials, and construction. Permitting, design, monitoring/surveying, nourishment, and mitigation costs are related cost items.

Mobilization/demobilization (mob/demob) for groin construction is estimated at $300,000. Mob/demob for groin construction typically requires several truckloads of materials. As a relatively recent example of mob/demob costs, the City of Folly Beach, SC had mob/demob costs for the complete rehabilitation for nine rock groins (and hauling in 72 percent new stone) that averaged $208,000 (City of Folly Beach, 2018). In terms of materials, armor stone, bedding stone and marine mattresses (Figure 9-2) will be used. Armor stone tonnage calculations are typically based on a 25 percent void ratio assumption.

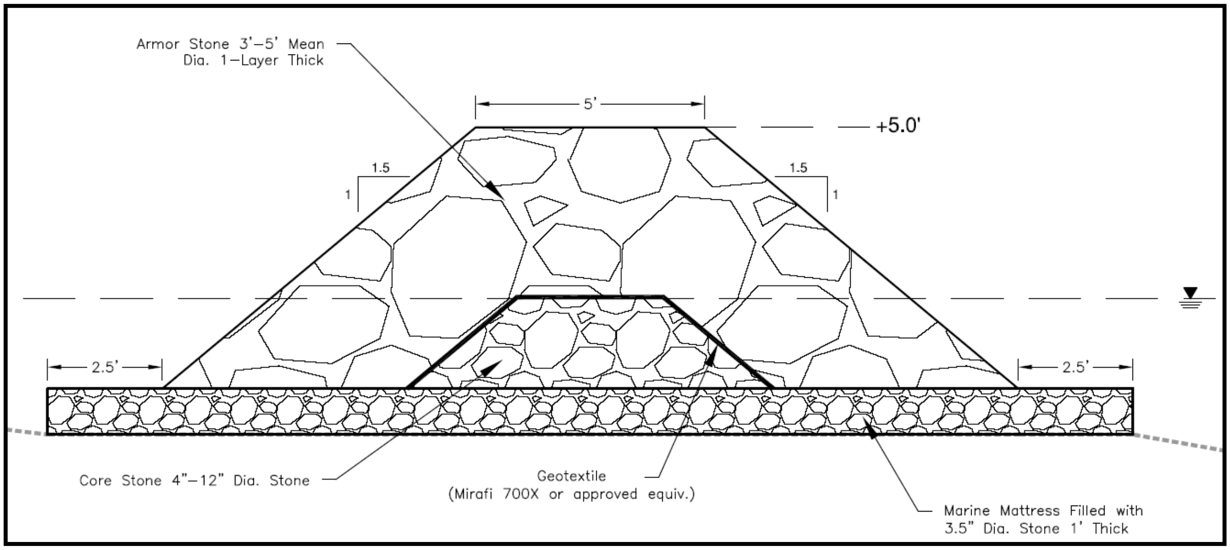


Figure 9-2. Typical Groin Cross-Section (source: USACE CEM, 2003)

Groin construction for several recent projects in South Carolina (Hunting Island, Daufuskie Island) have realized costs ranging between $1,000 and $1,500 per foot of groin length (Bloody Point POA, 2010; SC Dept of Parks, Recreation, and Tourism, 2007). These structures were built from land, which typically results in significant savings, versus water-based construction from barges or temporary trestles.

A 2012 terminal groin construction project occurred on Hilton Head Island where an approximately 1,000-foot rubble-mound terminal groin with T-head was installed for $1.67 million (Olsen Associates, 2012). The next two most competitive bids for the groin project were $2.55 million and $2.58 million (Olsen Associates, 2012). The total project cost included site preparation, sand excavation and backfilling, offsite assembly, transport, delivery and placement of approximately 190 stone-filled marine mattresses, installation of geogrid/fabric composite underlayment, and placement of approximately 12,000 tons of granite (or equivalent) armor stone. Additional work also included establishment of access and staging area, site restoration, demobilization, safety and security measures, permit compliance, final grading, and surveying (Town of Hilton Head, 2012). Cost per linear foot for the awarded bid was approximately $1,670.

At Bald Head Island, a 1,300-foot-long groin was recently constructed in 2015 with a low bid of $4.84 million ($3,723 per linear foot). Bald Head Island is only accessible by boat/ferry, therefore, some additional mob/demob can be expected. The Bald Head Island terminal groin also featured a seaward end in relatively deep water that necessitated a temporary trestle system (which the contractor may elect to do for the proposed terminal groin).

Figure 9-3. Bald Head Island Construction Trestle for Seaward Groin Construction (image source, Olsen Associates NCBIWA presentation, 2015).

In addition to the construction of a trestle, the use of standard barges or in some cases, jack-up barges, can be used in the nearshore area. Jack-up barges are more stable than regular barges in the surfzone. The use of trestles and jack-up barges increases groin construction costs.

The North Carolina Terminal Groin study (Moffatt & Nichol, 2010) proposed cost estimates for rubble-mound structures (i.e., $1,230/lf for a 450 ft groin on a mild sloping beach) similar to the recent South Carolina groin construction projects. To expedite groin construction, the beach nourishment component is often constructed immediately prior to groin construction. This allows for more work area that is unaffected by tides and waves.

The preferred NGroin is approximately 2,012 feet in length and features anchor, upland/beach and water elements (see Table 9-1). The water and beach elements will be the most expensive and were costed similar to the BHI terminal groin. The anchor section will be a more economical component and the anchor section can likely occur at a later time (based on shoreline monitoring). While the BHI terminal groin was constructed entirely of rock, sheetpile is recommended for the portions of the upper and anchor sections of the preferred NGroin.

Table 9-1. NGroin Section Lengths

Anchor	Upland	Water	Total
345	894	782	2,021

The proposed NGroin structure is estimated to cost $6,000,000, however, delaying the anchor section construction could allow construction costs to occur over several years. This would require an additional mobilization, however, it would be relatively minimal (about $50,000 to $100,000) for a typical heavy civil construction firm.

9.2 GROIN REMOVAL

Groin removal typically requires much less time and effort than groin construction (as with most construction vs. demolition projects). The North Carolina Terminal Groin study (Moffatt & Nichol, 2010) estimated that for rock or concrete armor groins, the cost of removal is approximately $500 - $1,500 per linear ft. The Hilton Head terminal groin had a “financially binding commitment” removal estimate of $300,000 (about $300/ft) (Creed, personal communication, 2010. Other recently permitted groin projects in SC, such as Hunting Island (five groins) and DeBordieu (three groins) required a $200,000 letter of financially binding commitment.

The BHI terminal groin removal costs was $2,000,000 and it is estimated that the total cost for the North Topsail Island Terminal Groin removal is proposed to be the same amount. Note that complete groin removal is a last resort and that nourishment and/or groin modification would represent initial mitigative steps.

9.3 BEACH NOURISHMENT

For the nourishment-only alternative, it is believed that a 2-year nourishment interval is required for the project area. Therefore, it is proposed that instead of using the ebb channel borrow area every 4 years with ~600,000 cy dredged, the ebb channel borrow area should be used every 2 years with ~300,000 cy dredged. Similar to many other NC inlets, the project area shoreline cannot be sustained by nourishment-only on a 4-year interval. Therefore the 2-yr dredging interval is provided in this analysis. This alternative would not allow for additional sections of beach to be nourished since it uses all ebb channel dredged material (in comparison to the groin and nourishment project).

Nourishment costs include a number of items; although dredge mob/demob and active pumping constitute the primary costs. Whether beach nourishment is considered independently or as a component of the terminal groin project, the preferred borrow area is the NRI channel. Nourishment costs are estimated at $12.50/cy (based on historical and recent projects of similar size and borrow area location).

The borrow area proximity to the nourishment area should allow for relatively competitive pricing however the project will require ocean certified dredges due to the COLREGs line location. Dredge mobilization for the proposed NRI project is estimated at $2,000,000 for the purposes of this document and is based on recent ocean-going cutterhead dredge projects.

In the future, increasing beach nourishment construction costs can be expected due to the following factors:

- Increased marine diesel fuel prices
- Increased environmental constraints (environmental windows, access restrictions/buffer, monitoring and mitigation related costs)
- Increased market demand and a limited US dredger fleet

Increased fuel costs are directly related to dredge mob/demob fees, which represent a significant portion of overall project costs. A relatively recent example is the $4 million mob/demob fee for the 2013 Carteret County nourishment project. The Town of Hilton Head has also summarized dredge mob/demob fees in relation to increasing fuel costs over the last two decades (Figure 9-4). It is noted that the inflation rate exhibited for Figure 9-4 is approximately 6%, which is relatively high. Dredging in the US can only be performed by US dredging companies due to the Jones Act. Unfortunately this means that the largest dredge companies (which are headquartered in Europe) cannot compete for dredging work at the project site. The US dredging fleet has expanded recently, with Great Lakes and Weeks adding new dredges, however there have been many instances over the last decade where dredger-demand greatly outstrips dredger supply (basically following any major hurricane event).

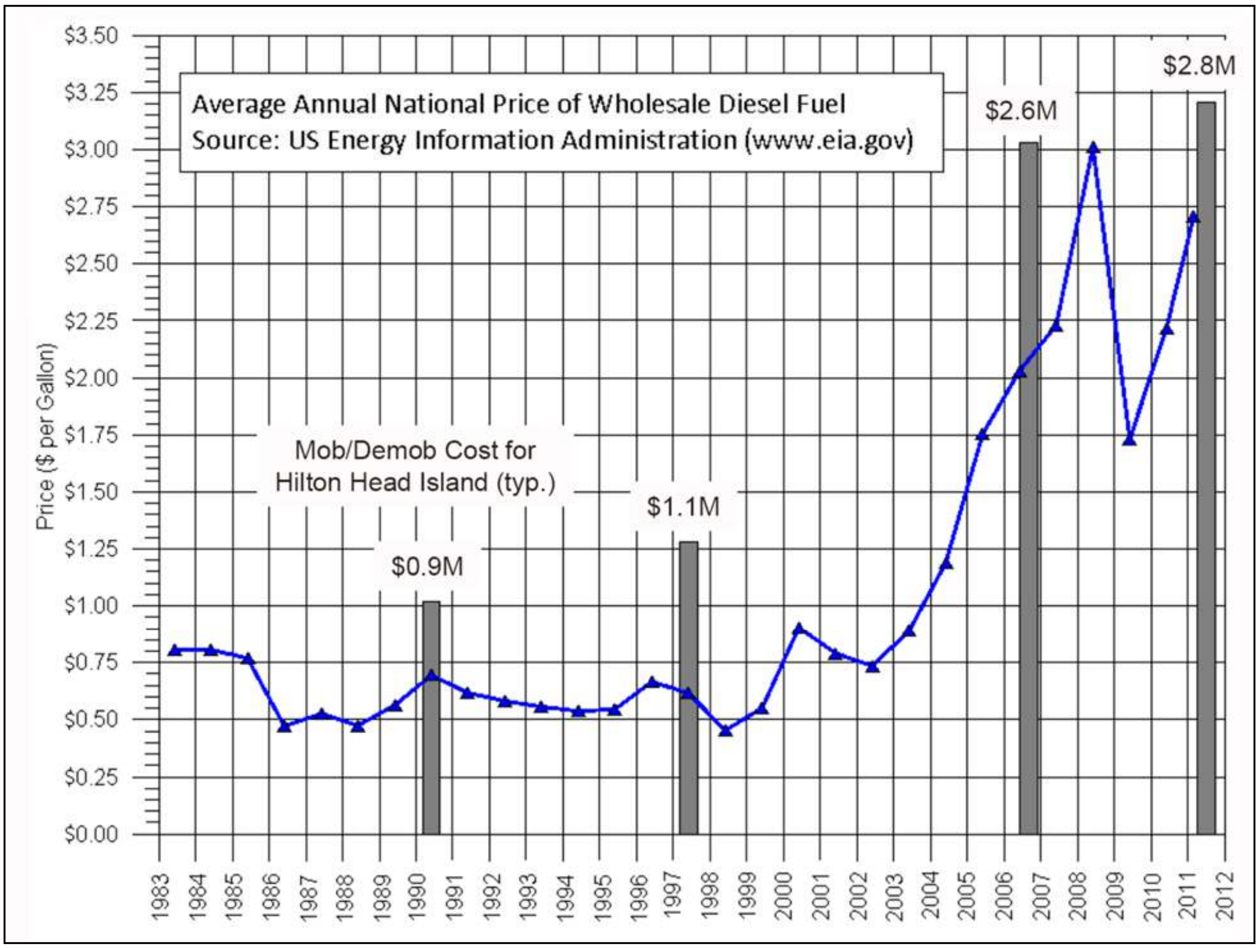


Figure 9-4. Average Annual National Price of Wholesale Diesel Fuel in Comparison to Hilton Head Nourishment Mob/Demob Costs (source: Olsen Associates, 2012).

9.4 NO-ACTION

The no-action plan refers to the continuation of current beach management practices along the northern end of NTB as well as continuation of the current navigation management practices within NRI. These measures include the NRI ebb channel, Cedar Bush Cut and New River/AIWW nourishment projects as well as the deployment of the sandbag revetment.

While the dredging and nourishment activities have been overall beneficial, they have not been able to prevent the loss of homes on the northeast end which necessitated a sandbag revetment for the dozens of imminently threatened homes in the project area.

The NC Terminal Groin Report (Moffatt & Nichol, 2010) developed two different economic categories for a general assessment of terminal groin feasibility:

1. 30-Year Risk Area (YRA)
2. Imminent Risk Property (IRP)

The 30-YRAs were defined by lines on aerial photography maps provided by the DCM. The maps are based on aerial photos from 2003 to 2009. Any land existing seaward of the lines is assumed to be at risk in the next 30 years. IRP infrastructure is located immediately adjacent to erosion control sandbags locations or between two nearby sandbag locations (Moffatt & Nichol, 2010). These lines were agreed upon by the Science Panel for use in the NC Terminal Groin Report assessment (refer to Moffatt & Nichol, 2010 for more information). Figure 9-5 presents the NRI figure from the terminal groin report.

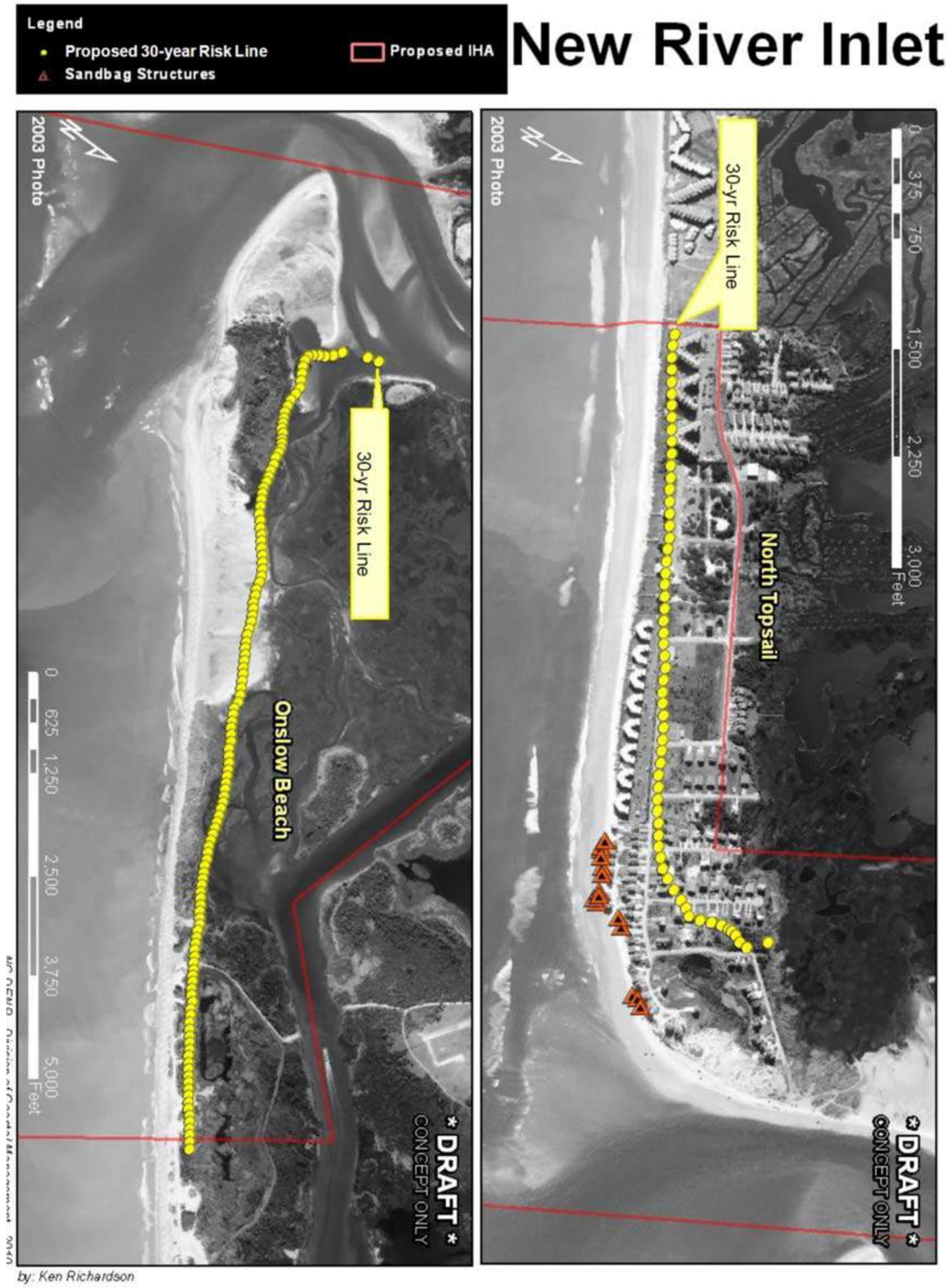


Figure 9-5: New River Inlet proposed 30-year risk line (source: M&N, 2010).

The Terminal Groin Study included the following economic values in determining IRP and 30 YRA costs:

- Residential property
- Commercial property
- Government property
- Road infrastructure
- Waterline infrastructure
- Sewer infrastructure
- Property tax base and revenues
- Recreation and environmental value

IRP and 30-YRA values for structures adjacent to NRI are presented in Table 9-2. As shown, almost $3 million in economic value was considered as imminent risk property when this report was published. More property is currently in imminent risk.

Table 9-2. Estimated Structure Costs adjacent to New River Inlet (source: M&N, 2010)

	30-yr Risk Area (YRA)		Imminent Risk Properties (IRP)	
Inlet Hazard Area	West of Inlet (Onslow Beach)	East of Inlet (North Topsail Beach)	West of Inlet (Onslow Beach)	East of Inlet (North Topsail Beach)
New River Inlet	None (undeveloped military land)	$70,281,548	None (undeveloped military land)	$2,914,211

Tables 9-3 and 9-4 are excerpted from the State Terminal Groin Report and itemizes 30-year and IRP values for NRI. These values were estimated in 2009.

Table 9-3. Economic Value at Risk within 30-yr Risk Lines at New River Inlet (source: Moffatt & Nichol Terminal Groin Report, 2010)

Value Type	**North Side of Inlet (Onslow Beach side)**	**South Side of Inlet (North Topsail Beach side)**
Residential Property Value		
Number of Parcels	None (undev. military land)	136 residential single fam. 240 condo units
Land Value	-----	$24,773,765
Structure Value	-----	$41,666,597
Other Value	-----	$377,331
Total Value	-----	$66,817,693
Commercial Property Value		
Number of Parcels	None (undev. military land)	None known.
Land Value	-----	-----
Structure Value	-----	-----
Other Value	-----	-----
Total Value	-----	-----
Government Property Value		
Number of Parcels	None (undev. military land)	None known.
Land Value	-----	-----
Structure Value	-----	-----
Other Value	-----	-----
Total Value	-----	-----
Road Infrastructure Value		
Type	None (undev. military land)	2-lane road w. 2' paved shoulders (no curb, gutter, parking or sidewalk)
Length (ft)	-----	4480
Replacement Cost / ft	-----	$568
Total Value	-----	$2,545,455
Waterline Infrastructure Value		
Type	None (undev. military land)	Typical
Length (ft)	-----	4480
Replacement Cost / ft	-----	$55
Total Value	-----	$246,400
Sewer Infrastructure Value		
Type	None (undev. military land)	Typical
Length (ft)	-----	4480
Replacement Cost / ft	-----	$150
Total Value	-----	$672,000
GRAND TOTAL VALUE	**None (undev. military land)**	**$70,281,548**

Property Tax Base within 30-yr Risk Lines	None (undev. military land)	$66,817,693
Property Tax Revenue within 30-yr Risk Lines	None (undev. military land)	$443,001 annually
Municipal Property Tax Base	Tax Exempt (military)	$1.50 billion (North Topsail Beach)
County Property Tax Base	$9.7 billion (entire Onslow County)	

Table 9-4: Economic Value at Imminent Risk at New River Inlet
(source: Moffatt & Nichol Terminal Groin Report, 2010)

Value Type	North Side of Inlet (Onslow Beach side)	South Side of Inlet (North Topsail Beach side)
Residential Property Value		
Number of Parcels	None (undev. military land)	37 parcels (of which 15 have structures)
Land Value	-----	$1,328,850
Structure Value	-----	$1,556,901
Other Value	-----	$28,460
Total Value	-----	$2,914,211
Commercial Property Value		
Number of Parcels	None (undev. military land)	None.
Land Value	-----	-----
Structure Value	-----	-----
Other Value	-----	-----
Total Value	-----	-----
Government Property Value		
Number of Parcels	None (undev. military land)	None.
Land Value	-----	-----
Structure Value	-----	-----
Other Value	-----	-----
Total Value	-----	-----
Road Infrastructure Value		
Type	None (undev. military land)	None.
Length (ft)	-----	-----
Replacement Cost / ft	-----	-----
Total Value	-----	-----
Waterline Infrastructure Value		
Type	None (undev. military land)	None.
Length (ft)	-----	-----
Replacement Cost / ft	-----	-----
Total Value	-----	-----
Sewer Infrastructure Value		
Type	None (undev. military land)	None.
Length (ft)	-----	-----
Replacement Cost / ft	-----	-----
Total Value	-----	-----
GRAND TOTAL VALUE	**None (undev. military land)**	**$2,914,211**

Property Tax Base of IRPs	Exempt	$2,914,211
Property Tax Revenue of IRPs	Exempt	$19,322 annually
Municipal Property Tax Base	Tax Exempt (military)	$1.50 billion (North Topsail Beach)
County Property Tax Base	$9.7 billion (entire Onslow County)	

As previously mentioned, the no-action alternative would rely on existing beach management programs where the sandbag revetment continues to grow in size and length. As a result, it is reasonable to assume that losses of homes will continue to occur. Therefore, the estimated losses between $2.9 million (IRP) and $70.2 million (30-YRA) are likely expected to occur over the next 30 years under this scenario.

The 2009 EIS also presented some relevant economic analysis including:

> *During the past year, 17 duplex structures located at the extreme north end of Town, which have a total tax value of over $17 million, have become imminently threatened.*
>
> *Two (2) of the imminently threatened duplexes were relocated to other parts of North Topsail Beach at the expense of the property owners. Six (6) of the remaining duplexes have been declared uninhabitable due to the loss of water, sewer, and electrical connections and were demolished in February 2009 at a cost to the Town of $2 million.*

Table 9-5 presents average annual economic impacts of the No-Action alternative from the 2009 EIS. The "North Section" includes Stations 950+00 to 1160+00 therefore includes the project study area.

Table 9-5: 2009 EIS average annual economic impacts of the No-Action Alternative.

Economic Impact	No Action Alternative		
	Central Section	North Section	Total
Damages & Losses			
Erosion & Storm Damages	$5,738,200	$17,688,400	$23,426,600
Rental Income Loss	$529,500	$3,709,800	$4,239,300
Reduction in Household Spending	$207,000	$5,437,600	$5,644,600
Total Damages & Losses	$6,474,700	$26,835,800	$33,310,500
Reduction in Tax Revenues			
Town Ad Valorem	$31,700	$115,500	$147,200
County Ad Valorem	$46,900	$172,000	$218,900
Sales Tax (Local & State)	$14,600	$380,600	$395,200
Accommodation Tax	$31,800	$222,800	$254,600
Total All Tax Revenues	$125,000	$890,900	$1,015,900

In addition to residential homes, principal elements of the Town's infrastructure include the streets, utilities, and public access parking areas that the Town owns and maintains. FEMA cannot help in mitigating the majority of damages that occur during hurricanes and major storm events due to COBRA designations. Of course the Town also must fund any repairs due to northeasters or other erosional events.

9.4.1 ECONOMIC LOSSES RELATED TO BEACH WIDTH

The NC BIMP conducted a study of losses attributed to 50 percent beach width loss and found that for NTB, the 2013-2014 estimated annual loss (including output/sales/ business activity) was $6.0 million. The losses calculated in the NC BIMP for NTB are provided in Table 9-6.

Table 9-6. Estimated Annual Losses based on 50 Percent Beach Width Reduction (source: 2016 NC BIMP)

	2013-2014 50% Beach Width Reduction			
Area	Loss in Annual Output/Sales/Business Activity (Total Impact)	Loss in Employment (Jobs)	Loss in Beachgoer Consumer Surplus	Loss in Shore/Bank Fishing Consumer Surplus
North Topsail Beach	$5,993,973	77	$396,528	$53,132

Assuming the proposed terminal groin will conservatively enhance approximately 4,500 feet of shoreline (Inlet shoulder down to 1125+00 nodal area), that the NTB shoreline is 11 miles long, and the estimated losses along the entire beach provided in Table 9-6 are uniform, losses of approximately $464,000 annually can be attributed to narrower beach conditions on the east end. Losses are likely larger due to the sandbag revetment and essentially no high-tide beach currently available.

In general, the no-action alternative has significant costs and economic consequences (direct and indirect) associated with it. Many communities, including NTB, have adopted this alternative in the past and do not consider it a viable/practicable alternative in the long-term. The erosion rates on the north end are too high for the current beach management practices to work effectively and economically.

9.4.2 ECONOMIC LOSSES RELATED TO INLET SHOALING

The NC BIMP (2016) also conducted an analysis based on the economic impacts of reduced dredging and increased shoaling in 6 shallow draft inlets along the NC coast. The NC BIMP didn't include NRI but did include several nearby inlets (New Topsail, Bogue, Carolina Beach). The shoaling scenarios assumed navigable channel depths of 4 feet for New Topsail Inlet. It should be noted that shoaling channel depths at NRI can commonly less than 4 feet with no marked channel. NC BIMP states: *If inlets shoal to four feet or less, it is assumed that oceangoing commercial fishing vessels may either (1) go out of business, (2) travel longer distances to other inlets before reaching the ocean (increasing fuel costs, decreasing ocean fishing time, and decreasing profits), or (3) change ports.*

Overall, annual commercial fishing losses from shoaling ranged between $200,000 and $1,700,000 for the inlets studied in the NC BIMP. Charter (For-Hire) vessels are also impacted by excessive shoaling and unsafe inlet navigation, where up to $2,500,00 of annual losses are anticipated. Recreational boating was also evaluated related to shoaled inlets and anticipated measurable impacts to local marinas and boat building. Based on Onslow County boater registration data from 2014, there are an estimated 3,215 recreational boaters that are potentially affected by shoaled inlet conditions.

9.5 ABANDON/RETREAT

The abandon/retreat alternative assumes that no erosion mitigation measures will occur. Therefore, no nourishment projects, no beach/sand scraping, and no sandbag deployment would occur. As a result, erosion would occur unabated and result in the loss of land, property and the many benefits associated with a healthy beach and dune system. Under current conditions, almost no dune and only sandbag revetments occur along the majority of the project shoreline. Figures 9-6 through 9-9 present photos of recent conditions along the project shoreline.

Figure 9-6: August 4, 2015 conditions (~Station 1160+00).

Figure 9-7: Sept 27, 2015 conditions along inlet shoreline at public parking lot (~Station 1170+00).

Figure 9-8: August 30, 2015 conditions (~Station 1150+00).

Figure 9-9: Roped off beach access along inlet shoreline (August 30, 2015)

The 2009 EIS performed a Buyout/Relocation alternative and found:

> *The Buy-Out/Relocation Alternative is similar to the No Action Alternative, except temporary sand bag revetments would not be used to assist in protecting threatened structures.*
>
> *Erosion and storm related damages are less for the Buy-Out Alternative than for the No-Action alternative primarily due to the assumed earlier demolition of the Topsail Reefs Condominiums and the Villa Capriani during the 30-year analysis period.*

Table 9-7 presents the buyout/relocation costs for the “Central” and “North” sections of shoreline (the “North” section includes the project study area).

Table 9-7: 2009 EIS average annual economic impacts of Buyout Alternative.

Economic Impact	Buy-Out Alternative		
	Central Section	North Section	Total
Damages & Losses			
Erosion & Storm Damages	$5,166,900	$13,501,100	$18,668,000
Rental Income Loss	$670,500	$6,144,100	$6,814,600
Reduction in Household Spending	$340,000	$8,961,500	$9,301,500
Total Damages & Losses	$6,177,400	$28,606,700	$34,784,100
Reduction in Tax Revenues			
Town Ad Valorem	$40,400	$150,200	$190,600
County Ad Valorem	$60,100	$223,900	$284,000
Sales Tax (Local & State)	$24,000	$627,300	$651,300
Accommodation Tax	$40,200	$368,500	$408,700
Total All Tax Revenues	$164,700	$1,369,900	$1,534,600

9.6 BENEFITS

Benefits are an important factor when evaluating beach management alternatives. The most basic benefit to the groin and nourishment alternative is that it will sustain a 4-year renourishment interval. In addition to sustainable 4-year nourishment intervals, the groin will provide damage reduction to the dune system and, subsequently, protect houses and property values. Additional benefits related to beach use include:

- Safer and more predictable inlet navigation
- Removal of sandbag revetment
- More years in between disruptions (pipelines and heavy equipment) on the beach
- More turtle nesting due to more stable dune and upper beach
- Available recreational beach at high tide (walking, fishing, etc.)

The beach and properties in the project area comprise a major economic and social resource for the Town of North Topsail Beach. Continued erosion (under no-action conditions) of the Phase 1 oceanfront will result in a reduced tax base due to the loss of homes as well as reduced tourism due to restricted beach access and recreation area.

An additional benefit to successful shoreline stabilization programs is reduced emergency costs (beach scraping, sandbagging, repairs to roads, beach access walkovers, light posts, etc.), damages to private property other than structures/contents, and post-storm recovery process can also be estimated at approximately $20,000/mile annually (USACE, 2012). The sandbag revetment construction cost was approximately $3,500,000 (occurring in phases by different groups) while annual maintenance costs can be estimated at $250,000 to $500,000.

9.6.1 RECREATION

Public access to the northeast end of NTB and NRI is a critical economic revenue source to the Town. Popular activities include, but are not limited to, surf fishing, swimming, surfing, walking, shell hunting, sunbathing, bird watching, and boating. The NC BIMP report estimated the 2013-2014 Beach Recreation Annual Total Impact Output for North Topsail Beach at $38.1 million, which accounted for 493 jobs. This extrapolates to approximately $2.9 million annually for 4,500 feet of shoreline on the north end, as well as about 38 jobs.

Currently, there are periods of significant loss of dry beach due to erosion, which limits many beach activities to low-tide periods. The proposed groin and nourishment project would make the beach more accessible during the year, particularly during times of high tide.

The terminal groin and dredged ebb channel will also aid in maintaining a more stable and predictable navigation channel. A channel is anticipated to align next to the terminal groin while the dredged channel will infill approximately 50% slower based on the presence of the terminal groin.

9.7 COST COMPARISON

In an effort to compare the nourishment-only to nourishment/groin alternatives, Table 9-8 presents a breakdown of annualized project construction-related costs over a 30-year period.

The conceptual construction cost table includes the following alternatives:

1. Bi-Annual Beach Nourishment
2. Groin and Nourishment (4-year renourishment interval)

Table 9-8. Conceptual Annualized Construction Cost Estimate over 30 years. NOTE - Annualized Benefits and Damage Costs are not included

ALTERNATIVE		2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032
Bi-Annual Nourishment (Nour. Only)		NOURISHMENT		NOURISHMENT		NOURISHMENT		NOURISHMENT		NOURISHMENT		NOURISHMENT		NOURISHMENT
CPI Boost	4.0%													
Dredge Interest	4.0%													
Discount Rate	8.0%													
Nourishment Interval	2 yrs													
Fill Volume		300,000		300,000		300,000		300,000		300,000		300,000		300,000
Unit Cost ($/cy)		$12.50		$13.52		$14.62		$15.82		$17.11		$18.50		$20.01
Estimated Fill Cost		$3,750,000		$4,056,000		$4,386,970		$4,744,946		$5,132,134		$5,550,916		$6,003,871
Mob/Demob		$2,000,000		$2,163,200.00		$2,339,717.12		$2,530,638.04		$2,737,138.10		$2,960,488.57		$3,202,064.44
Groin Construction Cost		$0												
Monitoring/Surveying/Permitting Coordination		$125,000	$65,000	$135,200.00	$73,116.16	$146,232.32	$79,082.44	$158,164.88	$85,535.57	$171,071.13	$92,515.27	$185,030.54	$100,064.51	$200,129.03
TOTAL Annual Cost		**$5,875,000**	**$65,000**	**$6,354,400**	**$73,116**	**$6,872,919**	**$79,082**	**$7,433,749**	**$85,536**	**$8,040,343**	**$92,515**	**$8,696,435**	**$100,065**	**$9,406,064**
TOTAL Present Value Annual Cost (2020)		**$5,875,000**	**$60,185**	**$5,447,874**	**$58,042**	**$5,051,801**	**$53,822**	**$4,684,523**	**$49,909**	**$4,343,947**	**$46,281**	**$4,028,132**	**$42,916**	**$3,735,278**
Annual Cost assuming 33.3% Local Match*		**$2,041,667**	**$65,000**	**$2,208,267**	**$73,116**	**$2,388,461**	**$79,082**	**$2,583,360**	**$85,536**	**$2,794,162**	**$92,515**	**$3,022,165**	**$100,065**	**$3,268,774**
Present Value Annual Cost (2020) assuming 33.3%Matcl		**$2,041,667**	**$60,185**	**$1,893,233**	**$58,042**	**$1,755,590**	**$53,822**	**$1,627,955**	**$49,909**	**$1,509,599**	**$46,281**	**$1,399,847**	**$42,916**	**$1,298,075**
TOTAL Cost assuming 33.3% Local Match*		**$58,690,000**												
TOTAL Present Value Cost (2020) w/33.3% Match*		**$19,690,000**		**$125,000**	**Beach Fill Monitoring**									
AVERAGE ANNUAL COST (Total/30yrs) w/33.3% Match*		**$1,960,000**		**$65,000**	**Annual Monitoring**									

*33.3% funding match does not include groin construction costs or monitoring

	2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042	2043	2044	2045	2046	2047	2048	2049
	NOURISHMENT		NOURISHMENT		NOURISHMENT		NOURISHMENT		NOURISHMENT		NOURISHMENT		NOURISHMENT		NOURISHMENT		NOURISHMENT	PRORATED NOURISHMENT
CPI Boost																		
Dredge Interest																		
Discount Rate																		
Nourishment Interval																		
Fill Volume	300,000		300,000		300,000		300,000		300,000		300,000		300,000		300,000		300,000	
Unit Cost ($/cy)	$20.01		$21.65		$23.41		$25.32		$27.39		$29.62		$32.04		$34.66		$37.48	
Estimated Fill Cost	$6,003,871		$6,493,787		$7,023,680		$7,596,812		$8,216,712		$8,887,195		$9,612,391		$10,396,762		$11,245,137	
Mob/Demob	$3,202,064.44		$3,463,352.90		$3,745,962.49		$4,051,633.03		$4,382,246.29		$4,739,837.58		$5,126,608.33		$5,544,939.57		$5,997,406.64	
Groin Construction Cost																		
Monitoring/Surveying/Permitting Coordination	$200,129.03	$108,229.78	$216,459.56	$117,061.33	$234,122.66	$126,613.53	$253,227.06	$136,945.20	$273,890.39	$148,119.92	$296,239.85	$160,206.51	$320,413.02	$173,279.36	$346,558.72	$187,418.96	$374,837.91	$202,712.34
TOTAL Annual Cost	**$9,406,064**	**$108,230**	**$10,173,599**	**$117,061**	**$11,003,765**	**$126,614**	**$11,901,672**	**$136,945**	**$12,872,848**	**$148,120**	**$13,923,273**	**$160,207**	**$15,059,412**	**$173,279**	**$16,288,260**	**$187,419**	**$17,617,382**	**$1,905,425**
TOTAL Present Value Annual Cost (2020)	**$3,735,278**	**$39,796**	**$3,463,714**	**$36,903**	**$3,211,894**	**$34,220**	**$2,978,382**	**$31,732**	**$2,761,847**	**$29,425**	**$2,561,054**	**$27,286**	**$2,374,859**	**$25,302**	**$2,202,201**	**$23,462**	**$2,042,096**	**$204,505**
Annual Cost assuming 33.3% Local Match*	**$3,268,774**	**$108,230**	**$3,535,506**	**$117,061**	**$3,824,003**	**$126,614**	**$4,136,042**	**$136,945**	**$4,473,543**	**$148,120**	**$4,838,584**	**$160,207**	**$5,233,413**	**$173,279**	**$5,660,459**	**$187,419**	**$6,122,353**	**$905,425**
Present Value Annual Cost (2020) assuming 33.3%Matcl	**$1,298,075**	**$39,796**	**$1,203,702**	**$36,903**	**$1,116,190**	**$34,220**	**$1,035,041**	**$31,732**	**$959,791**	**$29,425**	**$890,012**	**$27,286**	**$825,306**	**$25,302**	**$765,304**	**$23,462**	**$709,665**	**$97,177**

Table 9-8 (continued)

ALTERNATIVE		2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032
Nourishment & Groin (4yr Nour. Int.)		GROIN CONSTR. + NOUR.				NOURISHMENT				NOURISHMENT				NOURISHMENT
CPI Boost	4.0%													
Discount Rate	8.0%													
Nourishment Interval	4 yrs													
Fill Volume		300,000				300,000				300,000				300,000
Unit Cost ($/cy)		$12.50				$14.62				$17.11				$20.01
Estimated Fill Cost		$3,750,000				$4,386,970				$5,132,134				$6,003,871
Mob/Demob		$2,000,000				$2,339,717.12				$2,737,138.10				$3,202,064.44
Groin Construction Cost		$6,000,000												
Monitoring/Surveying/Permitting Coordination		$227,000	$132,000	$142,771.20	$148,482.05	$265,557.89	$160,598.18	$167,022.11	$173,702.99	$171,071.13	$92,515.27	$96,215.88	$100,064.51	$200,129.03
TOTAL Annual Cost		**$11,977,000**	**$132,000**	**$142,771**	**$148,482**	**$6,992,245**	**$160,598**	**$167,022**	**$173,703**	**$8,040,343**	**$92,515**	**$96,216**	**$100,065**	**$9,406,064**
TOTAL Present Value Annual Cost (2020)		**$11,977,000**	**$122,222**	**$122,403**	**$117,870**	**$5,139,509**	**$109,300**	**$105,252**	**$101,354**	**$4,343,947**	**$46,281**	**$44,567**	**$42,916**	**$3,735,278**
Annual Cost assuming 33.3% Local Match*		**$8,143,667**	**$132,000**	**$142,771**	**$148,482**	**$2,507,787**	**$160,598**	**$167,022**	**$173,703**	**$2,794,162**	**$92,515**	**$96,216**	**$100,065**	**$3,268,774**
Present Value Annual Cost (2020) assuming 33.3%Matcl		**$8,143,667**	**$122,222**	**$122,403**	**$117,870**	**$1,843,298**	**$109,300**	**$105,252**	**$101,354**	**$1,509,599**	**$46,281**	**$44,567**	**$42,916**	**$1,298,075**
TOTAL Cost assuming 33.3% Local Match*		**$41,260,000**		**$227,000**	**Construction Monitoring**									
TOTAL Present Value Cost (2020) assuming 33.3% Matcl		**$17,800,000**		**$132,000**	**Semi-Annual Monitoring**		**$125,000**	**Beach Fill Monitoring (year 2028 onward)**						
AVERAGE ANNUAL COST (Total/30yrs) w/33.3% Match*		**$1,380,000**		**$67,000**	**Annual Monitoring**		**$65,000**	**Annual Monitoring (year 2029 onward)**						
*33.3% funding match does not include groin construction costs or monitoring														

2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042	2043	2044	2045	2046	2047	2048	2049
NOURISHMENT				NOURISHMENT				NOURISHMENT				NOURISHMENT				NOURISHMENT	PRORATED NOURISHMENT
300,000				300,000				300,000				300,000				300,000	
$20.01				$23.41				$27.39				$32.04				$37.48	
$6,003,871				$7,023,680				$8,216,712				$9,612,391				$11,245,137	
$3,202,064.44				$3,745,962.49				$4,382,246.29				$5,126,608.33				$5,997,406.64	
$200,129.03	$108,229.78	$112,558.97	$117,061.33	$234,122.66	$126,613.53	$131,678.07	$136,945.20	$273,890.39	$148,119.92	$154,044.72	$160,206.51	$320,413.02	$173,279.36	$180,210.54	$187,418.96	$374,837.91	$202,712.34
$9,406,064	**$108,230**	**$112,559**	**$117,061**	**$11,003,765**	**$126,614**	**$131,678**	**$136,945**	**$12,872,848**	**$148,120**	**$154,045**	**$160,207**	**$15,059,412**	**$173,279**	**$180,211**	**$187,419**	**$17,617,382**	**$10,250,359**
$3,735,278	**$39,796**	**$38,322**	**$36,903**	**$3,211,894**	**$34,220**	**$32,952**	**$31,732**	**$2,761,847**	**$29,425**	**$28,335**	**$27,286**	**$2,374,859**	**$25,302**	**$24,365**	**$23,462**	**$2,042,096**	**$1,100,146**
$3,268,774	**$108,230**	**$112,559**	**$117,061**	**$3,824,003**	**$126,614**	**$131,678**	**$136,945**	**$4,473,543**	**$148,120**	**$154,045**	**$160,207**	**$5,233,413**	**$173,279**	**$180,211**	**$187,419**	**$6,122,353**	**$1,940,663**
$1,298,075	**$39,796**	**$38,322**	**$36,903**	**$1,116,190**	**$34,220**	**$32,952**	**$31,732**	**$959,791**	**$29,425**	**$28,335**	**$27,286**	**$825,306**	**$25,302**	**$24,365**	**$23,462**	**$709,665**	**$208,287**

As previously mentioned, it is believed that the only feasible nourishment-only alternative for the project area shoreline is a 2-year 300,000 cy nourishment and that the current 4-year 600,000 cy cycle is unsustainable. The analysis in Table 9-8 spans from 2020 to 2049 (30 years). A 4 percent inflation rate was assumed for the analysis and is presented as the Consumer Price Index (CPI) Boost in Table 9-8. This rate agrees with recent USACE GRR economics studies (USACE, 2012) and is typical when considering future nourishment-related costs (e.g., dredging, diesel fuel).

A discount rate is also provided in Table 9-8 and is used to "discount" cash flows in future years. This provides a present value of the money that a potential investment generated. This allows planners to get an idea of what a particular investment will generate in "today's cash" and compare across alternative investments.

Nourishment volumes are dependent on the renourishment interval and are based on historic shoaling, historic projects, and model results. Fill volumes for 2020 are estimated at 300,000 cy for all nourishment options.

Mob/demob costs for beach nourishments are estimated at $2,000,000 per event and remain constant for all alternatives. Groin construction is estimated at $6.0 million (see Section 9.1). No groin maintenance beyond ongoing nourishments was included. Note that most existing groin systems require little to no maintenance over the first couple of decades (Moffatt and Nichol, 2010). Some minor rock restacking may be needed and this can be assumed to occur in conjunction with nourishment events.

Table 9-8 also includes monitoring costs. It is generally assumed that monitoring related to a nourishment/groin project will require more effort than nourishment-only monitoring. However after an initial period of 5 years, it is assumed that groin-related monitoring costs can be reduced based on monitoring results.

Total and present value costs have "33.3% local match" columns that take into account the fact that the State's shallow draft dredging program will pay up to 66.7% of the beach nourishment costs of the preferred borrow area (which is the ebb shoal pivot channel) due to its dredging-for-navigation component. This match is only included for beach nourishment construction and it is not included for groin construction or monitoring (which will be funded by the Town and/or County and/or other funding sources).

Table 9-9 summarizes Table 9-8 results while also including non-construction related (e.g., recreation, damage losses, benefits) costs for the relocation/retreat and no-action alternatives. These non-construction-related costs were developed by the 2009 EIS, the NC Terminal Groin Report, and the NC BIMP. Annual costs from the 2009 EIS were taken from the "North Section" of shoreline (~20,000 feet) and pro-rated to the 4,500-foot project area shoreline. Annual costs from the NC terminal groin and BIMP reports were pro-rated from the entire 11 mile shoreline to the 4,500-foot project area shoreline.

The no-action alternative assumes 4-year interval dredging where 600,000 cy is placed. The no-action cost doesn't explicitly include sandbag revetment construction and maintenance costs. Instead these assumed to be included in the annual damages costs. For the 2-year nourishment interval, no damages/losses are assumed (similar to the groin alternative). The retreat/abandon and no-action alternatives are significantly more expensive in terms of damage and revenue loss.

As seen in Table 9-9 the groin alternative is the least expensive option. This analysis is conceptual in nature due to forecasting out to 2049 and the assumptions involved therein, however, it is clear that for a highly erosional area such as the north end, a groin will act to increase the nourishment interval and significantly reduce both long term construction-related costs (e.g., nourishment, monitoring) and total costs. Overall, the groin-and-nourishment alternative is approximately 42% more economical than the next closest alternative in terms of total annualized costs.

Table 9-9. Total Costs of Conceptual Alternatives

Alternative	30-Year Construction Cost	Average Annual Construction Cost	Annual Damages	Revenue Losses	NC BIMP Recreation Losses (50% beach width)	**Total Annualized Cost**
Retreat/Relocate/Land Acquisition	$0	$0	$6,436,508	$308,228	$464,409	**$7,209,144**
No-Action (4yr Interval, Sandbag Revetment, no dune, etc.)	$51,290,000	$1,710,000	$6,038,055	$200,453	$464,409	**$8,412,917**
Nourishment (2-yr Interval)	$58,690,000	$1,960,000	$0	$0	$0	**$1,960,000**
Groin and Nourishment (4-yr Interval)	$41,260,000	$1,380,000	$0	$0	$0	**$1,380,000**

10.0 SUMMARY

10.1 BACKGROUND

The Town of North Topsail Beach and Onslow County have been actively and independently performing NRI management activities for decades. Dredging for navigation and nearby shoreline placement (i.e., beneficial use of dredged material) of beach compatible material have only provided short-term navigation and shoreline benefits and a longer-term solution is required.

The Town of North Topsail Beach developed a comprehensive EIS in 2009 for its entire 11-mile shoreline. This EIS also included NRI management activities (dredging, beach fill placement, etc.). The engineering report presented herein includes many references to the 2009 EIS and builds off it. Onslow County has also recently taken a more active role in inlet management and navigation improvements to NRI. NRI is the primary ocean access to the majority of Onslow County for both commercial and recreational vessels. NRI connects with both New River Estuary and the AIWW, however, the inlet itself is hazardous to navigation. The USCG Coast Pilot states that navigating NRI "*is considered dangerous by local pilots, and entrance should not be attempted except under the most favorable conditions*."

10.2 ALTERNATIVES

The study presented herein describes the alternatives to improve navigation for NRI and shoreline stability for the northeast end (Phase 1 reach) of North Topsail Beach. Alternatives that were evaluated include:

- Nourishment-Only
- Terminal Groin
- Channel Dredging
- Jetties
- Inlet Relocation
- Structure Relocation/Retreat

The nourishment-only alternative is different from existing beach management practices in that it proposes a 2-year dredge/fill interval which dredges only 300,000 cy from NRI. The no-action alternative features a 4-year dredge/fill interval which dredges 600,000 cy from NRI. The no-action alternative is not sustainable where over 35 structures (representing over 200 residential units) are imminently threatened and currently 3,600-ft of shoreline is revetted with a sandbag system (see Figure 10-1).

Figure 10-1. Sandbags (shown in black) Continuous along ~3,300 feet from ~Station 1163+00 to ~1135+00. Another ~300 ft sandbag section is located just south of Station 1170+00

Additional alternatives, such as the NRI channel relocation and jetties were also assessed. NRI relocation would entail cutting through a portion of Onslow Beach, similar to the 2002 Mason Inlet relocation. Jetty (single and dual-jetty) systems were also conceptually assessed. Channel relocation and jetty construction alternatives are not preferred at this time due to the lengthy/costly environmental, design, and permitting process.

10.3 PREFERRED ALTERNATIVE

Modeling and analysis indicates that the preferred alternative is the north groin (or "NGroin") alternative with a concurrent nourishment program. This groin layout was chosen from six different groin configurations that were modeled (APTIM, 2016). The pending preferred alternative includes three primary components:

- Terminal Groin
- NRI Dredging and Beach Nourishment
- Monitoring

The preferred alternative involves the construction of a 2,021-ft-long terminal groin on the north end ocean beach at NRI and recurring beach nourishment of the adjoining ~5,100 linear-ft north end shoreline using the inlet outer bar channel as a borrow source. For purposes of maintaining navigability and acquiring beach fill, the Town will continue to dredge the outer bar channel to a depth of 16 ft and width of 500 ft as authorized by existing CAMA Major Permit 79-10 issued 10 January 2017.

Figure 10-2 presents a conceptual depiction of all three project components (terminal groin, nourishment, inlet borrow site). A terminal groin is being proposed based on its ability to keep sand along the northern NTB shoreline significantly longer than nourishment-only alternatives. The proposed terminal groin and inlet channel borrow site design were also evaluated for benefits to inlet navigation. Modeling of the proposed borrow site and groin alignment indicate that a more stable channel will occur. Thus, the proposed action is anticipated to provide some improvements in navigation safety.

The proposed terminal groin would consist of three main sections: anchor section, upland section, and in-water section. Although the in-water section and most of the upland section would require immediate construction, the anchor section could be constructed at a later date as a cost-saving measure. As proposed, the landward-most approximately 500-ft section of the terminal groin would only be constructed when the receding shoreline contacts an established threshold for a specified buffer distance from the anchor footprint. Beach nourishment events would place about 310,000 cy of sand along about 5,100 feet of shoreline (average of 61 cy/ft). The inlet channel borrow site will be the pivot channel that was slightly realigned after the initial 2013 realignment project. The borrow site is expected to yield about 600,000 cy (based on recent sedimentation and pre-project bathymetries); however, project nourishment events will only require about 50 percent of the total volume of material. The remainder of the material may be used for shoreline sections located south of the project area (based on need).

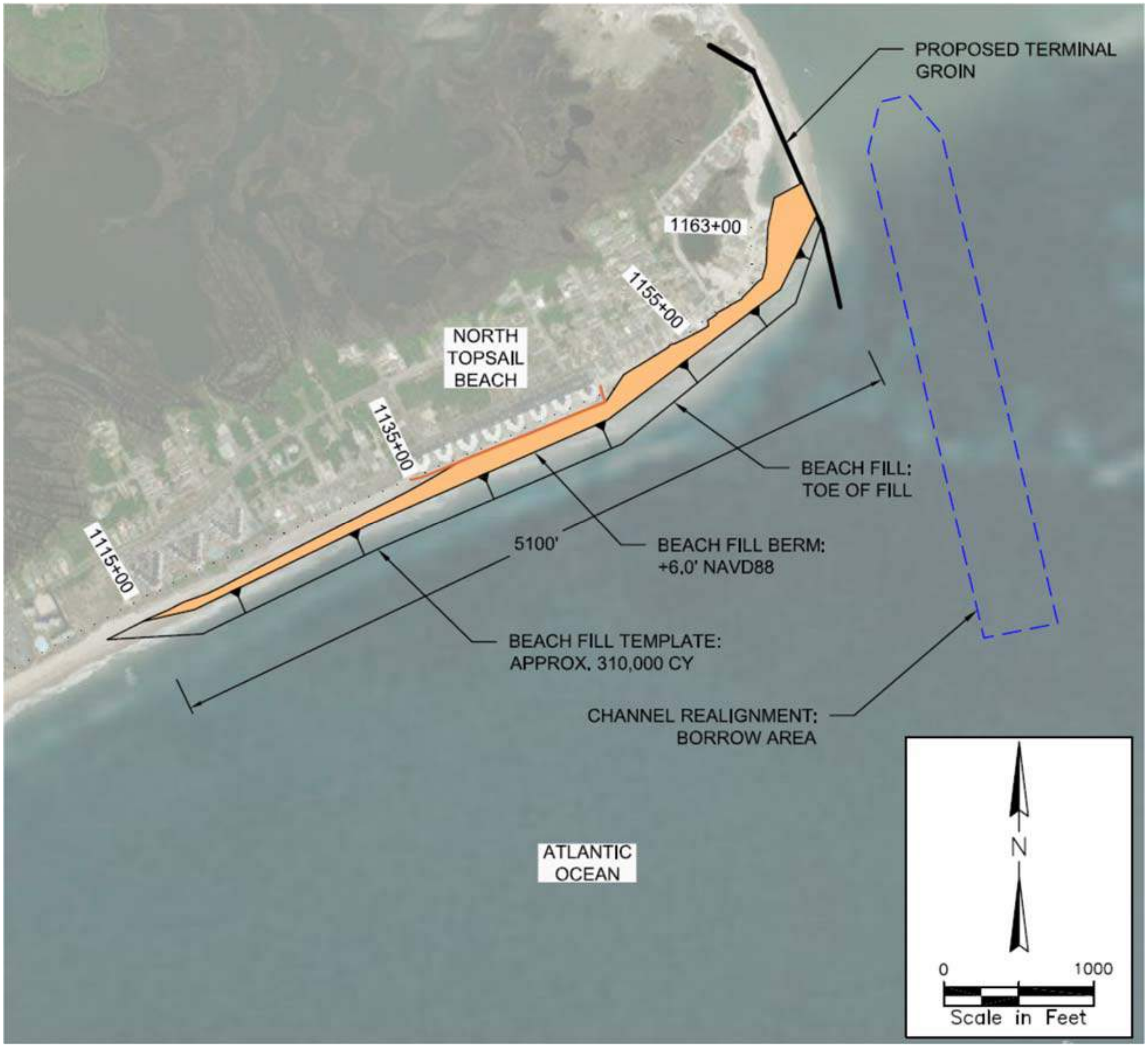


Figure 10-2. Preferred Terminal Groin, Inlet Dredging, and Beach Fill Placement Overview.

The terminal groin and nourishment will create a reoriented shoreline that will reduce sediment transport in the project area. The preferred alternative is shown to increase the nourishment interval from 2 years to 4 years, in comparison to the nourishment-only alternative. The proposed project was designed to deliver significant protection to the most vulnerable shoreline currently protected by sandbag revetments.

The preferred borrow area is a reusable borrow area that will create a deeper/larger navigation channel. The terminal groin will also decrease in-filling of the borrow area by approximately 50 percent.

"Downdrift" at the project areas refers to the estuarine shoreline along NRI as net sediment transport is into the inlet. The groin modeling showed relatively minor and localized effects to the NRI system however monitoring will occur as this inlet has been migrating to the southwest over the last century. A comprehensive monitoring program will be instituted to assess project-related effects to the NRI system and adjacent shorelines.

Costs

The 2016 NC BIMP report estimates that the 2013-2014 Beach Recreation Annual Total Impact Output for NTB was $38.1 million, which accounted for 493 jobs. Beach recreation is the primary economic engine for the Town of North Topsail while it also benefits Onslow County. The proposed terminal groin, dredging and nourishment program for NRI is estimated to result in substantial savings over the long-term. Over a 30-year period, the proposed project is estimated to result in over $17 million (30% reduction) in total savings when compared to existing beach management practices.

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Appendix A

2016 Terminal Groin Findings

COASTAL PLANNING & ENGINEERING OF NORTH CAROLINA, INC.

4038 MASONBORO LOOP ROAD, WILMINGTON, NC 28409 910-791-9494 PHONE 910-791-4129 FAX

August 31, 2016

Stuart Turille
Town of North Topsail Beach
2008 Loggerhead Court
North Topsail Beach, NC 28460

Subject: Findings of the Numerical Modeling/Design Analysis of a Terminal Groin

Dear Mr. Turille:

On July 9, 2015, you provided CPE-NC with notice to proceed on a numerical modeling/design analysis of a terminal groin adjacent to New River Inlet. This letter report serves as our summary of findings as described in Exhibit C of the agreement between CPE-NC and the Town of North Topsail Beach. The findings include an introduction to the project, a description of the methodology, results of the analysis, conclusions, and a description of a preliminary structural design, which includes cost estimates.

Introduction

Simulation of potential locations and lengths of a terminal groin on the north end of North Topsail Beach have been performed using the Delft3D model. Two locations, one designated as Northern and the other the Southern, were evaluated. Three (3) terminal groin lengths were also simulated for each terminal groin location. The two location options are shown on Figure 1. The nomenclature used for the various terminal groin options are:

Option 1- Northern Groin
Option 2- Northern Groin Extended 250 feet
Option 3- Northern Groin Extended 500 feet
Option 4- Southern Groin
Option 5- Southern Groin Extended 250 feet
Option 6- Southern Groin Extended 500 feet

Methodology

As part of a channel alternative analysis conducted by CPE-NC for the Town of North Topsail Beach to determine preferred channel alternatives for a proposed channel realignment project, a Delft3D model was setup and calibrated for the New River Inlet. A detailed discussion of the data used to drive the model, the model setup, and model calibration was included in the report titled *New River Inlet Channel Realignment Alternative Channel Modeling Study North Topsail Beach, NC* (CPE-NC, 2016). Using the same model setup developed for the channel alternative analysis, CPE-NC evaluated six (6) different terminal groin options as well as a no groin alternative. A 2-year simulation period was used to evaluate the different responses of the model to the various terminal groin options weighed against the model results obtained for the no groin alternative.

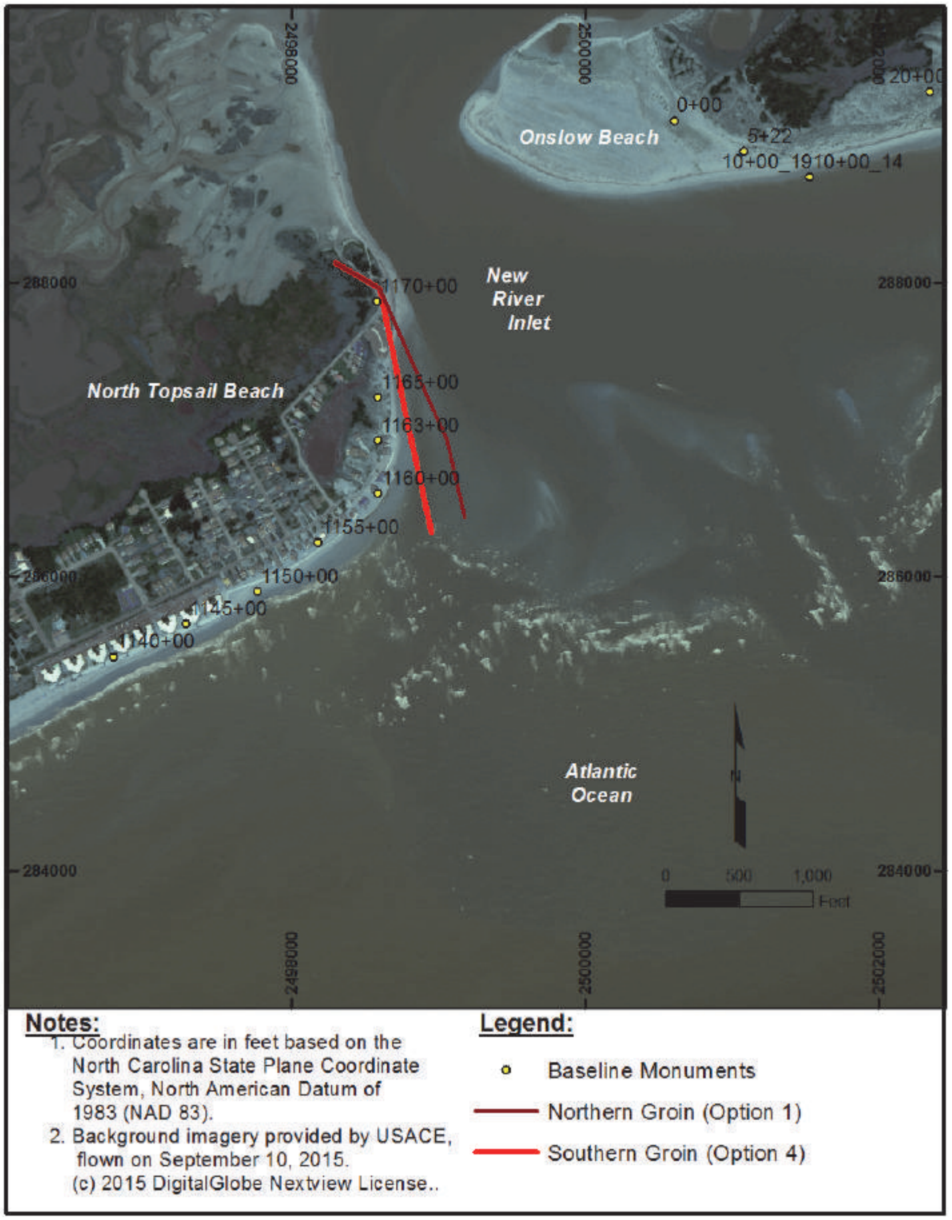


Figure 1. Map showing the Northern Groin (Option 1) and Southern Groin (Option 4).

Results

Results of model simulations were evaluated in terms of model-indicated volume changes along the north end of North Topsail Beach, changes in the ocean bar channel, resulting ebb shoal configuration, and sediment transport patterns.

Model-Indicated Volume Changes Along the North End of North Topsail Beach: The modeled volume changes along the north end of North Topsail Beach for the six (6) terminal groin options, measured relative to the modeled volume changes obtained for the no groin alternative, are provided in Figure 2. The results are given in terms of cubic yards per foot of distance along the north end of North Topsail Beach and covers the area from station 1120+00 (middle building of the St. Regis Resort) to station 1160+00 (approximately 275 ft. north of the intersection of River Dr. and New River Inlet Road). Positive volume changes indicate benefits with respect to the no groin scenario (i.e. due do the presence of the groin). Positive volume changes shown in Figure 2 can be indicative of increased accretion rates or decreased erosion rates. Although not present in Figure 2, negative volumetric changes would indicate more erosive/less accretional behavior of beach profile due to the presence of the analyzed structural options.

The results shown in Figure 2 take into account volume change across the entire beach profile for points south of station 1134+00. For the area north of station 1134+00, the volume changes shown on Figure 2 were limited to the area landward of the -6-foot NAVD88 contour in order to represent the direct impact of the terminal groin options on the nearshore profile and dry sand beach. The model indicated volume changes seaward of the -6-foot NAVD88 contour (the area within the polygon on Figure 3) were primarily associated with changes in the ebb tide delta caused by different responses of the inlet bar channels. As discussed below, the channel that formed immediately adjacent to the inlet side of the groins had a significant impact on volume changes in the nearshore area off the north end of North Topsail Beach. The inclusion of the volume changes in this area would have masked the true impact the groins had on the shoreline and dry sand beach.

The performance of the shorter groins (Options 1 and 4) are less effective when compared to the longer groin options, as the integrated amount of material 'below' the blue lines in Figure 2 are smaller. The other groin options show similar efficiency to hold sand and overall performance, although the distribution of the benefits along the north end of the island varies according to the position (northern/southern) and length (extended 250 feet or 500 feet) of the structure. Options 2 and 3 (extended northern groins) tend to concentrate the benefits immediately south of the structure, while for the southern groins, the positive effects are more evenly distributed along the north end of the island. For both the northern and southern groins, extended 500 feet, the benefits are concentrated closer to the structure, while for the extended 250-foot options, the benefits are more evenly distributed.

With the exception of Option 5 (the southern groin extended 250 feet), the model results show very little difference in the impact of the groin options on volume changes along the shoreline between station 1142+00 (located between buildings #4 and #5 of the Topsail Reefs) and station 1120+00 (located near the middle building of the St. Regis). Option 5 indicated larger positive volume differences in this section. The simulated volume change results also show Option 5 produced higher positive volume differences up to a point just north of station 1155+00 (located approximately 120 ft. northeast of Port Dr.). From this point north to the terminal groin, several of the other options show higher positive volume differences; however, Option 5 consistently produced a positive volume difference of approximately 50 cy/ft. in this area compared to the no groin alternative.

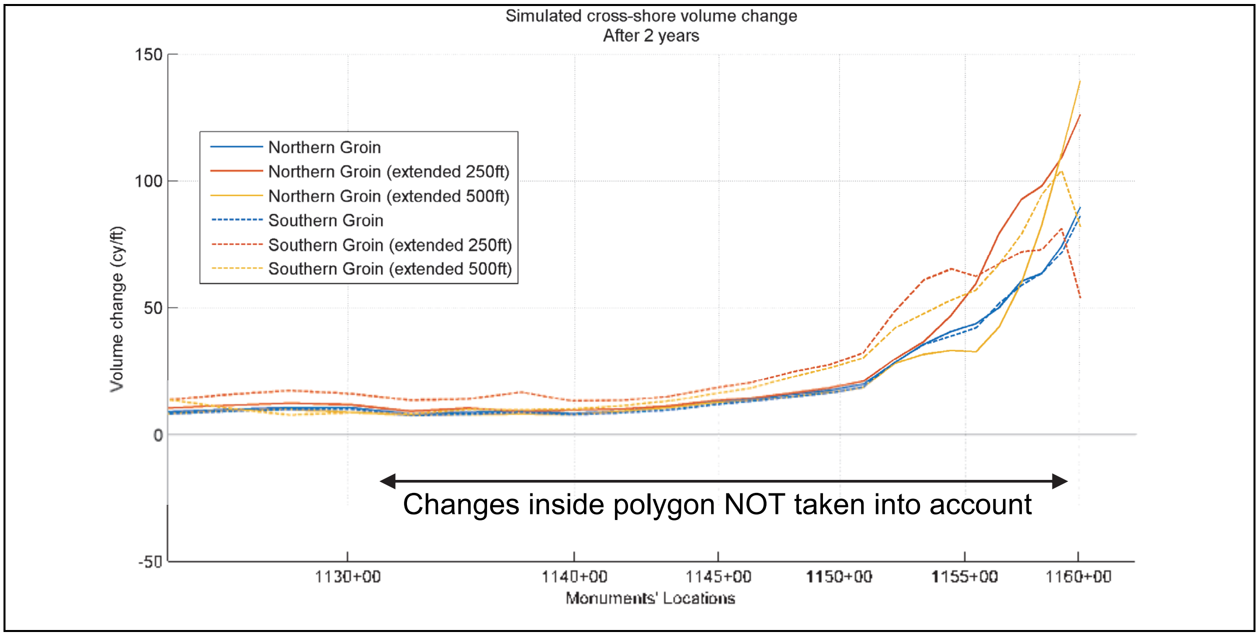


Figure 2. Graph showing a comparison of the difference in volume change between each of the six (6) groin alternatives relative to the no groin alternative in terms of cubic yards per foot.

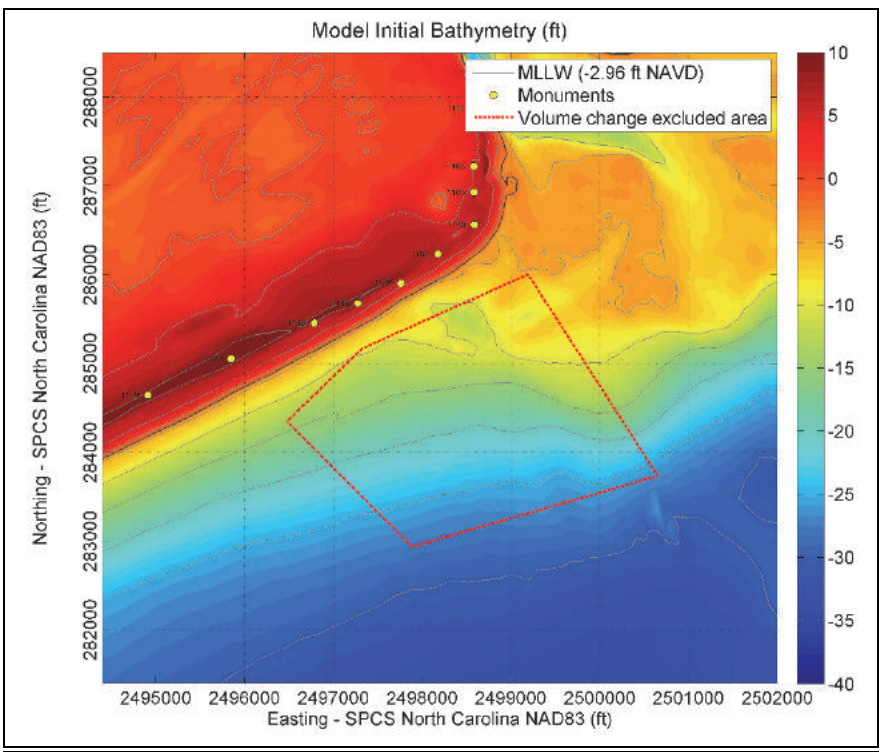


Figure 3. Map indicating the area in which volume change was not included in the results shown in Figure 3.

<u>Changes in the Ocean Bar Channel</u>: Bathymetric changes to the ebb tide delta of New River Inlet and changes in the channels passing through the delta relative to the no groin option are provided for all six terminal groin options in Figure 4. Positive values (warm colors) represent more accretion or less erosion of the ocean bar compared to the results of the "no groin alternative", while negative values (cool colors) indicate more erosion/less accretion than the 'no groin option'.

With exception of the channel that formed on the inlet side and immediately adjacent to the terminal groins for all terminal groin options, there were only minor differences in the response of the ebb tide delta and the channels passing through the delta associated with the terminal groin options. In all cases, the primary ebb channel appeared to experience some shoaling while the channel immediately to its south and projecting straight seaward appeared to deepen somewhat. The north side of the ebb tide delta also appeared to lose material around the seaward periphery with some of the material apparently migrating on shore and depositing in the secondary flood channel situated just offshore and parallel to the south end of Onslow Beach.

<u>Ebb Shoal Reconfiguration:</u> Qualitative analysis of change plots of the relative difference between each of the six (6) groin options and the no-groin alternative shows a buildup of a shoal at the outer or seaward end of the channel that formed immediately adjacent to the inlet side of the groin for all groin options. Options 1 and 4 shown in Figure 4 are the Northern and Southern Groin locations, respectively, while Options 2 and 5 are the Northern and Southern Groins extended 250 feet seaward. Based on these model results, extending the groins 250 feet seaward resulted in a larger build-up of sand on the south side of the inlet's ebb tide delta for both groin locations. Extending the groins 500 feet seaward (Options 3 and 6) appeared to move the sediment accumulation farther seaward with the build-up concentrated close to the seaward end of the structures. The concentration of sediment accumulation along the seaward ends of the 500-foot extension appears to be due to the longer structures preventing the outer end of the channel next to the groins from turning toward the southwest around the seaward ends of the groins.

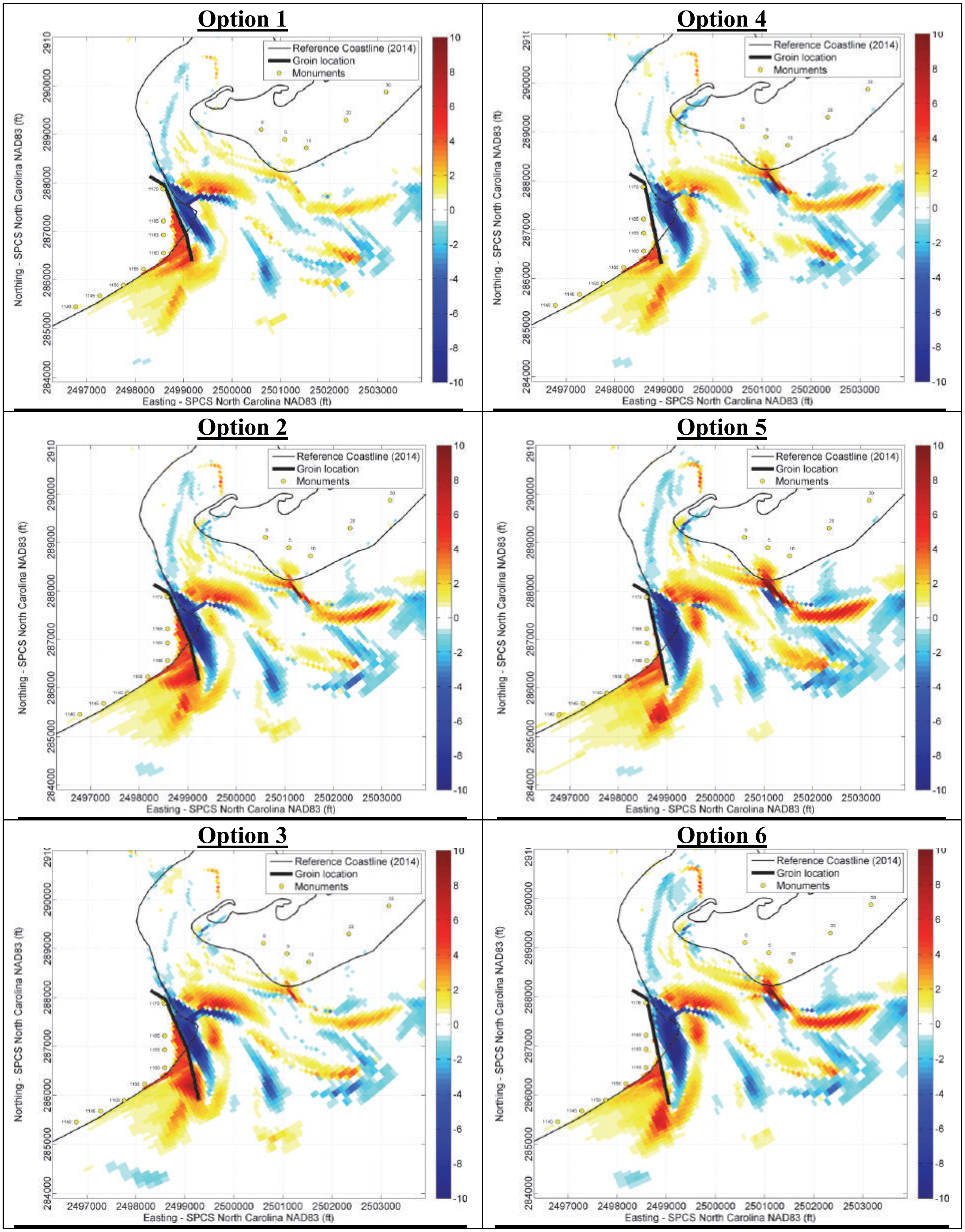


Figure 4. Plots showing the bathymetric differences between the 6 groin options and the no groin option after a 2 year simulation.

As stated earlier, all of the terminal groin options resulted in similar changes to the ebb tide delta, most notably volume losses from the seaward periphery of the delta offshore of Onslow Beach and the concomitant shoaling of the secondary flood channel running parallel to the southern end of Onslow Beach. Volume losses from this area of the ebb tide delta did appear to be larger for the Southern Groins (Options 4, 5, and 6) compared to the Northern Groins (Options 1, 2, and 3). However, there was no discernable difference in the response of the ebb tide delta between Options 5 and 6, i.e., extending the Southern Groin 250 feet produced essentially the same changes as the 500-foot extension.

In general, the types of changes in the configuration of the ebb tide delta observed from the model results are comparable to the changes the channel relocation project was attempting to induce. Thus, the terminal groins would seem to provide an added benefit of accumulating sediment on the ebb tide delta south of the inlet, which would in turn serve to diminish the severity of wave energy reaching the northern shoreline of North Topsail Beach.

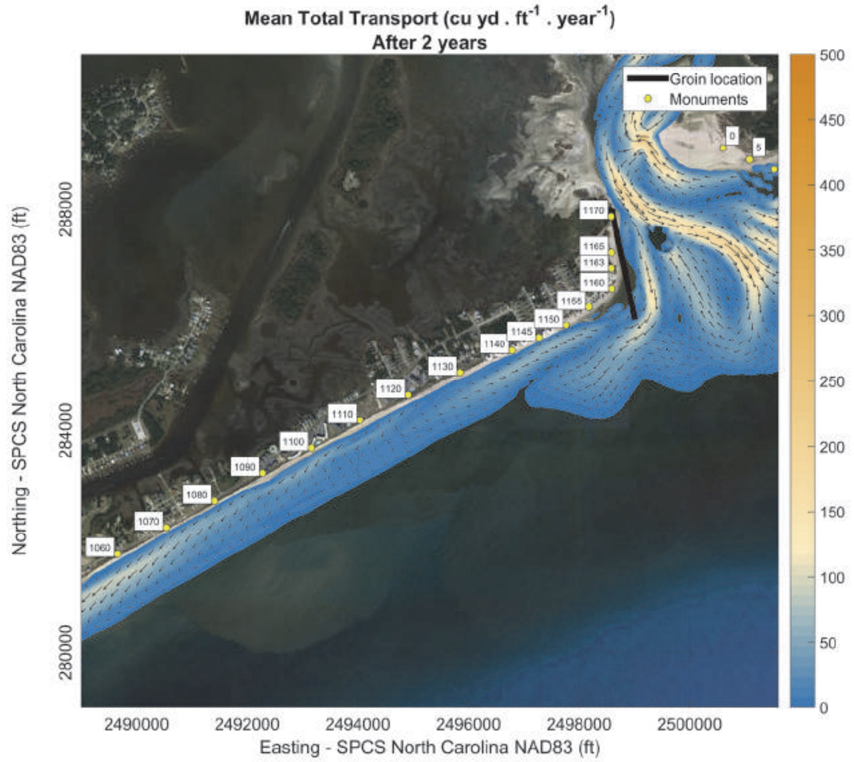
Sediment Transport Pattern: As discussed in the previous section on ebb shoal configuration, the groin lengths simulated in Options 2 and 5, which were 250-foot extensions of the Northern and Southern Groins, respectively, appear to balance the buildup of sand offshore of the structure by allowing the channel that forms on the inlet side of the terminal groin to wrap around the seaward end of the structure and distribute the sand over a broader area south of the groin relative to the sediment distribution obtained with the longer groin options (Options 3 and 6). Based on the model results, the Southern Groin provided the best results in terms of volume changes along the North Topsail Beach shoreline and the accumulation of sediment seaward of the north end of the island.

Of the three length options evaluated for the Southern Groin, the 250-ft. extension produced a larger build-up of material on the south side of the ebb tide delta by allowing the flow from the channel next to the groin to turn southeasterly and direct flow and hence sediment transport into the nearshore area immediately offshore of the north end of North Topsail Beach. This is demonstrated in Figure 5, which shows a comparison of the mean total sand transport in units of cubic yards per foot per year for Option 5 (Southern groin alignment extended 250 ft.) and Option 6 (Southern groin alignment extended 500 ft.). In this instance, the per foot unit is measured perpendicular to the direction of sediment transport.

The lighter shading in Figure 5 indicates higher rates of sand transport than the blue shading. While the differences in sediment transport shown for the two options is relatively small, the 250-ft. extension definitely allows the sediment to move south closer to shore.

During the July 26th, 2016 Board of Alderman Special Meeting, at which preliminary results of the terminal groin analysis were presented, a property owner voiced concerns about some of the graphics that show a reversal in the average total transport between stations 1120+00 and 1130+00. Specifically, the graphics indicate the mean total transport of sediment in this area is moving both north and south. The concern voiced by the property owner was that sand is moving out but no sand is moving in, which indicates the terminal groin is having a negative effect on this portion of the shoreline. Although the graphics provided do in fact show a reversal in this area for the recommended groin alternative, the same sediment transport pattern is produced when the model is run with the no-groin option as well as the other five (5) groin options. Figure 6 shows a comparison between the no-groin alternative and Alternative 5, which is the recommended alignment. The conclusion here is that given the model is not showing and difference in the sediment transport patterns in this vicinity when comparing groin and no-groin alternatives, the groin is not having a measurable impact on sediment transport patterns between stations 1120+00 and 1130+00.

Option 5



Option 6

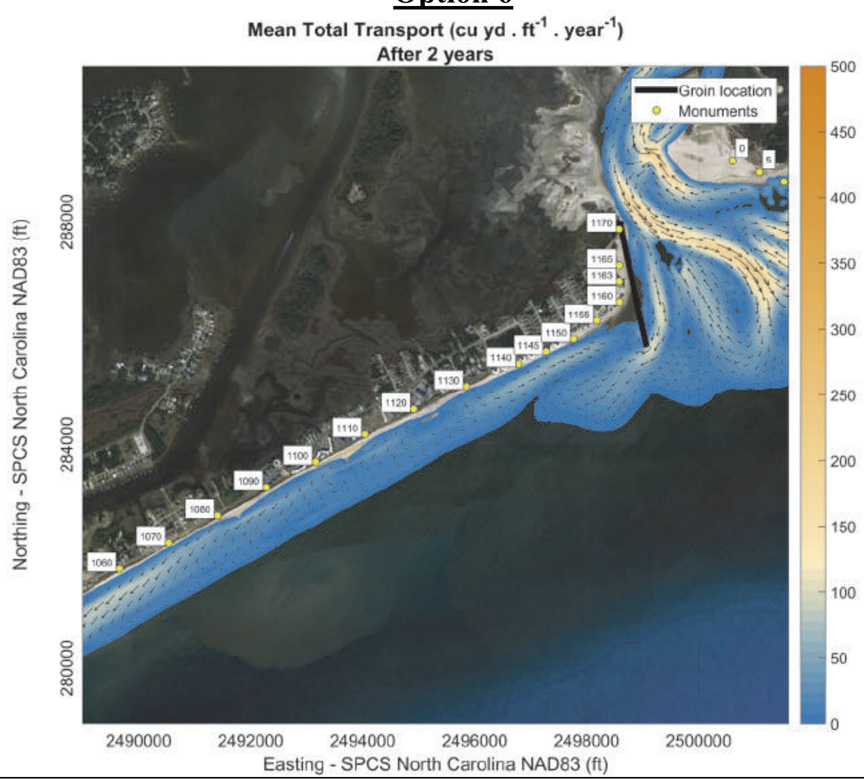


Figure 5. Sediment transport patterns for Groin Options 5 and 6.

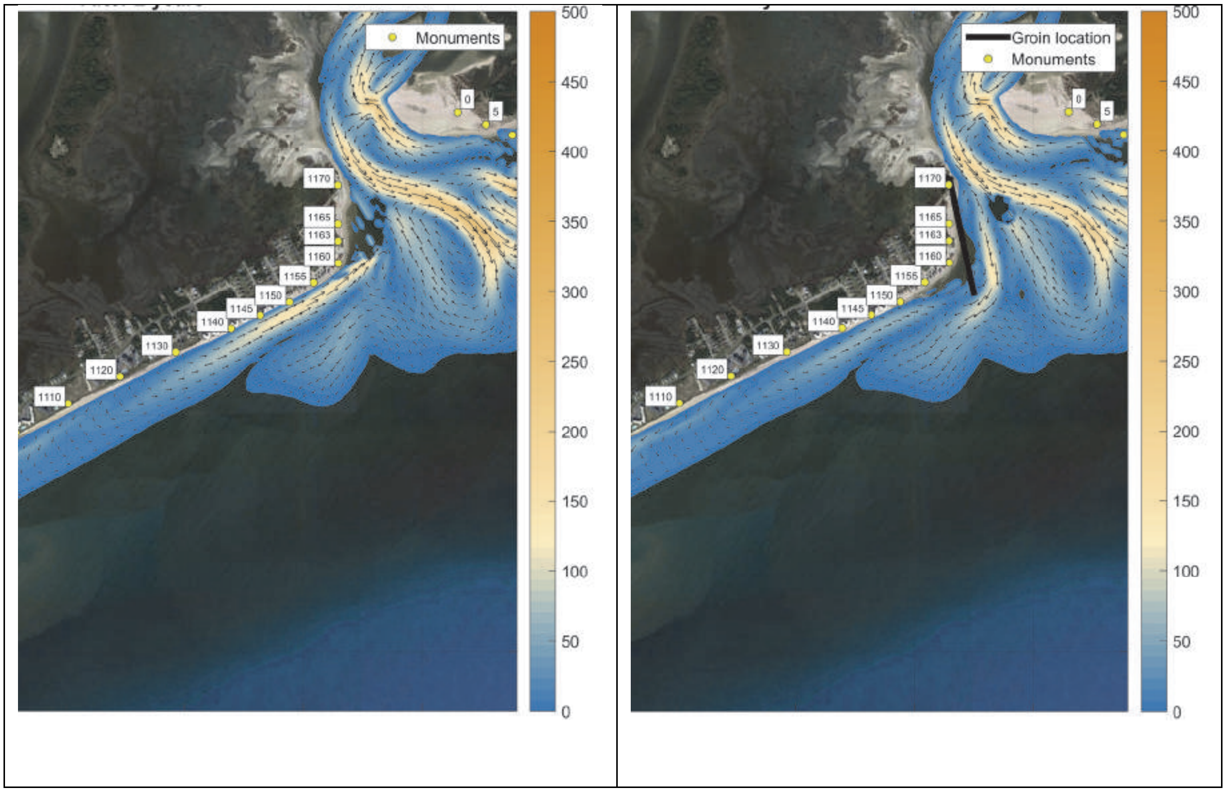


Figure 6. Mean total transport graphics showing a comparison between the no-groin alternative (left) and Alternative 5 groin (right). Scale indicates the mean total transport in cubic yards per foot per year.

Conclusions

Based on the model results discussed herein, the terminal groin option that appeared to provide the desired shoreline response south of the structure was Option 5, the Southern Groin Extended 250 feet. Option 5 outperforms the other five (5) simulated options along the majority of the shoreline south of the terminal groin. Although other alternatives show higher positive volume change immediately south of the groin, all of the options performed well in this area.

Modeled Shoreline Volume Changes for Option 5 versus Beach Fill Only. The Beach Fill Only Option and all of the terminal groin options were simulated using a uniform beach fill with a density of 46 cubic yards/linear foot (cy/lf). Figure 7 provides a comparison of model volume changes over a two-year model simulation period for the shoreline extending south from baseline station 1160+00 (south shoulder of New River Inlet) to baseline station 1050+00 (near Jenkins Way) for the Beach Fill Only Option and Terminal Groin Option 5.

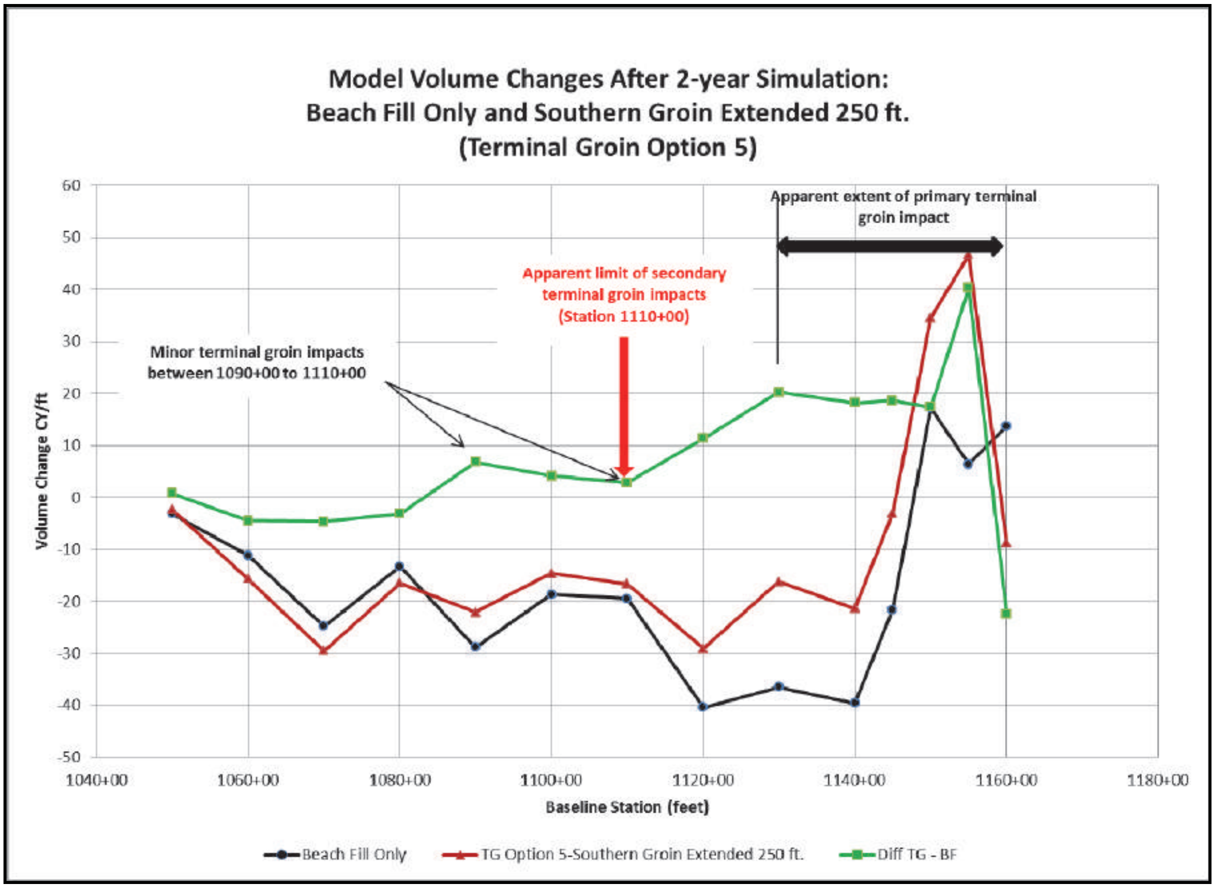


Figure 7. Model volume changes after 2-year simulation for the Beach Fill Option and Terminal Groin Option 5.

The differences in the volume changes for the Beach Fill Only and Terminal Groin are shown by the green line in Figure 7. Based on these preliminary results, the terminal groin appeared to have a direct positive impact on the shoreline that extended south to about baseline station 1130+00. Between baseline stations 1090+00 and 1110+00, there were some apparent positive secondary impacts of the structure on volume changes. The terminal groin did not have any appreciable impact on volume changes south of station 1090+00.

Between baseline stations 1150+00 and 1160+00, volume changes over the two-year simulation period were positive, i.e., the area accumulated sediment. This trend would not be expected to continue indefinitely since the structure, as discussed below, will be designed to limit the size of the accretion fillet by allowing littoral sediment pass through, over, and around the structure. Over the long-term, the 1,000-foot shoreline segment immediately south of the terminal groin would be expected to maintain a relatively stable position and should not require any appreciable amount of periodic nourishment in the future.

For the rest of the shoreline south of station 1150+00, the shoreline continued to erode under both the Beach Fill Only and Terminal Groin Option; however, relative volume changes for the Terminal Groin Option were less compared to the Beach Fill Only Option. This is demonstrated in Table 1 which provides a comparison of modeled volume changes within each 2,000-foot shoreline segment between stations 1150+00 and 1070+00. The relative comparisons are given as a percent reduction or increase in volume change associated with the Terminal Groin versus Beach Fill Only.

As shown in Table 1, the terminal groin produced significant reductions in volume losses south to about baseline station 1110+00 compared to Beach Fill Only. However, since volume losses continued to occur even with the terminal groin, the shoreline would require some level of periodic nourishment in the future to maintain the shoreline in a preferred position. Nevertheless, the volume of material that would be needed would be significantly less with the terminal groin compared to the no groin option.

Table 1. Relative shoreline volume changes associated with a terminal groin (Option 5) versus beach fill only along the north end of North Topsail Beach.

Shoreline Segment (baseline stations)	Relative Volume Change between Beach Fill Only and the Terminal Groin	Direction of Relative Change with Terminal Groin versus Beach Fill
1130+00 to 1150+00	69%	reduction
1110+00 to 1130+00	33%	reduction
1090+00 to 1110+00	21%	reduction
1070+00 to 1090+00	5%	increase

All of the previous discussion regarding the potential impacts of a terminal groin on volume changes is based on a limited number of model simulations. The final assessment of the potential impacts of a terminal groin on the North Topsail Beach shoreline will require additional detailed model simulations to assess the overall impacts of the proposed terminal groin and provide the information needed to address questions that will arise during the permitting process. For example, the model will be used to evaluate relative changes to the shoreline along Onslow Beach, changes in the interior shoals and intertidal areas, as well as changes in the marsh complexes. In addition, future model test would need to be made to evaluate navigation channel options that would be included with the terminal groin as well as evaluate a range of beach fill options for the north end of North Topsail Beach.

Preliminary Structural Design

The centerline of the terminal groin used in the model simulation and shown in Figure 1, included a rather long shore anchorage section that extended about 125 feet north of River Dr. and then angled inland approximately 320 ft. This put the landward end of the shore anchorage section over 250 feet from the inlet shoreline. At this preliminary stage in the design of the terminal groin, the shore anchorage section was shortened to terminate near River Dr. and then angle inland about 100 feet which positions the end point of the terminal groin about 220 feet from the inlet shoreline. This revised centerline for the proposed terminal groin, which was used to develop preliminary costs for the structure, is shown on Figure 8. In the final design, consideration must be given to the rate of movement of the inlet shoreline along the north end of North Topsail Beach to determine how far inland the anchorage section should extend to prevent possible flanking of the landward end of the terminal groin. Based on preliminary measurements of changes in the position of the inlet shoreline on the north end of North Topsail Beach, the rate of shoreline movement to the south in the vicinity of the proposed landward end of the terminal groin appears to be around 5 to 10 feet/year; however, the movement has not been constant as there have been periods in which the shoreline moved back to the north.

Figure 8. Centerline of proposed terminal groin on north end of North Topsail Beach (centerline station numbers are in feet measured from the landward end of the structure).

The total length of the terminal groin shown on Figure 8 is 1,900 feet. Based on shoreline and inlet conditions existing in 2016, the first 350 ft. of the structure (centerline station 0+00 to 3+50) would be constructed as a sheet pile wall protected on the inlet side by a rubble scour protection apron. The top of the sheet pile wall would be around +4.0 ft. NAVD88 while the top of the scour apron would be set at +2.0 ft. NAVD88. The top width of the scour protection apron would be 10 feet between centerline stations 0+00 and 1+00 and 20 feet between centerline stations 1+00 and 3+50. A typical cross-section of the shore anchorage section, designated as Section C-C is provided on Figure 9.

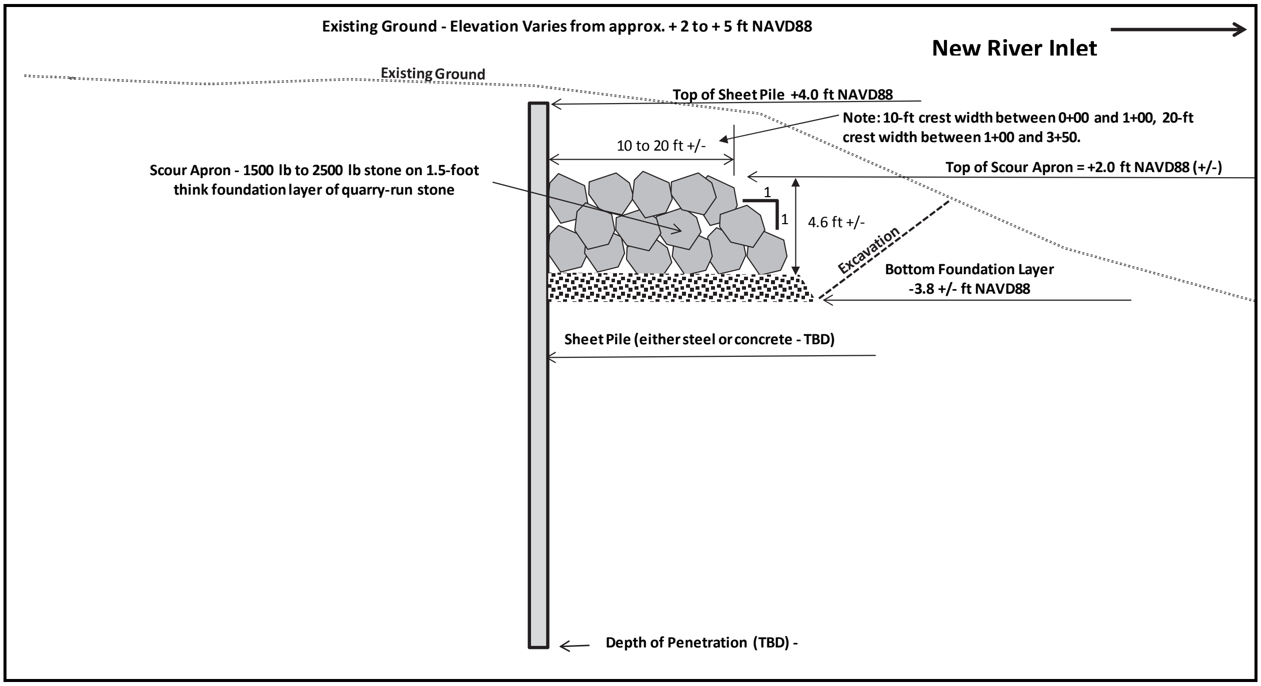


Figure 9. Typical terminal groin Cross-Section C-C-shore anchorage section-centerline stations 0+00 to 3+50.

From centerline station 3+50 to station 19+00 the terminal groin would be of rubblemound construction using two typical rubblemound cross-sections designated as B-B and A-A are shown on Figures 10 and 11, respectively. Section B-B would be used between centerline stations 3+50 and 11+50 and would be constructed with granitic armor stones weighing between 4.0 and 6.5 tons.

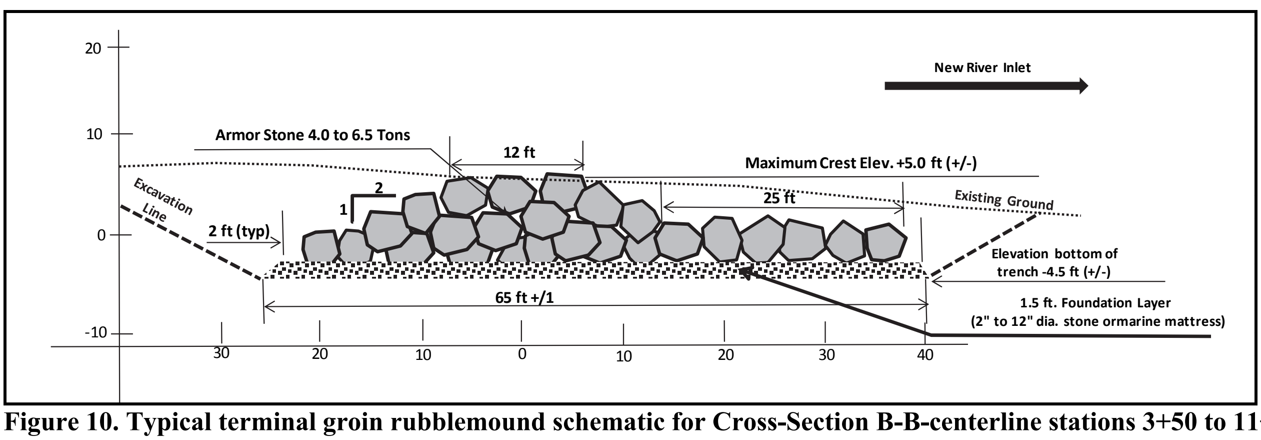


Figure 10. Typical terminal groin rubblemound schematic for Cross-Section B-B-centerline stations 3+50 to 11+50.

Cross-Section A-A would be used on the seaward portions between centerline stations 11+50 and 19+00 and would be constructed with granitic armor stones weighing between 7.5 and 12.5 tons. Both rubblemound cross-sections would include a 1.5-ft. thick bedding layer to protect against settling.

The overall design concept for both rubblemound sections is to allow littoral sediment to move past the structures toward New River Inlet once the accretion fillet on the south side of the structure is fully developed. To facilitate movement of material past the terminal groin, relatively large voids would be provided between adjacent stones to allow sediment to move through the structure. Also, the crest elevation would be set at +/- 5.0 ft. NAVD88 to allow sediment to move over the top of the structure. With the seaward end of the structure terminating in relatively shallow water (between -5.0 and -6.0 ft. NAVD88), littoral material would also be able to move around the seaward end of the structure.

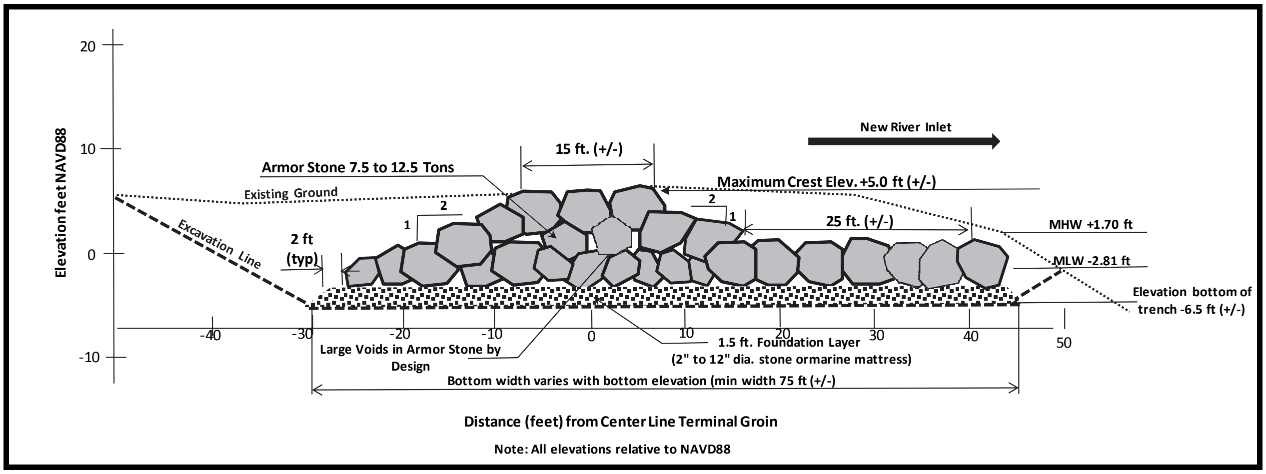


Figure 11. Typical terminal groin rubblemound schematic for Cross-Section AA-centerline stations 11+50 to 19+00.

All of the design features described above are preliminary and subject to change once the final design of the terminal groin is set.

Preliminary Cost Estimate: Initial construction of the terminal groin described above could range between $7 and $10 million. There would also be some future maintenance costs to repair the structure following severe coastal storms. In the absence of a detailed assessment of potential future damages, a preliminary estimate of the average annual cost for maintaining the terminal groin was estimated to be $20,000/year. This was based on the assumption an average of 0.5% of the armor stone would have to be replaced each year over the 30-year evaluation period. This does not mean maintenance would be needed every year only that over the full 30-year period, the total cost for maintenance would be approximately $600,000.

The final structural design of the terminal groin would be based on an optimization process that would minimize the average annual cost of the structure based on the initial construction cost (amortized over the 30-year evaluation period) and the expected future maintenance cost associated with storms that exceed the initial design conditions. In this regard, the initial design would be based on a range of design storm intensities (defined by a storm still water level and associated wave height) and the amount of future damage each initial design could experience based on the frequency future storms would exceed the initial design conditions. The final design would be based on the set of design conditions that result in the minimum average annual cost over the 30 year life of the project. That is, the total average annual cost for each design condition would be the sum of the initial cost amortized over a 30-year evaluation period plus the estimated average annual cost to maintain the structure determined from the frequency the design conditions could be exceeded over the 30-year evaluation period.

Construction of the terminal groin would have to be accompanied by a beach fill to initially create an accretion fillet south of the structure. A design for the accompanying beach fill has not been made, but based on similar terminal groin projects under consideration, the sources of sand available to the Town for the construction of the fill (ocean bar channel, Cedar Bush Cut, AIWW Crossing and DA-143) and whether the fill is placed in combination with construction of a beach nourishment project for a separate portion of the Town, the beach fill portion of the project could cost between $2 and $6 million which would push the total cost of the terminal groin option to between $9 and $16 million.

Please feel free to contact me with any questions or comments regarding the deliverables.

Very truly yours,

COASTAL PLANNING & ENGINEERING OF NORTH CAROLINA, INC.

Tom Jarrett, P.E.
Senior Vice President

cc: Carin Faulkner, Town of North Topsail Beach
Ken Willson, CPE-NC

Appendix B

2016 Channel Realignment Modeling

DRAFT

New River Inlet Channel Realignment Alternative Channel Modeling Study North Topsail Beach, NC

Coastal Planning & Engineering of North Carolina, Inc.

DRAFT

NEW RIVER INLET CHANNEL REALIGNMENT ALTERNATIVE CHANNEL MODELING STUDY NORTH TOPSAIL BEACH, NC

Prepared for:

Town of North Topsail Beach, North Carolina

Prepared by:

Coastal Planning & Engineering of North Carolina, Inc.

September 2016

NEW RIVER INLET CHANNEL REALIGNMENT ALTERNATIVE CHANNEL MODELING STUDY NORTH TOPSAIL BEACH, NC

EXECUTIVE SUMMARY

In July 2015, the Town authorized CPE-NC to conduct a numerical modeling study using the Delft3D morphological model to evaluate alternative channel alignments for the proposed 2016-2017 project. The Delft3D model was used as an engineering tool to evaluate relative differences in response of a system (beach and inlet) to channel modification. Due to the inability to predict weather and sea conditions well into the future, the Delft3D model is not a predictive model. Rather, the model results are only given in terms of the relative difference in the model's response to man-induced changes associated with a known set of input parameters such as tides, waves, and winds compared to modeled changes under "existing" conditions using the same set of input parameters.

In this study, model simulations of alternate channel alignments were used to evaluate model-indicated volumetric changes along the adjacent shorelines, changes in the ocean bar channel (channel orientation, shoal volumes, channel depths, etc.), volumetric changes on the ebb tide delta, and sediment transport patterns. The model results were also used to assess relative differences in flow patterns from one option to another, differences in the significant wave heights that would impact the shoreline, and potential changes in the volume of water that would pass through various channels within the system.

Initial simulations included a no action alternative, the 2013 channel configuration, Alternative 1, and Alternative 2. Following the initial simulations, Alternative 3, which involved pivoting the 2012/2013 channel clockwise 17^{o}, was simulated. These alternatives are listed and briefly described below. Furthermore, the 2013 channel with beach fill comparable to fill densities placed during the Phase 1 project as well as several variations of Alternatives 2 and 3 were simulated and assessed.

- The No Action Alternative simulation is one that does not dredge any channel at the ocean bar of the New River Inlet or place any fill along the Phase 1 project area.

- The 2013 Channel is 500 ft. wide with a bottom elevation of -18.0 ft. NAVD88. Simulation of the 2013 Channel Configuration did not include any beach fill placed along the Phase 1 project area.

- Alternative 1 is a channel oriented parallel to the 2013 Channel and situated approximately 850 ft. to the northeast of the 2013 Channel. The Alternative 1 channel configuration did not include any beach fill placed along the Phase 1 project area. The Alternative 1 channel is 500 ft. wide with a bottom elevation of -18.0 ft. NAVD88.

- Alternative 2 is a channel that has a landward orientation similar to Alternative 1 and also begins approximately 850 ft. northeast of the 2013 Channel. However, the channel curves toward the southwest and terminates in the vicinity of the seaward termination point for the 2013 Channel. The Alternative 2 channel did not include any beach fill placed along the Phase 1 project area. The Alternative 2 channel is 500 ft. wide with a bottom elevation of -18.0 ft. NAVD88.

- Alternative 3 is a channel that is in a similar location as the 2013 Channel; however, the orientation of the channel has been pivoted approximately 17° clockwise. The Alternative 3 channel configuration did not include any beach fill placed along the Phase 1 project area. Similar to Alternatives 1 and 2, the Alternative 3 channel is 500 ft. wide with a bottom elevation of -18.0 ft. NAVD88.

Based on the results of the Delft3D model analysis, the following are the primary findings of the study:

- Although Alternatives 1 and 2 have less direct impact on the dredging of the established southwest lobe of the ebb shoal, Alternative 3, the 250 ft. and 500 ft. shifted versions of Alternative 3, and the 2013 channel configurations result in a greater buildup of sand on the southwest lobe of the ebb shoal fronting North Topsail Beach.
- Model simulations show similar channel widths and depths after 2 years for Alternatives 1, 2, and 3. The secondary alternatives (shallower, narrower, and shallower and narrower versions of Alternative 2) resulted in greater shoaling of the simulated channels after 2 years.
- Alternative 3 and Alternative 3-shifted 250 ft. simulations resulted in the most favorable beach performance along the north end of North Topsail Beach between stations 1140+00 and 1160+00.
- High rates of erosion of the sand placed as part of the Phase 1 project along the north end of North Topsail Beach are due to the creation of a shoreline alignment out of equilibrium with existing conditions. The unnatural alignment of the shoreline was quickly reworked to resume an orientation similar to the pre-project orientation. This has been observed from numerous navigation maintenance projects during which disposal material placed on the north end of North Topsail Beach was reworked over the course of weeks to months until the shoreline alignment resumed a pre-project alignment. The most recent example of this was the project constructed in March and April 2016.
- All simulated channel alternatives show similar transport patterns through the channel and along the southeast lobe of the ebb shoal (Onslow Beach side). Simulated results of Alternative 3 and Alternative 3-shifted 250 ft. direct the transport in a more preferable location on the ebb shoal to promote the preferred reconfiguration. Simulated results of Alternative 3 and Alternative 3-shifted 250 ft. also show a reduced sediment transport gradient on the north end of North Topsail Beach which may result in a slowing of the sand transport to the spit on the north end of North Topsail Beach.
- Simulated sediment transport patterns and erosion/sedimentation patterns suggest most of the material filling in the channel was derived from adjacent shoals and the interior inlet system not the beach.
- The 2013 channel and Alternative 3 had the greatest reduction in significant wave height (H_s) north of Oyster Lane for wave case #8, which represents the wave case resulting in

the greatest sediment transport compared to all other representative wave conditions selected for the morphology simulations.
- None of the channel alternatives had a significant impact on the tidal prism of New River Inlet.

Based on the findings listed above, the geomorphic analysis conducted by CPE-NC as part of the EIS (2009), and the monitoring conducted by CPE-NC since the Phase 1 project was completed (CPE-NC, 2014; CPE-NC, 2016a and CPE-NC, 2016b), the following recommendations are provided to the Town of North Topsail Beach for its consideration:

- Alternative 3 is the recommended channel alternative for the next channel realignment project.
- Proceed with obtaining permit modifications to allow for the construction of Alternative 3 for the next scheduled channel realignment project.
- Run model simulations to identify an optimal beach fill for the north end for the next channel realignment project.
- Determine the effects of reducing the recommended channel depth and width as a contingency if project costs exceed available funds.

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NEW RIVER INLET CHANNEL REALIGNMENT ALTERNATIVE CHANNEL MODELING STUDY NORTH TOPSAIL BEACH, NC

1.0 INTRODUCTION

The Town of North Topsail Beach completed Phase 1 of its beach and inlet management plan in February 2013. Phase 1 included relocating the main bar channel of New River Inlet to a preferred position and alignment with deposition of the dredged material along approximately 7,730 feet of the Town's shoreline south of New River Inlet. The intent of the bar channel relocation was to induce a build-up of material on the south side of New River Inlet which would eventually result in accretion along the northern portion of the Town's shoreline. CPE-NC stated in the engineering report associated with the Final Environmental Impact Statement (EIS) that predictions of the actual time for the shoreline between Buildings #5 and #6 of Topsail Reefs and the south shoulder of New River Inlet (Stations 1140+00 to 1160+00) to respond to the new channel cannot be made with a high degree of certainty; however, significant accretion should occur within 5 years with full recovery occurring within 15 years following the channel relocation (USACE, 2009). These projections were based on the implementation of a channel maintenance program that would maintain the channel in the preferred location.

During the first two years following completion of Phase 1, fill placed along the northern portion of the project from baseline station 1130+00 north to the inlet experienced high rates of erosion. Essentially all of the fill material placed in this area eroded with most of the material being transported by natural processes to the north. A large portion of this naturally transported sand accreted on the southern shoreline of New River Inlet in the form of a sand spit. The erosion of the fill material placed seaward of the homes north of Topsail Reef during the Phase 1 project left these structures in imminent danger comparable to the conditions of the structures prior to the construction of the Phase 1 project. This prompted the Town of North Topsail Beach to construct a sand bag revetment to provide temporary erosion control along this section of shoreline.

In March 2015, CPE-NC provided the Town with a contingency report, which provided recommendations for modifications to the existing long-term inlet management strategy associated with the Town's long-term beach and inlet management program. One of the four (4) recommendations described in the Contingency Plan was the modification of the channel alignment for the 2nd channel realignment event scheduled for the 2016-17 environmental dredging window.

In July 2015, the Town authorized CPE-NC to conduct a numerical modeling study using the Delft3D morphological model to evaluate alternative channel alignments for the proposed 2016-2017 project. The Delft3D model was used as an engineering tool to evaluate relative differences in response of a system (beach and inlet) to channel modification. Due to the inability to predict weather and sea conditions well into the future, the Delft3D model is not used as a predictive model. Instead, the model results are only given in terms of the relative difference in the model's response to simulated channel alternatives compared to its response to the No Action Alternative (i.e., existing condition). In this regard, all of the channel alternatives as well as the No Action

Alternative were evaluated in the Delft3D model using the same set of input parameters such as tides, waves, and winds. Therefore, any change in the model's response to the simulated channels compared to the No Action Alternative are solely due to changes induced by the simulated channels.

1.1 Study Area

The study area is the New River Inlet and adjacent shorelines, located in Onslow County, North Carolina (Figure 1). New River Inlet is hydraulically connected to the Atlantic Intracoastal Waterway and the New River through Cedar Bush Cut. New River Inlet separates the Town of North Topsail Beach to the southwest and Onslow Beach to the northeast. Onslow Beach is controlled by the U.S. Marine Corps as part of M.C.S. Camp Lejeune. The modeling study primarily focused on the ocean bar channel and ebb shoal configuration as well as the northern portion of North Topsail Beach from US Army Corps of Engineers (USACE) baseline stations 1170+00 (north end parking lot) to 1130+00 (Bottlenose Blvd.).

1.1.1 Modeling Scenarios and Results

The results of the simulations of the No Action Alternative and the various channel alternatives in the Delft3D model were used to evaluate; (a) model-indicated volumetric changes along the adjacent shorelines, (b) changes in the ocean bar channel (channel orientation, shoal volumes, channel depths, etc.), (c) volumetric changes on the ebb tide delta, (d) sediment transport patterns (e) relative differences in flow patterns from one option to another, (f) differences in the significant wave heights that would impact the shoreline, and (g) potential changes in the volume of water that would pass through various channels within the system.

All simulations were run using the April 2015 beach and inlet bathymetry/topography as the initial condition. One-year and two-year simulations were run for each alternative. Initial simulations included a No Action Alternative, the 2013 channel configuration, Alternative 1, and Alternative 2. Following the initial simulations, Alternative 3, which involved pivoting the 2012/2013 channel clockwise 17^{o}, was simulated along with two variations of the pivoted channel located closer to the north end of North Topsail Beach. Figure 2 provides a visual comparison of the alternatives. These alternatives and their variations that were simulated are described below.

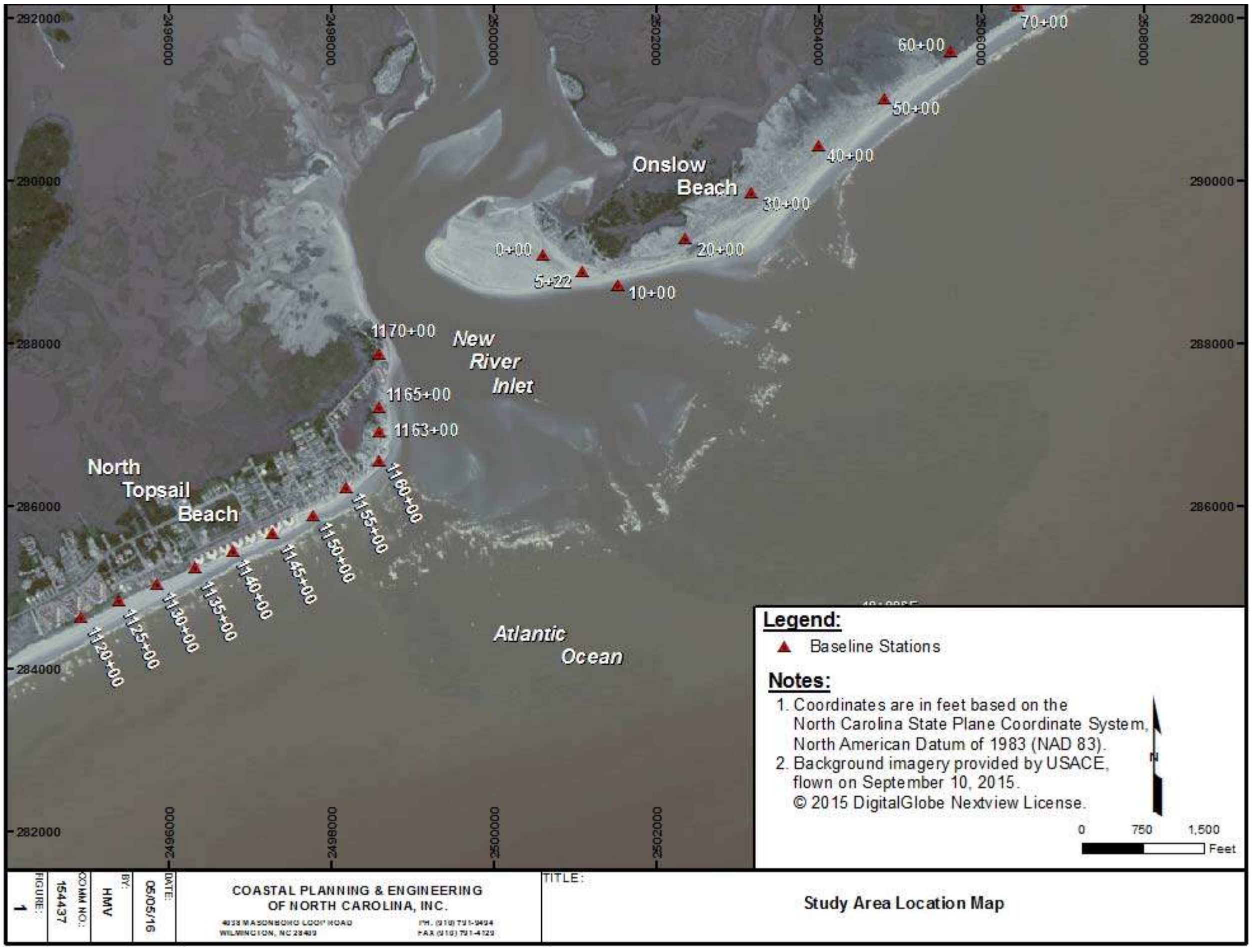

Figure 1: Study Area Location Map

No Action Alternative

The No Action Alternative simulation did not involve a dredged channel across the ocean bar of the New River Inlet or the placement of fill along the Phase 1 project area. This alternative is primarily used for comparison purposes to determine impacts of dredging a channel verses not dredging a channel. Figure 3 shows the initial bathymetry for the No Action Alternative.

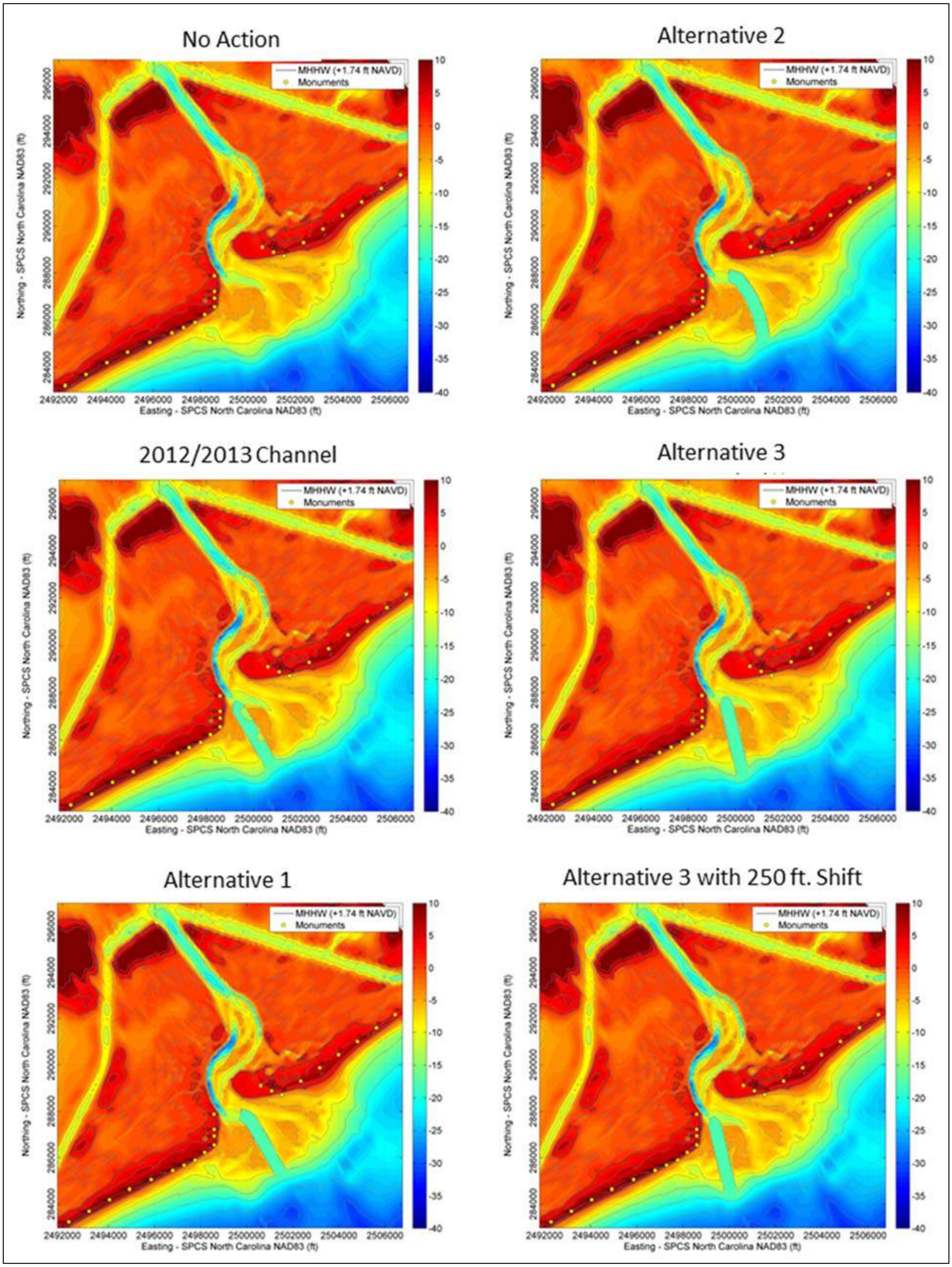

Figure 2: Initial Bathymetry for the tested Alternatives (Vertical datum is NAVD88)

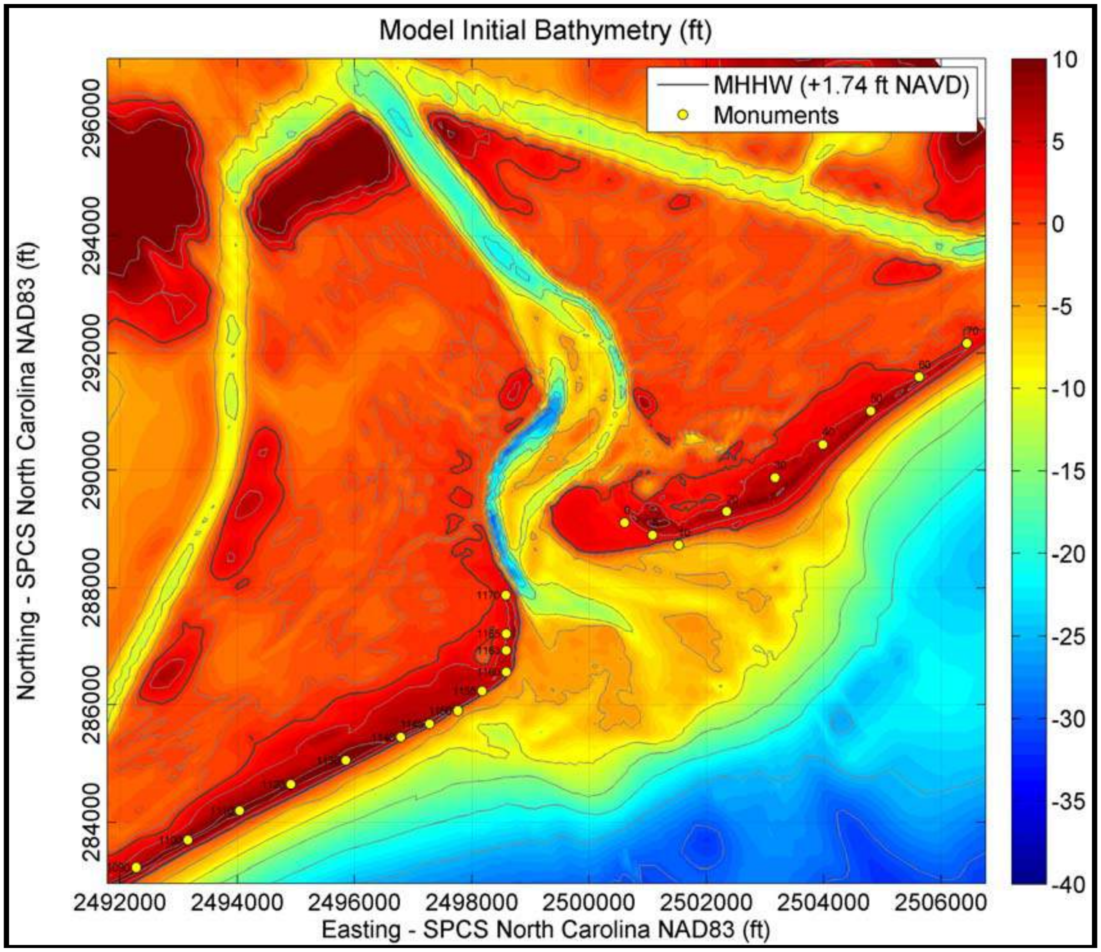

Figure 3: Initial Bathymetry for No Action Alternative (Vertical datum is NAVD88)

2013 Channel Configuration

The simulated 2013 Channel was 500 ft. wide and approximately 3,450 ft. long, with a bottom elevation of -18.0 ft. NAVD88. Simulation of the 2013 Channel did not include any beach fill placed along the Phase 1 project area. This alternative was simulated to provide a relative comparison of the differences in the inlet and shoreline responses for the other channel alternatives simulated in the model study, none of which included beach fill. Figure 4 shows the initial bathymetry for the 2013 Channel Alternative. Based on the April 2015 bathymetry, approximately 678,000 cy would need to be removed to construct this channel.

Simulation of the 2013 Channel without the beach fill resulted in very little volume change on the upper part of the beach profile, i.e., landward of the -6-foot NAVD88 contour in the area north of station 1130+00, which is located south of Building #8 of the Topsail Reef Condominiums. This modeled shoreline response differed from what was observed following the relocation of the inlet channel, as documented in the three monitoring reports for the Phase 1 project (CPE-NC, 2014; CPE-NC, 2016a & CPE-NC, 2016b), which indicated most of the fill placed north of station 1130+00 was lost during the first two years after placement. Therefore, a subsequent simulation of the 2013 Channel Alternative was run with a beach fill comparable to the fill placed during Phase 1.

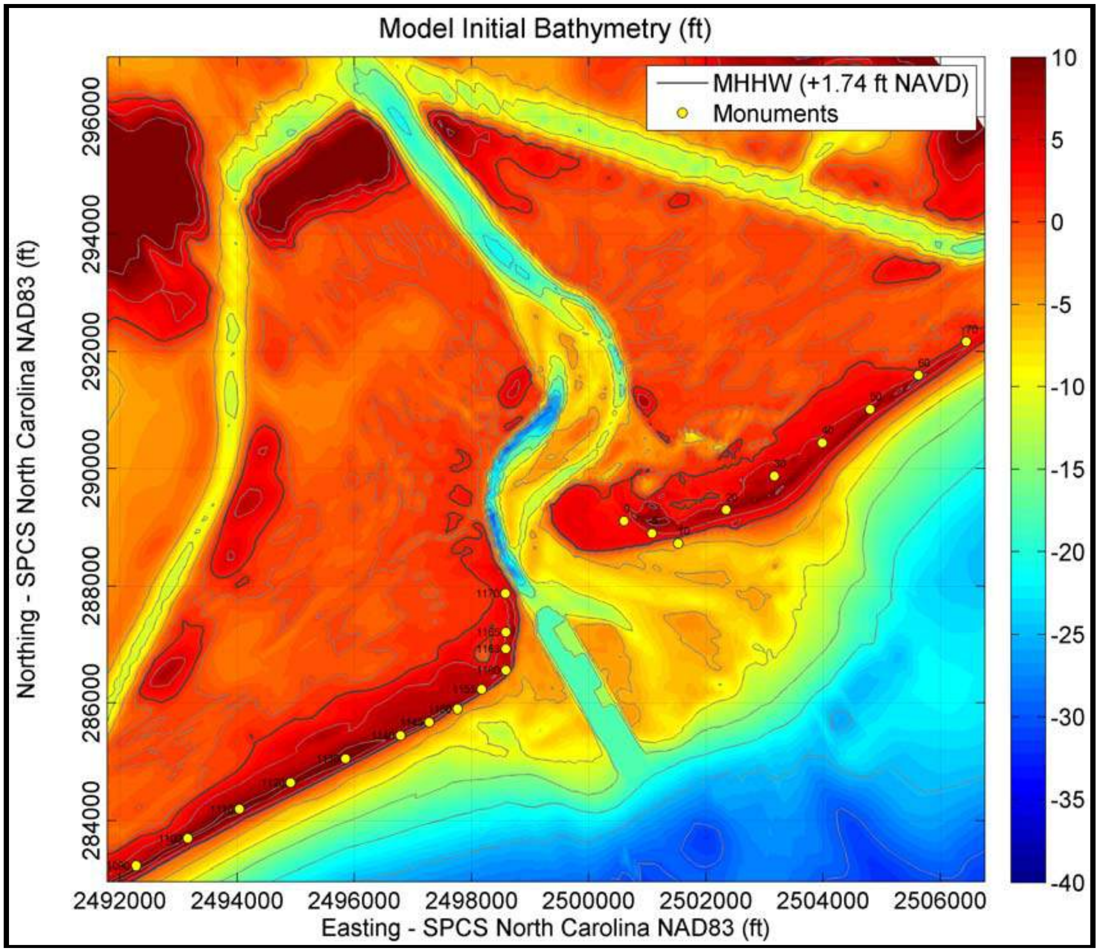

Figure 4: Initial Bathymetry for 2013 Channel Alternative (Vertical Datum is NAVD88)

The beach fill constructed during the 2013 channel relocation had an alignment that differed significantly from the natural shoreline. This is illustrated in Figure 5 in which the red-dashed line represents the shoreline alignment prior to the beach fill and the red solid line approximates the position and alignment of the shoreline immediately following placement of the fill. As is readily apparent, the beach fill created a seaward protuberance or bulge in the shoreline. During the April 13, 2016 Board of Alderman Special Meeting, Dr. William Cleary identified the alignment of the beach fill as one of the factors contributing to the rapid rate of loss of the beach fill.

The majority of the beach fill material that was eroded from the placement area appeared to be transported by natural processes to the north where it deposited on the south shoulder of New River Inlet in the form of an elongated sand spit (Figure 5). Some of the material that formed the sand spit was eventually used to construct the Town's sandbag revetment along the shoreline north of Topsail Reef. The sandbag revetment was authorized by North Carolina Division of Coastal Management and Dept. of the Army permits and constructed between December 13, 2014 and February 22, 2015.

Figure 5. Schematic showing post-construction shoreline orientation (solid red line) and natural shoreline (dashed red line).

In order to differentiate the shoreline response associated with the impacts of the 2013 Channel and the response due to the orientation of the shoreline following the placement of the fill, the following model simulations were conducted and shoreline changes were compared:

- No Action
- Phase 1 Beach Fill Only
- 2013 Channel Without Beach Fill
- 2013 Channel With Beach Fill

The volume changes along the north end of North Topsail Beach measured landward of the -6-foot NAVD88 contour for these simulations at the end of 2 years are shown on Figure 6. Volume changes were determined every 100 feet along the shoreline from bed level integration at initial and final simulation times using Matlab. For this evaluation, the volume changes above the -6-foot NAVD88 contour were assumed to provide a reasonable proxy of changes in the visible or dry sand beach.

Volumetric changes above the -6-foot NAVD88 contour along the north end of North Topsail Beach obtained for these simulations are provided in Figure 6. Apart from the impacts of the fill alignment and channel on shoreline volume changes, one of the significant results of these simulations was the relatively small volume losses produced by the model for simulations that did not include a beach fill. This indicates that the existing shoreline orientation (i.e., the shoreline without beach fill) has approached a state of equilibrium with this equilibrium state dictated by the existing offshore bathymetry and the effects the offshore bathymetry has on the magnitude and direction of wave energy that reaches the shoreline.

With regard to the impacts of the channel on shoreline changes, comparison of the model results for the Beach Fill Only simulation (blue line in Figure 6) with the results obtained for the 2013 Channel with the same beach fill (orange line in Figure 6) provides a direct measure of the channel impacts since both simulations included the beach fill. Therefore, any difference in the model results would have been due solely to the channel. As shown on Figure 6, the two-year volume losses from Stations 1160+00 to 1140+00 were less with the 2013 Channel compared to the Beach Fill Only alternative.

Results for the two simulations that did not include a beach fill, the No Action Alternative (black line in Figure 6) and the 2013 Channel (red line in Figure 6), indicated the 2013 Channel induced more accretion north of station 1145+00 compared to No Action.

Based on these model results, the primary cause of the rapid volume loss from the 2013 beach fill was the orientation of the shoreline created by the fill not the existence of the 2013 channel.

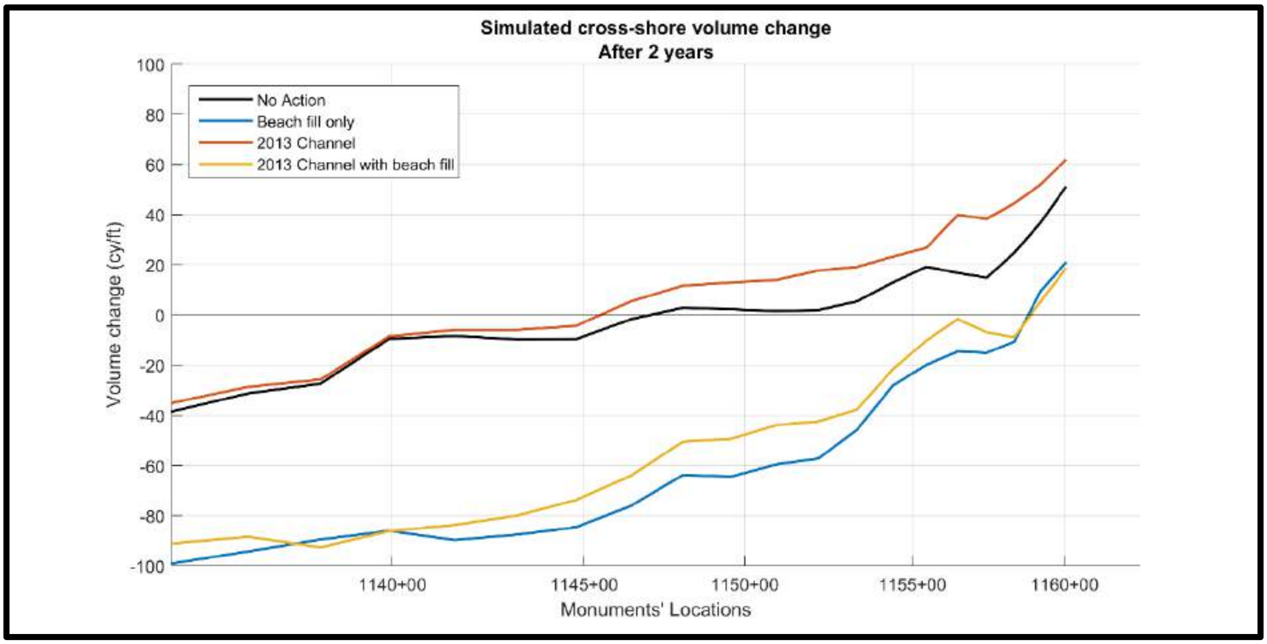

Figure 6. Cross-shore volume change for the 2012/2013 Channel without beach fill.

As further evidence supporting that the channel was not responsible for erosion of the beach fill, volumetric shoaling in the 2013 channel without the beach fill computed from the one-year model results totaled 229,000 cy, while shoaling in the channel with the beach fill totaled about 281,000 cy. This is 22.7% greater than the shoaling without the beach fill, which suggests that the beach fill is not the major source of the shoaled material; instead, sand is transported from areas other than the beach fill (i.e. adjacent shoals and the interior inlet system).

Alternative 1 – Parallel Channel

Alternative 1 is a channel oriented parallel to the 2013 Channel and situated approximately 850 ft. to the northeast of the 2013 Channel. The Alternative 1 channel configuration did not include any beach fill placed along the Phase 1 project area. The Alternative 1 channel is 500 ft. wide and approximately 3500 ft. long, with a bottom elevation of -18.0 ft. NAVD88. Figure 7 shows the initial bathymetry for Alternative 1. Based on the April 2015 bathymetry, approximately 677,000 cy would need to be removed to construct this channel.

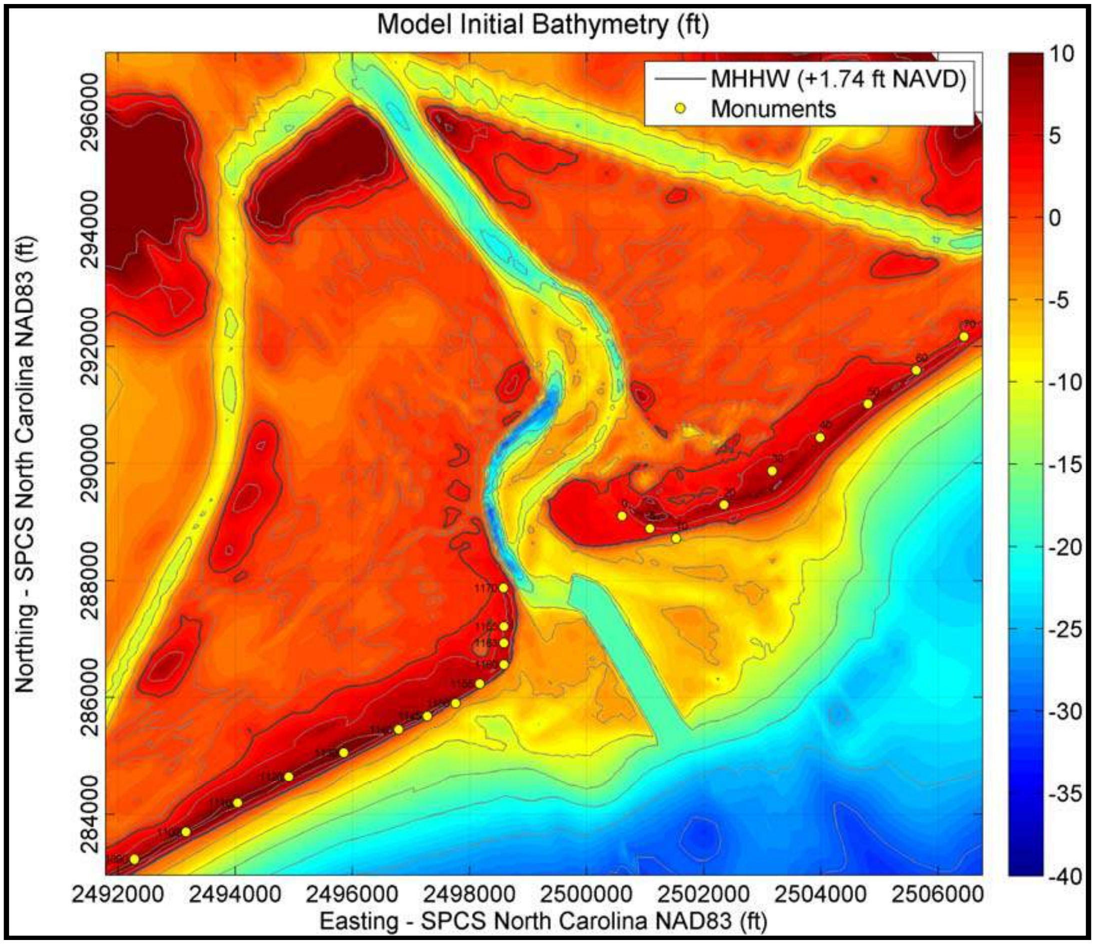

Figure 7: Initial Bathymetry for Alternative 1

Alternative 2 – Curved Channel

Alternative 2 is a channel that has a landward orientation similar to Alternative 1 and also begins approximately 850 ft. northeast of the 2013 Channel. However, the channel curves toward the southwest and terminates in the vicinity of the seaward termination point for the 2013 Channel. The Alternative 2 channel did not include any beach fill placed along the Phase 1 project area. The Alternative 2 channel is 500 ft. wide and approximately 3700 ft. long, with a bottom elevation of -18.0 ft. NAVD88. Figure 8 shows the initial bathymetry for Alternative 2. Based on the April 2015 bathymetry, approximately 681,000 cy would need to be removed to construct this channel.

Three modified versions of Alternative 2 were simulated; namely, a shallower channel (500 ft. wide at a depth of -15 ft. NAVD88), a narrower channel (400 ft. wide at a depth of -18 ft. NAVD88), and a narrower-shallower channel (400 ft. wide at a depth of -15 ft. NAVD88). These modified versions of Alternative 2 did not include beach fill placed along the Phase 1 project area.

The modified versions of the Alternative 2 channel did not produce the same volume build-up on the south side of the inlet as the original Alternative 2 channel. Also, the smaller channels experienced more rapid shoaling within the first year with depths over the outer portion of the ebb tide delta after one year being significantly less than the depths provided by the original Alternative 2 channel. As a result, no further consideration was given to the smaller version of the channel.

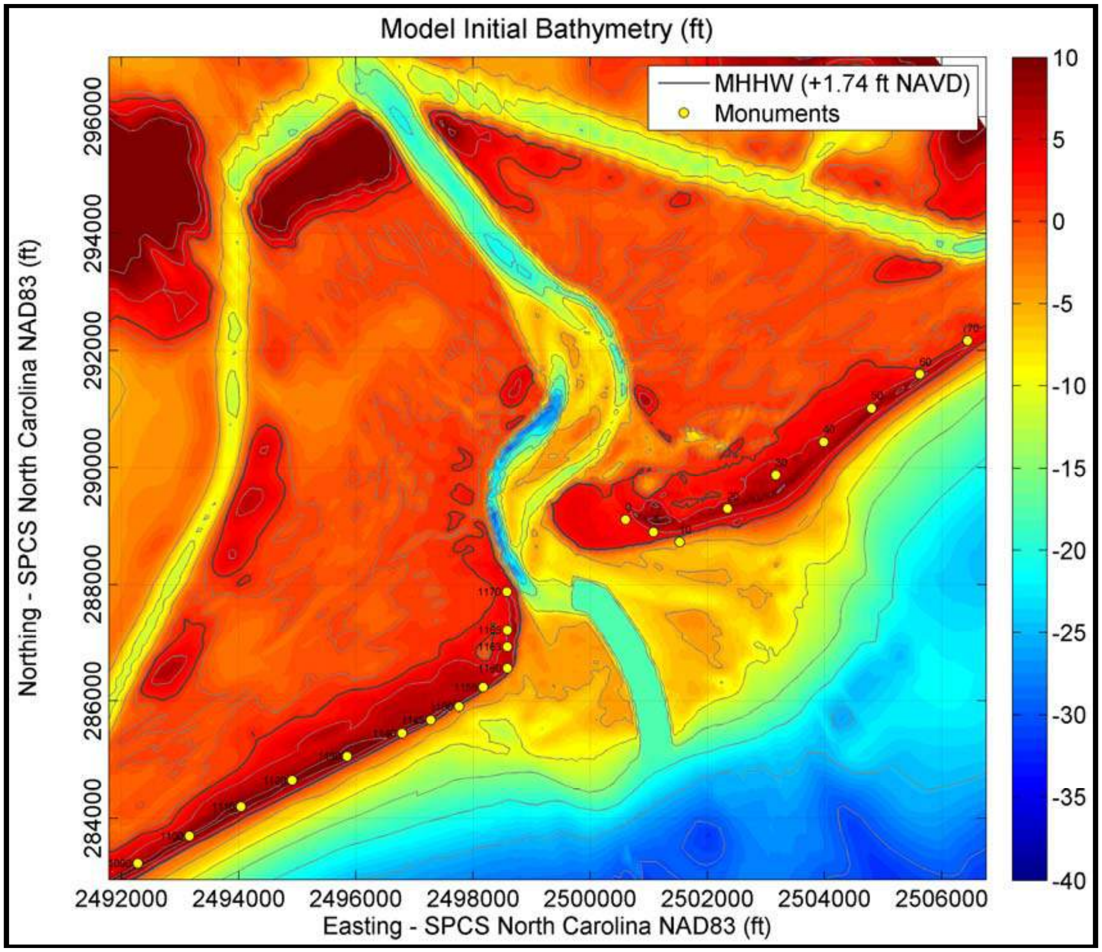

Figure 8: Initial Bathymetry for Alternative 2

Alternative 3 – Pivot Channel

Alternative 3 is a channel that is in a similar location as the 2013 Channel; however, the orientation of the channel has been pivoted approximately 17° clockwise. The Alternative 3 channel configuration did not include any beach fill placed along the Phase 1 project area. Similar to

Alternatives 1 and 2, the Alternative 3 channel is 500 ft. wide and approximately 3,450 ft. long, with a bottom elevation of -18.0 ft. NAVD88. Figure 9 shows the initial bathymetry for Alternative 3. Based on the April 2015 bathymetry, approximately 722,000 cy would need to be removed to construct this channel.

Two modified versions of Alternative 3 were simulated with both options maintaining the same orientation, width, and depth but with their positions shifted 250 ft. and 500 ft. to the west-southwest, respectively. These simulations were in response to comments made during the April 13, 2016 Board of Alderman Special Meeting, during which Dr. William Cleary suggested that the historic orientation of the 1987 channel was closer to the north end of North Topsail Beach than the proposed Alternative 3 (pivot) channel. The Board requested CPE-NC consider a channel that was closer to the 1987 alignment. These modified versions of Alternative 3 did not include beach fill placed along the Phase 1 project area.

Following initial model simulation for the modified pivot channel, the 500-ft shift toward the north end of North Topsail Beach was judged to be too close to development on the extreme north end of North Topsail Beach as the 500-ft shift resulted in higher volume losses off the extreme north end of the island compared to the 250-ft. shift. Therefore, the 500-ft shift was eliminated from detailed discussion in this report. The position of the 250-ft shift in the Pivot Channel relative to the pivoted channel for Alternative 3 is shown in Figure 10. Based on the April 2015 bathymetry, construction of the Alternative 3 Channel shifted 250 feet closer to North Topsail Beach would require the removal of approximately 724,000 cy of material.

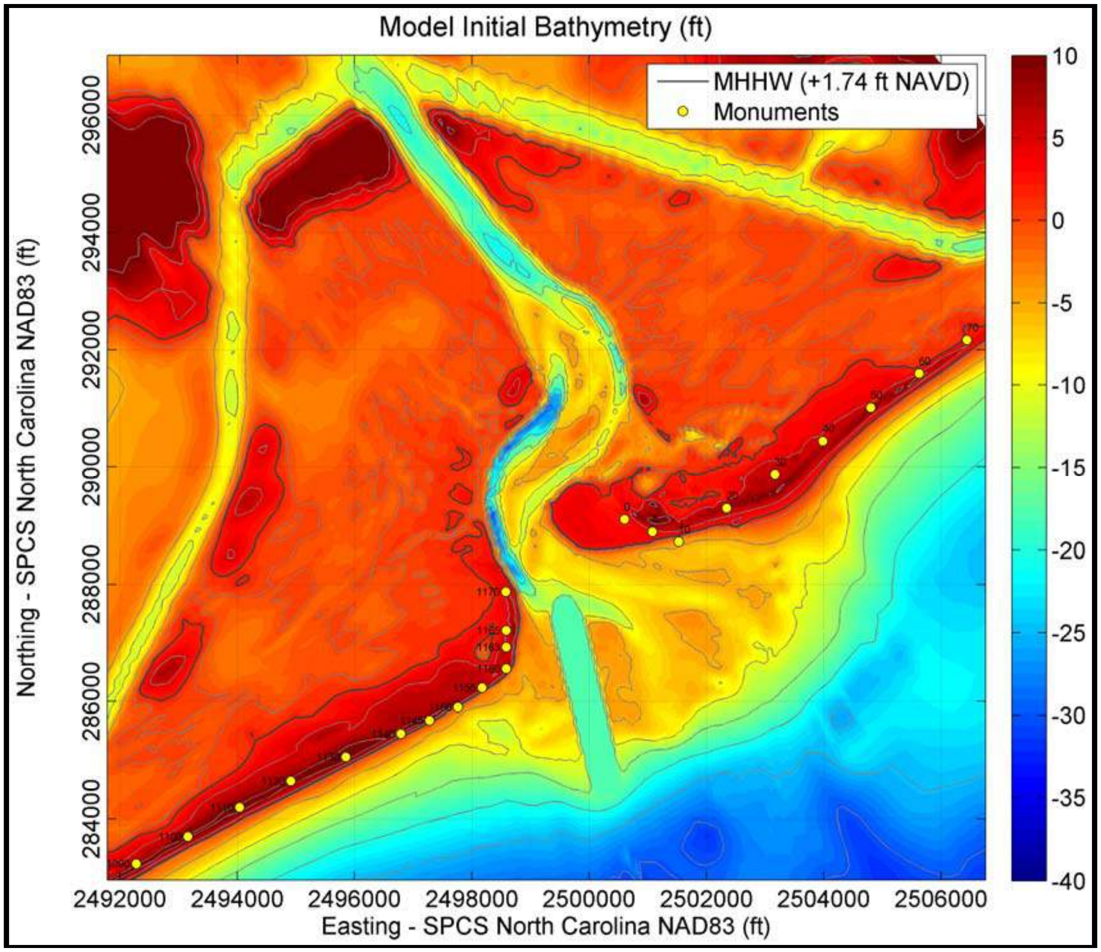

Figure 9: Initial Bathymetry for Alternative 3.

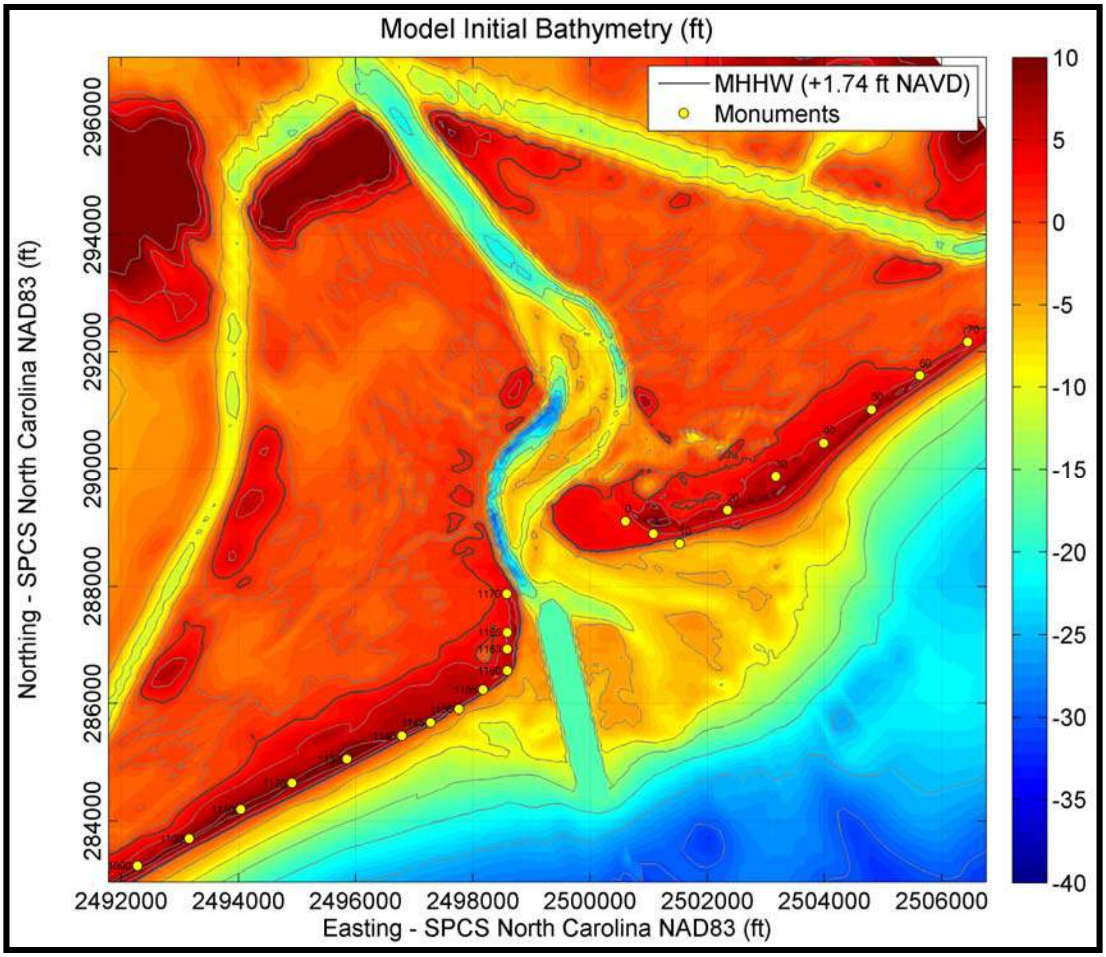

Figure 10: Initial Bathymetry for Alternative 3 with 250-foot west-southwest shift.

2.0 NUMERICAL MODELING METHODOLOGY

The primary modeling tool used in this investigation is the Delft3D morphological modeling package (Deltares, 2011). This package consists of two models, which are coupled together to determine changes in a topographic and bathymetric surface based on the effects of waves, water levels, winds, and currents.

Wave propagation from the offshore to the nearshore area is estimated using the Simulating Waves Nearshore Model (SWAN 40.72ABCDE, Delft University of Technology, 2008). Delft3D-FLOW utilizes the output waves from SWAN and varying water levels offshore to determine the resulting currents, water levels, sediment transport, erosion, and deposition. Based on the estimated erosion and deposition at each time step, the Delft3D-FLOW model calculates the subsequent elevations of the topographic and bathymetric surface and sends the updated bathymetry back to the SWAN model. Typical time steps in Delft3D-FLOW range from 1 second to 60 seconds, while wave propagation estimates in SWAN are performed every 1 to 3 hours. Given the interaction between waves and tidal currents near New River Inlet, Delft3D is among the best tools to evaluate how the various design alternatives perform and impact the adjacent beaches.

The modeling frame used to evaluate the performance of various engineering alternatives is built gradually, starting with the calibration/verification of the wave model against measured wave data. The wave and flow model are then coupled and its results compared with current and water level data measured during one month at different locations through the New River Inlet. Finally, the combined wave-flow model is applied to compute sediment transport and beach/inlet morphology changes with these results compared with measured erosion/sedimentation rates at the project site.

2.1 Model Grids

Five grids were created to evaluate wave propagation, flow, sediment transport, erosion, and deposition along the study area. These grids include the following:

- Regional Wave Grid, Intermediate Wave Grid and Local Wave Grid;
- Regional Flow Grid and Local Flow Grid.

Figure 11, Figure 12 and Figure 13 show the extents of the model grids, while grid characteristics are summarized in Table 1. Detailed descriptions of the various grids used in the model study are provided below.

The Regional Wave Grid was used to examine wave propagation from the open ocean to the nearshore region between Cape Fear and Cape Lookout. The offshore boundary of the Regional Wave Grid roughly follows the -500-foot depth contour. The Local Wave Grid was used to examine wave propagation from a depth of -30 feet NAVD88 to New River Inlet and the nearshore area along North Topsail Beach and Onslow Beach, extending approximately 30,000 feet northwest and southwest of New River Inlet. An Intermediate Wave Grid was used to facilitate the transition from the coarse Regional Wave Grid, that has a resolution of roughly 6,000 feet, to the finer Local Wave Grid, that has a resolution of approximately 100 feet in shallower areas; the resolution of the Intermediate Wave Grid is on the order of 1,000 feet.

The Regional Flow Grid was used to reproduce the hydrodynamics of the New River, capturing the tidal prism of the inlet. The grid extends from the coast to Jacksonville, NC, with horizontal resolution ranging from 25 to 890 feet. With the maximum resolution in the order of 50 feet, the Local Flow Grid was nested into the Regional Flow domain and is intended to simulate currents, sediment transport, erosion, and deposition within New River Inlet and the nearshore area along North Topsail Beach and Onslow Beach. The nesting approach is adopted to make the longer term morphology simulations computationally feasible. Except for the removal of grid lines along the northeastern and southwestern ends of the grid to facilitate a stable coupling between SWAN and Delft3D-FLOW, the Local Flow Grid is identical to the Local Wave Grid.

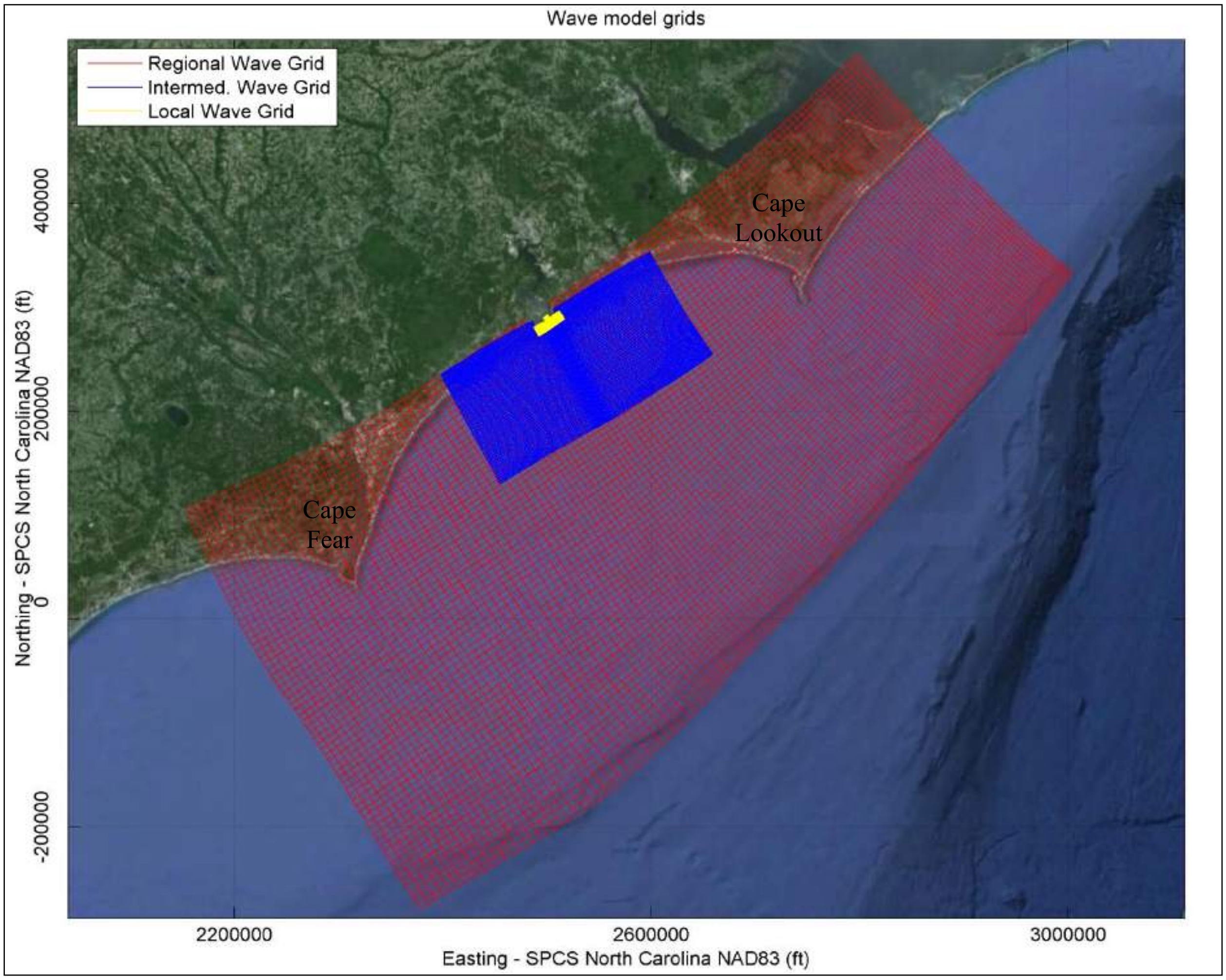

Figure 11. Wave Model Grids.

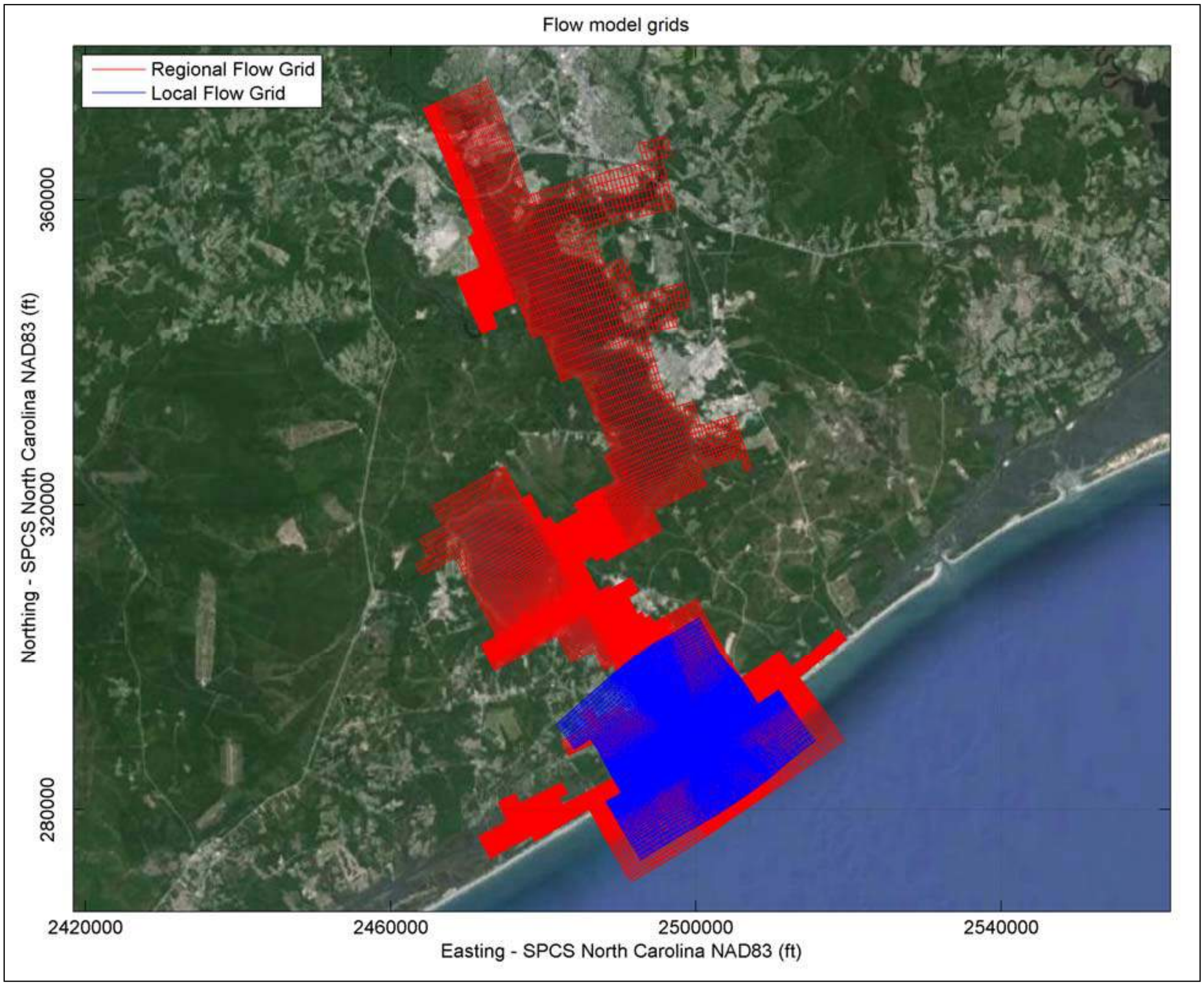

Figure 12. Flow Model Grids.

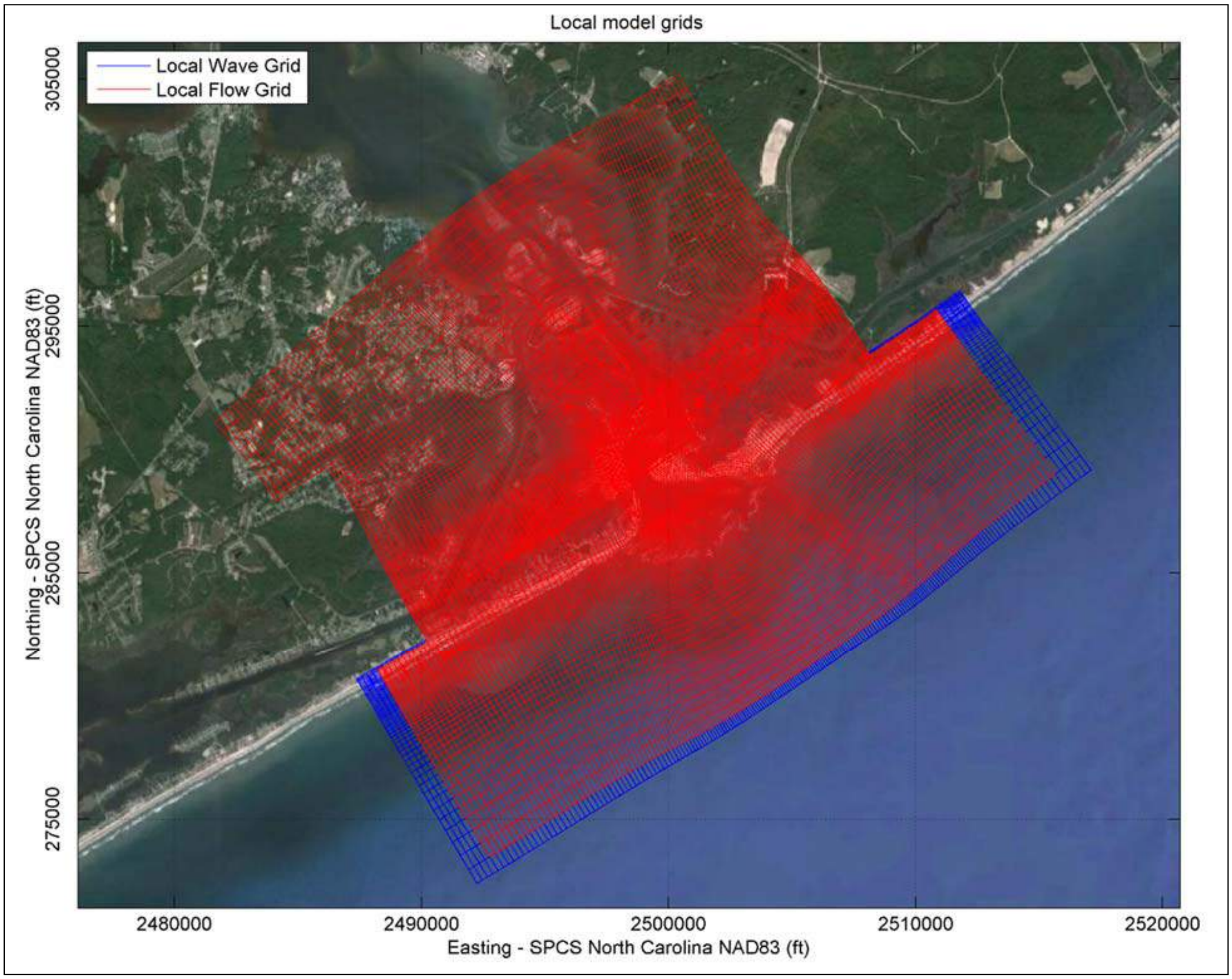

Figure 13. Local Model Grids.

Table 1. Model Grid Characteristics

	Number of Cells		Resolution (ft)		Orthogonality (°)		Smoothness	
Grid	**Longshore**	**Cross-shore**	**Min**	**Max**	**Min**	**Max**	**Min**	**Max**
Regional Wave	121	69	3989	8514	0.0	0.01	1.00	1.07
Intermed. Wave	165	76	928	1772	0.0	0.04	1.00	1.07
Local Wave	198	120	52	433	0.0	0.04	1.00	1.18
Regional Flow	322	400	26	889	0.0	0.03	1.00	1.16
Local Flow	189	161	52	436	0.0	0.04	1.00	1.20

The model grids generally follow the guidelines established by Deltares (2011) for smoothing and orthogonality. Smoothing values represent the change in cell size between adjacent grid cells. For example, a smoothing value of 1.1 indicates that the cell sizes between adjacent grid cells increase by 10%. The maximum smoothing value recommended by the model's developer is 1.2. Orthogonality is equivalent to the angle between the longshore and cross-shore grid lines, which should be at least 87.7° (cos $\alpha < 0.04$) within the area of interest. As shown in Table 1, all grids follow the guidelines for smoothing and orthogonality established by Deltares (2011).

2.2 Wave Modeling

2.2.1 Modeling Approach

The numerical wave model Delft3D-WAVE (SWAN) is applied to propagate wave conditions from the border of the continental shelf to the nearshore areas adjacent to the New River Inlet. In the nearshore areas, the wave model communicates and interacts with the flow/sediment transport model. Therefore, an appropriate representation of the wave regime along the study area is essential to reproduce the coastal processes that are the object of this study.

The Simulating Waves Nearshore Model (SWAN) was developed at Delft University of Technology (Netherlands) and can be used to simulate the evolution of random, short-crested wind-generated waves in coastal waters, estuaries, tidal inlets and lakes and has been validated and verified successfully in a range of complex laboratory and field experiments (Deltares, 2011).

The waves are described using the two-dimensional wave action density spectrum, even when non-linear phenomena dominate (e.g., in the surf zone). The model is capable of transforming offshore wave data into nearshore, taking into account processes such as wave refraction and diffraction due to the presence of shoals, channels or obstacles; wave generation by wind; wave dissipation by depth-induced breaking, white-capping and bottom friction; non-linear wave-wave interaction; and wave propagation through obstacles.

Inputs to the SWAN model include the bathymetry, the grid coordinates, the bottom friction factor, the wave breaking coefficients, the diffraction coefficients, the wind velocities, and the height, period, direction, and directional spreading values of each wave case. Outputs from the SWAN model include the wave height, wave peak period, and wave direction.

In order to access the consistency of the wave model developed for the current application, results of the computations are compared with wave data measured at three different locations along the modeled area in May 2012:

- NDBC 41036 (NDBC, 2015) - 30 miles offshore in 100 feet of water
- USACE 190 (USACE, 2015) - 3.8 miles offshore in 40 feet of water
- WHOI 09 (WHOI, 2015) - 1.0 miles offshore in 30 feet of water

These wave data are available through the National Data Buoy Center (NDBC), the USACE Field Research Facility (FRF), and the Woods Hole Oceanographic Institution (WHOI). During the wave model calibration period (May 2012, Figure 14) a wide range of distinct wave conditions were observed, representing a consistent sample of the wave climate adjacent to the study area. The conditions ranged from mild and energetic sea states (3 ft – 10 ft) with a prominent energy decay/dissipation observed across the continental shelf, while waves propagate from deeper to shallower waters. The narrow peak on the significant wave height (H_s) record observed after 05/30/2012 is associated to the passage of Tropical Storm Beryl that moved from SW to NE with its center being approximately 13 miles offshore North Topsail Beach on 05/30/2012. Over the one-month records, the peak wave period (T_p) varied from 3 to 11 seconds, encompassing both wind-sea and longer period swell waves. Waves approached the study area from different directions (PDir), with most conditions ranging from 60° (ENE) and 240° (WSW). Due to wave refraction effects, directional variability in the onshore gauges is lower.

Three computational grids are used to simulate wave propagation from deep water (i.e. Regional Wave Grid) to the nearshore area of interest (i.e. Local Wave Grid). A grid with intermediate coverage and resolution (i.e. Intermediate Wave Grid) is used to facilitate coupling between the regional and local domains (Section 2.1).

The bathymetric definition for the three computational grids was developed using data from different sources:

<u>Regional and Intermediate Wave Grids</u>:

1. 1927 Coastal Region Soundings (NCEI) between Topsail Beach and Onslow Beach (NOAA, 1927);
2. 1974 Coastal Region Soundings (NCEI) between Figure Eight Island and Onslow Beach (NOAA, 1974a);
3. 1974 Coastal Region Soundings (NCEI) between the New River Inlet and Atlantic Beach (NOAA, 1974b);
4. U.S. Coastal Relief Model (NCEI, 2015).

<u>Local Wave Grid:</u>

1. May 2012 LARC hydrographic data collected in New River Inlet (USACE, 2012a);
2. June 2012 Phase 1 pre-construction profile data collected for North Topsail Beach (CPE-NC, 2012b).
3. October 2012 Phase 1 pre-construction profile data collected for Onslow Beach (Gahagan & Bryant Associates, 2012).
4. 2014 Atlantic Coast LIDAR (NOAA, 2014).
5. January 2013 hydrographic data collected in New River Inlet (USACE, 2013a);
6. August 2005 hydrographic data collected in New River Inlet (CPE-NC, 2005);
7. August 2012 profile data collected for North Topsail and Onslow Beach (CPE-NC, 2012a);
8. 1927 Coastal Region Soundings (NCEI) between Topsail Beach and Onslow Beach (NOAA, 1927);
9. U.S. Coastal Relief Model (NCEI, 2015).

The vertical datum of the sources was converted to NAVD88 and interpolated to the computational points of the grids. The resulting bathymetry surfaces are given in Figure 15, Figure 16 and Figure 17.

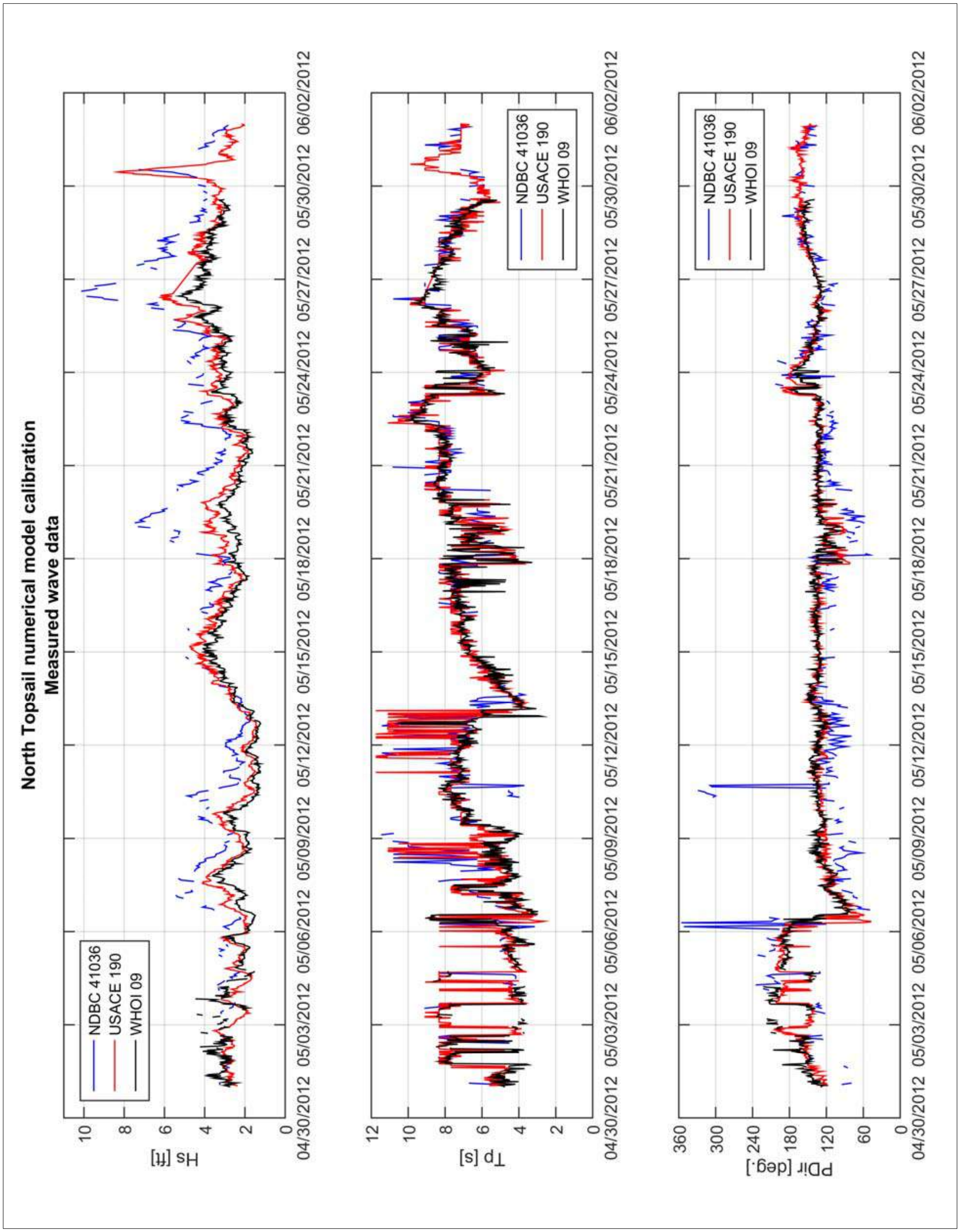

Figure 14. Measure wave data records used in the wave model calibration.

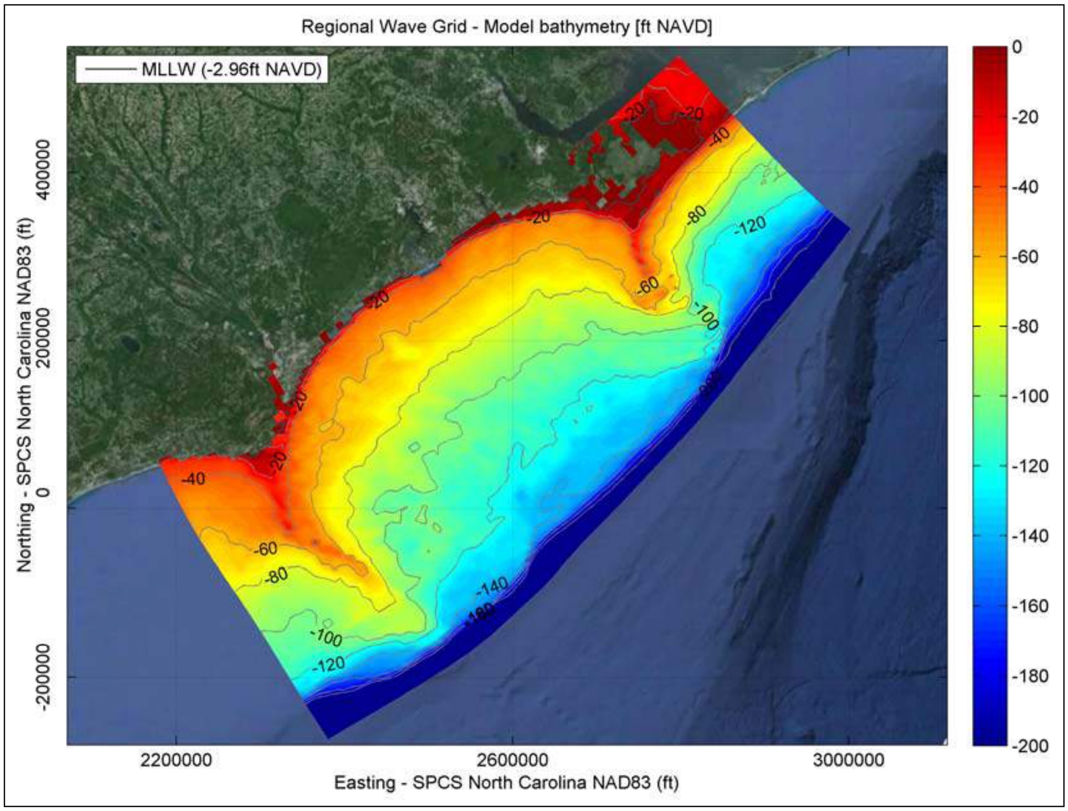

Figure 15. Regional Wave Model bathymetry.

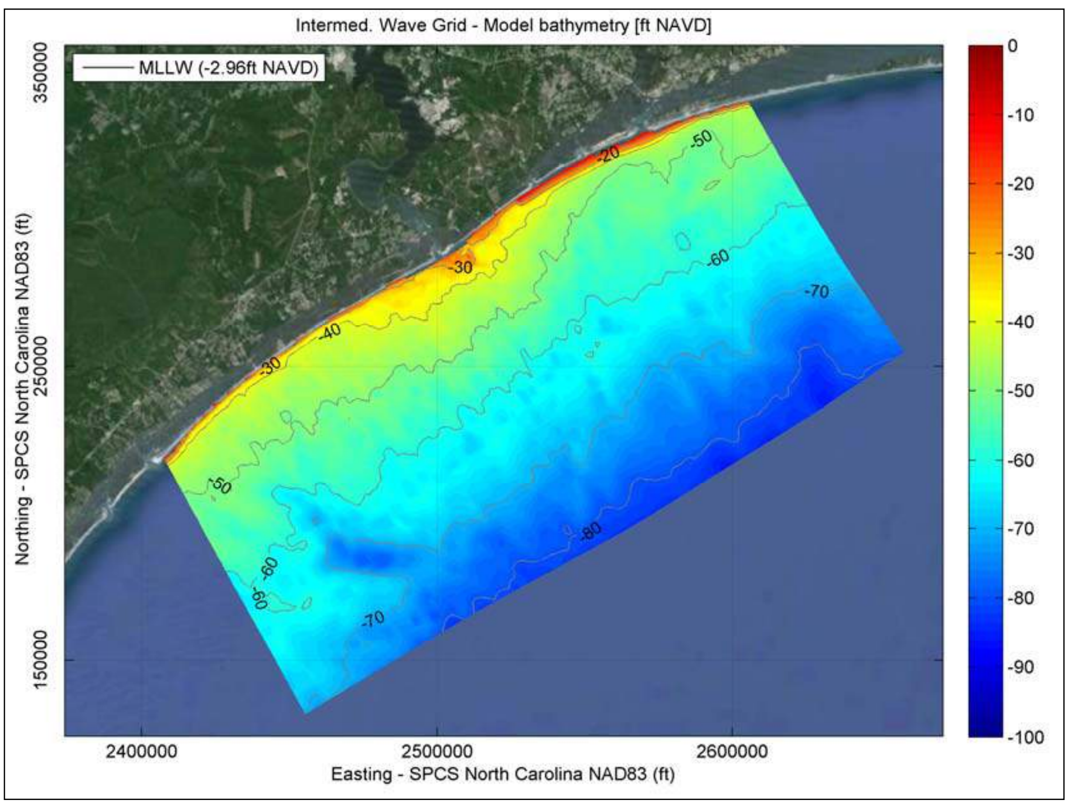

Figure 16. Intermediate Wave Model bathymetry.

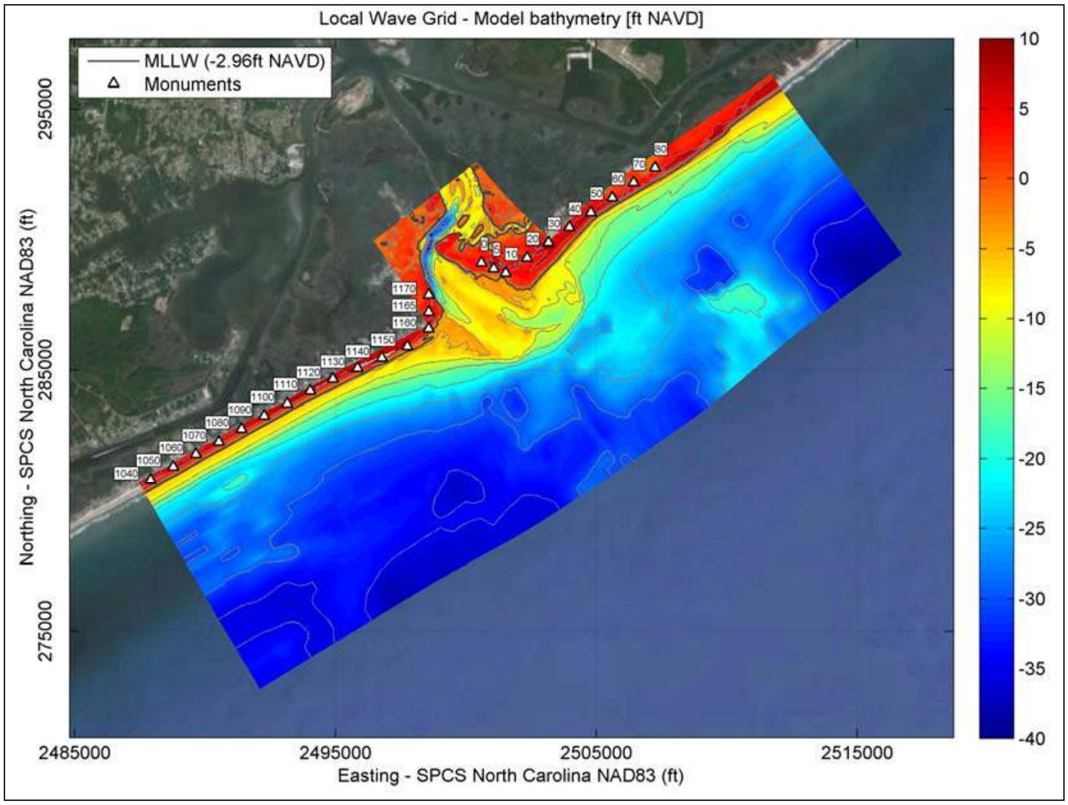

Figure 17. Local Wave Model bathymetry.

Time-varying wave conditions were imposed at the boundaries of the Regional Wave Grid. The boundary definitions for the Intermediate and Local Wave Grids are defined based on model results for the immediately larger domain (i.e. Regional and Intermediated Wave Grids, respectively). Different wave data sources were tested during the calibration of the wave model. This subject is covered in the next section. In addition to the regional boundary conditions, time- and space-varying wind fields were provided for the whole modeled area in order to account for local generation of waves.

2.2.2 Calibration

The calibration of the wave model consisted of testing different offshore wave boundary data. Boundary conditions were defined using parametric hindcast wave data such as significant wave height (H_s), peak wave period (T_p), and peak wave direction (PDir). These wave data were available through USACE Wave Information Studies (WIS, 2015) and NOAA Wavewatch III (WW3, 2015). WIS data for station number 63507 was used, located at 33° 45’ 0” N, 76° 35’ 2.4” W, at a depth of approximately 500 feet, available with a temporal resolution of 1 hour. WW3 data were obtained for the same location as the WIS point, with a temporal resolution of 3 hours. The wave model was calibrated by comparing model results with measured wave data from USACE 190, located approximately 3.8 miles offshore in approximately 40 feet of water and WHOI 09, located approximately 1.0 mile offshore in approximately 30 ft of water. The model grid boundaries and wave measurement locations are shown in Figure 18 and Figure 19.

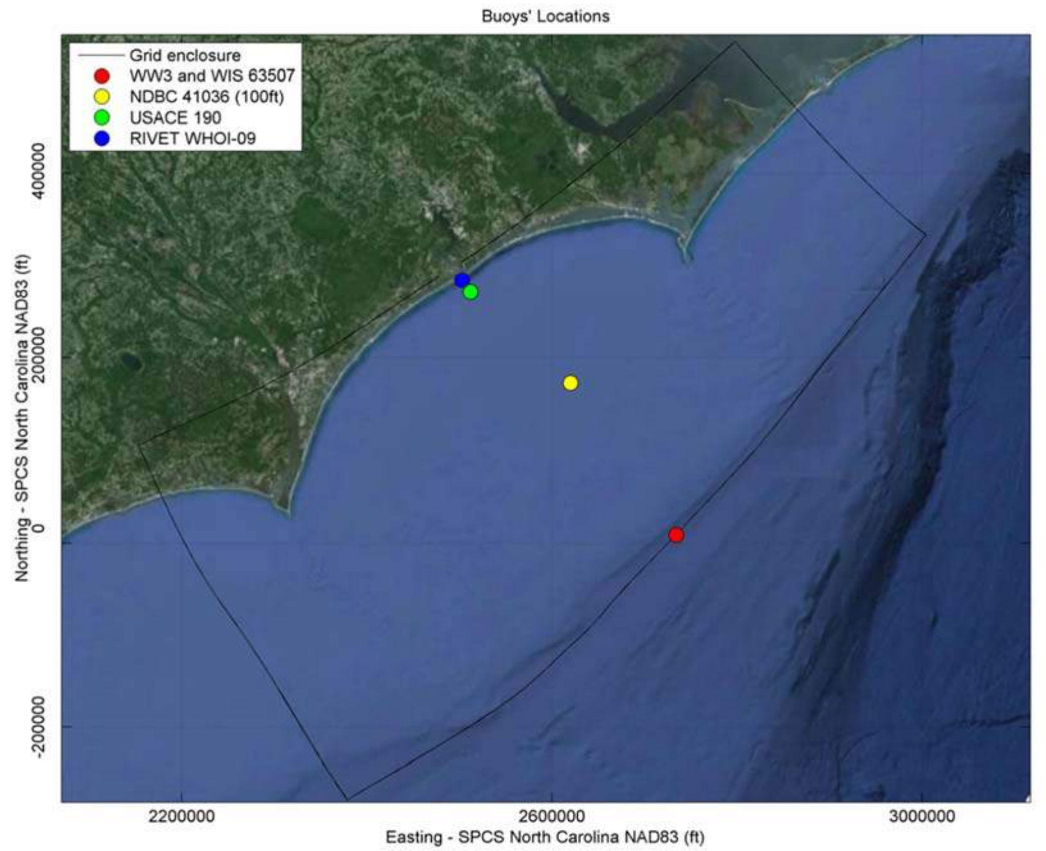

Figure 18. Regional wave grid boundary and wave data locations.

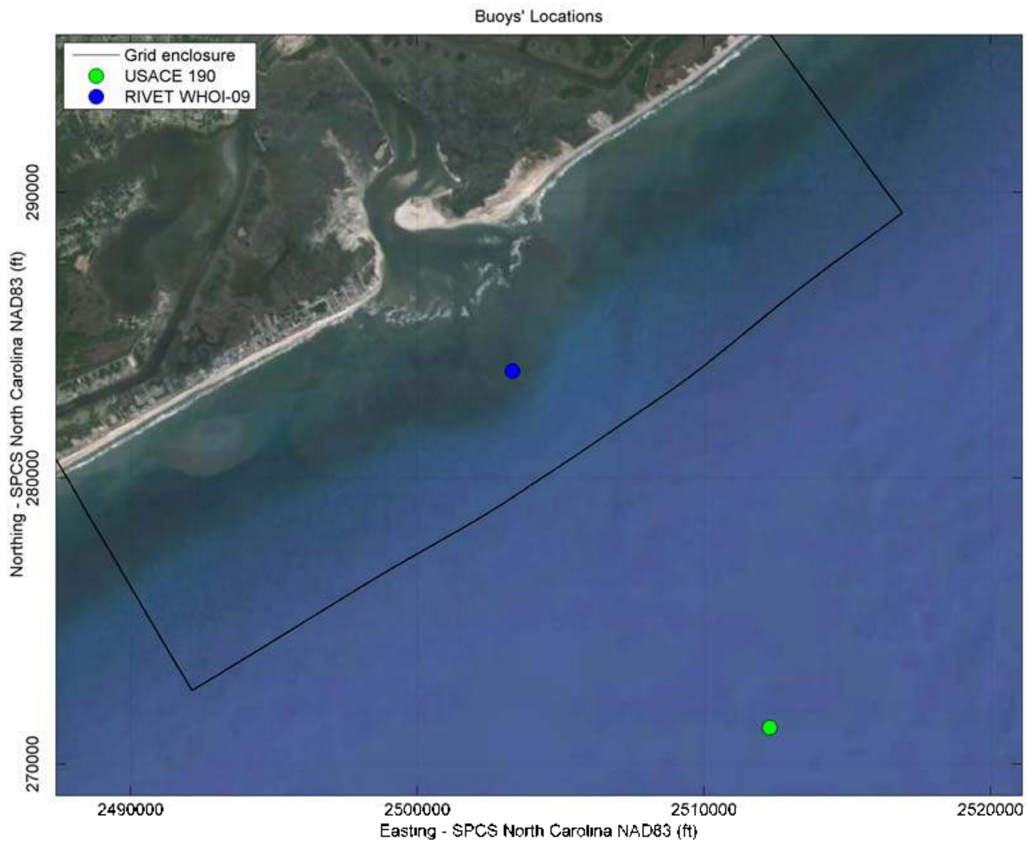

Figure 19. Intermediate wave grid boundary and wave data locations.

Spatially varying wind data used in the wave calibration was defined using the NCEP North American Regional Reanalysis (NARR, 2015). The NARR project is NCEP's high resolution combined model and assimilated dataset, covering 1979 to near present and is provided 8-times daily. Example of wind fields are provided in Figure 20.

Figure 21 presents a comparison of the time series of wave parameters associated with the WIS and WW3 databases for the calibration period (May 2012), and indicate good correspondence between the two sources.

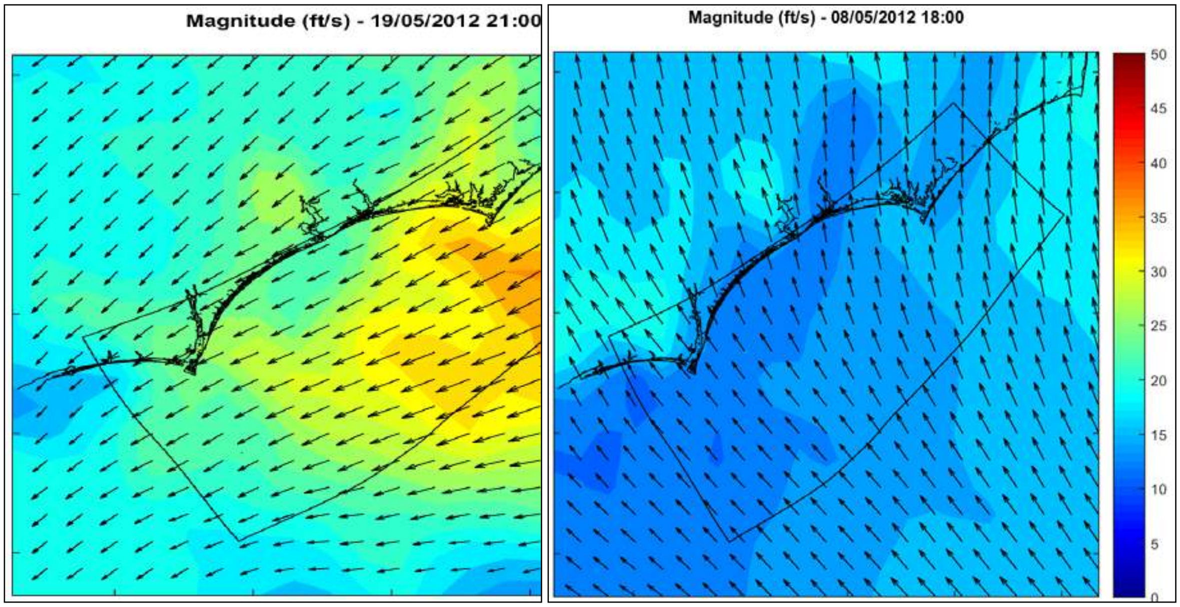

Figure 20. Example of NOAA/NARR wind fields along the study area. The rectangle contour indicates the extent of the Regional Wave Grid.

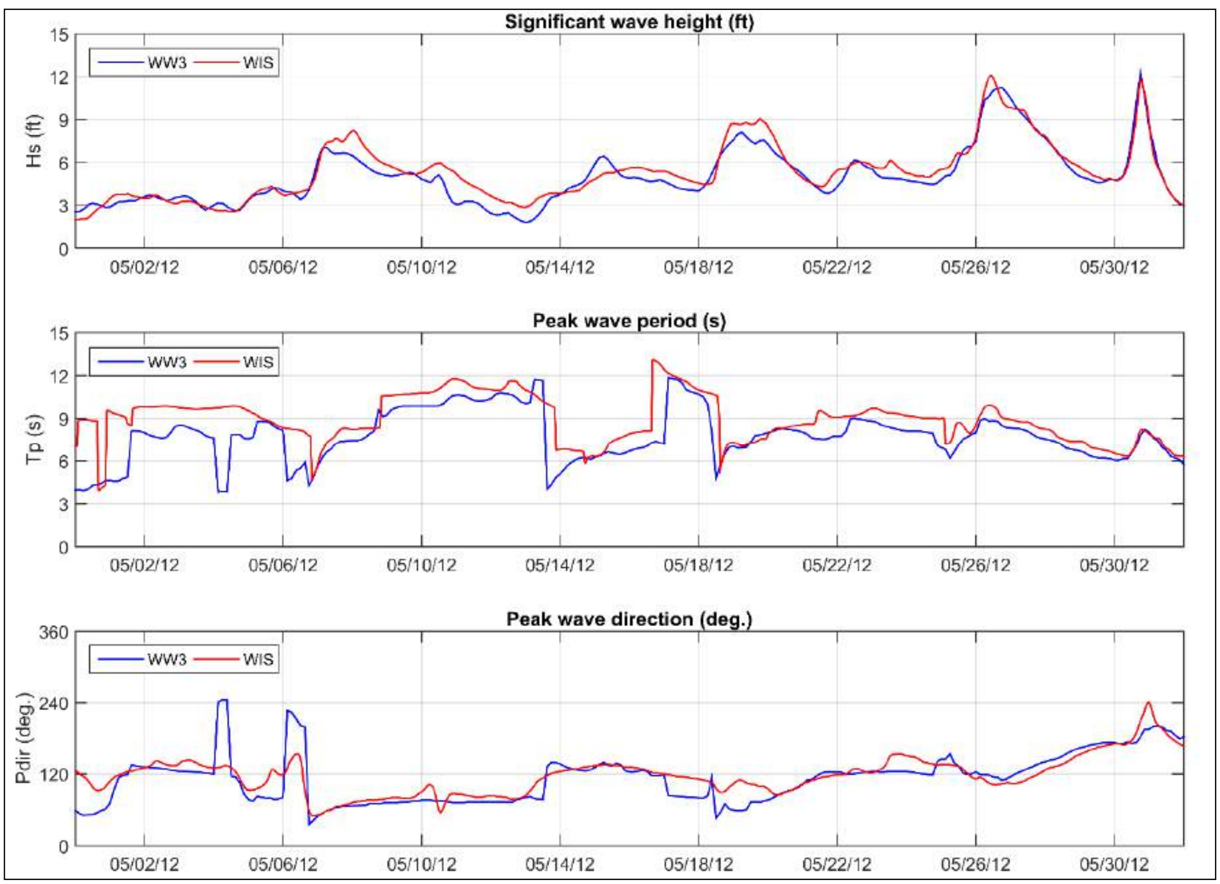

Figure 21. Comparison of NOAA Wavewatch III and USACE WIS offshore parametric wave data during the calibration period.

Comparisons of measured and simulated wave parameters for the deeper point (NDBC 41036, 30 miles offshore and in 100 feet of water) and nearshore locations (USACE 190 - 3.8 miles offshore/40 feet of water; WHOI 09 – 1.0 mile offshore/30 ft of water) are given in Figure 22, Figure 23 and Figure 24, respectively. These correspond to the simulations using the NOAA/WW3 parametric wave data along the Regional Wave Grid boundaries, which resulted in a slightly better correlation with the measured data relative to the results obtained using the WIS wave data. The percent value provided in the figures next to the RMSE value is the RMSE value divided by the range of variation in the measured data, providing a measure of the error with respect to the natural variability of the analyzed parameter.

Figure 22, Figure 23 and Figure 24 indicate that the wave model is able to reproduce the significant wave height (H_s) decay while waves propagate from deep to shallow waters. This effect is mainly related to wave energy dissipation due to bottom friction effects along the relatively wide and shallow continental shelf, which is properly handled by the Delft3D-WAVE/SWAN model. Waves generated by Tropical Storm Beryl – narrow peak with duration of approx. 3 hours, observed on 05/30/2012 – were reproduced by the model at the outer point (NDBC 41036), but underestimated in the nearshore location USACE 190. The differences between measured and simulated wave data at station ‘USACE 190’ is likely attributed to limitations of the wind database, which is known to underestimate small spatial scale cyclonic wind speeds. Comparisons of measured and modeled wave data during TS Beryl at the nearshore location WHOI 09 are not possible as the equipment was retrieved from the sea one day before the passage of the storm (Figure 14).

The peak wave period and peak wave direction are reasonably well reproduced during the simulated month. It is highlighted that especially for multi-modal wave spectra (e.g. superposition of swell wave condition(s) and wind sea), relatively small deviations in the wave energy distribution between the sea states may result in the selection of a different peak and therefore large differences in peak wave parameters. Such deviation does not necessarily mean, however, that the overall sea state is misrepresented.

Table 2 summarizes the configuration of the Delft3D-WAVE (SWAN) model. It should be noted that all default model parameters were employed except for the JONSWAP Peak Enhancement Factor, which was calculated using NDBC 41036 wave data collected between 2006 and 2014. Example of the wave model results at regional and local scales are provided in Figure 25 and Figure 26.

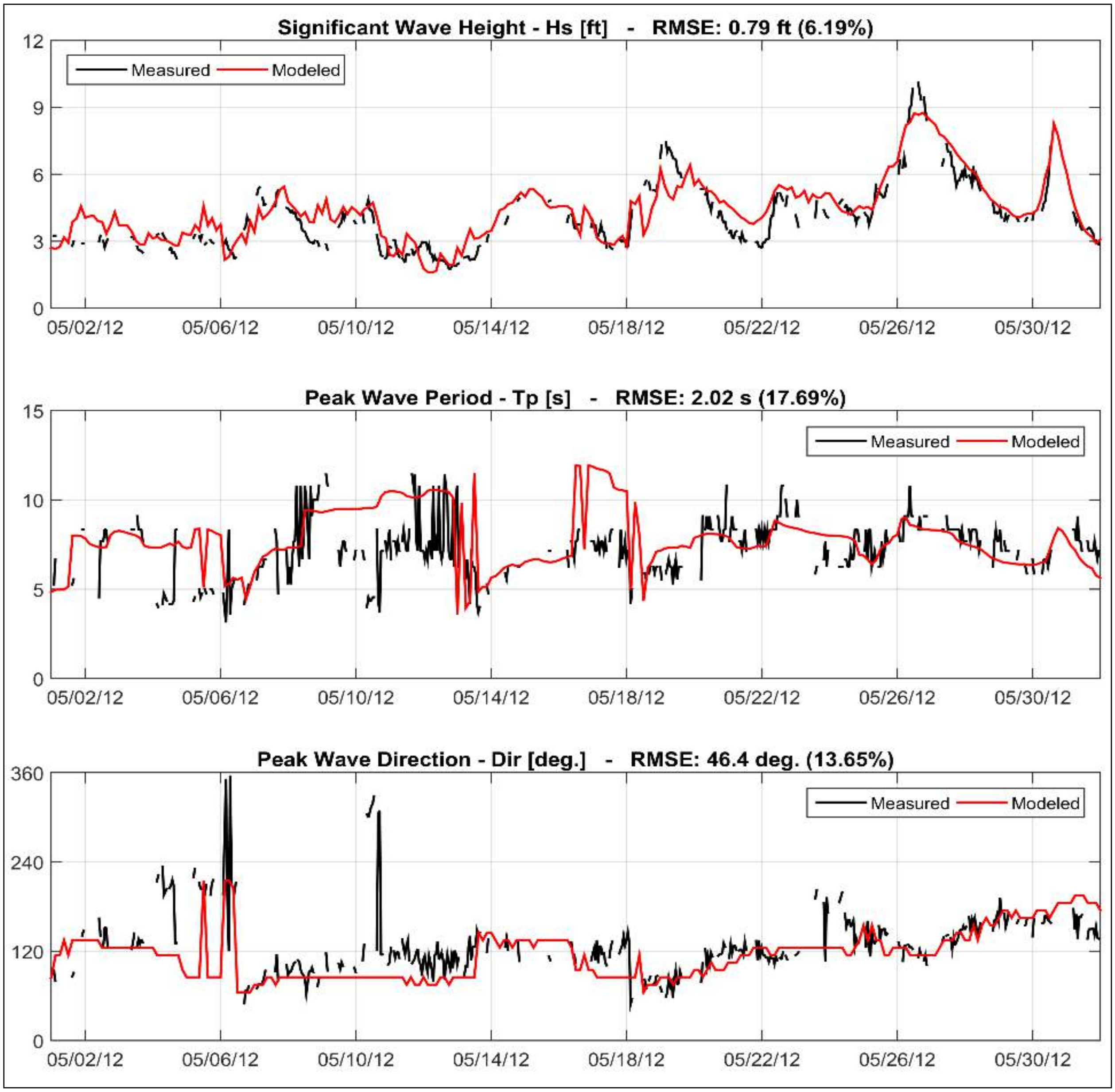

Figure 22. Comparison of measured and modeled wave data: NDBC 41036 - 30 miles offshore/100 feet of water. Offshore boundary condition: NOAA/WW3 parametric wave data.

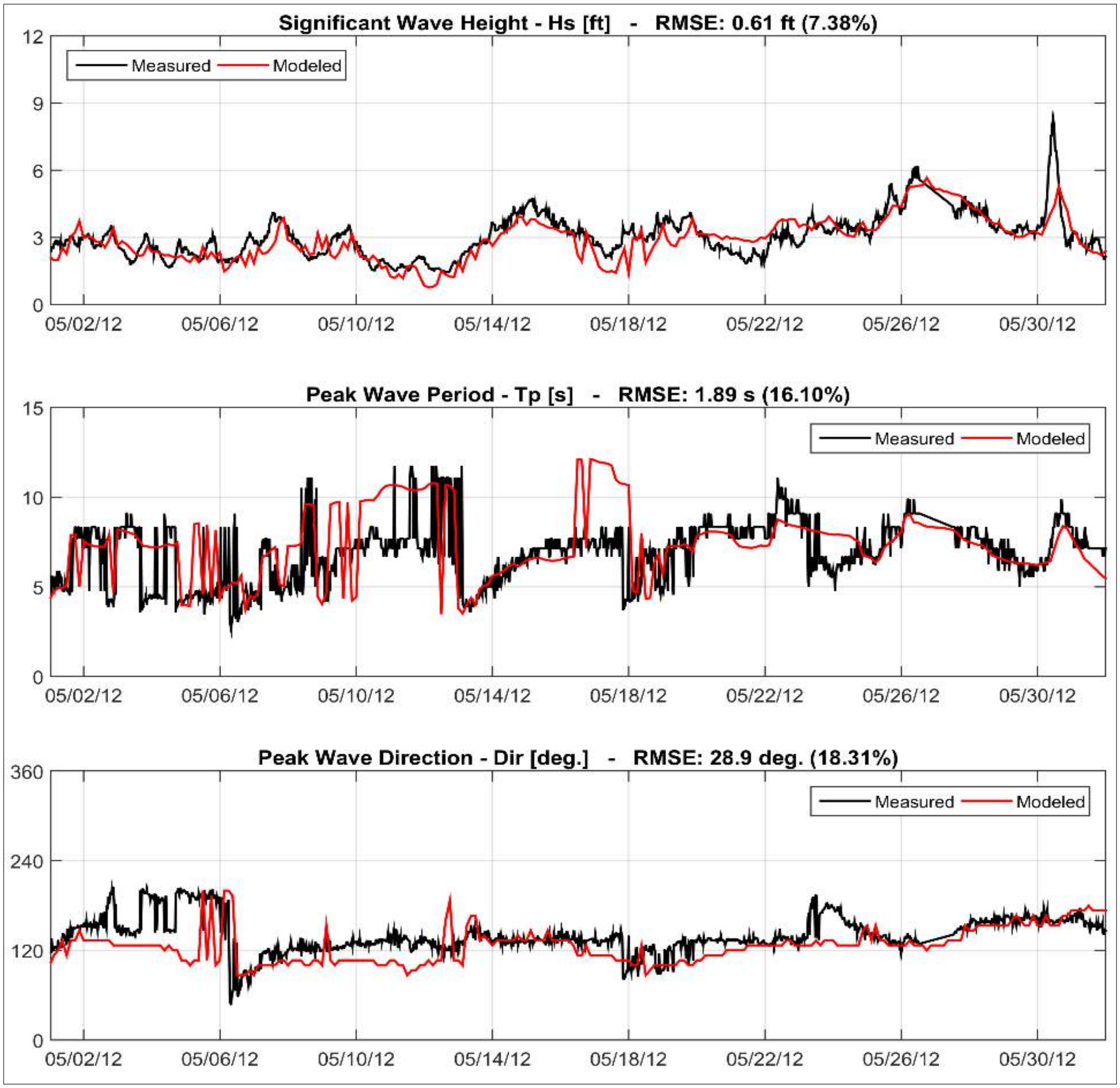

Figure 23. Comparison of measured and modeled wave data: USACE 190 - 3.8 miles offshore/40 feet of water. Offshore boundary condition: NOAA/WW3 parametric wave data.

Table 2. Summary of the Delft3D-WAVE (SWAN) model parameters.

SWAN Wave Transformation Model Parameters:	Min.	Default	Max.	Best Run Parameters
Breaking Parameter Gamma (Hb/db)	0.55	0.73	1.2	0.73
Breaking Parameter Alpha	0.1	1	10	1
JONSWAP Friction Value (m^2/s^3)	0	0.067	None	0.067
Triads - Energy Transfer from low to high frequencies in shallow water	-N/A-	Off	-N/A-	Off
Diffraction:	-N/A-	Off	-N/A-	Off
Diffraction Smoothing Coefficient	0	0.2	1	0.2
Diffraction Smoothing Steps	1	5	999	5
Wind Growth	-N/A-	On	-N/A-	On
JONSWAP Peak Enhancement Factor (for input parametric wave conditions)	-N/A-	3.3	-N/A-	1.6

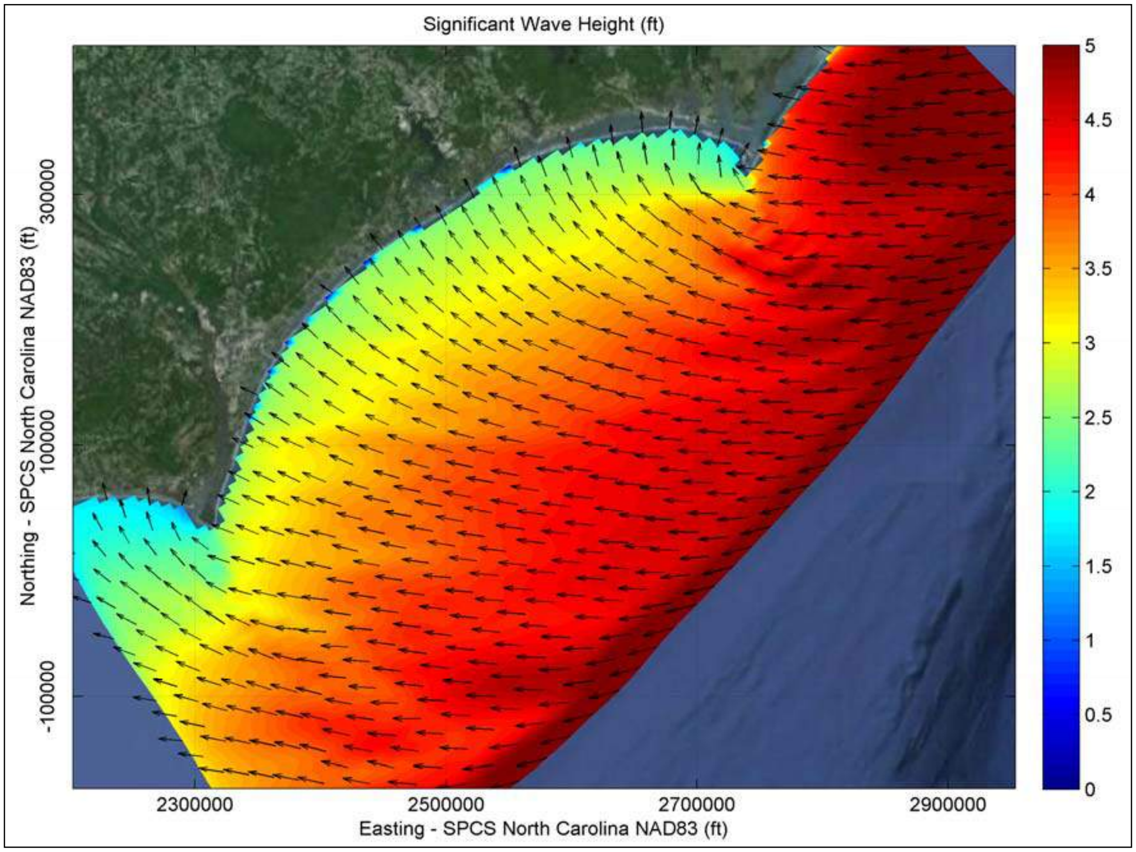

Figure 25. Example of Regional Wave model result.

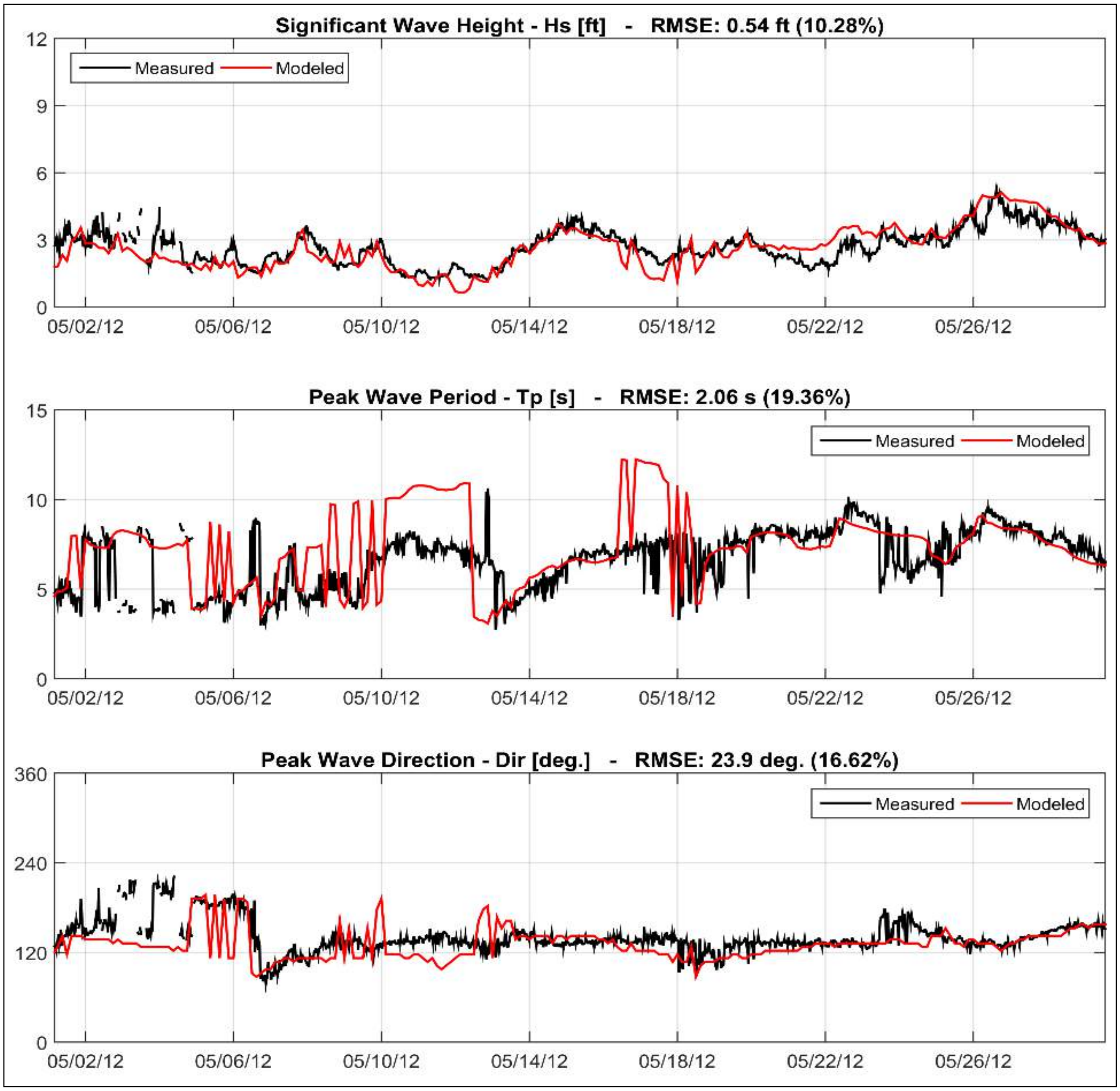

Figure 24. Comparison of measured and modeled wave data: WHOI 09 – 1.0 mile offshore/30 feet of water. Offshore boundary condition: NOAA/WW3 parametric wave data.

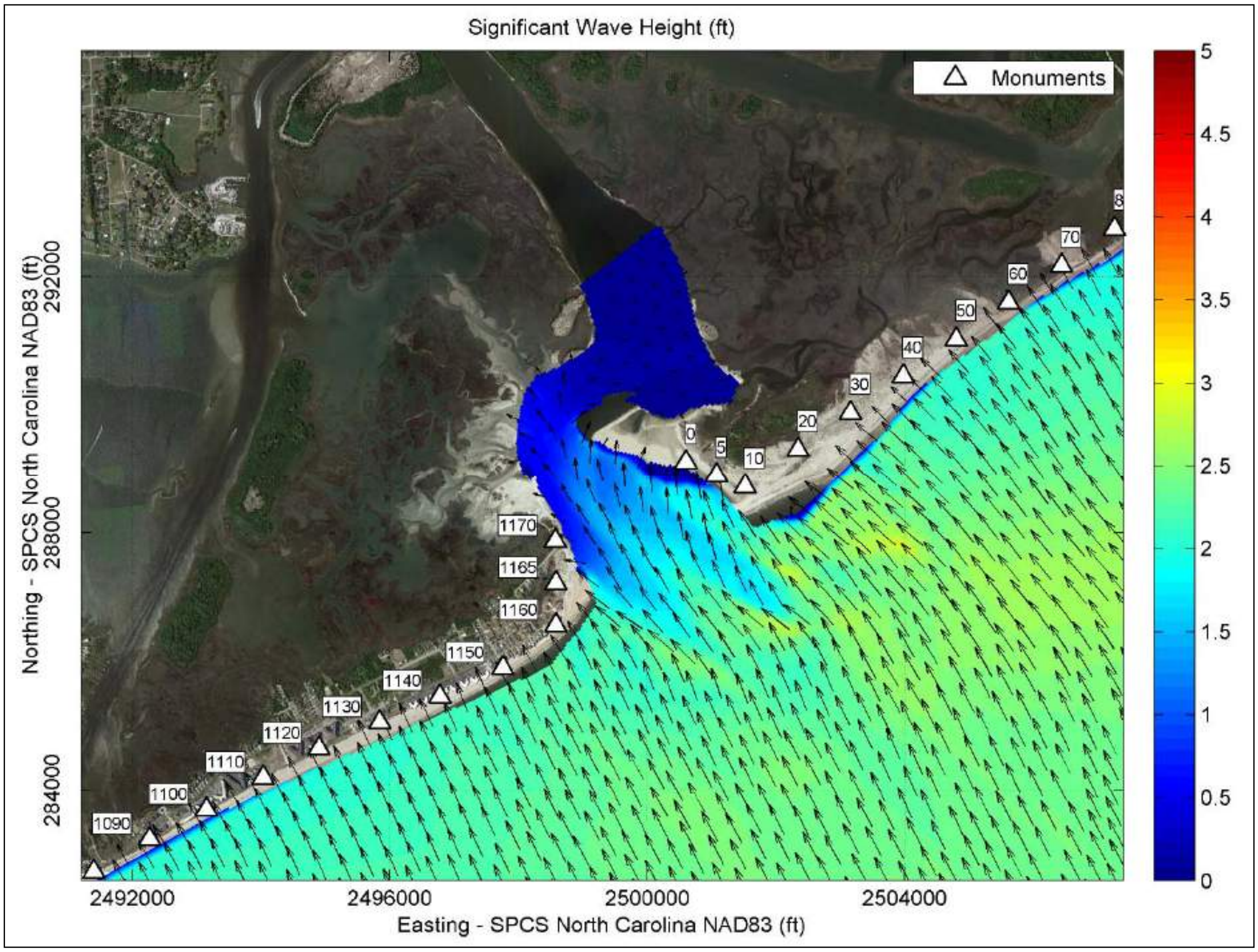

Figure 26. . Example of Local Wave model result.

For the morphology modeling of the New River Inlet and adjacent beaches, the wave and wind climate are schematized into representative conditions, requiring a wave and wind data time series (i.e. NOAA/WW3 parametric wave data and the associated wind speeds/directions).

In order to select a representative point to obtain a wind data time series from the time-dependent spatially varying wind fields (NOAA/NARR), tests are performed using homogeneous wind fields associated with five different points along the continental shelf (Figure 27, Table 3). The results of these tests are compared with the results of the original wave calibration for the most nearshore locations (USACE 190 and WHOI 09), which are closest to the study area (Table 4, Figure 28 and Figure 29).

Similar to the procedure adopted in the wave model calibration (Figure 22, Figure 23 and Figure 24), results of wave simulations using homogeneous wind fields are compared with wave measurements. As spatially varying winds better represent the wind forcing, the model is not expected to become more accurate after the simplification, although occasional improvement of results can be accounted in the analysis. The attempt of this verification is not necessarily matching the error obtained when using spatially varying winds, but seeking a minimal increase of errors due to the simplification of the wind representation.

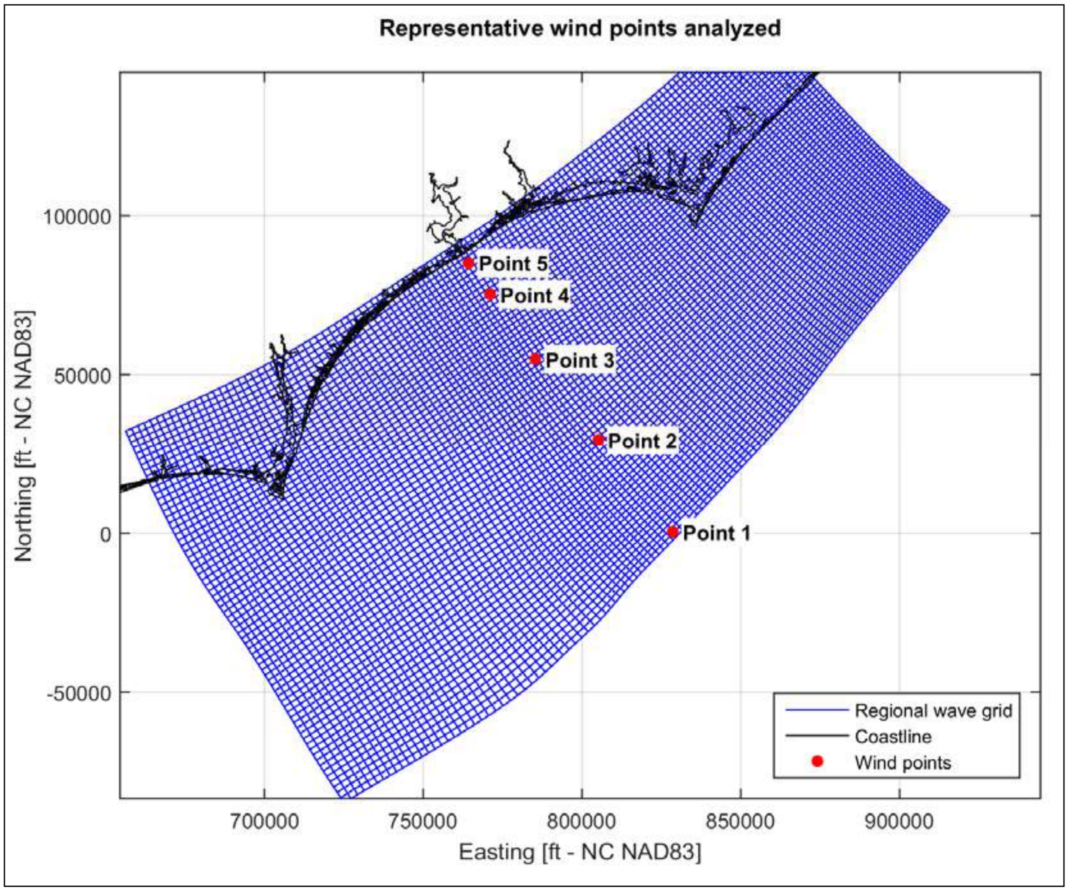

Figure 27. Wind points considered in the tests with homogeneous wind fields.

Table 3. Coordinates of the wind points considered in the wave model tests.

Wind point	Coordinates (NAD83)	
	Longitude	Latitude
#1	76.637° W	33.730° N
#2	76.883° W	33.996° N
#3	77.092° W	34.229° N
#4	77.243° W	34.415° N
#5	77.315° W	34.505° N

The summary of Root Mean Square Errors (RMSe) calculated from the comparison of modeled and measured wave data indicate the optimal performance is achieved when wind data associated with Point 3 is adopted for the whole simulation domain. RMS errors are similar to the ones associated with the reference run, using the spatially varying wind fields. The consistency of the simulation using homogeneous wind fields (Point 3) is confirmed in Figure 28 and Figure 29, which show results nearly identical to the simulations with spatially varying winds. This verification supports the use of the wind data time series associated with Point 3 in subsequent phases of the study.

Table 4. RMSe of the simulations using different representative points. The reference results correspond to the original calibration run using spatially varying wind fields (NOAA/NARR). Blue colors indicate smaller errors; red colors indicate higher errors.

Station	Parameter	Ref.	Representative points				
			P1	P2	P3	P4	P5
USACE 190	H_s (ft)	0.61	0.60	0.59	**0.60**	0.69	0.71
	T_p (s)	1.89	2.06	2.04	**1.92**	2.05	2.11
	Dir (°)	28.9	27.9	28.2	**27.6**	30.3	31.2
WHOI-09	H_s (ft)	0.54	0.61	0.59	**0.55**	0.59	0.61
	T_p (s)	2.06	1.95	1.95	**2.12**	2.33	2.37
	Dir (°)	23.9	25.5	25.0	**24.1**	25.7	25.1

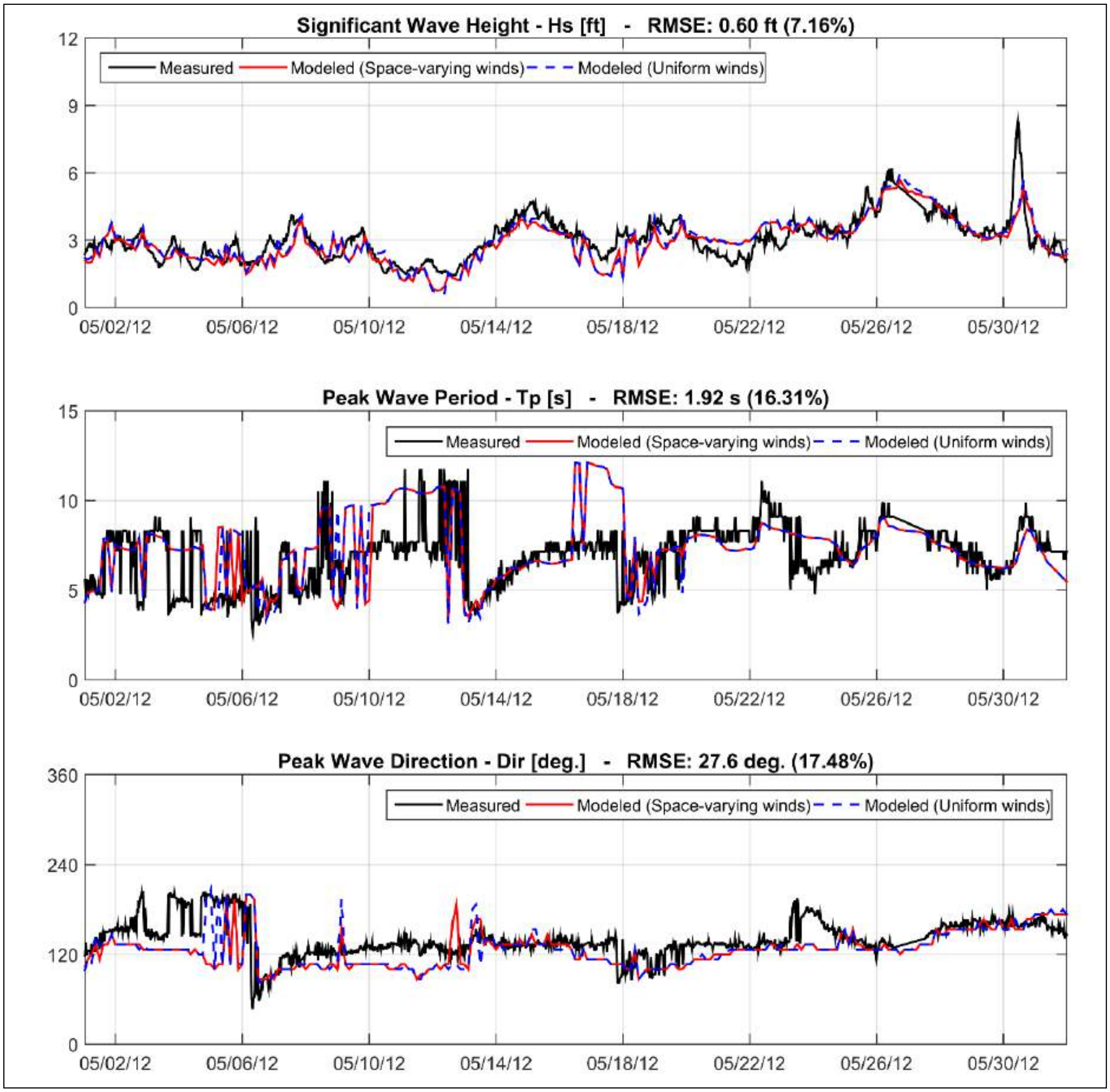

Figure 28. Comparison of measured and modeled wave data with spatially varying and uniform wind fields (representative wind Point 3): USACE 190 - 3.8 miles offshore/40 feet of water.

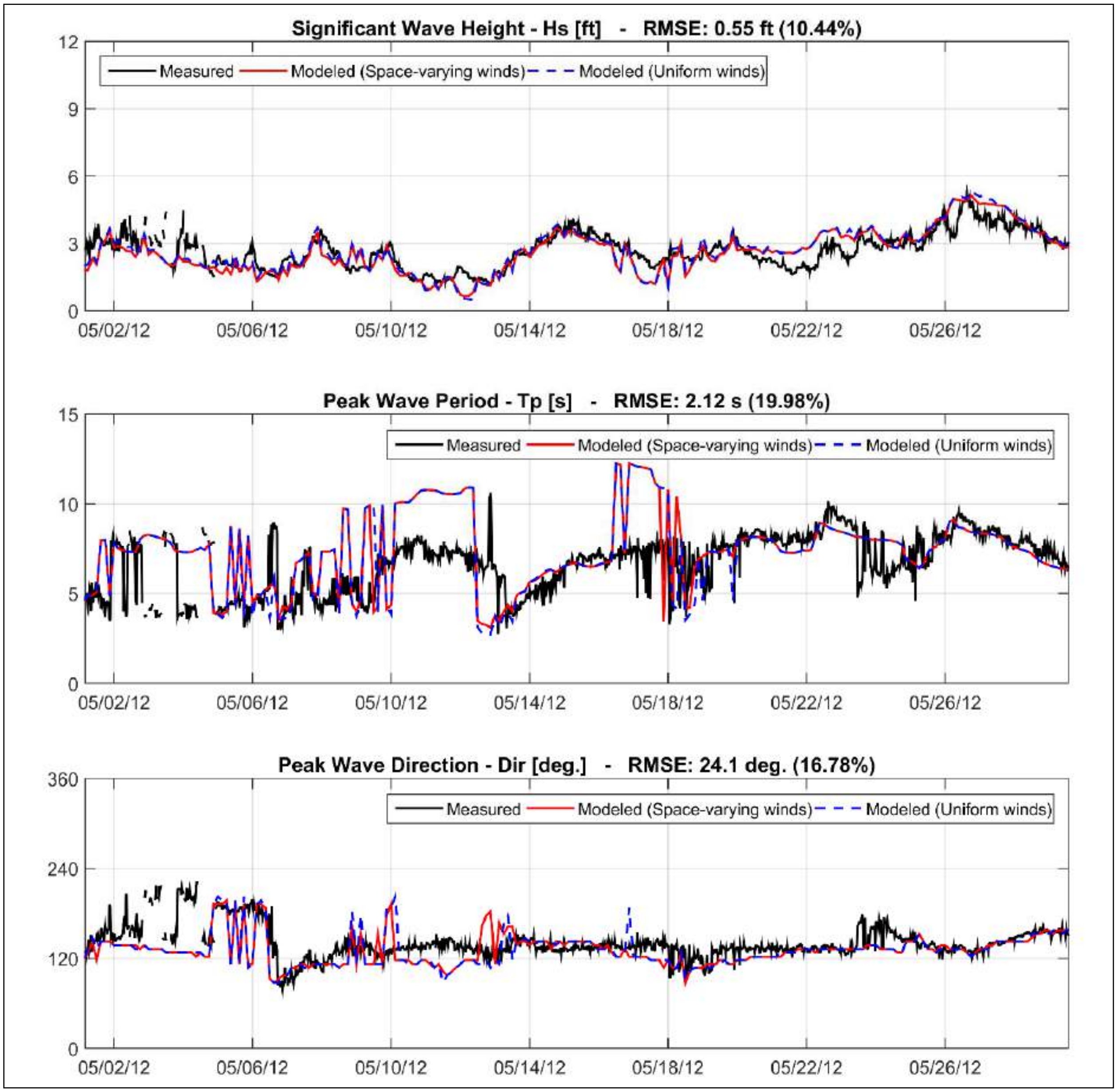

Figure 29. Comparison of measured and modeled wave data with spatially varying and uniform wind fields (representative wind Point 3): WHOI 09 – 1.0 mile offshore/30 ft of water.

2.3 Hydrodynamic Modeling

2.3.1 Modeling Approach

The objective of the hydrodynamic modeling is to reproduce the patterns of the water flow through the New River Inlet, controlled by the ebb and flood variability of the currents and water level. Wind effects, wave-induced alongshore currents, as well as currents driven by radiation stress gradients are also taken into account in the hydrodynamic simulations. These are the driving processes of the evolution of the morphology of the inlet and the adjacent beaches, which is the final piece of the modeling study and is covered in Section 2.4 of this document.

Riverine flow rate of fresh water out of the estuaries were not required as an additional forcing condition in the hydrodynamic simulations due to the relatively small freshwater flow volumes compared to the tide generated flow volumes through the inlet. In this regard, based on measurements reported by the US Geological Survey for a gauge located below the US Highway 17 bridge at Jacksonville, NC, between October 2008 and September 2013 (5-year record), the average freshwater discharge from New River was 233 cubic feet/second (cfs) (USGS, 2016). Peak tidal discharges through New River Inlet are on the order of 30,000 cfs. Therefore, freshwater discharge into the system has minimal impact on the hydrodynamics of the inlet or the connecting channels.

Water level and current data were collected as part of the RIVET study between April 16 and May 26, 2012. Both Woods Hole Oceanographic Institution (WHOI, 2015) and Scripps Institution of Oceanography (SIO, 2015) collected and processed oceanographic data for the RIVET study, which was funded by the Office of Naval Research (ONR). These data were used in the calibration process to adjust the model's parameters and overall setup/configuration, as well as verify the adequacy of the model forcing scheme, so that it reproduces the measured data with a high degree of confidence. Current and water level data were compared at 17 locations monitored during the RIVET study (11 - WHOI[1] ADCP gauges and 6 - SIO[2] ADCP gauges, Figure 30). Appendix A includes graphics developed to show the comparison of the simulated and measured current and water level data at these 17 locations.

[1] WHOI: Woods Hole Oceanographic Institution.
[2] SIO: Scripps Institution of Oceanography.

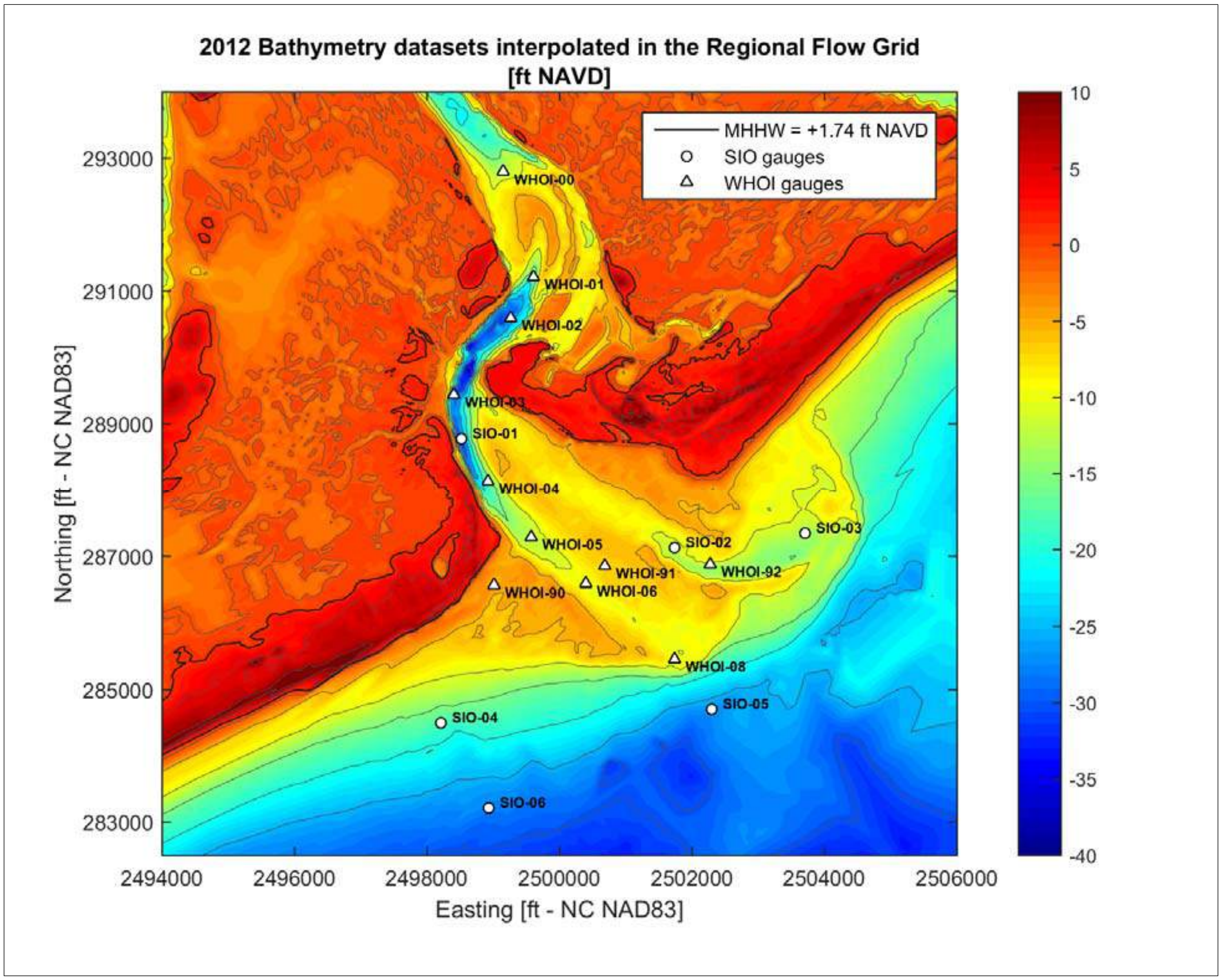

Figure 30. Location of the ADCP Gauges used in the hydrodynamic model calibration (current and water level data).

An offline nesting approach was applied in order to optimize the computational time. Two grids are used in the hydrodynamic simulations (grid specifications are provided in Section 2.1):

- Regional Flow Grid: covers a larger area and includes the whole Stones Bay and New River Inlet;
- Local Flow Grid: covers a smaller area, centered at the area of interest.

The regional grid provides boundary conditions for the Local Flow Grid (i.e. water discharges through the 'local' bayside boundaries) every 10 minutes.

2.3.2 Regional Flow Model

The specifications of the computational grid used for the regional hydrodynamic simulations is provided in Section 2.1. The vertical space is divided into 6 sigma vertical layers with variable fractions of the local water depth at each computational point: 10%, 25%, 30%, 20%, 10%, 5% (from surface to bottom). The regional flow bathymetry surface used in the model calibration was created using the following data sources, which are listed in the order of preference used to populate the model grid:

1. June 2012 Phase 1 pre-construction profile data collected for North Topsail Beach (CPE-NC, 2012b).
2. October 2012 Phase 1 pre-construction profile data collected for Onslow Beach (Gahagan & Bryant Associates, 2012).
3. May 2012 LARC hydrographic data collected in New River Inlet (USACE, 2012a).
4. April 2012 as-built hydrographic survey of New River Inlet and Cedar Bush Cut (USACE, 2012b).
5. 2010 Atlantic Coast LIDAR (USACE, 2010).
6. 1979 New River Estuary Soundings (NOAA, 1980).
7. Morehead City, NC DEM (NOAA)/North Carolina Floodplain Mapping Program DEM (Grothe et al, 2012).
8. U.S. Coastal Relief Model (NCEI, 2015).

The resulting interpolated bathymetry to the regional grid is delineated on Figure 31.

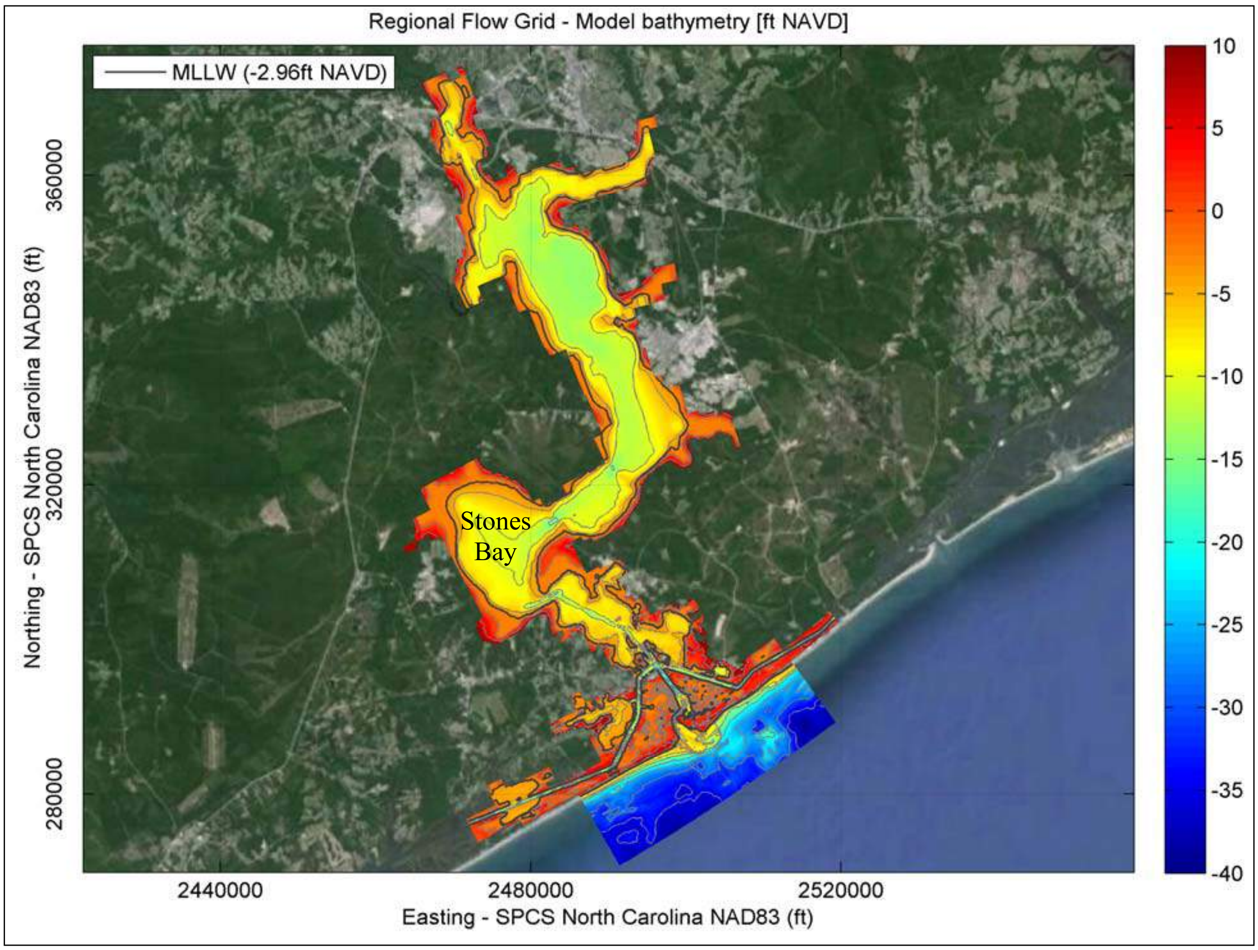

Figure 31. Initial bathymetry of the hydrodynamic calibration period interpolated to the regional grid.

The tidal boundary conditions of the offshore portion of the grid were defined based on water level data collected by NOAA at Station 8658163 (Wrightsville Beach) (NOAA, 2015). Over the entire domain of simulation, the model was run with wind and atmospheric pressure data from the NCEP NARR project (North Atlantic Regional Reanalysis) (NARR, 2015). The flow model was run in parallel communication with the calibrated wave model (described in Section 2.2) so that outcomes from the flow model are incorporated in the wave simulations (e.g. water levels, currents) and wave model results are used as inputs for the hydrodynamic simulations (e.g. radiation stresses, enhanced bottom friction).

To calibrate the hydrodynamic model, a series of simulations were made by changing only two model calibration parameters: the horizontal eddy viscosity coefficient and the Chezy bottom roughness coefficient. Three values of each parameter were tested, resulting in a matrix of 9 simulations. The set of parameters that resulted in the best overall comparison with measured data were chosen as the calibrated configuration. Table 5 shows the mean Root Mean Square Errors (RMSE) from the comparison of measured and modeled the current speed for the 17 ADCP locations and for each simulation. Errors associated with water levels are minimal for all simulations, and therefore were not used for ranking the model performance. Using the horizontal eddy viscosity of 3 m^2/s along with a Chezy value of 50 $m^{0.5}/s$ resulted in the smallest overall error.

Table 5. RMSE (ft/s) values for several calibration runs on the regional domain

		Viscosity (m^2/s)		
		0.5	**1**	**3**
Chezy ($m^{0.5}/s$)	**50**	0.491	0.456	**0.403**
	65	0.499	0.457	0.403
	80	0.500	0.451	0.405

The relation between the New River Inlet morphology and the tidal prism was verified using the calibrated Regional Flow model setup. Simulations were performed using different bathymetric configuration of the New River Inlet region, defined based on soundings collected by CPE-NC in 2005 and 2015 as well as data collected by the FRF in 2012 (Figure 32, Figure 33 and Figure 34). The model configuration, boundary conditions and simulated time frame are exactly the same for the three simulations.

The modeled water lever variations in the inner areas of the inlet were compared to evaluate the influence of the inlet morphology in the tidal prism (Figure 35). The differences between the results of the simulations are minor, indicating the consistency of the offline nesting approach (i.e. the regional flow model must not be revisited every time a new inlet bathymetric configuration develops).

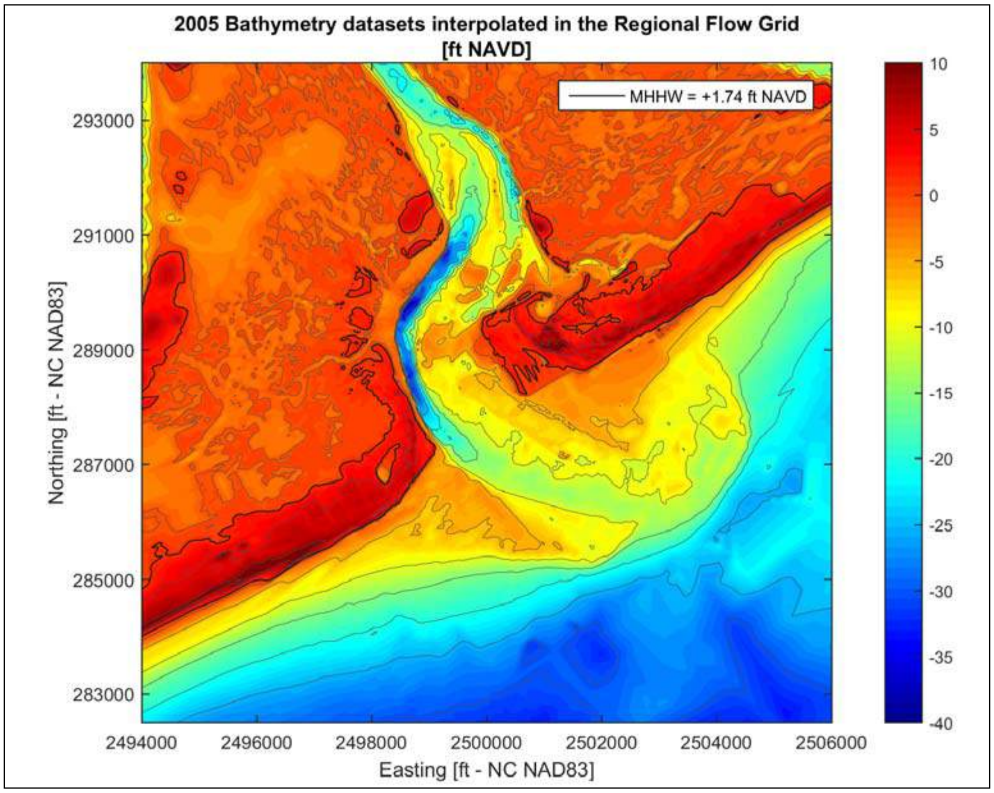

Figure 32. 2005 inlet bathymetry interpolated to the Regional Flow Grid.

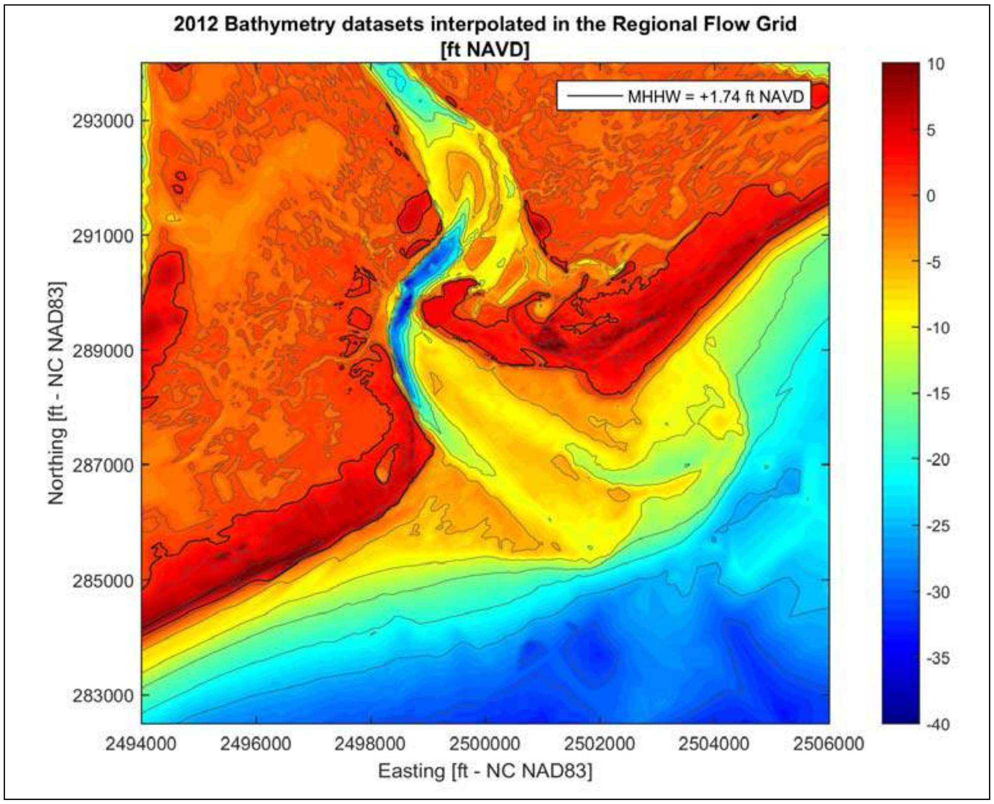

Figure 33. 2012 inlet bathymetry interpolated to the Regional Flow Grid.

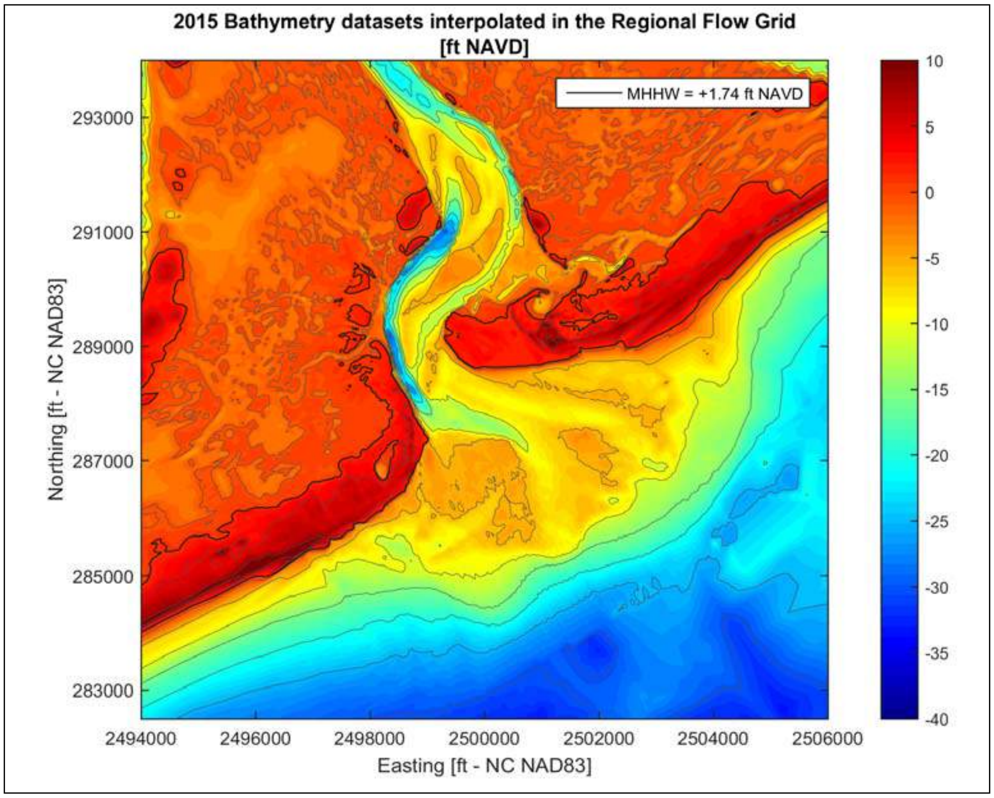

Figure 34. 2015 inlet bathymetry interpolated to the Regional Flow Grid.

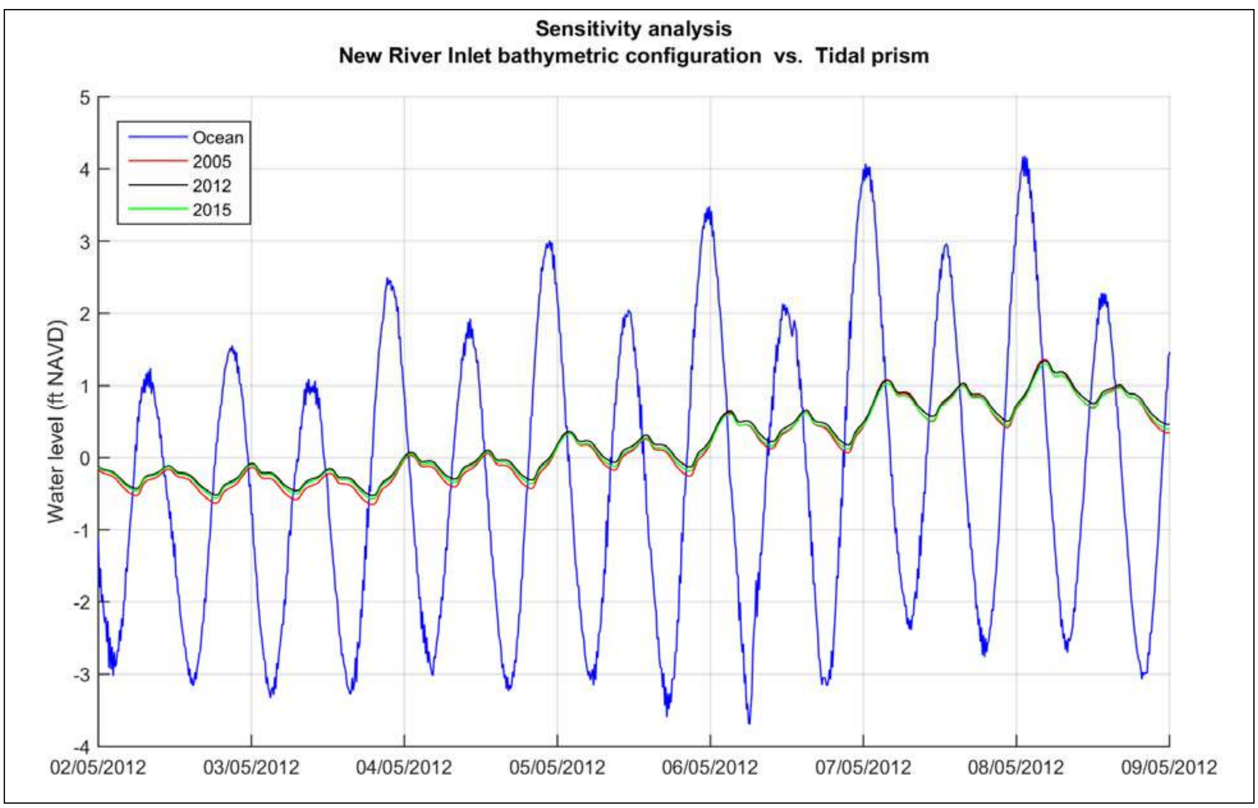

Figure 35. Verification of the influence of the inlet morphology in the water level variations of the New River and Stones Bay.

2.3.3 Local Flow Model

The local grid is centered at New River Inlet's ebb shoal, and extends about 1.5 miles offshore, and around 2.5 miles northward and southward, in the alongshore direction. Details of the grid specifications are given in Section 2.1. The vertical space for the Local Flow model is divided into 10 sigma vertical layers with variable fractions of the local water depth at each computational point: 20%, 20%, 15%, 12%, 10%, 8%, 6%, 4%, 3% and 2% (from surface to bottom).

The Local Flow model used for the detailed flow, sediment transport and morphology simulations considers a greater number of vertical layers (10 layers) with respect to the Regional Flow model (6 layers). The Regional Flow Model was used for hydrodynamics only (i.e. no sediment transport/morphology). The reason for the greater number of vertical layers is that sediment transport is computed based on bed shear stresses associated to the bottom layer velocity. In order to have a good representation of the near-bed flow velocity (i.e. logarithmic flow profile) and therefore shear stress and transport, the model user's manual recommends a bottom layer thickness of 3% of the water depth or less. Because the thickness of sigma layers must be increased gradually, 10 layers are required to cover the whole water depth. Since sand transport is not calculated in the Regional Flow model simulations, the ≤3% layer at the bottom is not a requirement.

The bathymetry definition for the local grid was defined based on the same set of bathymetry data used to create the bed surface of the Regional Flow model (Section 2.3.2). The result of the interpolation to the model grid points is given in Figure 36 and Figure 37.

The Local Flow model was forced with time-varying water levels along the offshore boundary and water discharges in three bay-side boundaries: ICWW southwest, ICWW northeast and New River (northwest). Similar to the Regional flow model, the water levels are defined based on water level data collected by NOAA at Station 8658163 (Wrightsville Beach) (NOAA, 2015). The water discharges through the inner boundaries result from the Regional Flow model, which makes the Local Flow model 'nested' into the Regional one.

Following the recommendation from Delft3D model developers, a time variable bed roughness coefficient is used in the local flow model considering that the local flow model setup is the base for the sediment transport and morphology simulations. This model configuration was relatively recently developed and implemented in the Delft3D model, using a bed roughness predictor based on the evolution of bed forms such as ripples and mega ripples over time.

The comparison of the Local Flow model results with the measured water level and current data at the 17 locations (Figure 30) is similar to the Regional Model, indicating a very good performance of the numerical model. In this case, the average RMSE calculated from the comparisons between measured and simulated current speeds over the calibration period was 0.411 ft/s. The model definitions used in the configuration of the Regional and Local Flow models are summarized in Table 6. Examples of current fields computed with the Local Flow model are given in Figure 38, for flood and ebb tide conditions.

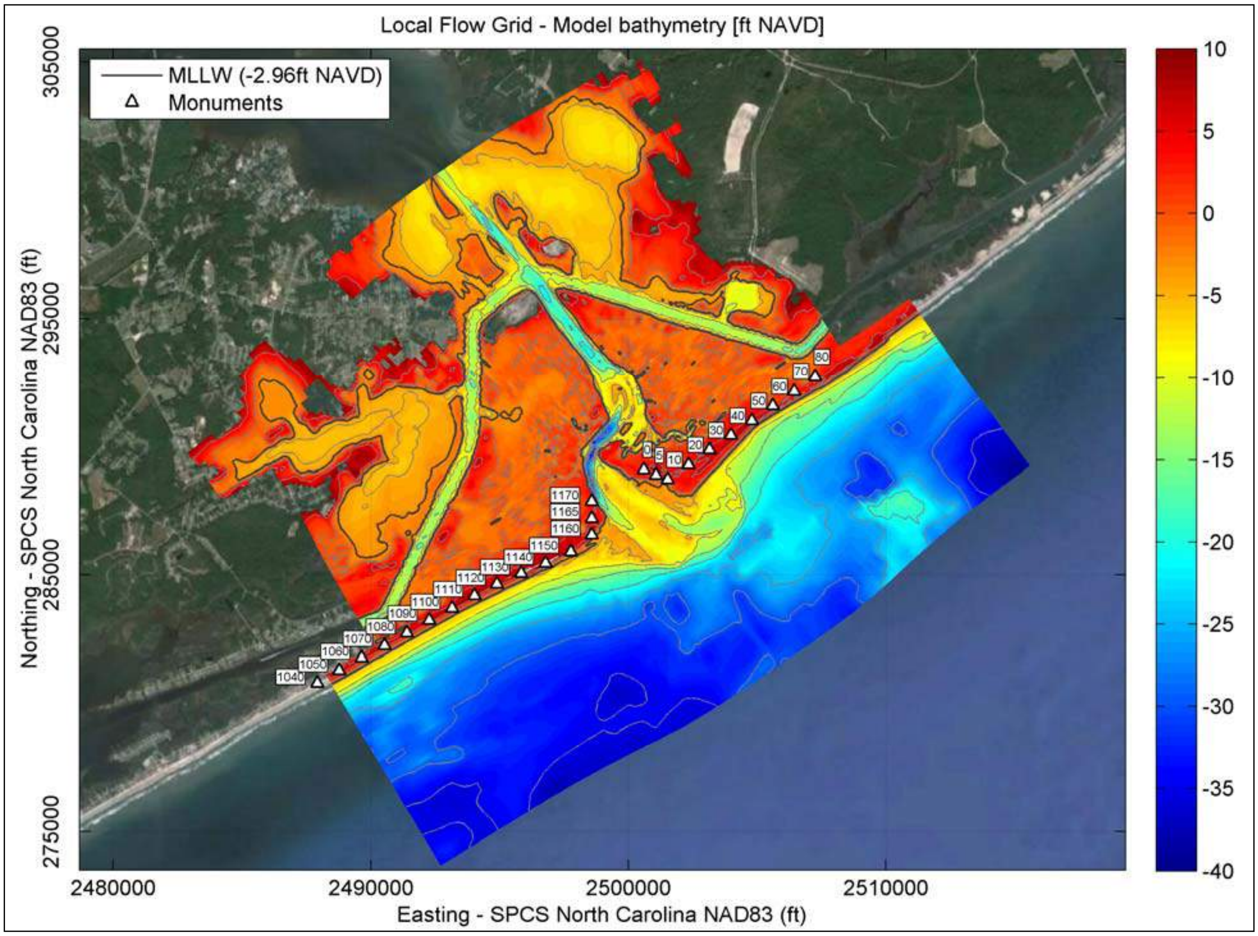

Figure 36. Bathymetry definition of the hydrodynamic calibration period (2012) interpolated to the local grid.

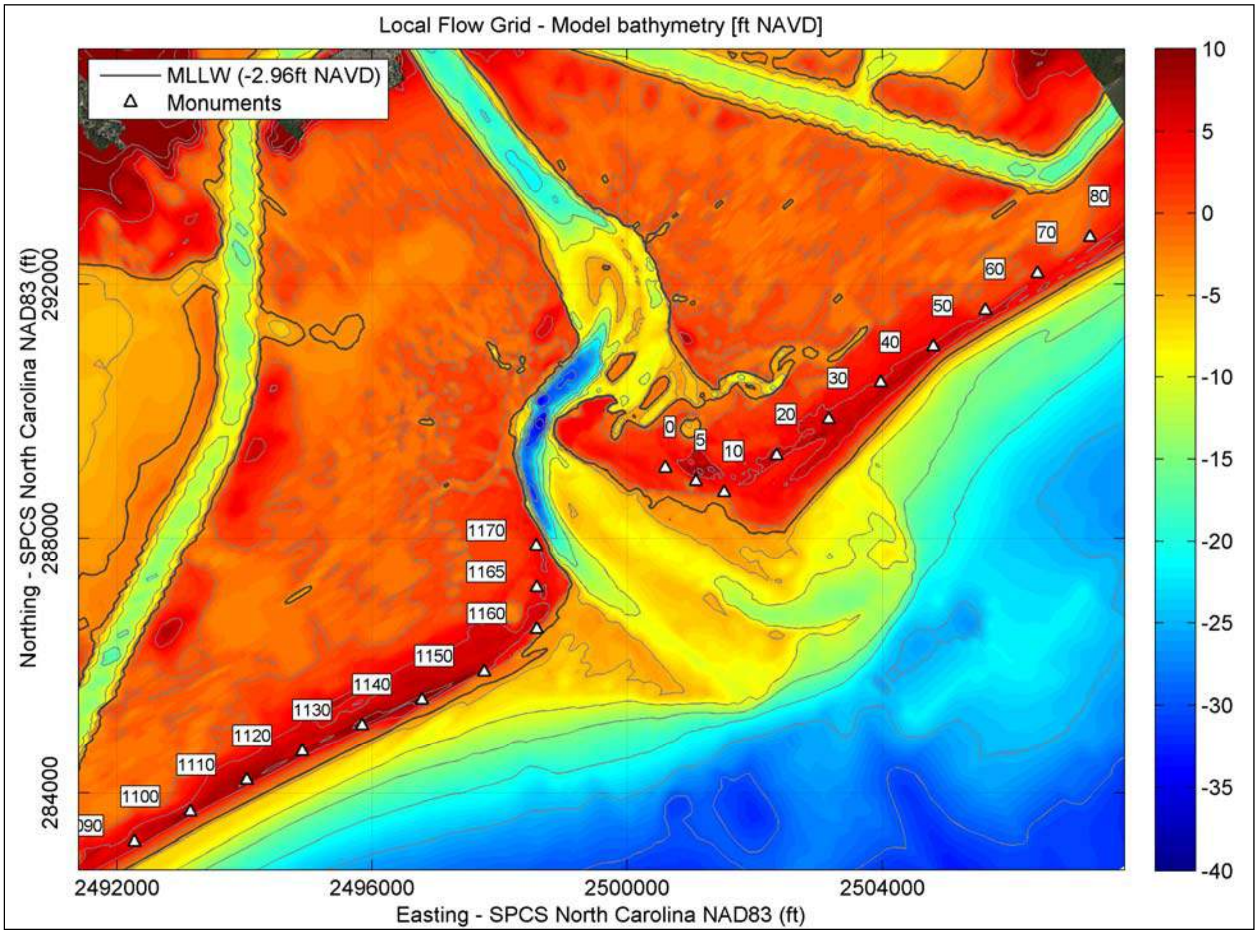

Figure 37. Bathymetry definition of the hydrodynamic calibration period (2012) interpolated to the local grid – Detail zoom in the New River Inlet and adjacent beaches.

Table 6. Summary of the Regional and Local Flow model definitions.

		Default Value	Regional	Local
Number of Layers		1	6	10
Wind Drag Coefficients	**0 m/s**		0.00063	
	100 m/s		0.00723	
Horizontal Eddy Viscosity		10	3	3
Model for 3D Turbulence			k-epsilon	
Vertical Eddy Viscosity			0	
Gravity (m/s^2)			9.81	
Water Density (kg/m^3)			1025	
Chezy (roughness)		65	50	Time variable (Trachytopes)

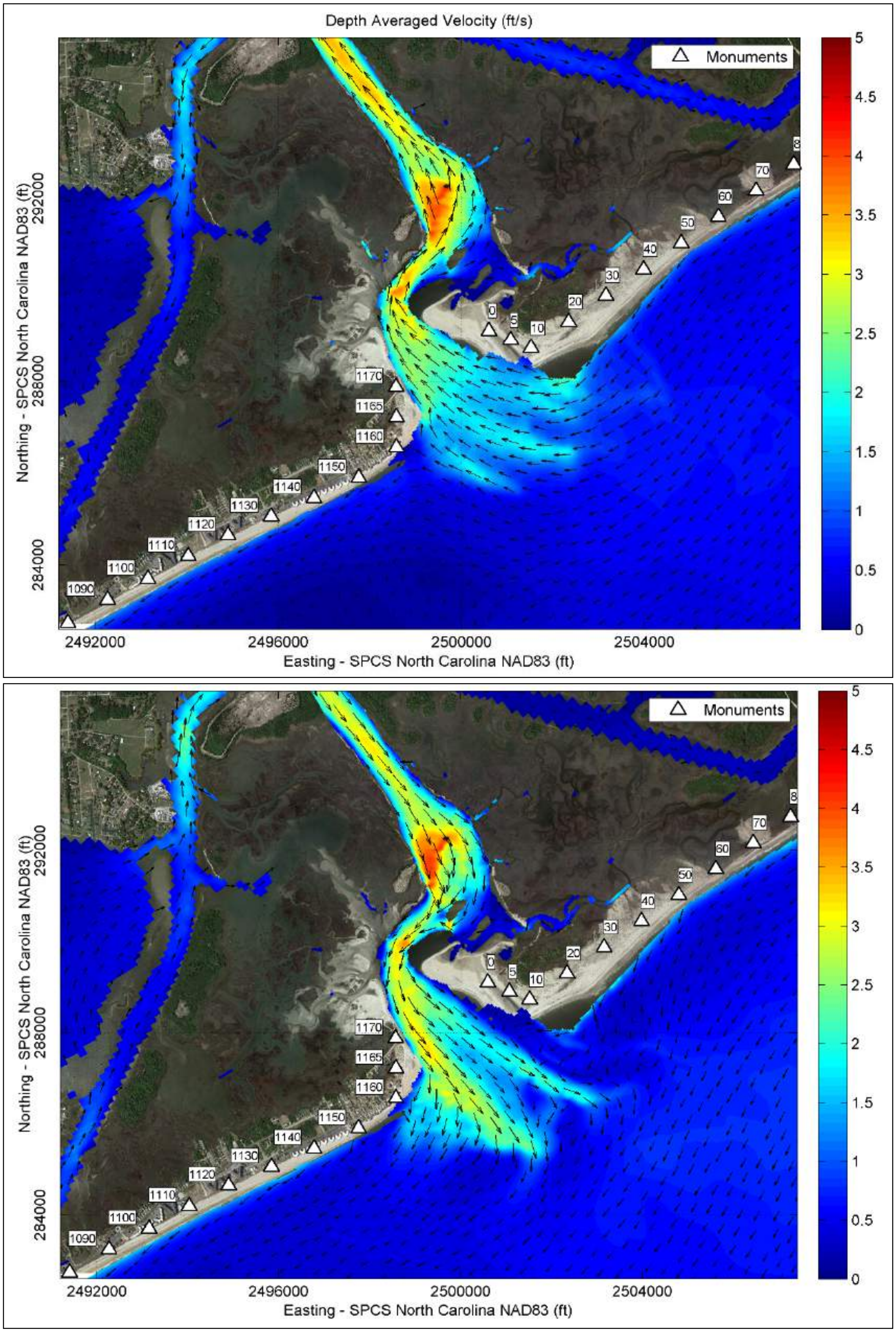

Figure 38. Current maps for flood (top) and ebb (bottom) tidal conditions.

2.4 Morphology Modeling

The primary modeling tool in this investigation is the Delft3D morphological modeling package (Deltares, 2011). This packages consists of two models, which are coupled together to determine changes in a topographic and bathymetric surface based on the effects of waves, water levels, winds, and currents. Wave propagation from the offshore to the nearshore area is estimated using the Simulating Waves Nearshore Model (SWAN 40.72ABCDE, Delft University of Technology, 2008). Delft3D-FLOW utilizes the output waves from SWAN, along with the varying water levels offshore and the bathymetry, to determine the resulting currents, water levels, sediment transport, erosion, and deposition. Based on the estimated erosion and deposition at each time step, the Delft3D-FLOW model calculates the subsequent elevations of the topographic and bathymetric surface and sends the updated bathymetry back to the SWAN model. The time step used in the morphological simulations is 12 seconds, while wave propagation estimates in the SWAN model are performed every 2 hours. Given the interaction between waves and tidal currents near New River Inlet, Delft3D is the best means of evaluating the performance and impact of terminal groin and beach fill alternatives along the Town's beach.

2.4.1 Modeling approach

To best calibrate a morphological model, it is important to select a calibration period where man –made features (i.e. dredging, fill placement, coastal construction) are not included. After thoroughly reviewing the dense and lengthy New River Inlet and North Topsail Beach construction history, no beach fill or inlet dredging occurred between May 8, 2013 and July 5, 2014. Also, beach profiles were available for May 2013 and April 2014. As such, these dates delineate the bounds of the morphology calibration period. This period was selected as it was after the February 2013 Phase 1 construction (channel relocation and beach nourishment), before the December 2014 sand bag revetment construction, and between the May 8, 2013 and July 5, 2014 New River Inlet maintenance dredging and side cast operations.

The main processes observed during the calibration period were the prominent beach erosion that occurred along the north end of North Topsail Beach, the formation of a spit towards the New River Inlet, and the consequent bending of the dredged channel northward.

The numerical grid for the morphology modeling is the same as the one from the Local Flow simulation, with 10 vertical sigma layers, as recommended on Delft3D-FLOW user manual. The bathymetry data from 2013 was interpolated to the grid and used as the initial conditions for the morphology calibration simulations. The morphological calibration model surface was created based on the following sources, which are listed in the sequence used to populate the model grid:

1. New River Inlet & Cedar Bush Cut Hydrographic Survey from April 17-19 and 30, 2013 (USACE, 2013b).
2. April 2013 profile data collected for North Topsail Beach (CPE-NC, 2013b).
3. April 2013 profile data collected for Onslow Beach (CPE-NC, 2013b).
4. May 2013 Atlantic Intracoastal Waterway and Cedar Bush Cut Hydrographic Survey (USACE, 2013c).
5. April 2013 Profile Data of New River Inlet Disposal Area (CPE-NC, 2013a).
6. 2014 Atlantic Coast LIDAR (NOAA, 2014).

7. 1927 Coastal Region Soundings (NCEI) between Topsail Beach and Onslow Beach (NOAA, 1927);
8. U.S. Coastal Relief Model (CRM) (NCEI, 2015).

The final 2013 bathymetry interpolated to the morphology grid can be seen on Figure 39.

The Regional Flow model provided the boundary conditions used in the morphology simulations: at the offshore boundary, time-varying water levels were prescribed; through the inner open boundaries (ICWW south and north, and New River), discharges were defined (i.e. offline nesting of the Local domain into the Regional Flow domain). In morphology simulations, the Delft3D Flow module is coupled with the Wave module, so that wave effects over currents and water level can be taken into account when calculating sediment transport.

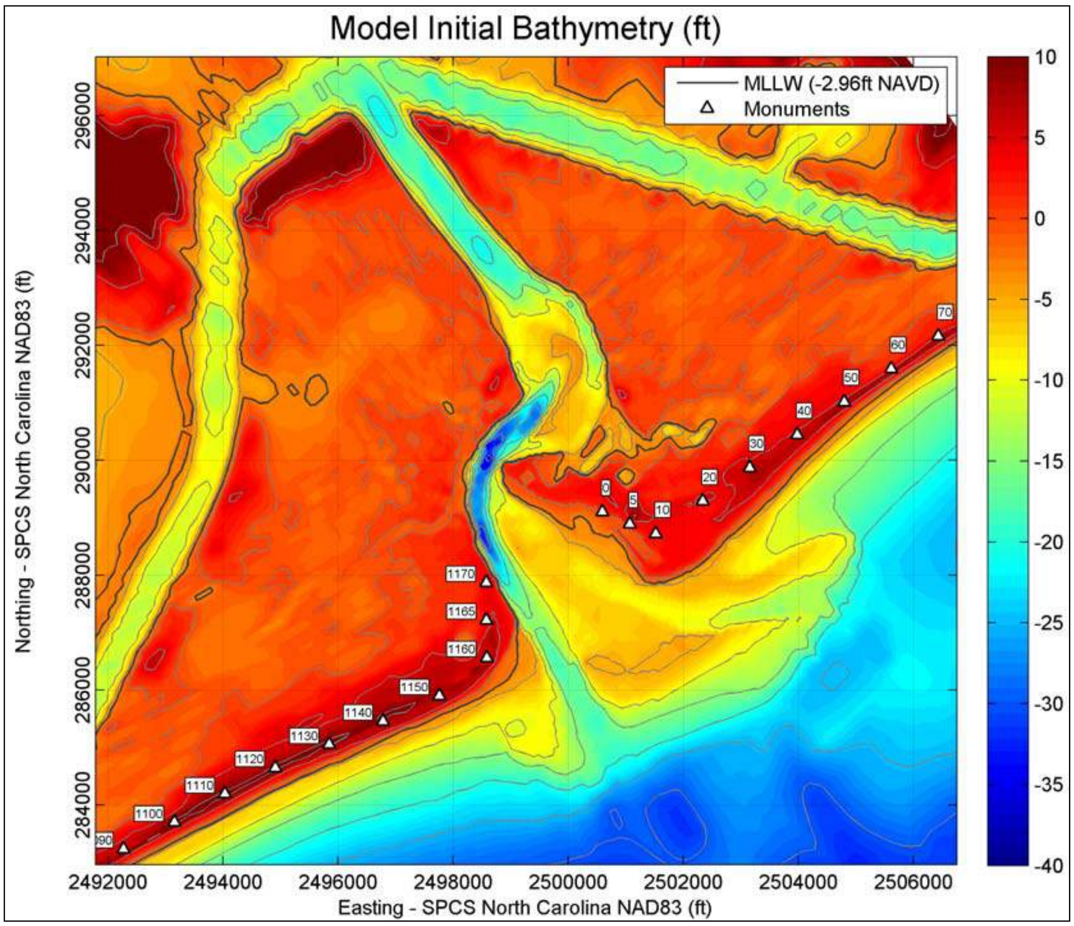

Figure 39. Morphology calibration model initial bathymetry as interpolated to the numerical grid.

The time scales of morphology processes (days-years) are substantially longer than the ones of the hydrodynamic processes reproduced in the Delft3D-FLOW model (seconds-hours). In order to fill the gap between those different time scales and make it computationally feasible to use process-based morphology models for longer term simulations (i.e. years), a morphological acceleration factor (*morfac*) is applied in the Delft3D model. Essentially, the gradients of sediment transport and morphology changes occurred at each flow/hydrodynamic time step (in the order of 10 seconds) are multiplied by *morfac* (in the order of 10). This makes the morphology model time step '*morfac* times' higher than the hydrodynamic time step.

For the application of the morphological acceleration technique based on the *morfac* to the simulation of a whole year of morphological changes, it is necessary to apply input reduction (described below) to all the forcing variables of the model: water level, wave and wind conditions.

The wave reduction was made by selecting a number of representative wave conditions from the full wave time series for the period (i.e. May 2013 to April 2014), defined based on the deep-water NOAA Wavewatch III hindcast wave data. The range of incoming wave directions between 60° and 220°, with a significant wave height of 0.5m or higher, was used to select 12 main wave cases. To select the wave cases, the energy of each wave case was calculated from the wave heights, based on the linear wave theory:

$$E = \frac{\rho g H_{rms}^2}{8}$$

The total energy of all the cases were summed and divided by 12, and limits were calculated for the Significant Wave Height and Peak Wave Period to group together wave cases that sum up to a same amount of energy. Each box on Figure 40 corresponds to one of these groups, and the wave case selected for that box has the same amount of energy as the mean energy of the cases inside the box. The peak wave period and peak wave direction of each case is calculated from the mean of all the cases in each group. Together, these 12 conditions represent approximately 67% of all wave cases in that period.

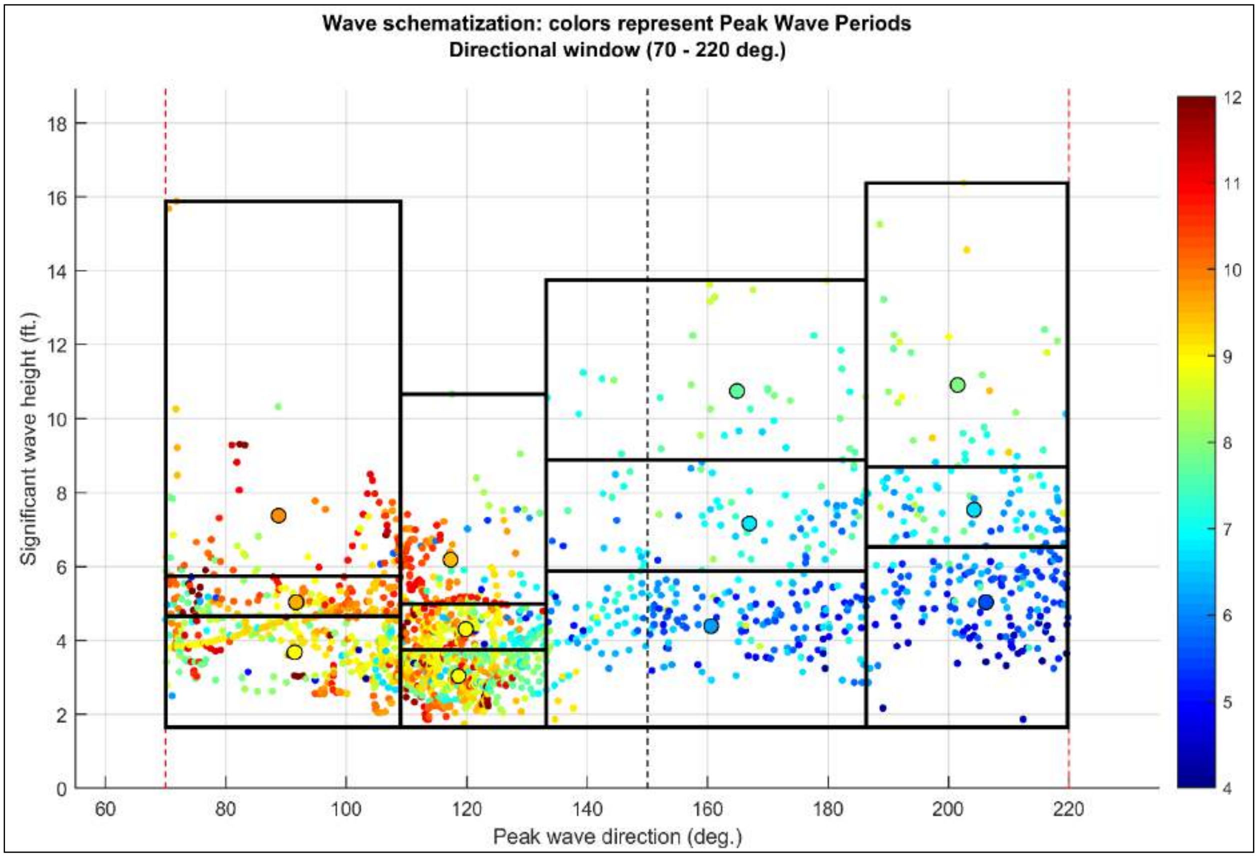

Figure 40. Wave cases that occurred during the calibration period (smaller colored dots), along with the selected cases for the morphological simulations with reduced input (larger colored dots). The wave cases inside the black boxes sum up to the same amount of energy.

The remaining 33% of the simulated time was represented by four additional cases. Three of them were selected in the same manner as the 12 main representative wave conditions, but using wave directions outside the 60° to 220° range. The last wave case represents calm conditions (i.e. situations with no waves, only tide affecting the study area).

For each representative wave case, a *morfac* value was assigned that depends on the frequency of occurrence of the group that wave case represents.

The wind schematization was made by assigning the mean wind speed and direction associated with the group of individual wave conditions represented by each representative wave cases, i.e. all the wave cases within the black boxes on Figure 40.

Table 7 shows all the simulated wave cases, as well as the frequency of occurrence during the calibration year and the morphological factor used in the calibration.

Table 7. Morphological model wave cases selected by the input reduction, as well as the associated wind speed and direction and the frequency of occurrence of each case. H_s is the significant wave height, T_p is the peak wave period and Dir_p is the peak wave direction.

Case #	H_s (ft)	T_p (s)	Dir_p (°)	Spreading (cosine power)	Wind speed [kn]	Wind direction [°]	Occurrence [days/yr]	Morfac
1	3.7	8.96	91.4	13.41	8.5	221.9	38.4	18.37
2	5.0	9.59	91.7	11.03	8.1	22.4	20.5	19.62
3	7.4	9.80	88.8	6.65	10.9	31.3	9.5	9.12
4	3.0	8.95	118.6	15.81	7.0	191.7	58.0	27.74
5	4.3	8.94	119.7	11.22	9.2	188.5	28.6	27.37
6	6.2	9.39	117.4	7.74	11.3	147.7	13.9	13.25
7	4.4	6.24	160.2	4.00	9.5	180.0	26.7	25.49
8	7.2	6.79	167.0	4.00	14.1	187.8	9.9	9.50
9	10.8	7.63	164.9	4.76	16.2	197.1	4.6	4.37
10	5.0	5.55	206.2	4.00	11.4	228.0	20.9	19.99
11	7.6	6.69	204.2	4.00	13.6	236.8	9.3	8.87
12	11.0	7.97	201.1	4.09	16.8	252.2	4.4	4.25
13	6.5	5.98	228.7	4.59	12.4	250.4	23.1	22.12
14	5.5	8.16	57.5	5.77	10.1	27.1	54.8	26.18
15	9.9	8.51	59.1	4.00	16.8	39.8	17.1	16.37
16	1.3	9.00	112.0	22.00	11.7	225.0	25.2	24.12

In addition to the schematized wave cases, the tides must also be schematized to run the morphological model. The main purpose of reducing tidal data is to replace the complex pattern of the real tide in the study area by a simplified tide, also called morphological tide. The morphological tide produces the same residual sediment transport and morphological pattern of changes that the actual tide produces (Lesser, 2009).

Tidal data reduction to a sinusoidal tide with constant periodicity allows each wave case to be propagated by at least one full tidal cycle. Thus, all wave cases are influenced by the same tidal amplitude and phase.

For the tidal reduction, a sensitivity analysis was conducted considering a diurnal morphological tide with a period of 24 hours, a semi-diurnal morphological tide with a period of 12 hours, and third morphological tide composed of the two prior tides (the so called M_2C_1, where the effects of a diurnal tide are added to the main semidiurnal component). The tidal signal of these three (3) different morphological tides can be seen on Figure 41.

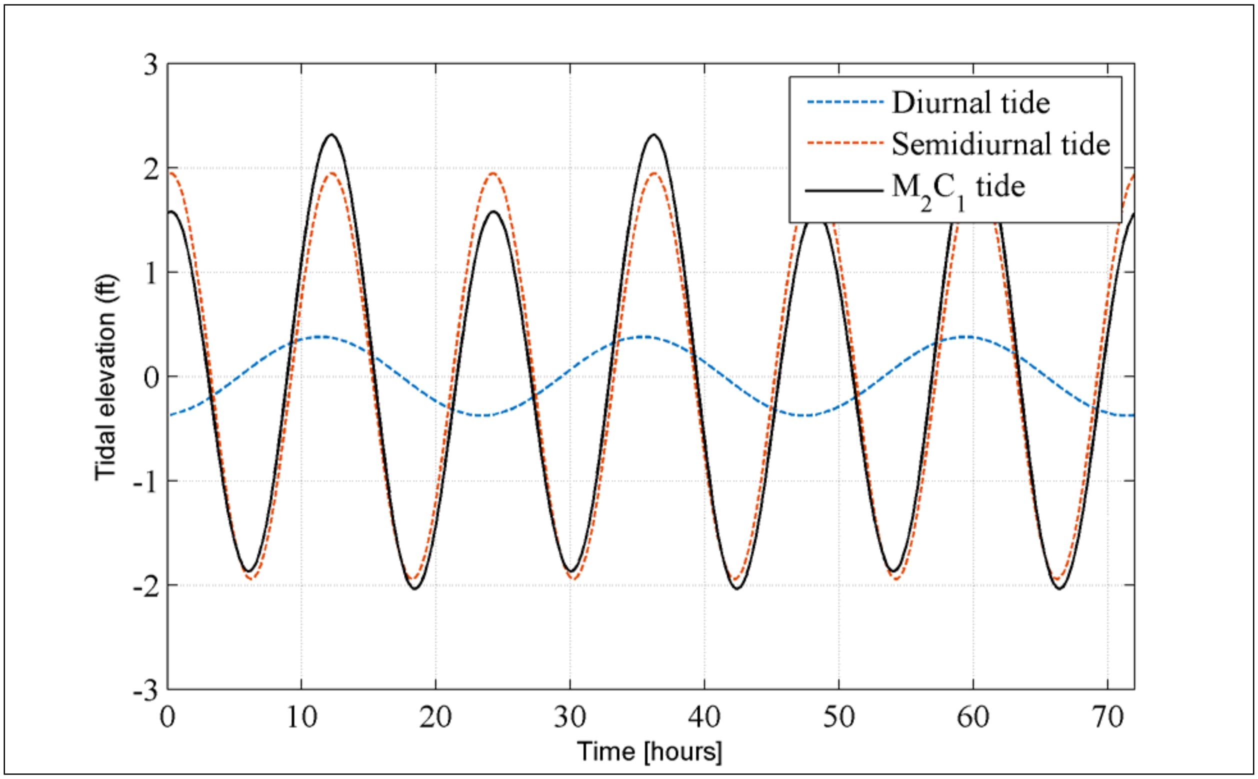

Figure 41 Morphological tides signals simulated for the sensitivity analysis

By comparing the results of these simulations (Figure 42), it is observed that the semidiurnal tide alone is able to satisfactorily represent the main patterns of sediment transport in the region when compared to a 31-day brute force simulation that included the full water level time series, with neap and spring tides. When considering morphological tide with only a diurnal component, practically no morphological changes are found, as expected.

In order to create the boundary conditions to the Local model used for the morphology simulations, the Regional Flow model was run for several days using the schematized tides. From these simulations, both the water level (offshore boundary of Local domain) and discharge time series (bayside boundaries) were derived.

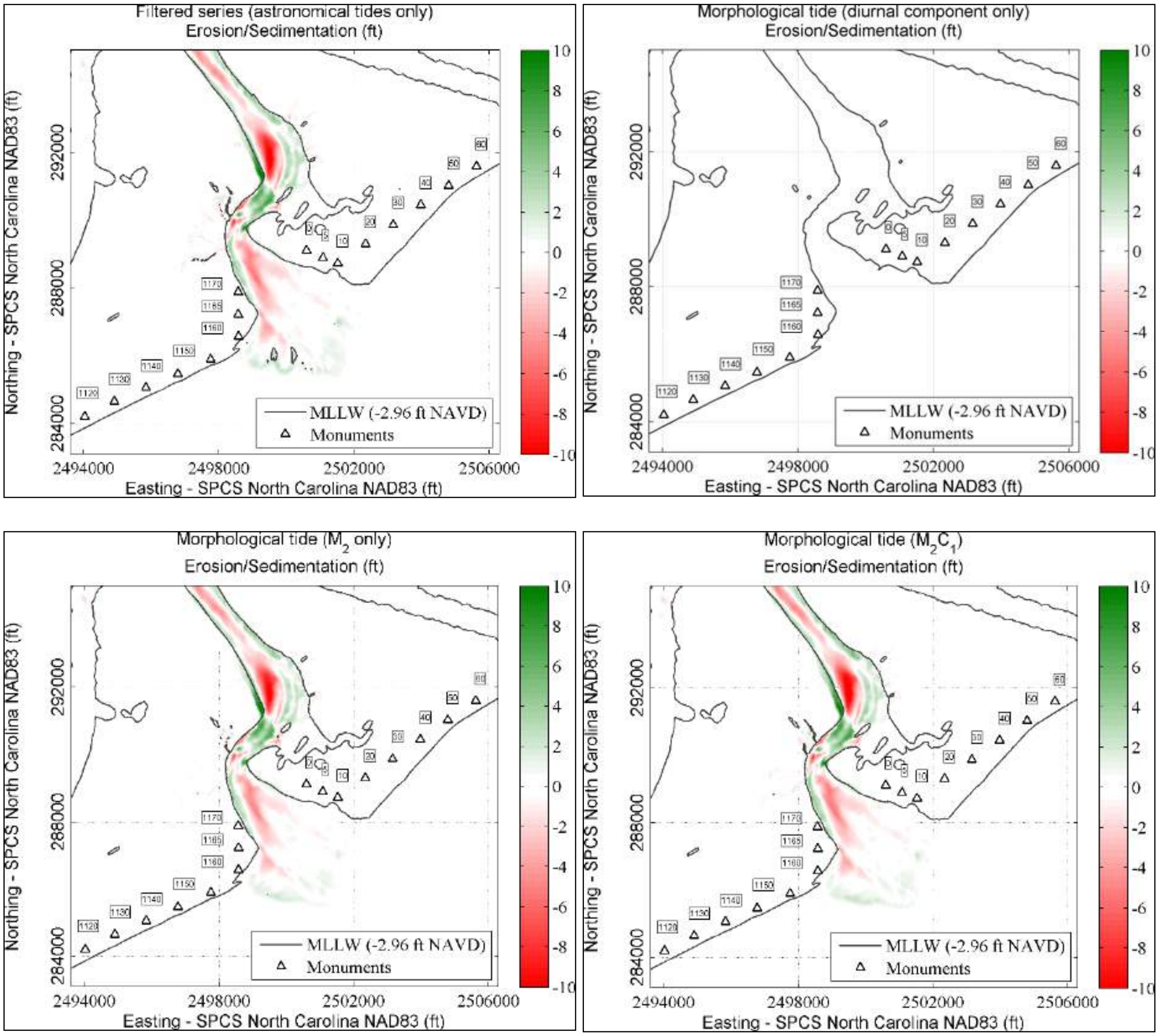

Figure 42. Erosion/sedimentation of 4 tests with different tidal boundary conditions: the original 31 days astronomical tides (top left), a morphological tide composed of just a diurnal component (top right), a morphological tide composed of just a semidiurnal component (bottom left), and the so called M_2C_1 morphological tide, composed of both a diurnal and a semidurnal component (bottom right)

2.4.2 Calibration

The morphology calibration was based on qualitative analysis of the model final bathymetry, the patterns and quantification of erosion and sedimentation in the inlet, and the integrated volume change along cross sections orthogonal to the coastline.

Several morphology simulations were performed for the calibration period in which different parameters of the model were tested and sensitivity analyses were performed. One of the parameters that considerably improved results was the transverse bed gradient factor for bedload transport (called AlfaBn). By adjusting this parameter, the depth of the inlet channels remained more stable and realistic (instead of overestimated scour using the default setting), as shown by the blue line on Figure 43.

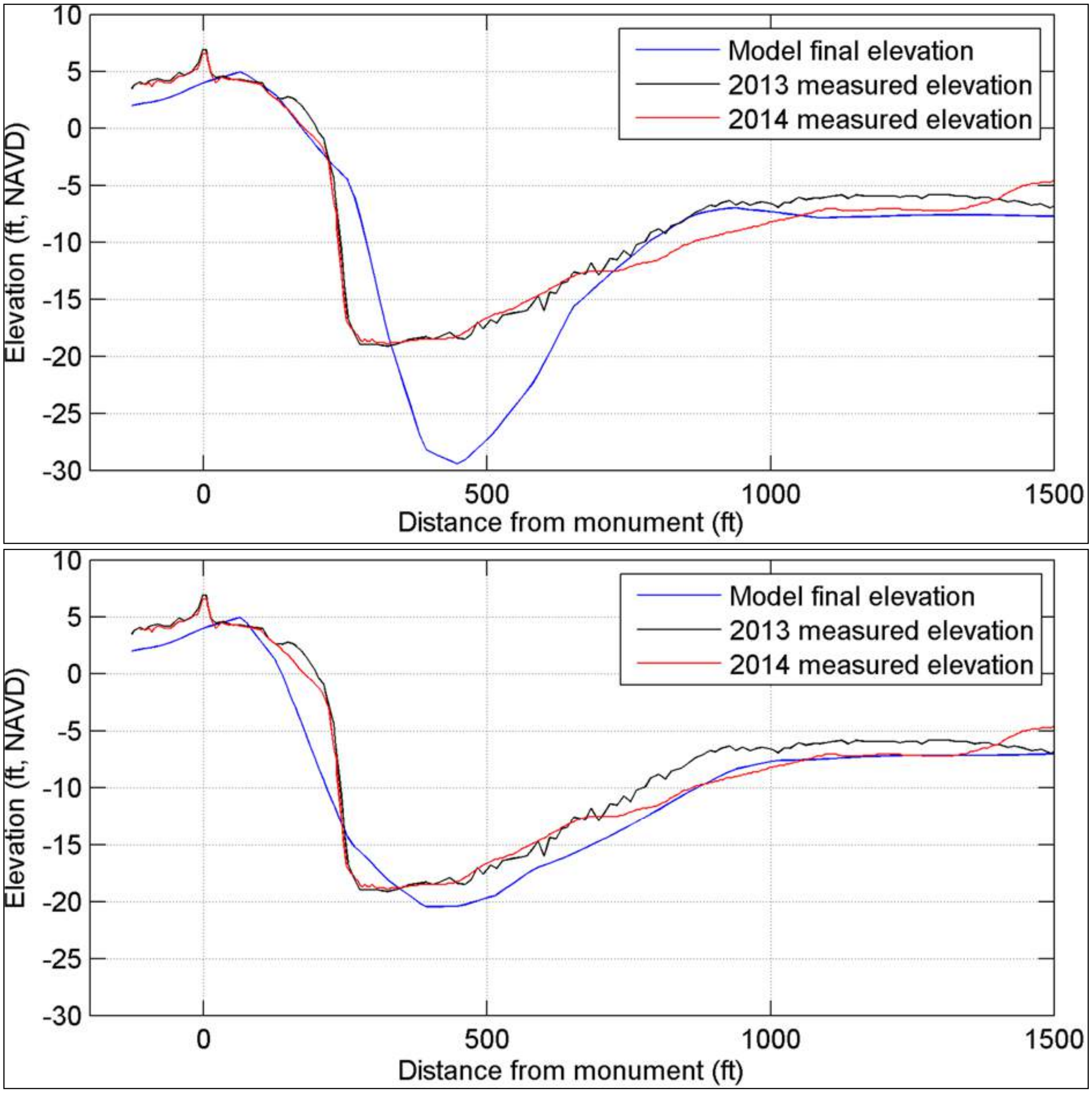

Figure 43. Curves showing scouring of the channel using two different transverse bed gradient calibration factors: the default value (top), and the value of the calibrated model (bottom).

Other parameters were important to calibrate the model's behavior along the beaches adjacent to the New River Inlet (i.e. North Topsail and Onslow Beach). For instance:

- The suspended and bedload transport factors along with the Horizontal Eddy Diffusivity coefficient were adjusted to better approximate the modeled and measured beach profiles' shape at the end of the simulation.

- Three different transport formulas were tested: the default Van Rijn formula (Van Rijn, 1993), the Soulsby &Van Rijn formulation (Soulsby, 1997), and the newest Van Rijn formulation (Van Rijn, 2007). The default one performed better.
- Multiple grain sizes were analyzed to test the model's sensitivity. Two different grain sizes were utilized for the calibrated model: 0.18 mm at the beach and 0.35 mm at the inlet. Mean grain size for the beach at North Topsail Beach is 0.23 mm and the mean grain size of the material within the channel dredged in 2012/2013 had a mean grain size of 0.39 mm (Finkl, *et al*., 2009)
- Instead of using a constant bed roughness coefficient in time and space, the Trachytopes approach was used so that the bed roughness is calculated every few time steps based on the dune heights predicted by the Van Rijn (2007) formulation.

The final parameters used in the calibrated model can be seen in Table 8.

Table 8. Calibrated morphology model parameters

		Default Value	Calibrated value
Horizontal eddy diffusivity		10	0.5
Current-related sediment transport factor	**suspended (Sus)**	1	1
	bedload (Bed)	1	1
Wave-related sediment transport factor	**suspended (Susw)**	1	0.04
	bedload (Bedw)	1	0.04
Maximum dry cell erosion factor (ThetaSD)		0.5	1
Transverse bed gradient factor (AlfaBn)		1.5	15
Bed roughness		Chezy = 65 $m^{1/2}/s$	Time variable (Trachytopes)

The final calibrated model's results after the one-year morphology simulation can be seen on Figure 45 to Figure 47, along with measured data.

Looking at the model and measured final bathymetries (April 2014, Figure 45) and comparing them to the initial bathymetry (May 2013, Figure 39), it is possible to notice the main features present in the 2014 measured bathymetry were reproduced by the numerical model, such as the bended channel and the spit that formed at the north end of North Topsail Beach.

The erosion and sedimentation maps (Figure 46) show that the main patterns of sediment transport were captured by the model. It includes: i) the higher beach erosion near the north end of North Topsail Beach and at the south end of Onslow Beach; ii) the landward movement of the banks at the northern lobe of the ebb shoal; and iii) the sedimentation inside the dredged channel, as well as the side scouring which resulted from the bending of this channel.

Besides this qualitative analysis, a quantitative analysis was made based on the volume change in certain areas in the inlet, shown in Figure 44.

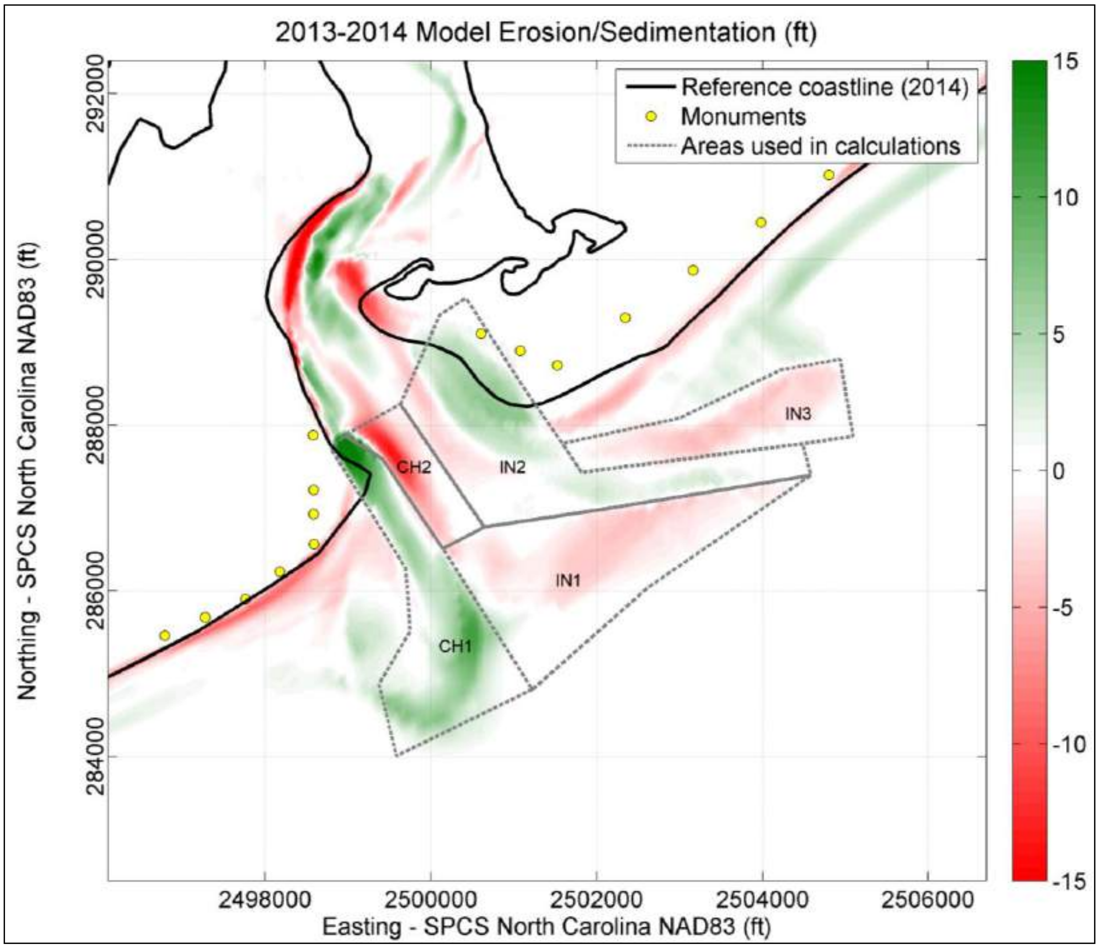

Figure 44. Areas defined for volume change analysis in the inlet region

The total volume change inside each of those five (5) areas shown in Figure 44 was calculated for the measured data and the simulated data. The results can be seen on Table 9. In general, the simulated values are very close to the measured ones, indicating that the regions of erosion and sedimentation are well captured by the model.

Table 9. Measured and simulated volume changes inside defined areas. Negative values indicate that more erosion occurred in an area.

Area Name	Volume change (1000 cu yd)		Percent Difference from Measured
	Measured	Simulated	
CH1	499.21	504.56	1.1%
CH2	-107.87	-141.71	31%
IN1	-150.48	-218.67	45.3%
IN2	172.09	230.77	34.1%
IN3	-132.04	-147.81	12.0%

Concerning the cross-shore volume change along North Topsail Beach, Figure 47 illustrates that the model accurately represents the regions of erosion, stability and sedimentation, as well as the order of magnitude of the volume changes all along the beach, although underestimation of the erosion is noticeable in some regions.

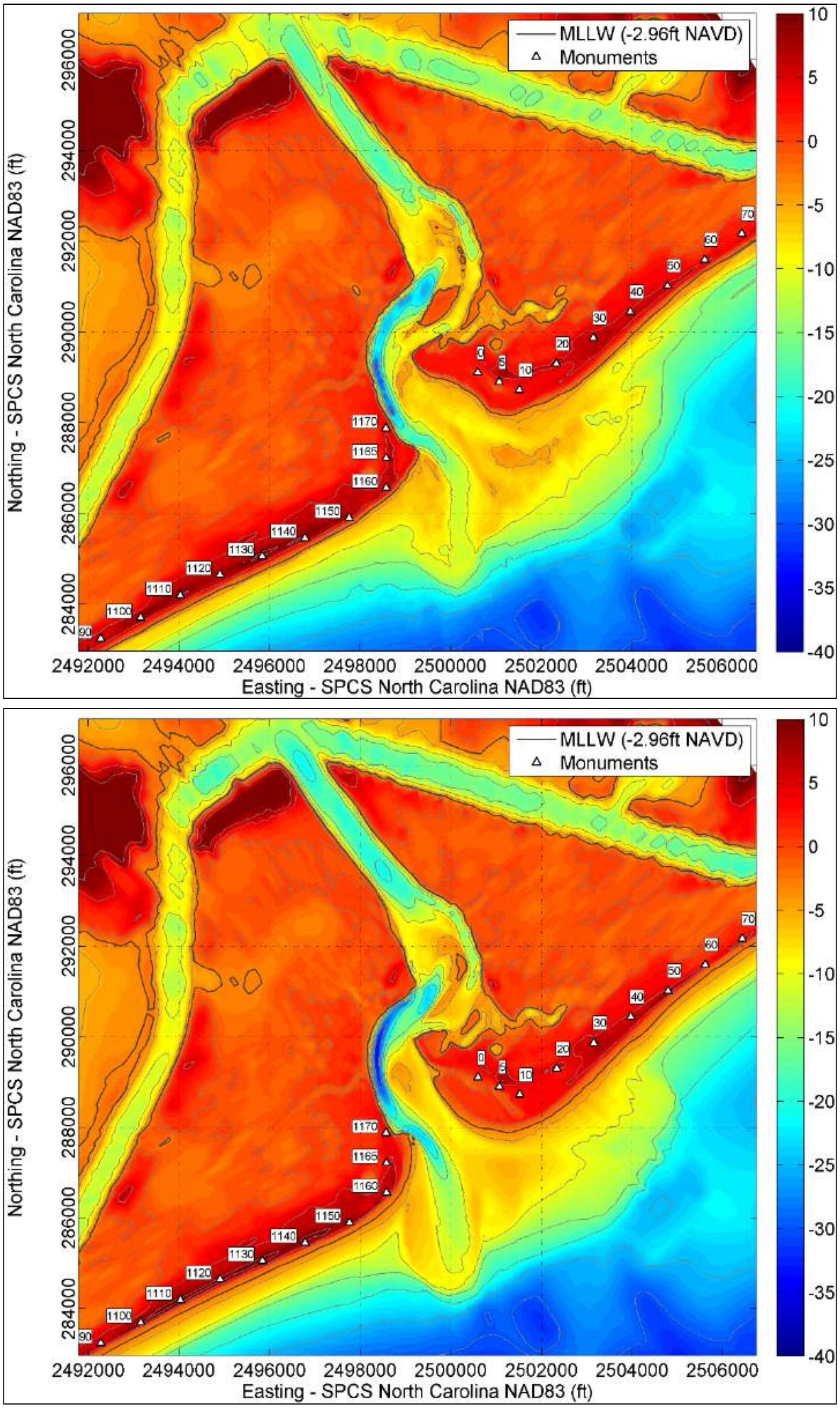

Figure 45. 2014 Measured bathymetry (top) and final bathymetry of the calibrated morphology model (bottom), after 1 year.

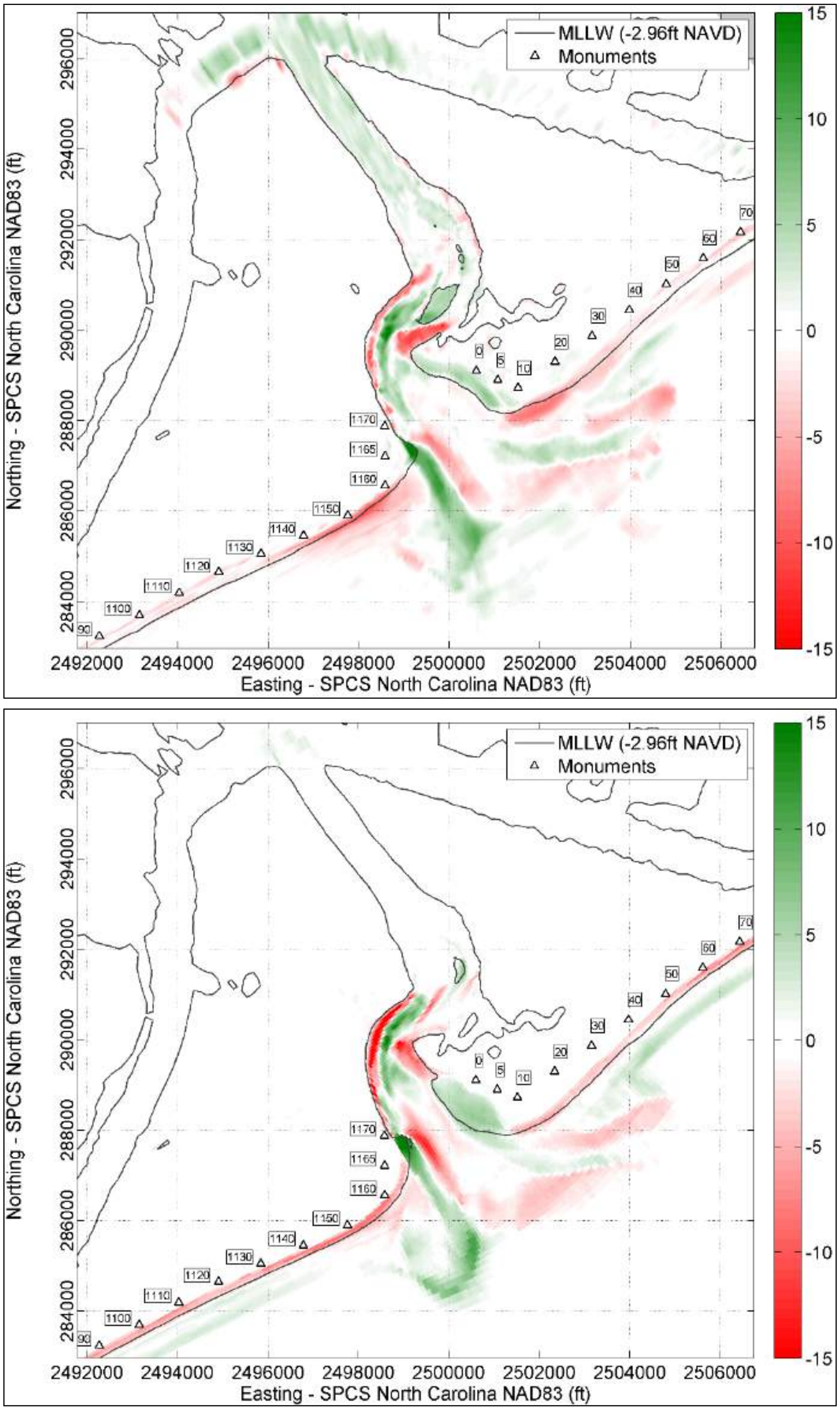

Figure 46. Measured (top) and simulated (bottom) erosion/sedimentation maps of the calibrated morphology model, after 1 year. Red colors indicate erosion (final bathymetry deeper than initial bathymetry).

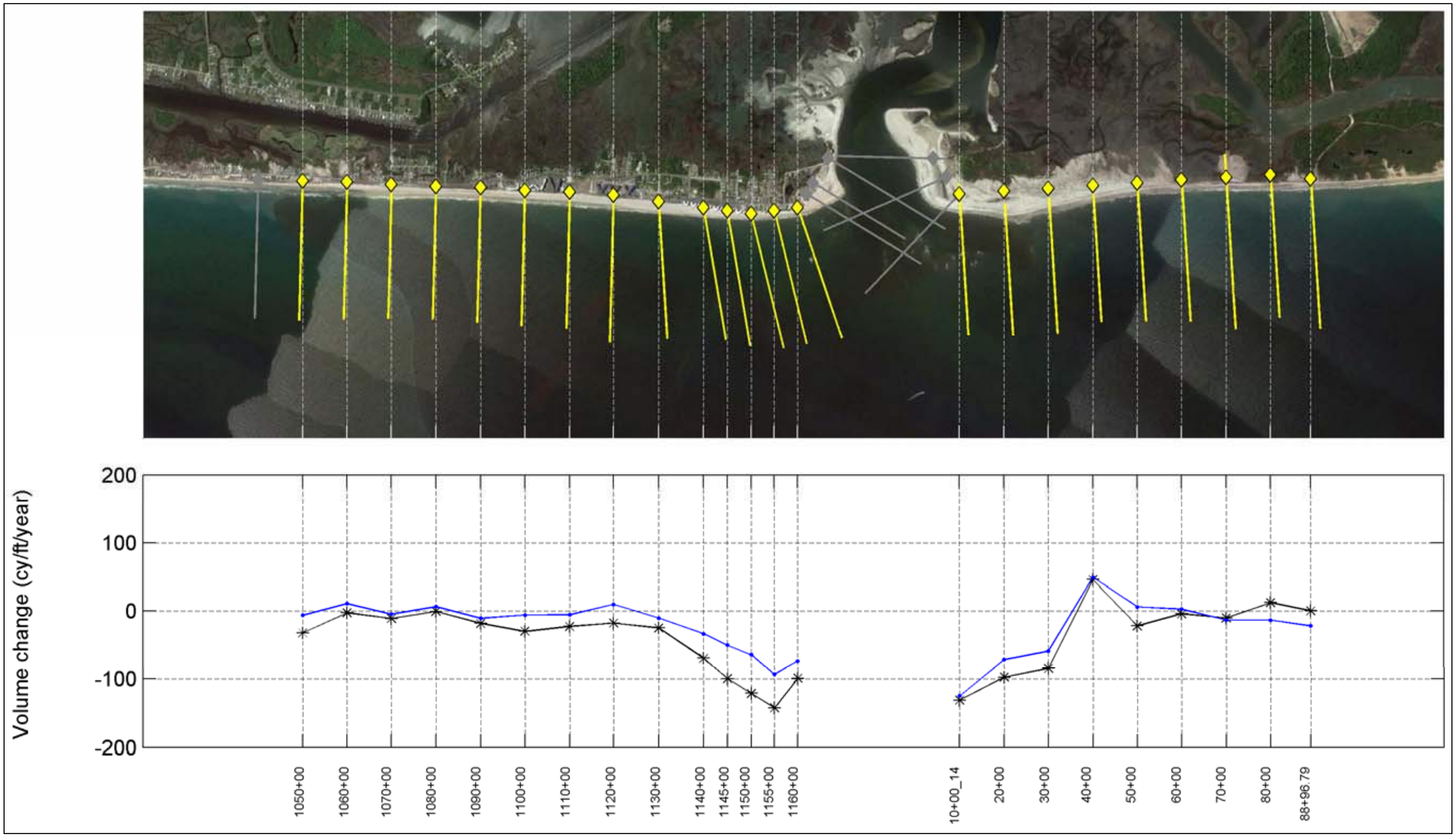

Figure 47. Simulated (blue line) and measured (black line) integrated cross-shore volume change.

2.4.3 Verification

Although the calibration process took into account one year of morphological changes, all of the models during that process simulated a period of 2 years. The second year was used as verification to make sure that the calibrated parameters can be useful in all models in that region for alternative performance evaluation, not just for the period of time analyzed during the calibration, which is where any project equilibration is expected to occur.

During this second year, some sidecasting dredging operations took place and the spit that was formed during 2013/2014 was also dredged. The dredging of the spit was taken into account by the model by manually removing the spit and restarting the simulation from that point. However, sidecast operations could not be inserted into the model, which introduced some uncertainties in the results.

The measured bathymetry in April 2015 can be seen on Figure 48, along with the simulated bathymetry from the verification run. From 2014 to 2015, the main channel starts bending eastward, and a secondary channel is created. The model also bends the channel eastward, and a secondary channel also appears. Although not precisely in the same location and depth, these changes in the channel are in good agreement with measured conditions.

On Figure 49, the cross-shore volume change of the verification run is shown together with the measured volume change for the same period, and the model performs reasonably well in that aspect, capturing the low magnitude of the erosion on the north end of North Topsail Beach and the higher erosion rates at Onslow Beach.

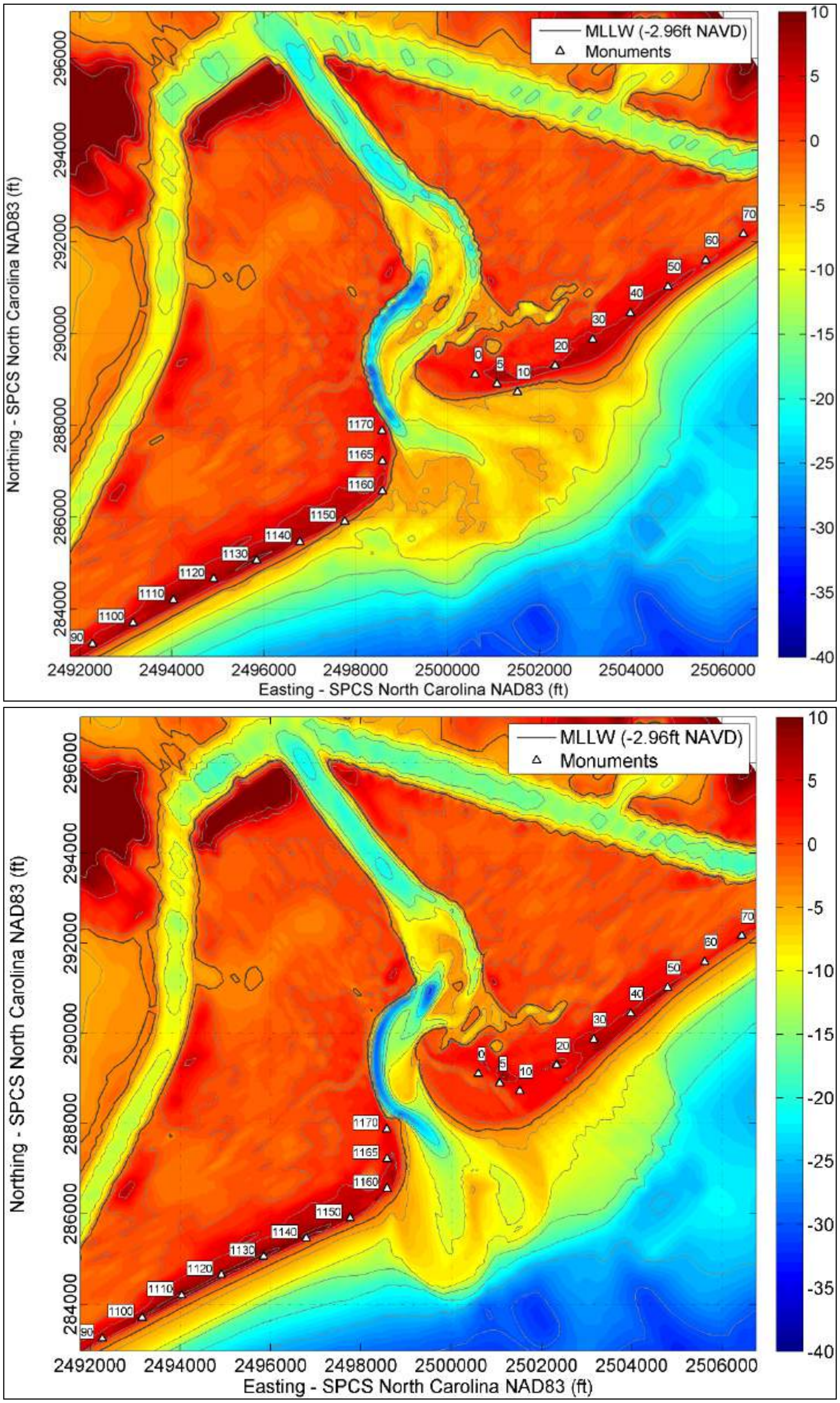

Figure 48. 2014 Measured 2015 bathymetry (top) and final bathymetry of the verification morphology model (bottom).

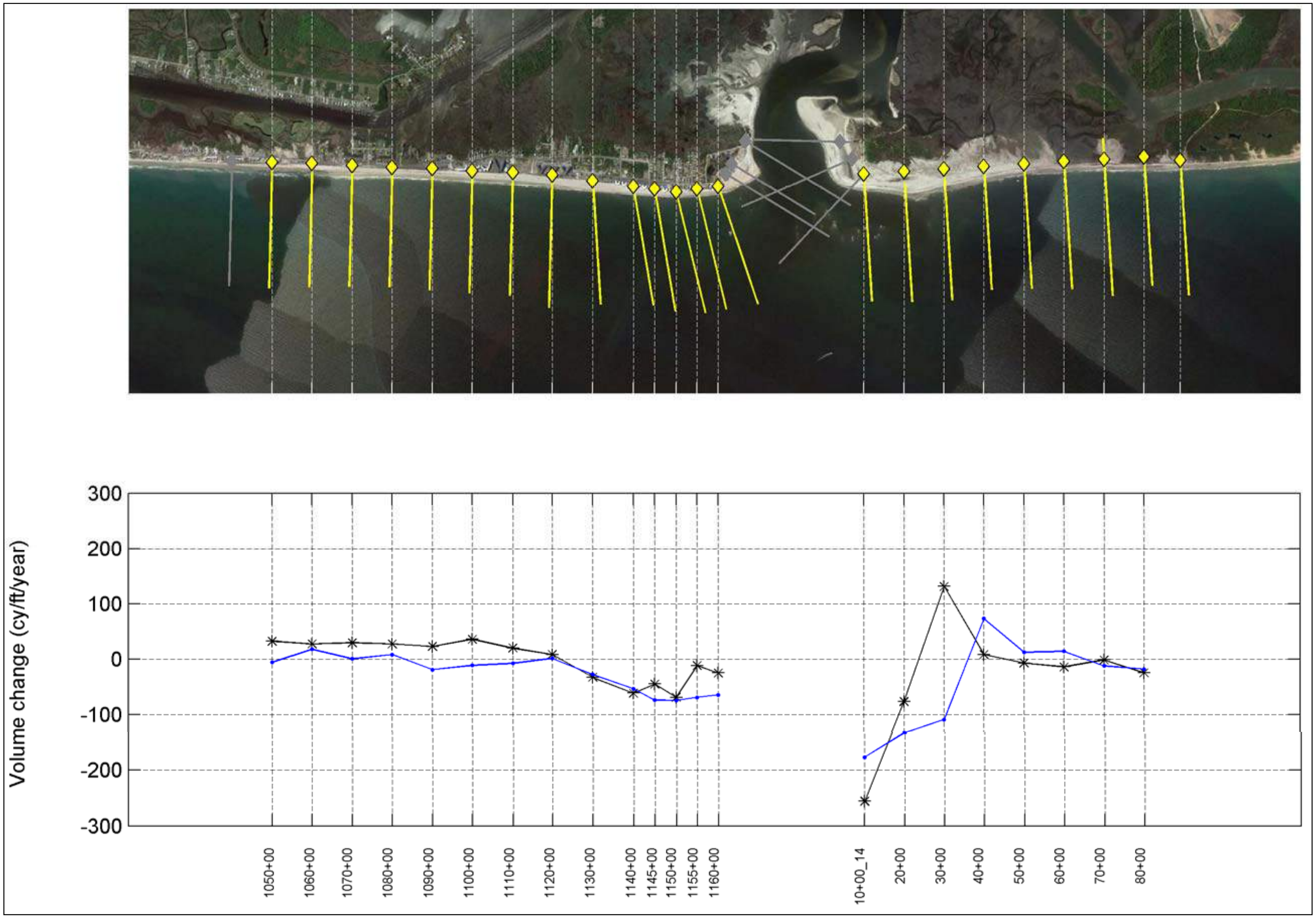

Figure 49. Simulated (blue line) and measured (black line) integrated cross-shore volume change from April 2014 to April 2015.

3.0 RESULTS

Results of the model simulations were evaluated based on six (6) primary criteria:

- Ebb Shoal Direct Impact – The portion of the ebb shoal that would be dredged through during construction of any of the alternatives.
- Ebb Shoal Buildup – The relative buildup of the south side of the ebb shoal over the 2-year simulation following construction of the alternative.
- Channel Depth and Width – The relative depth and width of the channel after the 2-year simulation period.
- Volume Change (North End of North Topsail Beach) – The measured volume changes generated over the 2-year simulations for each alternative.
- Sediment Transport Patterns – The relative differences between sediment transport patterns on the ebb shoal and along the adjacent inlet shorelines on both the Onslow Beach and North Topsail Beach side.
- Wave Energy Sheltering Effects – The relative differences in the reduction of wave energy output from the model during specific wave conditions along the north end of North Topsail Beach, the south end of Onslow Beach, and the ebb shoal.

In addition to these six criteria, all of the model results for alternative channels, as well as the No Action alternative, were evaluated to determine the tidal flow (tidal prism) through the inlet throat and through Cedar Bush Cut. In this regard, maintaining the tidal exchange through the inlet to something comparable to existing conditions would assure that there would be no measurable negative impacts on the estuarine habitats inside the inlet.

3.1 Ebb Shoal Direct Impact

The design concept for the New River Inlet channel realignment project was to realign the channel and cause the ebb tide delta of New River Inlet to reconfigure with a build-up of material on the south side and deflation of the north side. This concept is illustrated in Figure 50, which is taken from the engineering appendix to the 2009 EIS (CPE-NC, 2009). The design concept described in that report, stated that once the south side of the ebb tide delta fully responds to the new bar channel position and alignment, a process that could take 5 to 15 years, the reconfigured ebb delta should provide a protective buffer between offshore wave forces and the project shoreline.

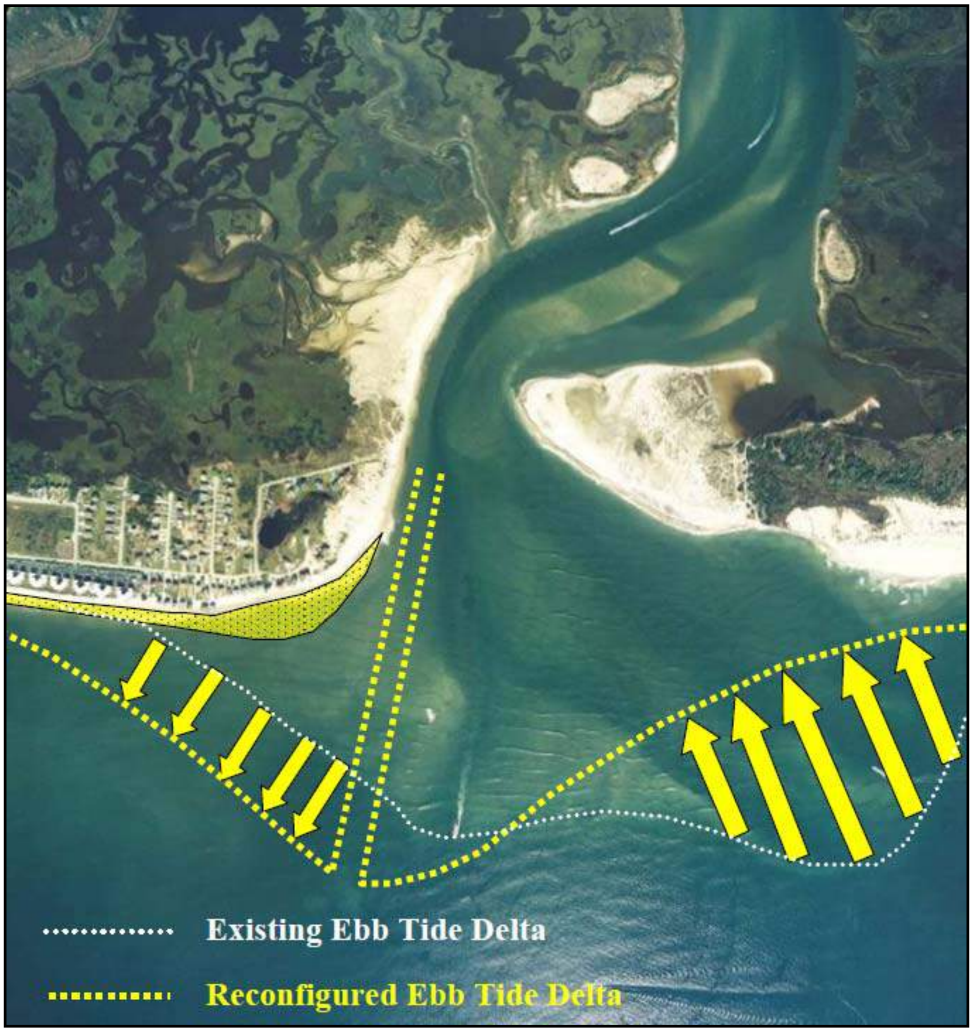

Figure 50. Conceptual illustration of the ebb shoal reconfiguration in response to channel realignment. (Photo is from October 2003)

Monitoring data collected over the course of two years following the construction of the initial channel realignment in 2013 shows the ebb shoal response to the channel realignment was consistent with the anticipated response. Figure 51 shows a difference plot representing changes that occurred between June 2012 (Pre-Construction) and April 2015. This captures the cumulative changes that took place in the first 2 years following construction.

On the Onslow Beach side of the figure, between the seaward ends of profiles 30+00 to 10+00 SW, the red shaded areas on the seaward edge of the shoal indicate losses in elevation (deepening) of the seafloor ranging from 2 to 8 ft. while the green shaded areas landward of the red areas represent sediment accumulations which were also in the range of 2 to 8 ft. Most of the sediment accumulation appears to be related to the onshore migration of the material eroded from the seaward edge of the delta as the north side of the ebb tide delta appears to collapse in response to the new flow pattern across the ebb tide delta created by the new channel. On the North Topsail Beach side of the ebb shoal there is a large buildup of material on the outer portion of the ebb shoal between Sta. 1145+00 and 1160+00. The observed trends in the reconfiguration of the ebb shoal are generally consistent with the conceptual shoal reconfiguration illustrated in Figure 50.

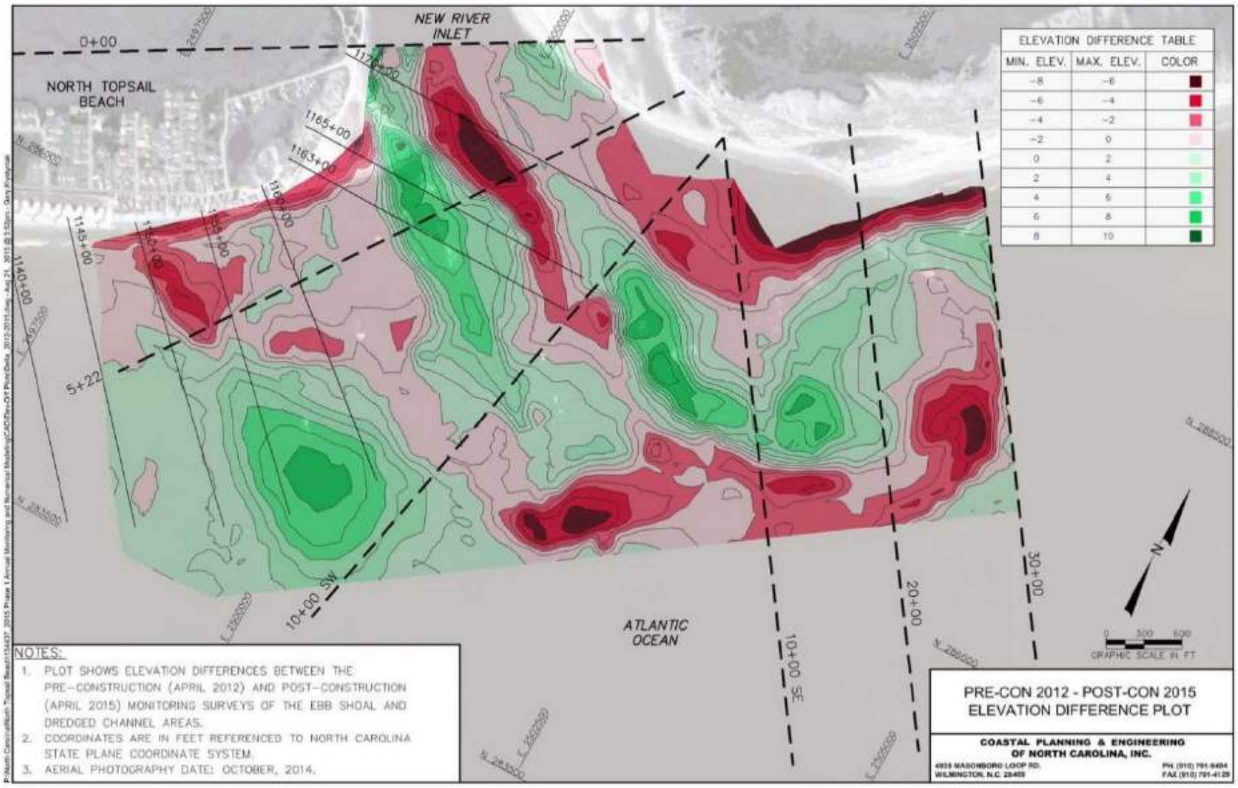

Figure 51. Difference Plot showing the changes in elevation on the ebb shoal between June 2012 and April 2015

Since the overall goal of the channel relocation project is to induce changes in the configuration of the ebb tide delta of New River Inlet that would result in a reduction in the volume of material north of the channel and a buildup of material on the south side, consideration was given to minimizing the direct impact of the new channel construction on the volume of material contained in the southwest lobe of the delta. The area of the ebb tide delta identified as the southwest lobe is shown in the first panel on Figure 52 for the No Action Alternative.

The impacts resulting from the excavation of the channel during construction of the various alternative channel designs on the southwest lobe of the ebb tide delta, referred to as direct impacts, are shown in Figure 52. Alternatives 1 and 2 have the least direct impact on the southwest lobe of the ebb shoal; however, as described below, the more northeastern positions of the Alternatives 1 and 2 channels provided the least amount of buildup on the south side of the inlet over the 2 year simulation period compared to the two options considered for Alternative 3 (Pivot Channel).

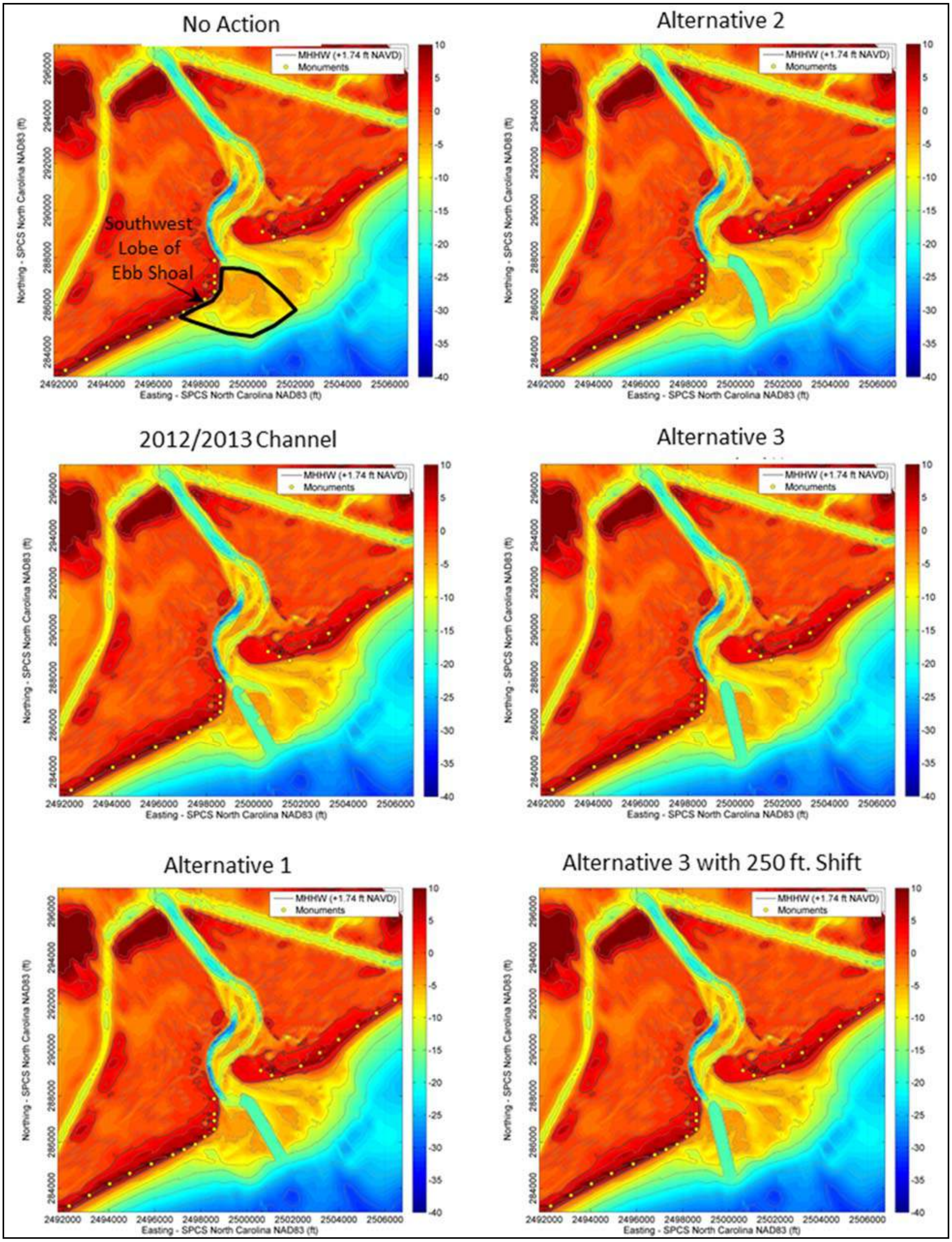

Figure 52. Initial bathymetry with alternative channel configurations showing the initial impact on the southwest lobe of the ebb shoal. Depth is shown in feet referenced to NAVD88.

3.2 Ebb Shoal Buildup

Results of the simulations for the various channel alternatives were used to evaluate the relative buildup of material within the area indicated by the polygon on the southwest side of the ebb tide delta, designated as NTBL in Figure 53. . The only difference between these simulations is the deepening of the channel alternatives within its footprint. Outside of the channel footprint, the initial bathymetric configuration used for each channel alternative was exactly the same.

Accumulation of material within this area is one of the primary desired responses to the channel realignment as this portion of the ebb shoal provides the most wave sheltering effect to the north end of North Topsail Beach. Figure 53 shows a comparison between the simulated bathymetry after 2 years for the No Action bathymetry and alternative channel configurations evaluated. Both the outline of the simulated channel and a polygon generally representing the southwest side of the ebb shoal are shown in Figure 53. Figure 54 shows areas where sediment was eroded (red shaded areas) and accreted (green shaded areas) over the two-year simulation period for the various alternatives. Table 10 summarizes the measured net volume changes within a fixed area denoted as NTBL, which is represented by the polygon shown in Figure 53 and Figure 54 for the various channel alternatives. The net volume takes into account the volume of material that would have been removed from the NTBL polygon to initially construct the channel.

Table 10. Modeled net volume changes in the NTBL area on the southwest side of New River Inlet

Alternative	Model Volume Change in NTBL (1000's CY)
No Action	41.61
2013 Channel	113.81
1	-45.67
2	-24.37
3-Pivoted 2013 Channel	198.29
3-Pivoted 2013 Channel w/ 250-ft Shift	247.76

The three channel alternatives that resulted in a net accumulation of material within the NTBL area were the 2013 Channel, Alternative 3-Pivoted 2013 Channel, and Alternative 3-Pivoted 2013 Channel shifted 250 to the west-southwest. The largest net accumulation occurred with the Pivoted Channel shifted 250 feet to the west-southwest.

The simulated year 2 results for Alternatives 1 and 2 show a buildup of sand adjacent to the channels; however, the more northern position of these two channels relative to Alternative 3 resulted in accumulations occurring north of the targeted NBTL area while volumetric changes within the NTBL area were negative, i.e., the area experienced a net loss of sediment.

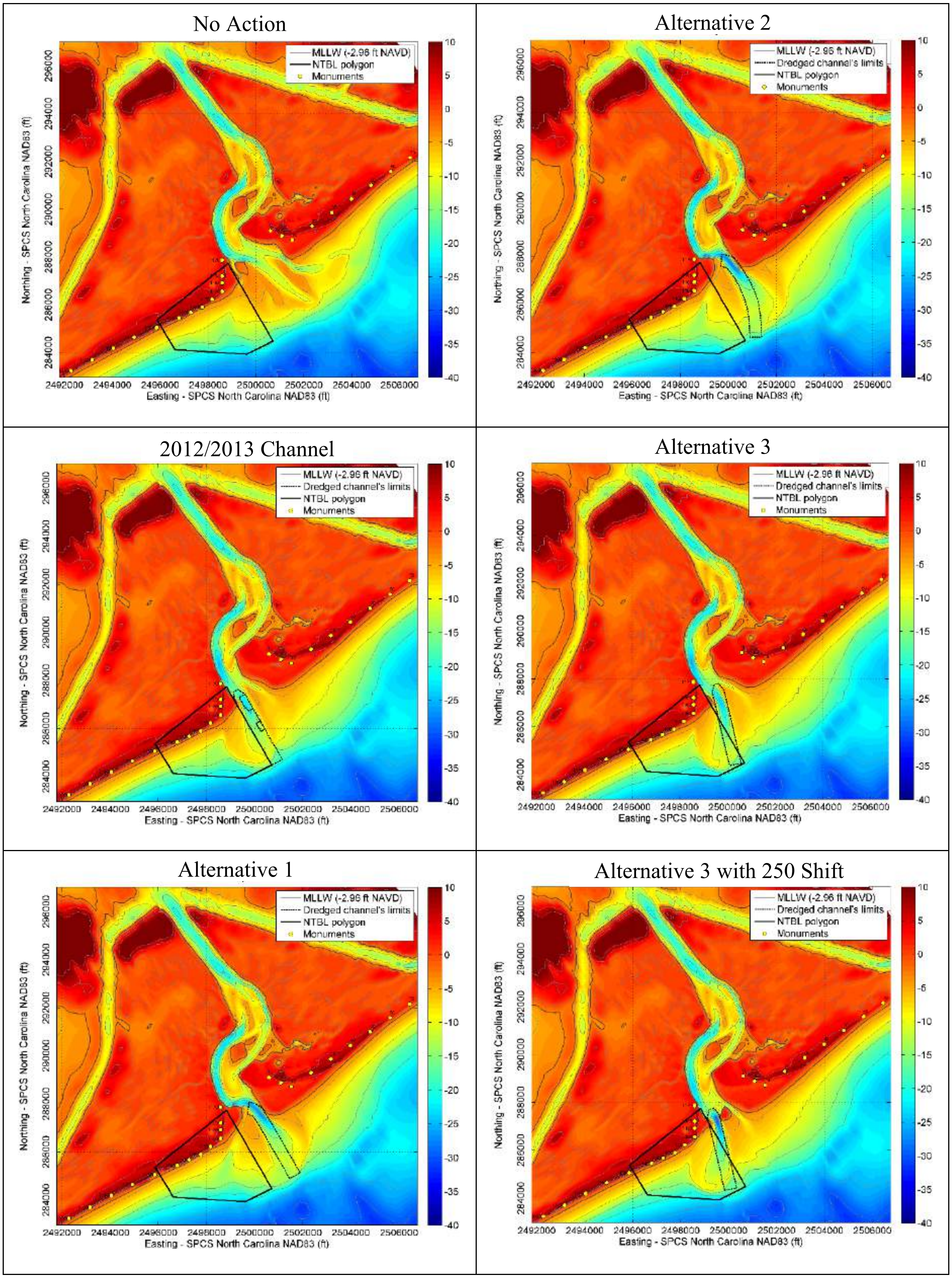

Figure 53. Year 2 simulated bathymetry with alternative channel configurations showing the shoaling within the targeted portion (NTBL) of the ebb shoal and the relative comparison of channel width and depth.

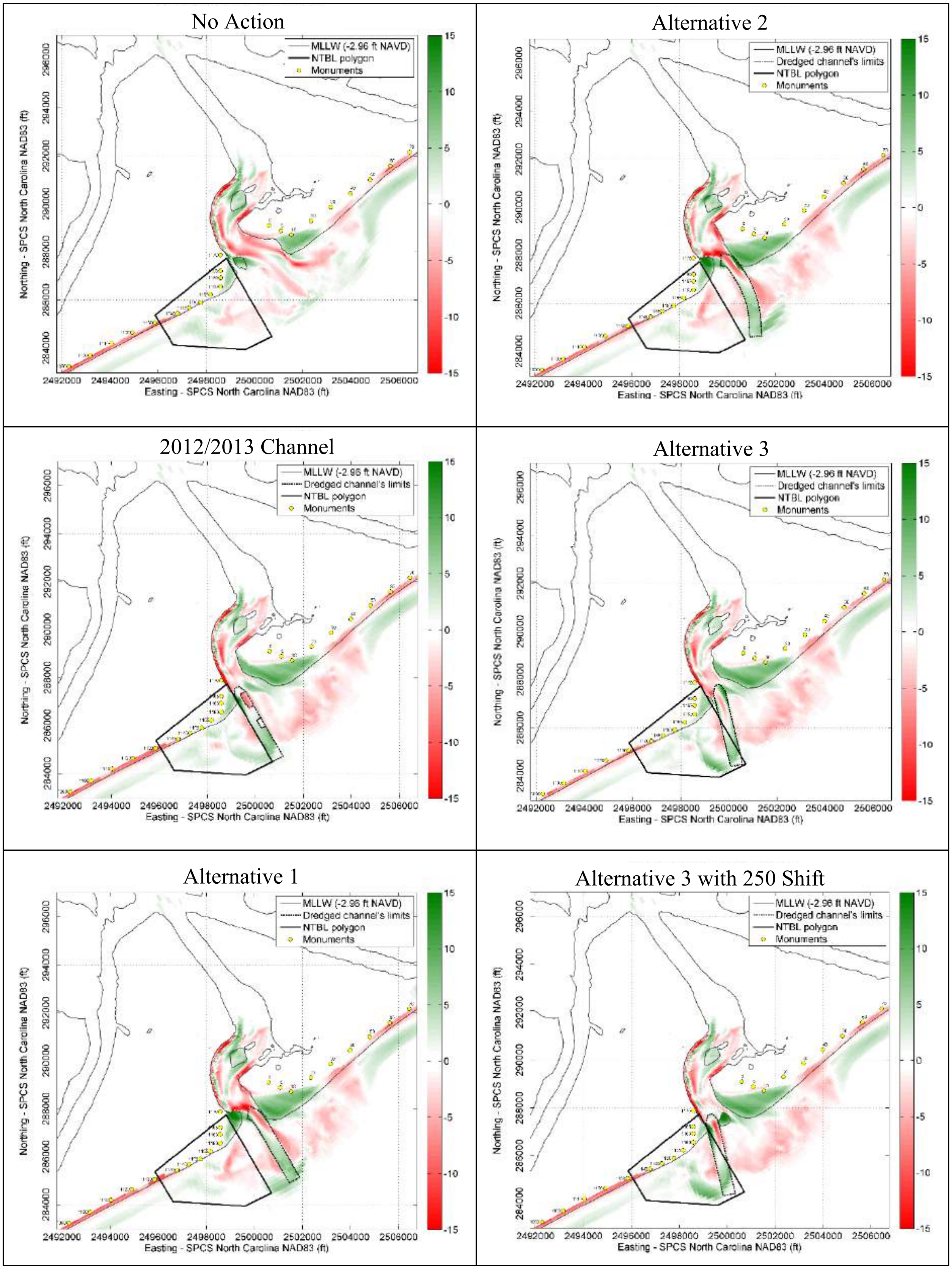

Figure 54. Erosion and Sedimentation after 2 years for Alternatives. Polygon shown denotes the extent of the NTBL area, referenced in Table 10.

3.3 Channel Depth and Width

Results of the simulations were also used to evaluate the relative depth and width of the channels after 2 years. The channel realignment concept is premised on the need to maintain the ocean bar channel in a preferred alignment and with sufficient depths and width to carry the majority of the tidal flow.

After the 2-year simulation period, Alternatives 1, 2, and 3 as well as Alternative 3 with the 250-foot channel shift maintained a wider channel across the ocean bar compared to the 2013 channel alignment (Figure 53). Although simulations for Alternatives 1 and 2 resulted in slightly wider channels, as noted above, the shoal seaward of the north end of North Topsail Beach (NTBL polygon) experienced a net loss of material, while Alternative 3 shifted 250-feet west-southwest, and the 2013 Channel caused sediment to accumulate in the polygon.

3.4 Volume Change – North End of North Topsail Beach

Model simulated volume changes were determined for each channel alternative in the areas along both North Topsail Beach and Onslow Beach landward of the -6-foot NAVD88 contour, between the -6-foot and -24-foot NAVD88 contours, and the entire profile landward of the -24-foot NAVD88 contour. Examples of the volume changes for Alternative 3 with and without beach fill were shown in Figure 6.. However, given the focus of this investigation, which is to minimize impacts to North Topsail Beach, only the volume changes along the north end of North Topsail Beach landward of the -6-foot NAVD88 contour are included in this discussion. Again, volume changes above the -6-foot NAVD88 contour serve as a proxy to represent changes in the recreational beach.

Volume changes landward of the -6-foot NAVD88 contour obtained after one year for all of the channel alternatives evaluated, including Alternative 3 with a 500-foot west-southwest shift and the 2013 Channel with beach fill, are provided in Figure 55. The shoreline area included in this figure extends from New River Inlet south to approximately Building #1 of Topsail Reef, which is situated near baseline station 1135+00. The volume changes reported on this figure are given in terms of cubic yards/linear foot of shoreline/year (cy/lf/yr).

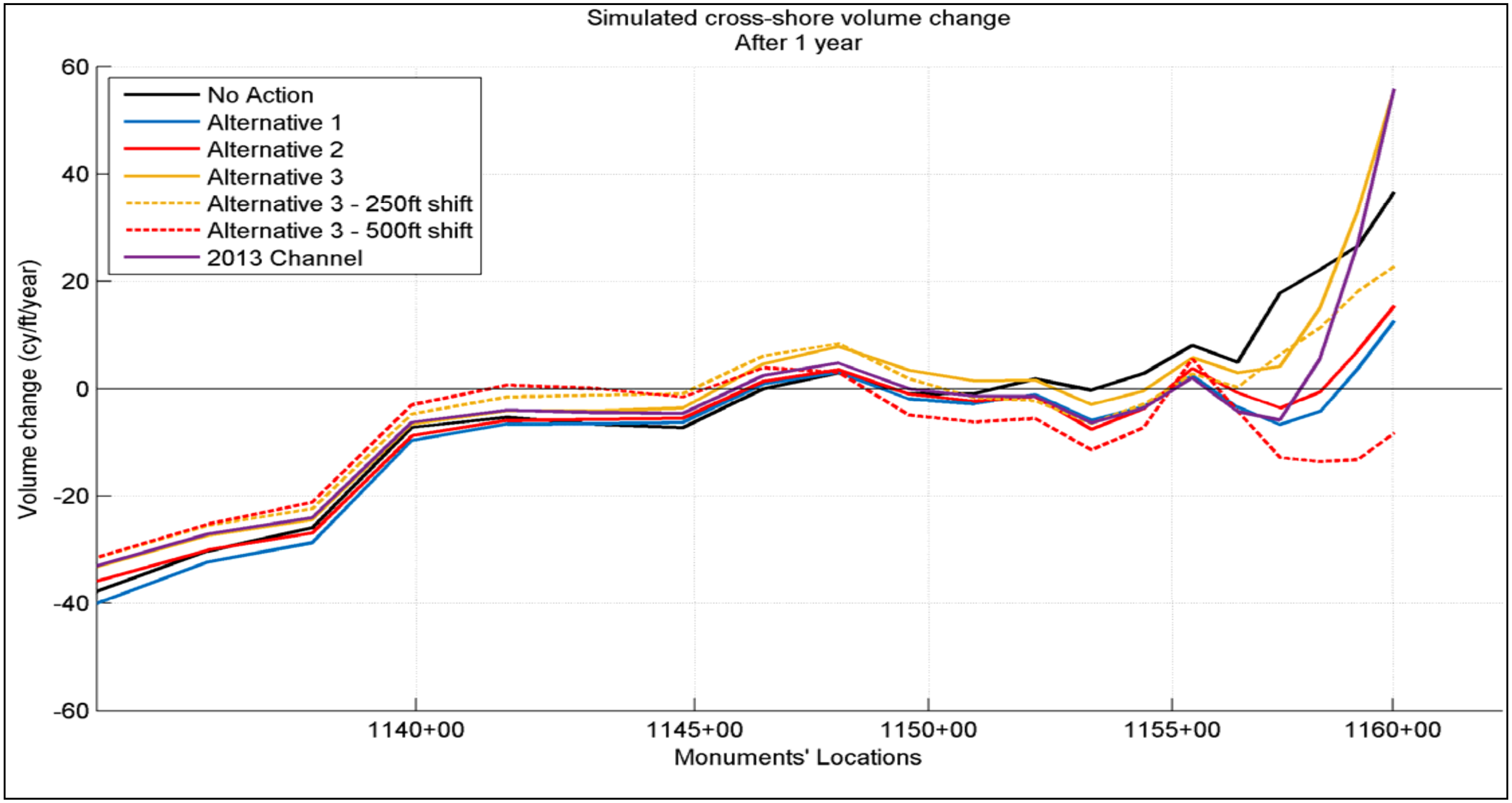

Figure 55. Graph showing the simulated volume change landward of the -6-foot NAVD88 contour along the north end of North Topsail Beach after 1 year. Simulated volume change is shown in cubic yards per foot per year.

Initial simulations of the No Action alternative, the 2013 channel, and alternative channels 1 and 2 showed very little difference in the shoreline response between Stations 1140+00 (between Buildings #5 and #6 of Topsail Reefs) and 1155+00 (2334 New River Inlet Road). Simulated volume change rates for those same channel configurations were all less than 20 cy/ft/yr in the area landward of the -6-foot NAVD88 contour.

Based on the volume changes shown in Figure 55, Alternative 3 produced positive volume changes above the -6-foot NAVD88 contour in the area north of baseline 1155+00. South of Station 1155+00, modeled volume changes above the -6-ft. NAVD88 contour were similar for all of the channel alternatives evaluated.

An additional alternative was simulated to represent what was constructed in 2013. Both the 2013 Channel and a comparable sized beach fill were constructed along the north end of North Topsail Beach. The intent of this simulation was to verify that the model would respond in a similar way to the way the inlet system actually responded following the 2013 initial channel realignment project. The "2013 Channel with beach fill" curve (black) in Figure 55 shows the simulated volume change measured for this alternative. The simulation showed much greater rates of volume change when fill was placed along the north end in a similar configuration as what was placed during the 2013 project. In comparing the "2013 Channel" results (maroon) with the "2013 Channel with beach fill" results (black), the simulation with beach fill show volume change rates between 2 and 7 times higher between Stations 1135+00 and 1155+00.

3.5 Sediment Transport Patterns

Simulated sediment transport patterns were also analyzed and compared to determine which channels configuration would best promote the preferred reconfiguration of the ebb shoal. Figure 56 shows the mean sediment transport over the first year of the simulation for the No Action alternative. The lighter colors correspond to higher sediment transport rates. Arrows show the average direction sediment is moving over the course the one-year simulation. The patterns reflected in Figure 56 are reflective of a channel configuration similar to the one present in October 2015 at New River Inlet, shown in Figure 57. The model suggests that sediment transport patterns toward the outer portion of the ebb shoal are highest in the channel areas. Sediment transport along the rest of the ebb shoal are either moving back toward the mouth of the inlet or moving along shore as sediment is bypassed around the inlet.

Sediment transport patterns for each of the alternative channel configurations were evaluated to assess which alternative simulation best promoted the intended reconfiguration of the ebb shoal. Figure 58 shows the comparison between the measured mean sediment transport after a 1-year simulation for the No Action and alternative channel configurations. All of the alternatives generally promote transport of sediment to the apex of the dredged channel and the migration of sediment landward along the northeast lobe of the ebb shoal (Onslow Beach side of the shoal). However, the position of the apex of the 2013 and Alternative 3 channels being farther to the southwest than the Alternative 1 and 2 channels, may promote a greater buildup to the southwest lobe of the ebb shoal as desired. This is evidenced in the bathymetric surfaces generated from the 2-year simulation shown in Figure 53.

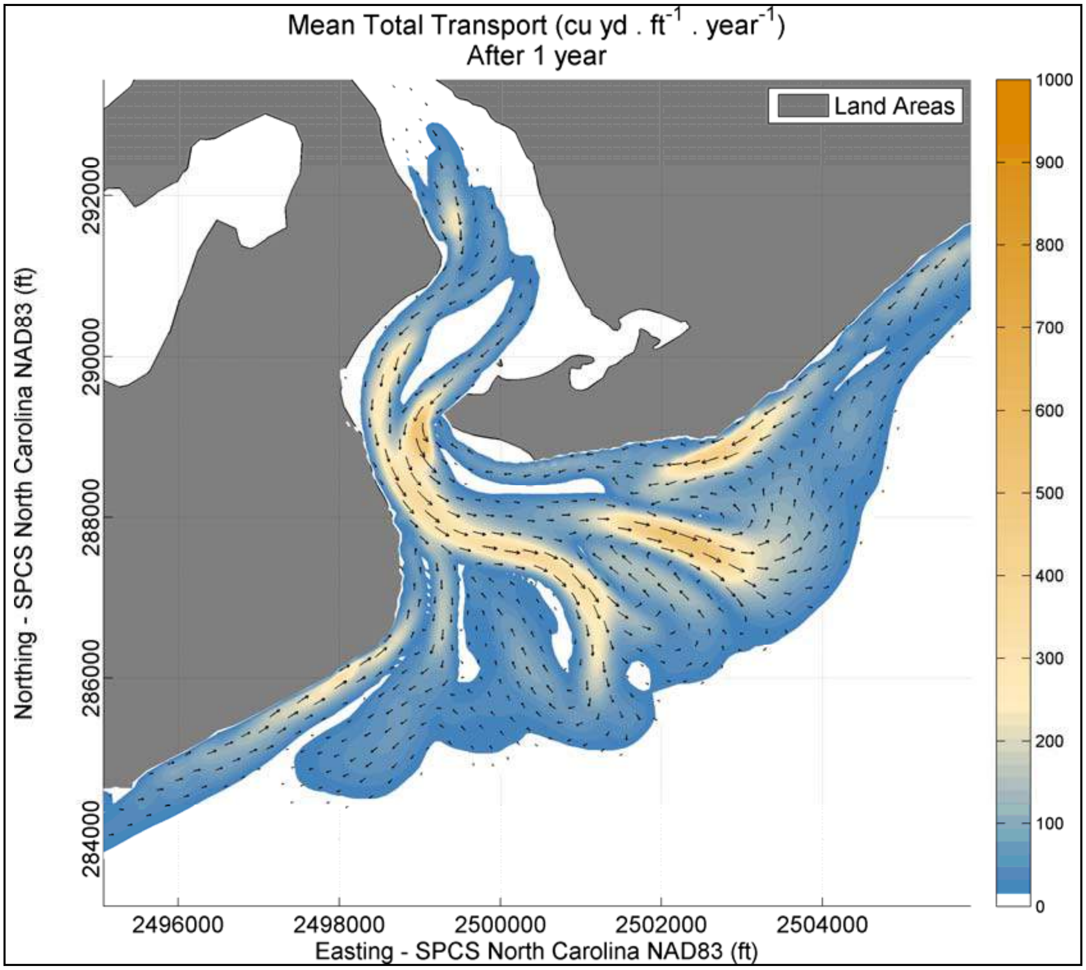

Figure 56. Map showing simulated sediment transport patterns for the no action alternative. Simulated mean transport is shown in units of cubic yards per foot per year. Arrows indicate the direction of the transport.

Sediment transport patterns were also used in conjunction with erosion/sedimentation change plots to evaluate the primary origins of the sand shoaling in the channel. Figure 59 shows a comparison between the sediment transport patterns and the erosion/sedimentation change plot for the 1-year simulation of the 2013 channel configuration. The erosion/sedimentation plot shows erosion in red and sedimentation in green. Erosion on either side of the channel suggests some of the sand filling in the channel is coming from these regions. The erosion/sedimentation plot does not show significant erosion of the nearshore portion of North Topsail Beach that would be expected if sand was moving from the beach to fill the channel. The sediment transport map suggests that a high rate of transport is coming from inside the inlet system into the channel.

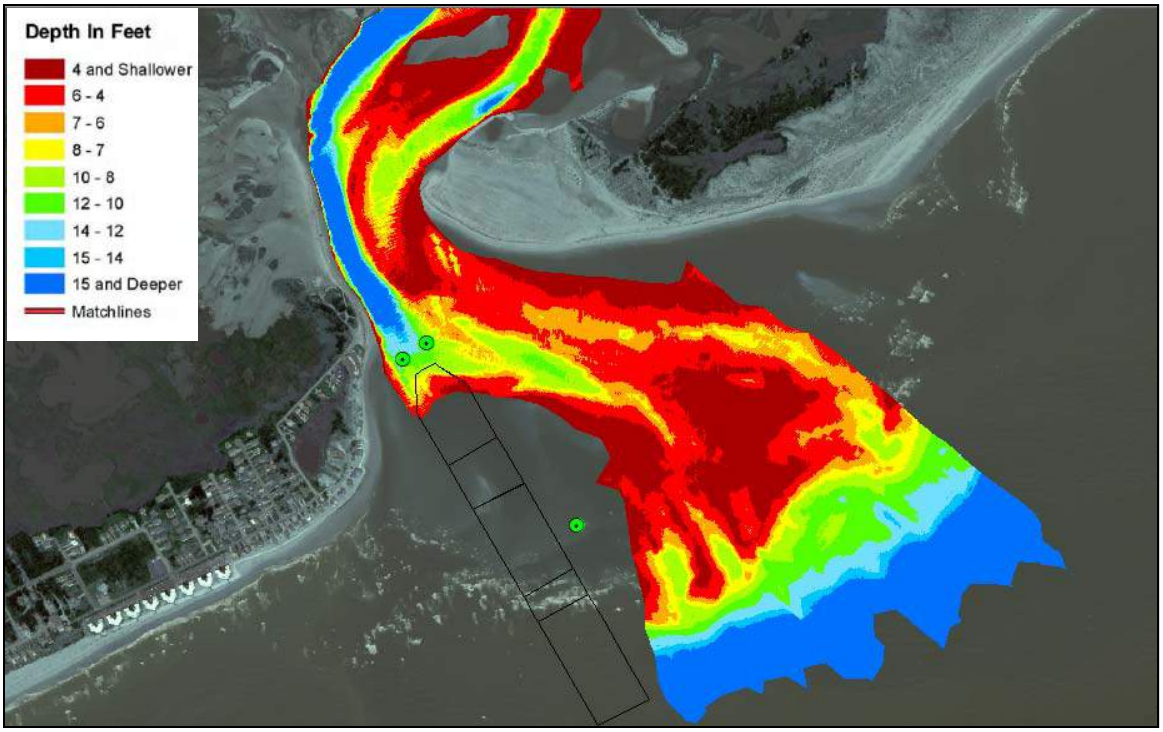

Figure 57. Map showing the condition survey conducted by the US Army Corps of Engineers in October 2015. Background imagery provided by USACE, flown on September 10, 2015. © 2015 DigitalGlobe Nextview License.

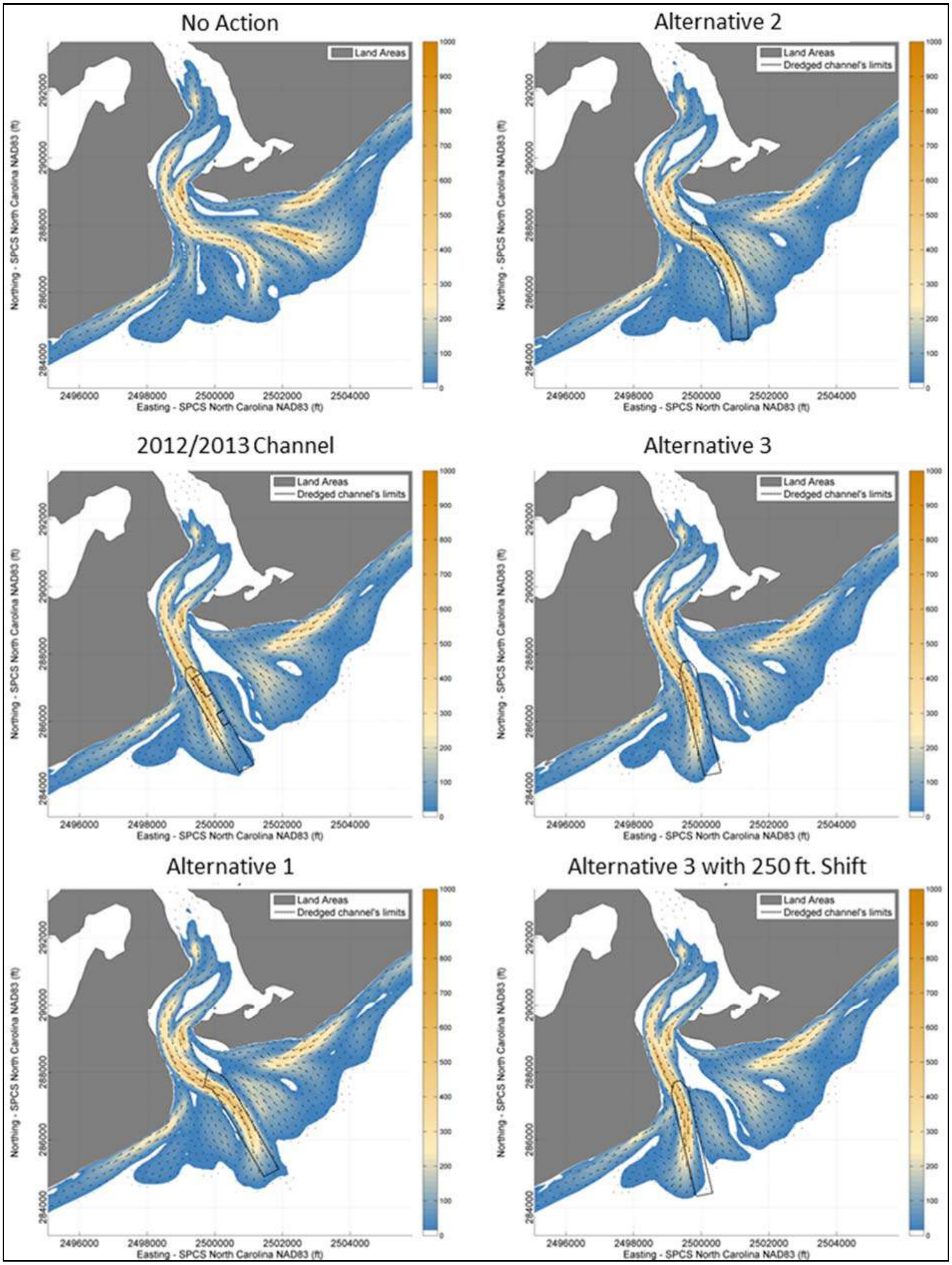

Figure 58. Simulated sediment transport patterns for the alternative channel configurations showing the mean total transport over a 1 year simulation in cubic yards/ft./year.

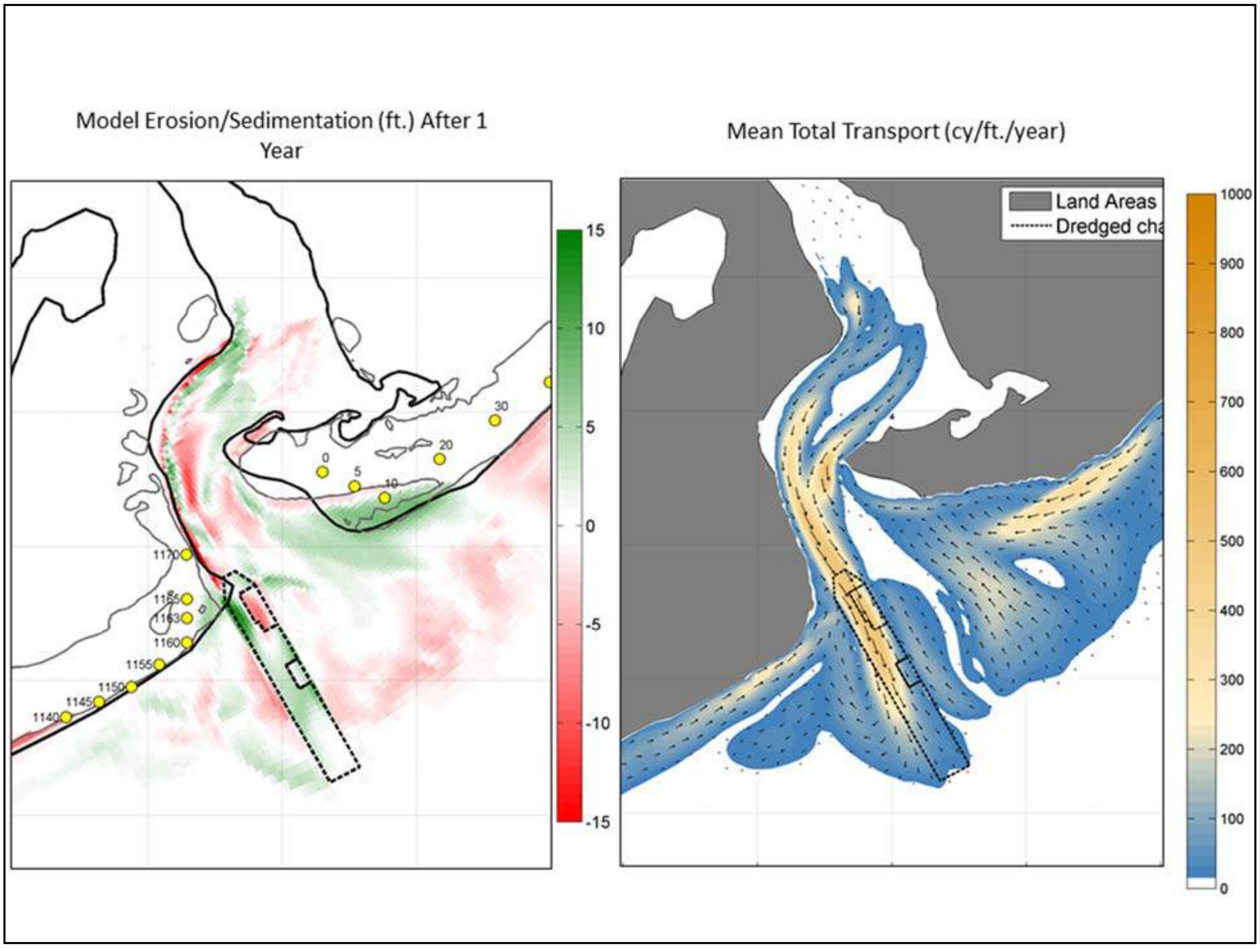

Figure 59. (Left) Map showing erosion/sedimentation change plot for the 1 year simulation of the 2013 channel configuration; and (right) map showing the sediment transport patterns for the 1 year simulation of the 2013 channel configuration.

3.6 Wave Energy Sheltering Effects

Results of the Delft3D model simulations were used to assess the wave sheltering effect of the ebb shoal configurations for each of the alternative channel configurations. Sixteen (16) schematized wave cases were evaluated during the model simulations. Out of the 16 wave cases, case #8 was determined to have the highest transport potential based on the frequency of such an event and the wave characteristics. Case #8 has a significant wave height (H_s) of 9.3 ft., a peak wave period (T_p) of 9.33 seconds, and a peak wave direction (D_p) of 122 degrees (waves approaching from the east-southeast). A transect was created running generally parallel to shore across the project area. Figure 60 shows the location of the transect (black dotted line). Significant wave heights (H_s) were extracted from the model at each location along the transect for each of the different alternative channel configurations and plotted to create the graph in Figure 61.

Figure 61 shows the data extracted between Station 1120+00 (middle building at the St. Regis) and the inlet for each of the alternative channel configuration. The graph shows that for wave case

#8, Alternatives 1, 2, and the No Action alternative, configurations have similar significant wave heights between Station 1150+00 (Oyster Lane) and the north end of the island. The 2013 channel and Alternative 3 (pivot channel) had the greatest reduction in H_s north of Oyster Lane for wave case #8. South of Oyster Lane, the model simulations showed a slight reduction in H_s for the 250 ft. and 500 ft. shifted versions of Alternative 3; however, the reduction in H_s for these two alternatives do not provide as great a reduction north of Oyster Lane as do the 2013 channel and Alternative 3 (pivot channel).

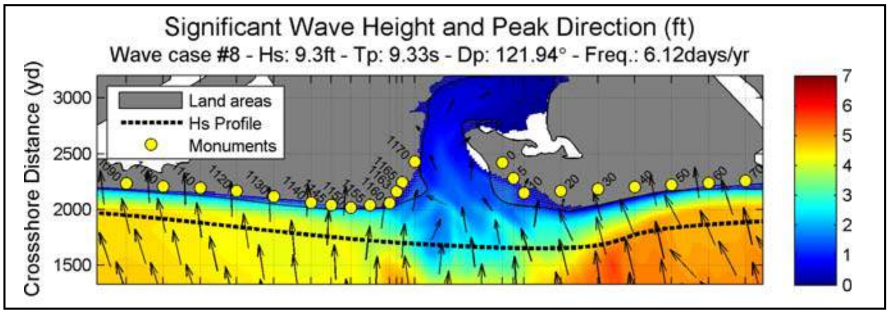

Figure 60. Map showing the location of the transect (black dotted line) along which significant wave height (H_s) data were extracted to evaluate wave sheltering.

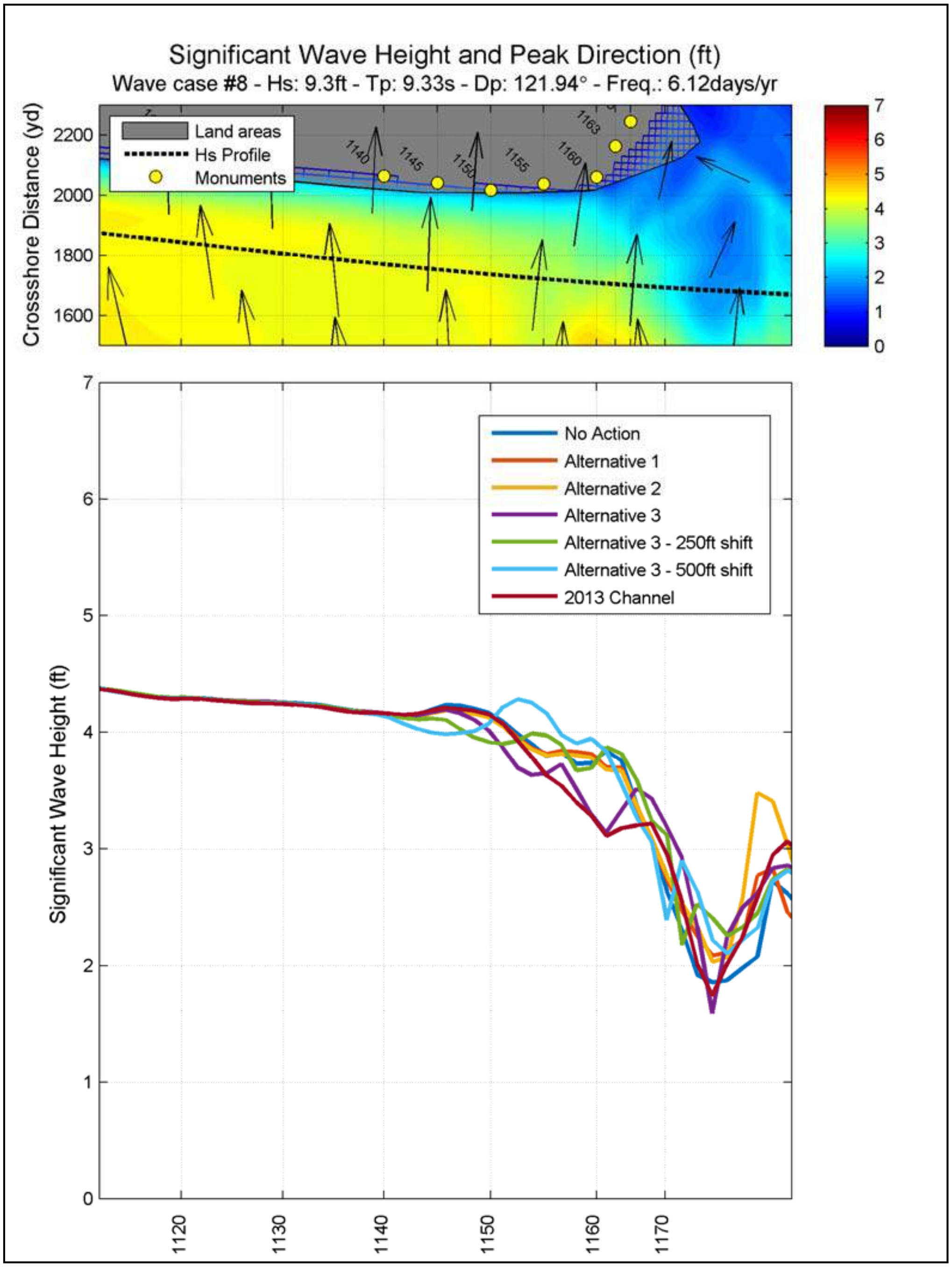

Figure 61. Graphic showing the comparison of simulated significant wave height (H_s) throughout the study area. Significant wave heights shown in the lower graph are based on data extracted along the transect shown in the map at the top of the figure.

3.7 Tidal Prisms

The volume of water flowing through the New River Inlet throat as well as the volume of water flowing through Cedar Bush Cut (Cross-Sections A & B in Figure 62, respectively) during the ebb cycle was computed by averaging all of the ebb flows simulated for the period between May 2, 2012 to May 31, 2012 for all channel alternatives using the Regional flow domain so that inland boundary conditions do not affect the water flow through the inlet. The volume of water flowing out of an inlet during the ebb phase of the tidal cycle is referred to as the tidal prism. The results are provided in Table 11.

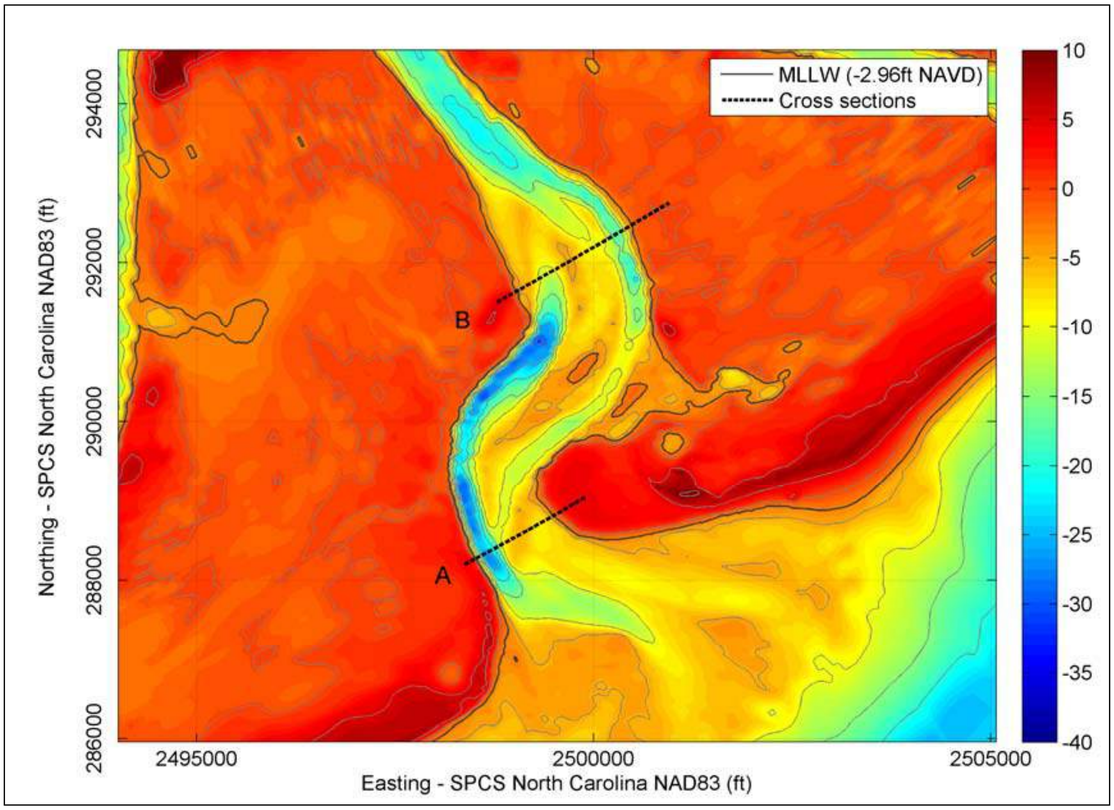

Figure 62. Cross-Sections used to compute model tidal prisms.

Table 11. Average Ebb Tidal Prism for Alternatives.

Alternative	Mean Tidal Prism (10^6 ft^3)		Percent Difference Relative to No Action	
	A	B	A	B
No Action	509.8	503.0	--	--
Alternative 1	526.0	519.2	3.2%	3.2%
Alternative 2	521.1	514.4	2.2%	2.3%
Alternative 3	521.1	514.4	2.2%	2.3%
Alternative 3-250 ft. Shift	524.9	518.1	3.0%	3.0%
2013 Channel	526.8	519.8	3.3%	3.3%

For all of the alternatives including No Action, the volume of water flowing through Cross-Section A (the inlet throat) was consistently about 1.3% greater than the flow measured across Cross-Section B in Cedar Bush Cut. This difference in flow volume is to be expected given the volume of water that empties into the area between the two cross-sections through tidal creeks behind the north end of North Topsail Beach and the south end of Onslow Beach that does not pass through Cross-Section B in Cedar Bush Cut.

Alternatives 2 and 3 had the least impact on the tidal prisms relative to the No Action Alternative with flow volume increases in the range of 2.2 to 2.3%. All the other alternatives resulted in volume increases of between 3.0 and 3.3% relative to No Action. These relatively small differences in flow volume for all the channel alternatives considered implies there would be essentially no measurable difference in impacts on the estuarine environment associated with a change in the tidal regime of New River Inlet that could be caused by any of the channel alternatives evaluated.

4.0 CONCLUSIONS AND RECOMMENDATIONS

A numerical modeling analysis of the New River Inlet ocean bar and adjacent shorelines was completed using Delft3D to assess alternative channel configurations for the proposed channel realignment project. The model was used to evaluate model-indicated volumetric changes along the adjacent shorelines, changes in the ocean bar channel (channel orientation, shoal volumes, channel depths, etc.), volumetric changes on the ebb tide delta, and sediment transport patterns. Simulations included a No Action alternative, the 2013 channel configuration, three (3) primary alternatives (Alternatives 1, 2, and 3), and 5 secondary alternatives. The primary alternatives were all 500 ft. wide with a bottom depth of -18 ft. NAVD88. The secondary alternatives included a 400 ft. wide -18 ft. NAVD88 version of Alternative 2, a 500 ft. wide -15 ft. NAVD88 version of Alternative 2, a 400 ft. wide -15 ft. NAVD88 version of Alternative 2, and two modified versions of Alternative 3, one shifted 250 ft. and one shifted 500 ft. toward the north end of North Topsail Beach.

Based on the results of the Delft3D model analysis, the following are the primary findings of the study:

- Although Alternatives 1 and 2 have less of a direct impact on the dredging of the established southwest lobe of the ebb shoal, Alternative 3, the 250 ft. and 500 ft. shifted versions of Alternative 3, and the 2013 channel configurations result in a greater buildup of sand on the southwest lobe of the ebb shoal fronting North Topsail Beach.
- Model simulations show similar channel widths and depths after 2 years for Alternatives 1, 2, and 3. The secondary alternatives (shallower, narrower, and shallower and narrower versions of Alternative 2) resulted in greater shoaling of the simulated channels after 2 years.
- Alternative 3 and Alternative 3-shifted 250 ft. simulations resulted in the most favorable beach performance along the north end of North Topsail Beach between Stations 1140+00 and 1160+00.
- High rates of erosion of the sand placed as part of the Phase 1 project along the north end of North Topsail Beach are due to the creation of a shoreline alignment out of equilibrium

with existing conditions. The unnatural alignment of the shoreline was quickly reworked to resume an orientation similar to the pre-project orientation. This has been observed from numerous navigation maintenance projects during which disposal material placed on the north end of North Topsail Beach was reworked over the course of weeks to months until the shoreline alignment resumed a pre-project alignment. The most recent example of this was the project constructed in March and April 2016.

- All simulated channel alternatives show similar transport patterns through the channel and along the southeast lobe of the ebb shoal (Onslow Beach side). Simulated results of Alternative 3 and Alternative 3-shifted 250 ft. direct the transport in a more preferable location on the ebb shoal to promote the preferred reconfiguration. Simulated results of Alternative 3 and Alternative 3-shifted 250 ft. also show a reduced sediment transport gradient on the north end of North Topsail Beach which may result in a slowing of the sand transport to the spit on the north end of North Topsail Beach.
- Simulated sediment transport patterns and erosion/sedimentation patterns suggest that most of the material filling in the channel is not coming from the beach, but rather the adjacent shoals and the interior inlet system.
- The 2013 channel and Alternative 3 had the greatest reduction in H_s (significant wave heights) north of Oyster Lane for wave case #8, which represents the wave case resulting in the greatest sediment transport compared to all other wave cases.
- None of the channel alternatives had a significant impact on the tidal prism of New River Inlet.

Based on the findings listed above the following recommendations are provided to the Town of North Topsail Beach for its consideration:

- Based on the analysis described in this report, the geomorphic analysis conducted by CPE-NC as part of the EIS (2009), and the monitoring conducted by CPE-NC since the Phase 1 project was completed (CPE-NC, 2014; CPE-NC, 2016a and CPE-NC, 2016b), Alternative 3 is the recommended channel alternative for the next channel realignment project.
- Proceed with obtaining permit modifications to allow for the construction of Alternative 3 for the next scheduled channel realignment project.
- Run model simulations to identify an optimal beach fill for the north end for the next channel realignment project
- Determine the effects of reducing the recommended channel depth and width as a contingency if project costs exceed available funds

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Comparison Plots of the
Simulated and Measured Current and Water Level Data at
17 Locations Used During the Flow Calibration
Regional and Local Flow Model Calibration Plots

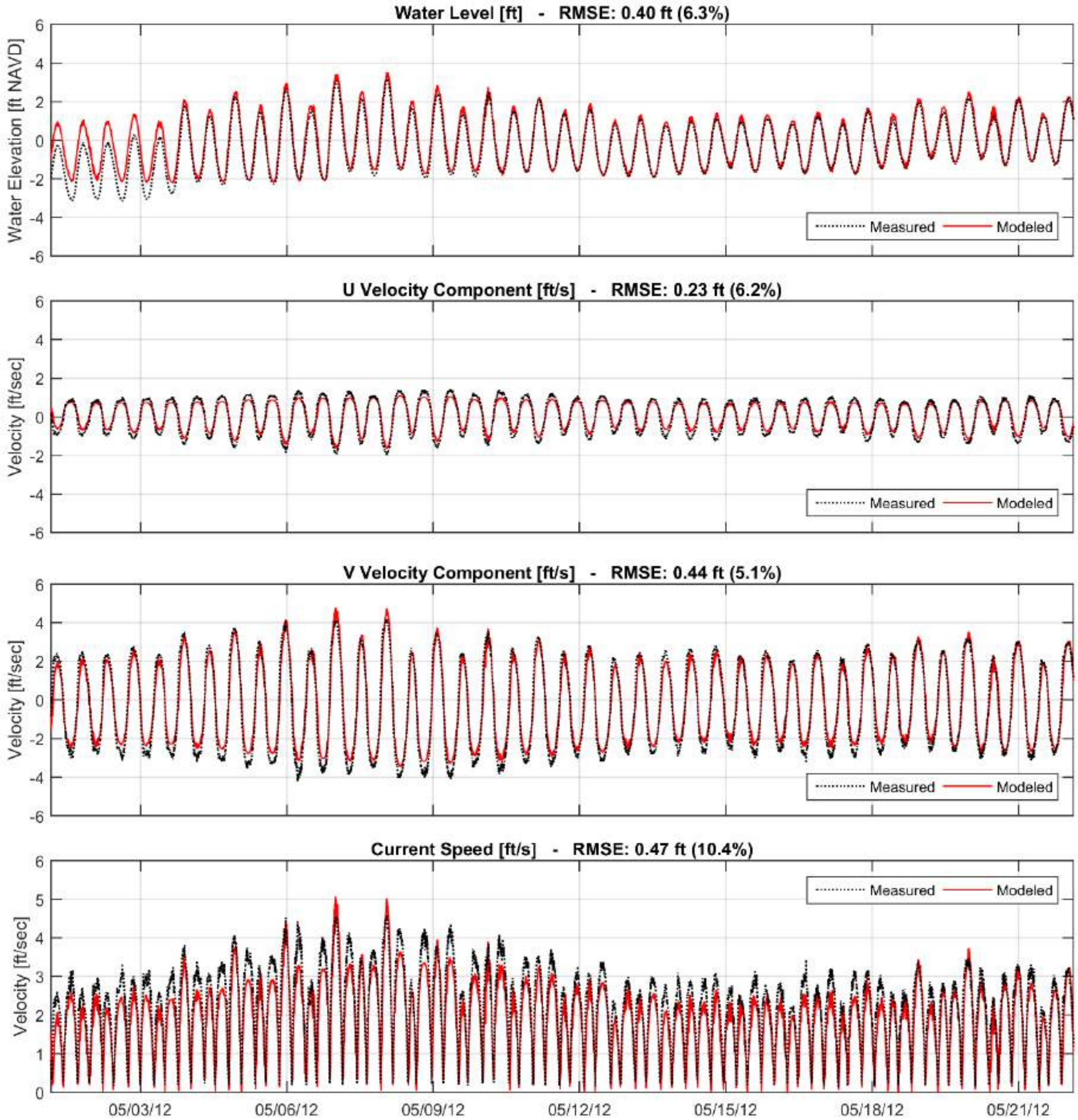
APPENDIX A

Comparison Plots of the Simulated and Measured Current and Water Level Data at 17 Locations Used During the Flow Calibration

Regional and Local Flow Model Calibration Plots

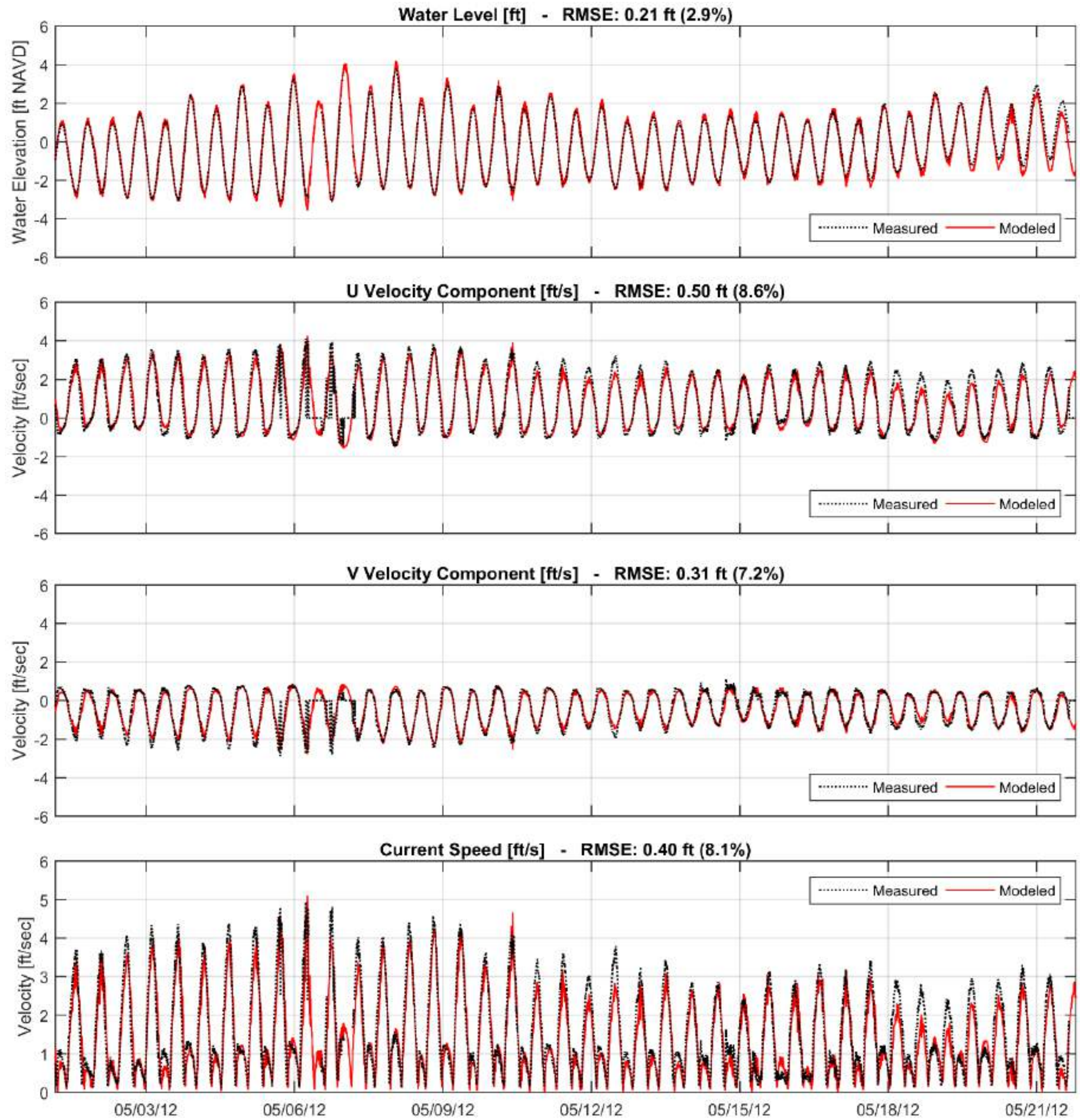
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

SIO Gauge – SIO-01



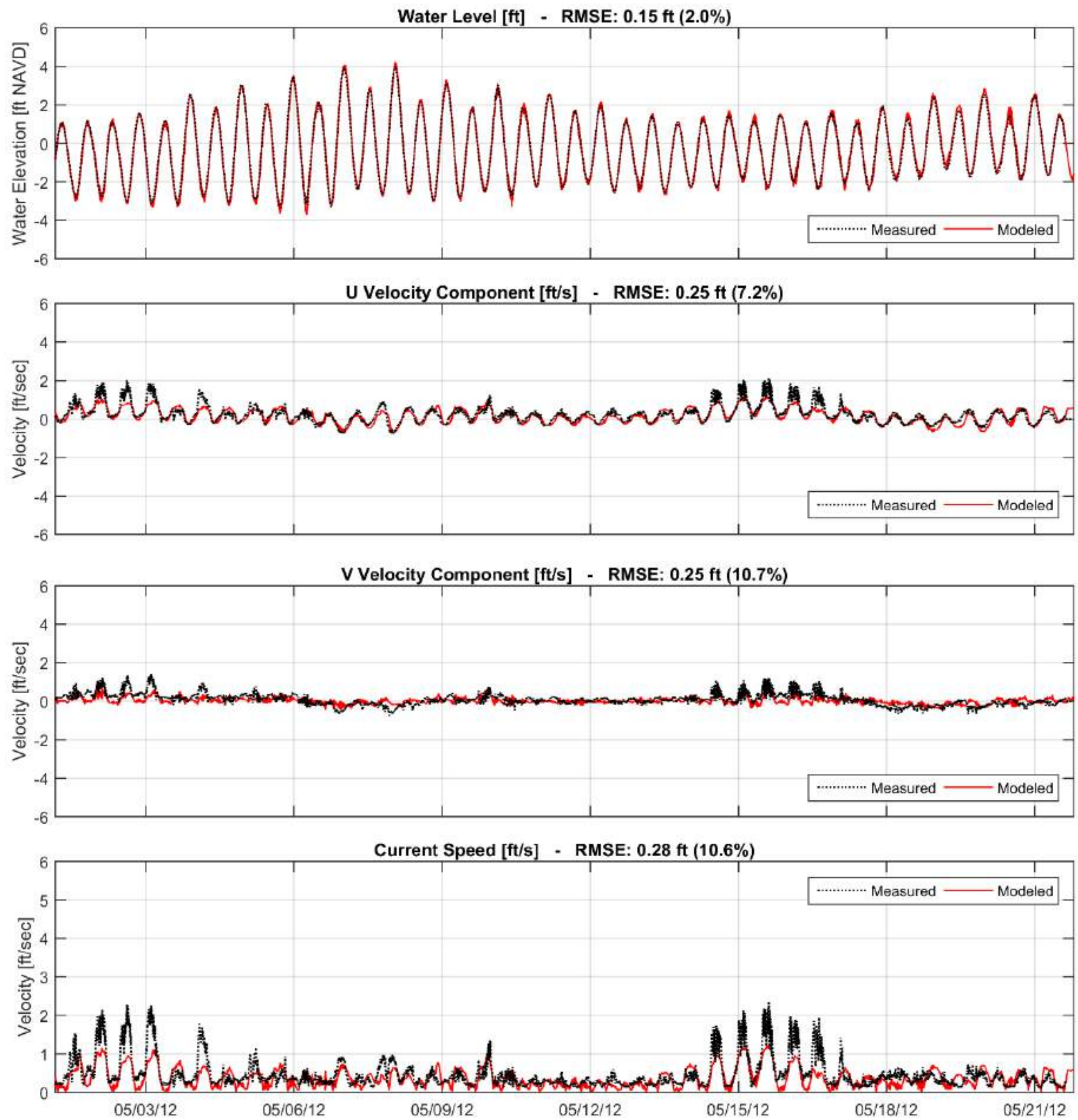
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

SIO Gauge – SIO-02



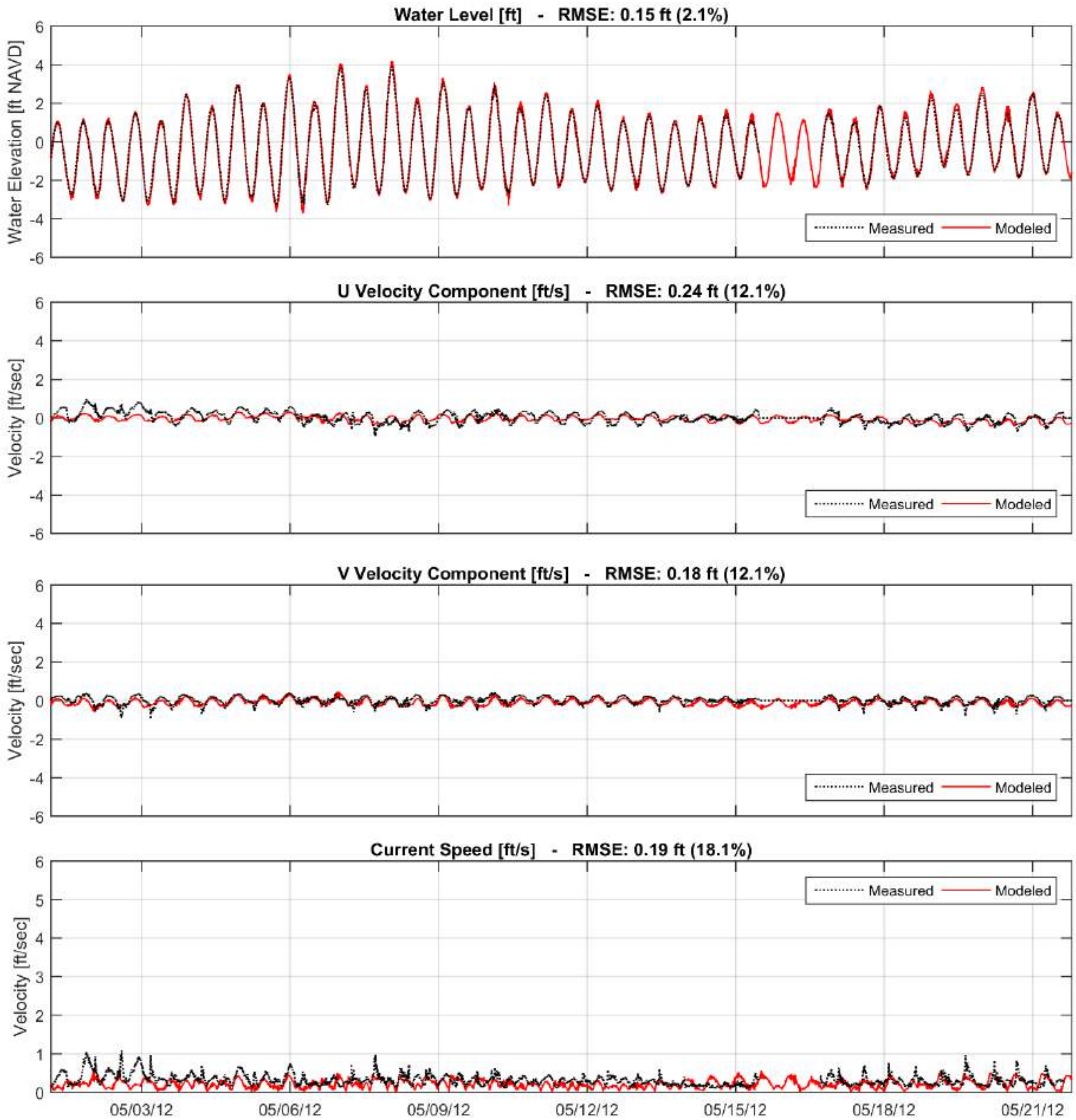
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

SIO Gauge – SIO-03



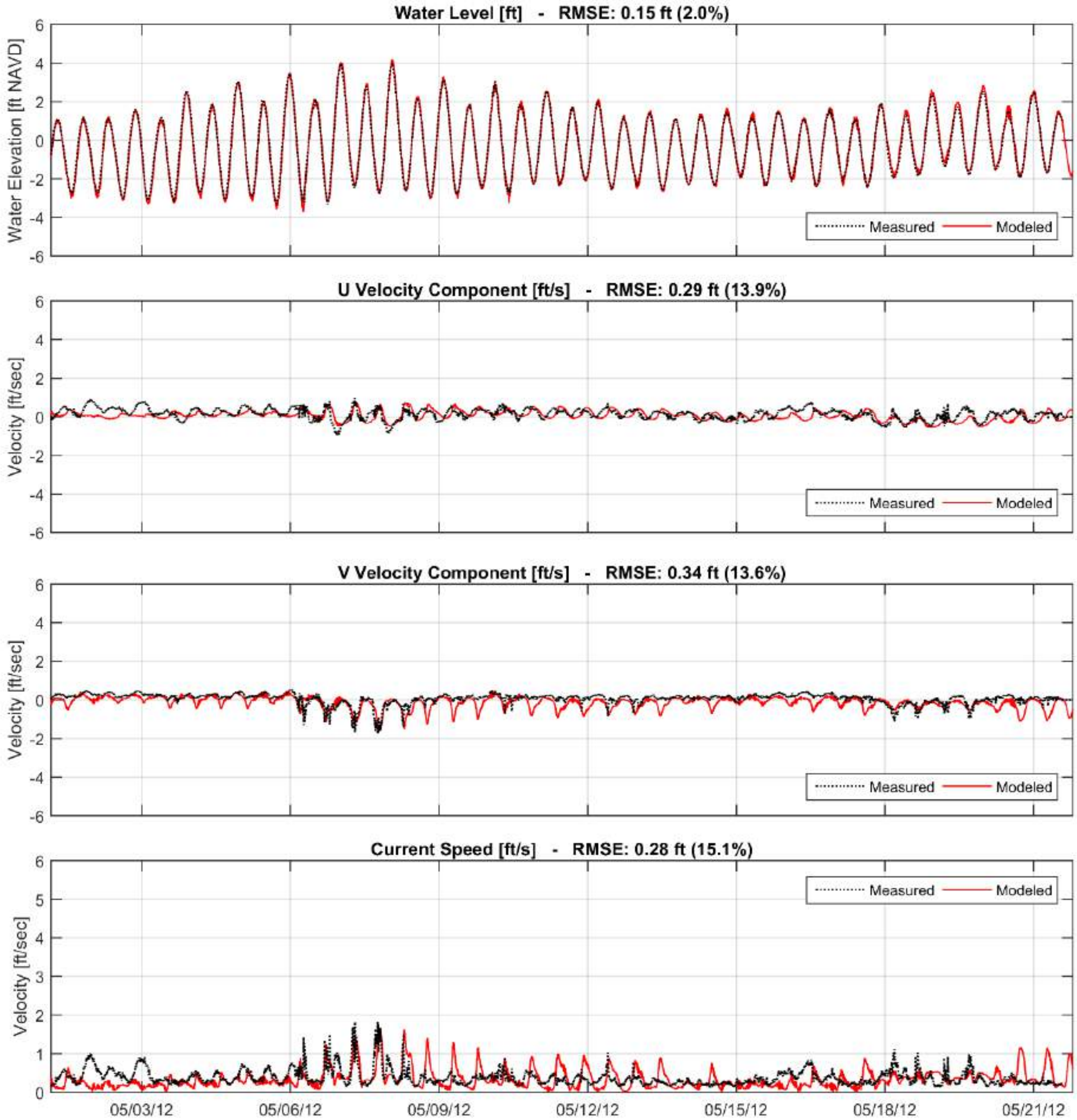
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

SIO Gauge – SIO-04

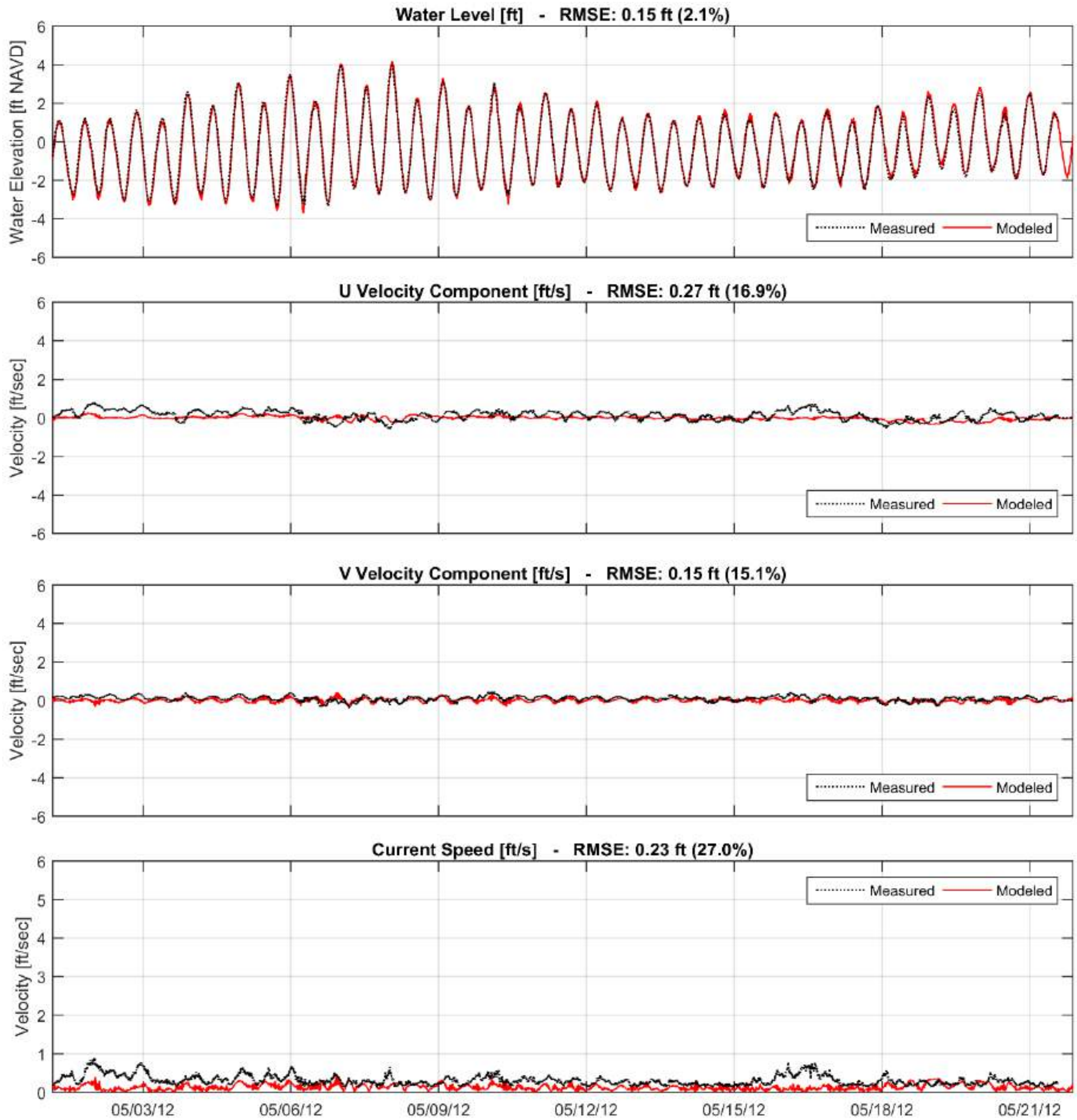


Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

SIO Gauge – SIO-05

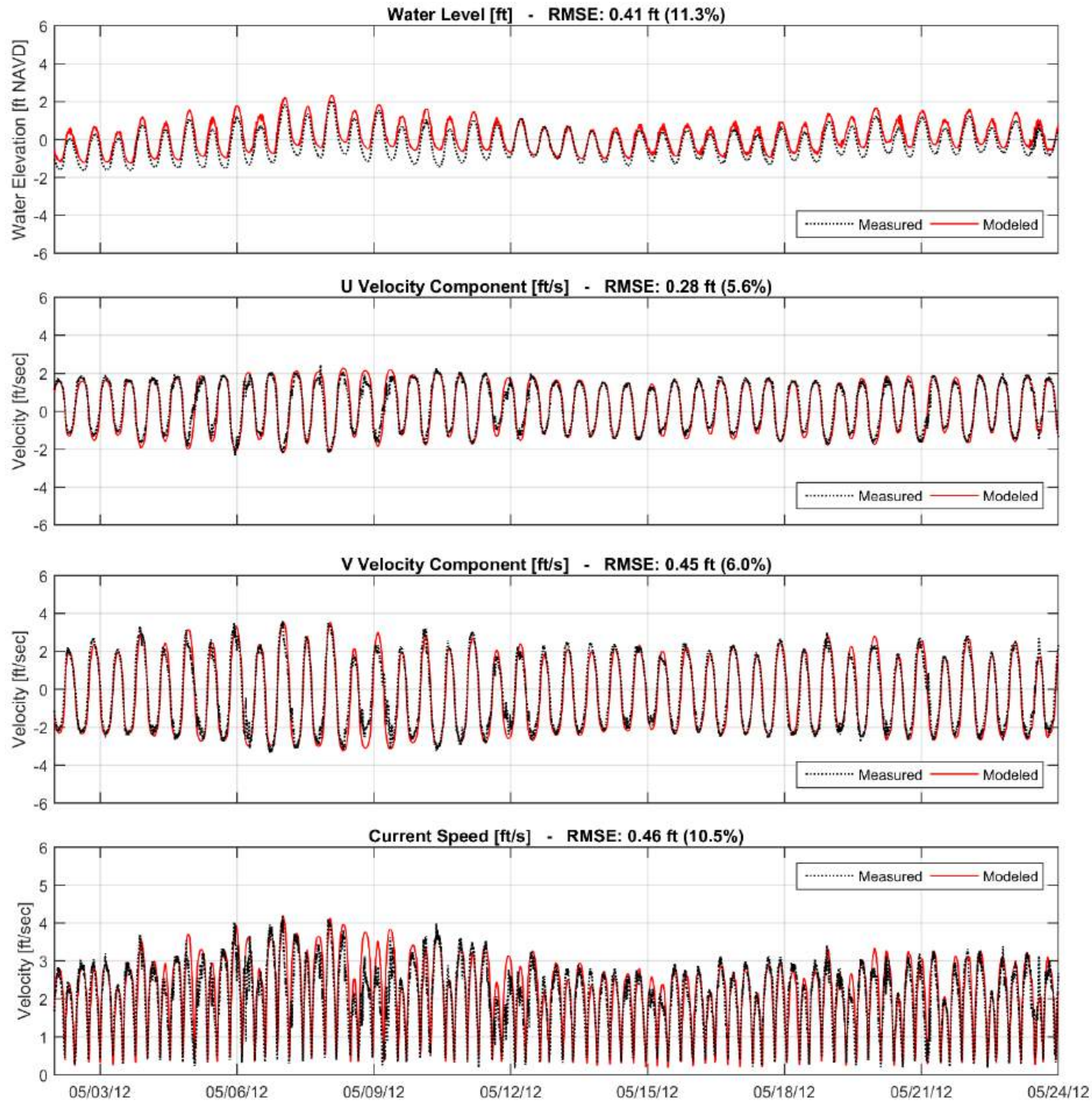


SIO Gauge – SIO-06



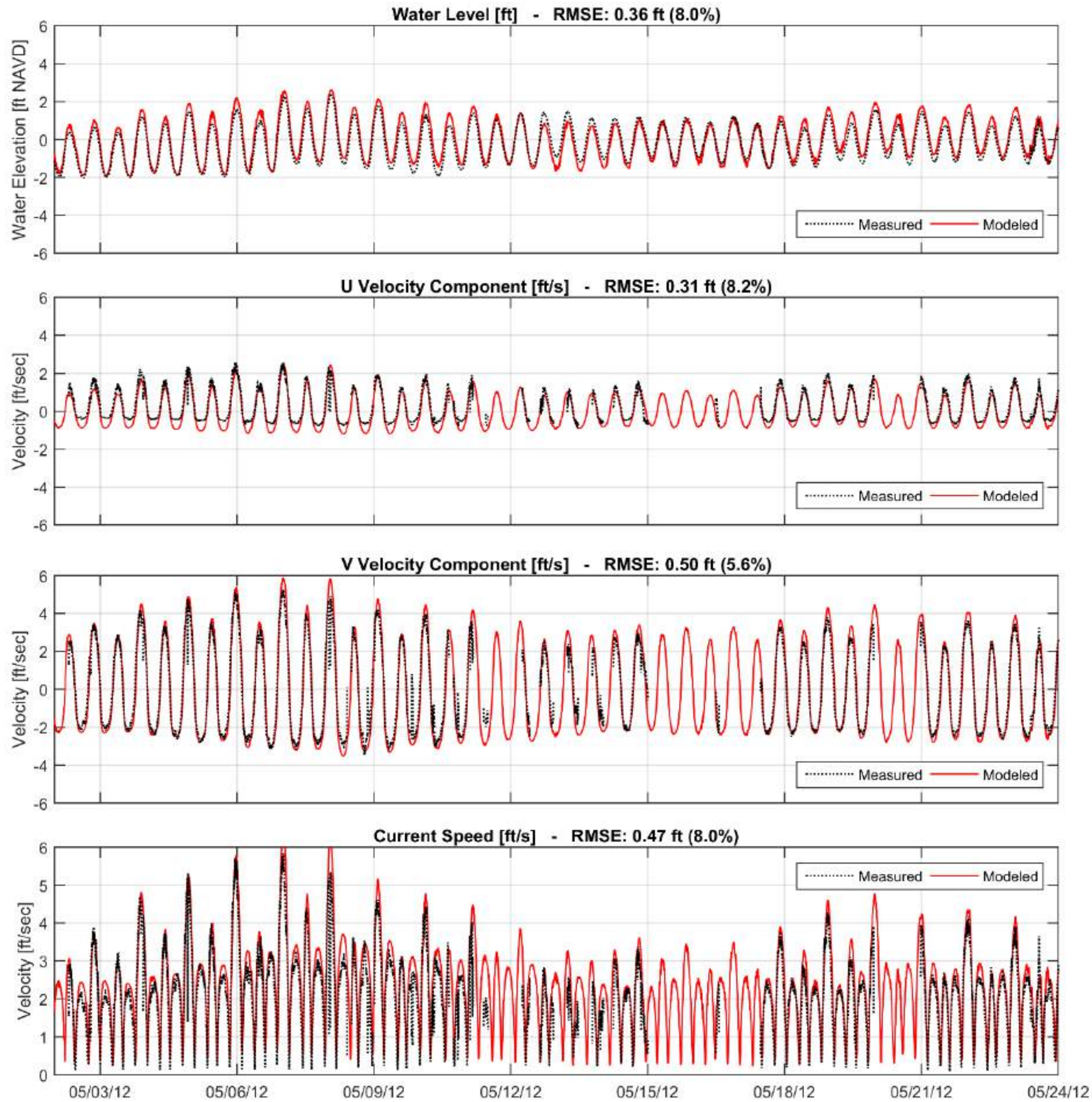
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-00



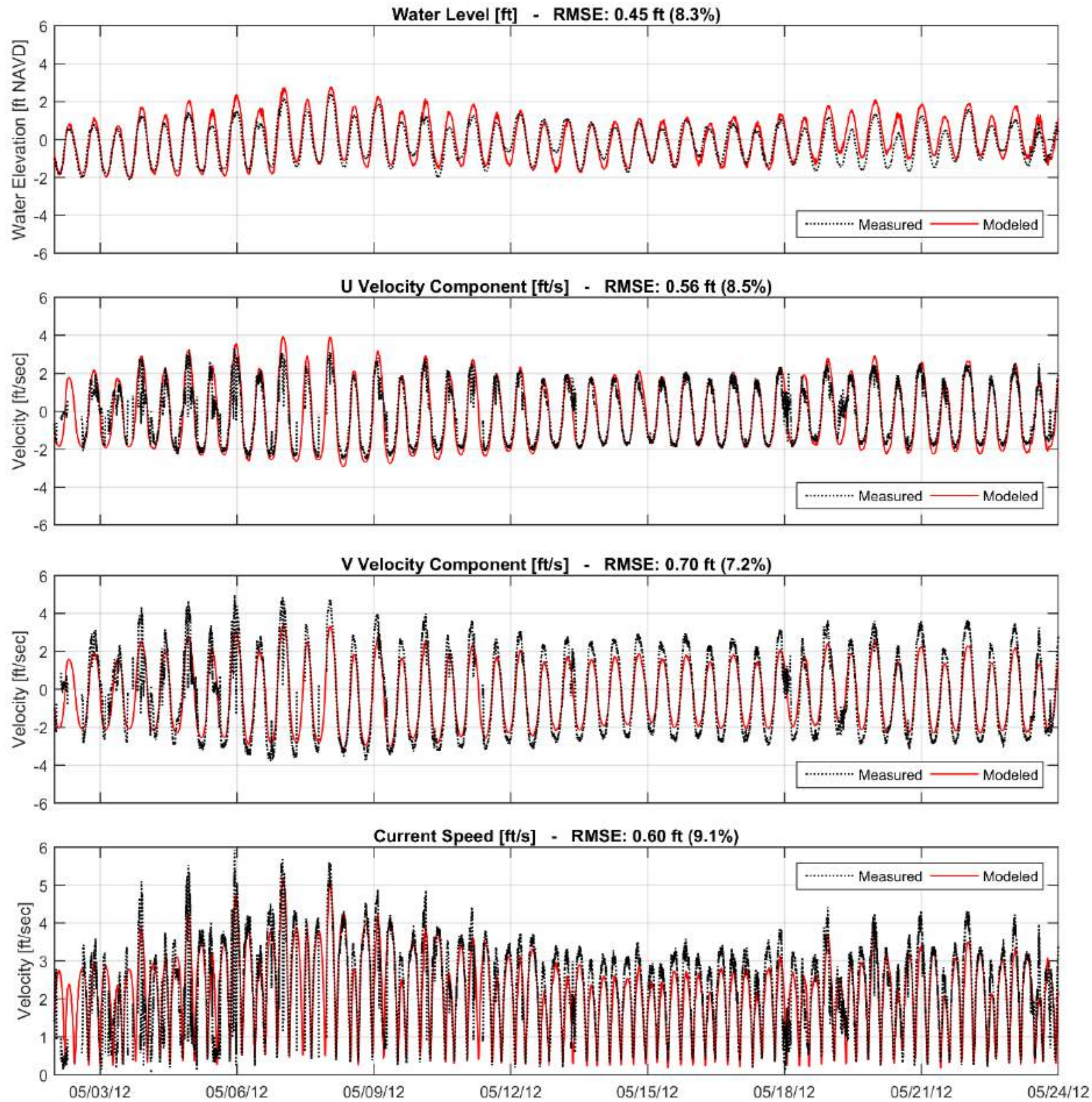
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-01



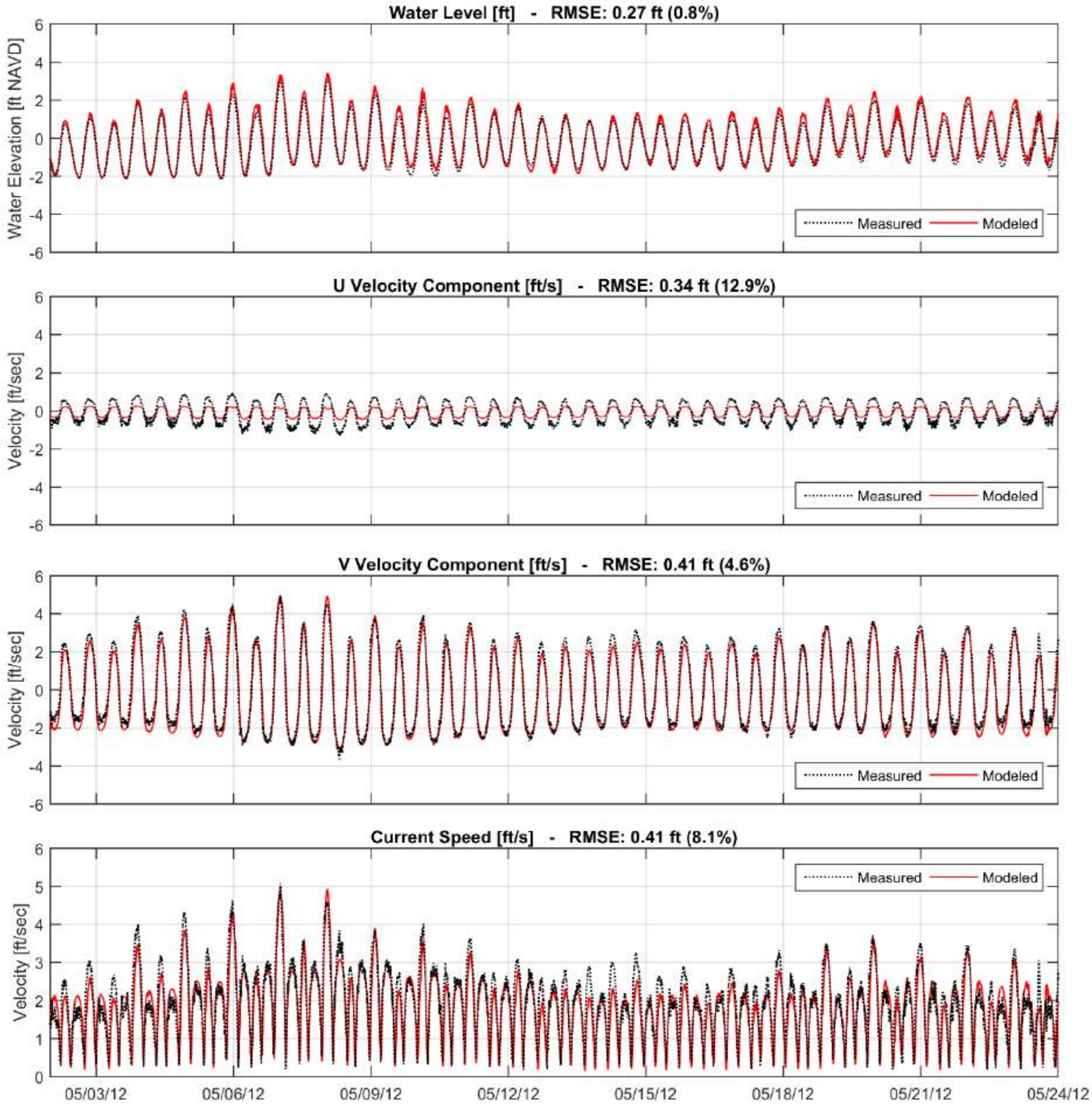
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-02



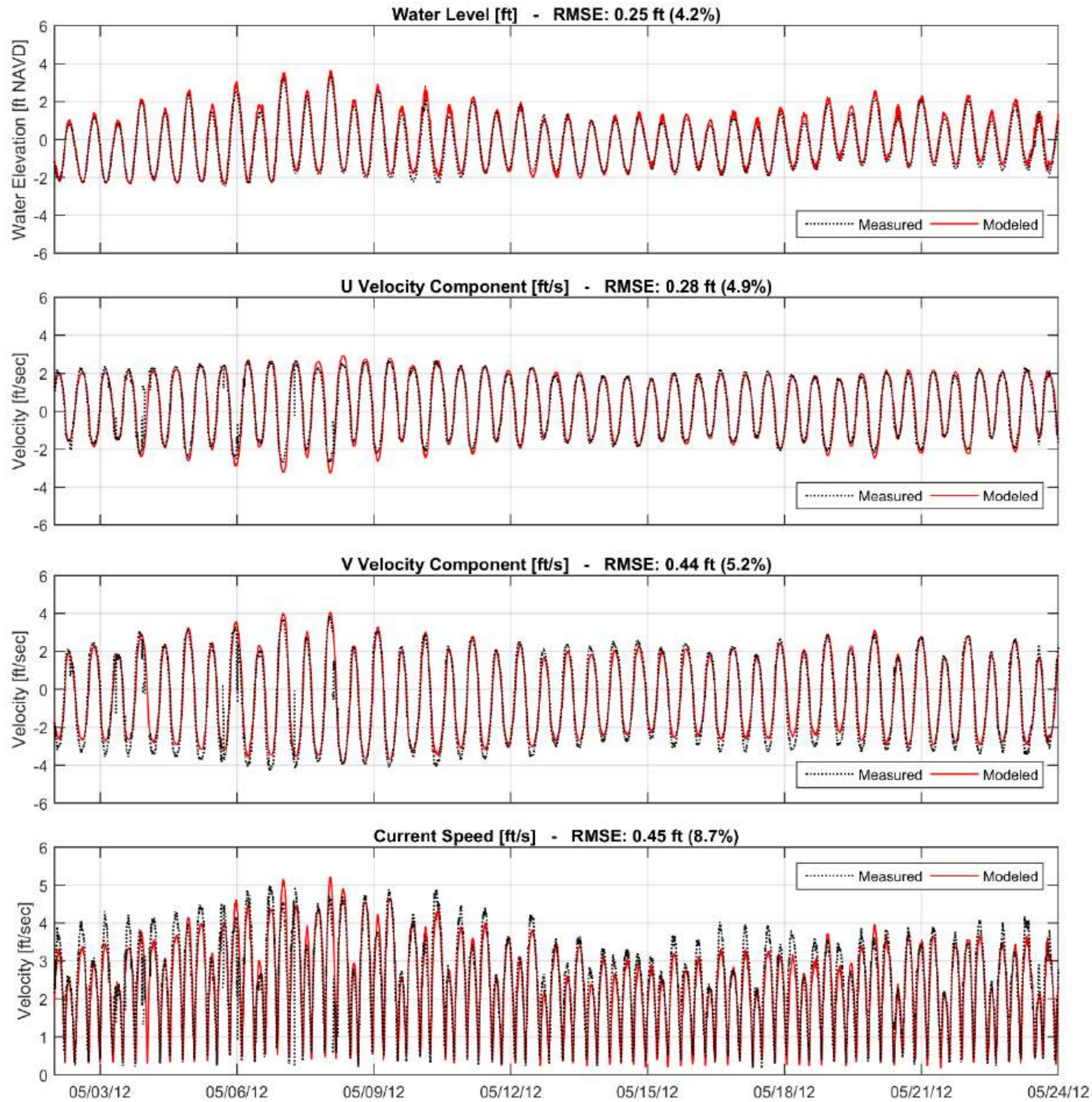
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-03



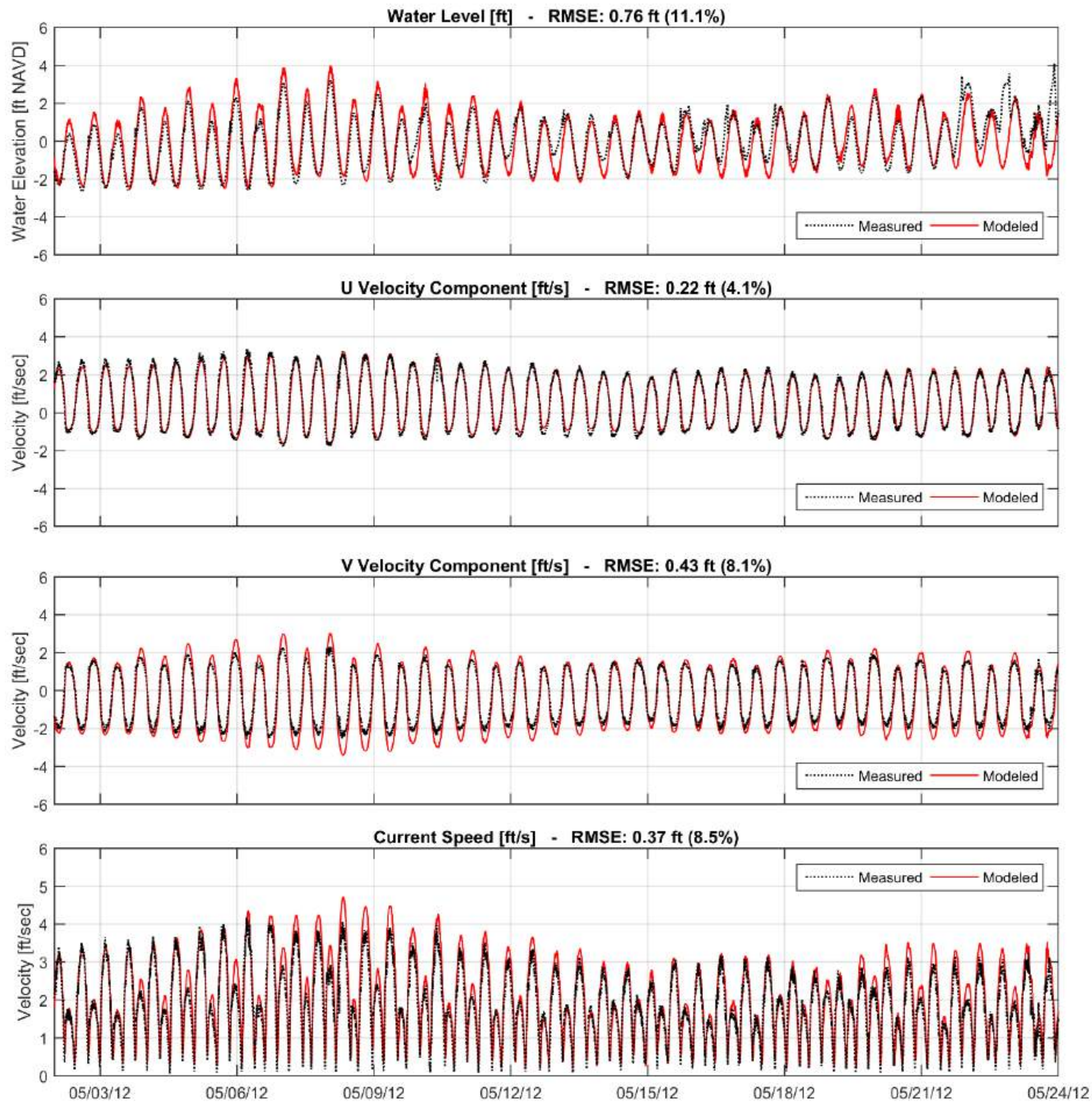
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-04



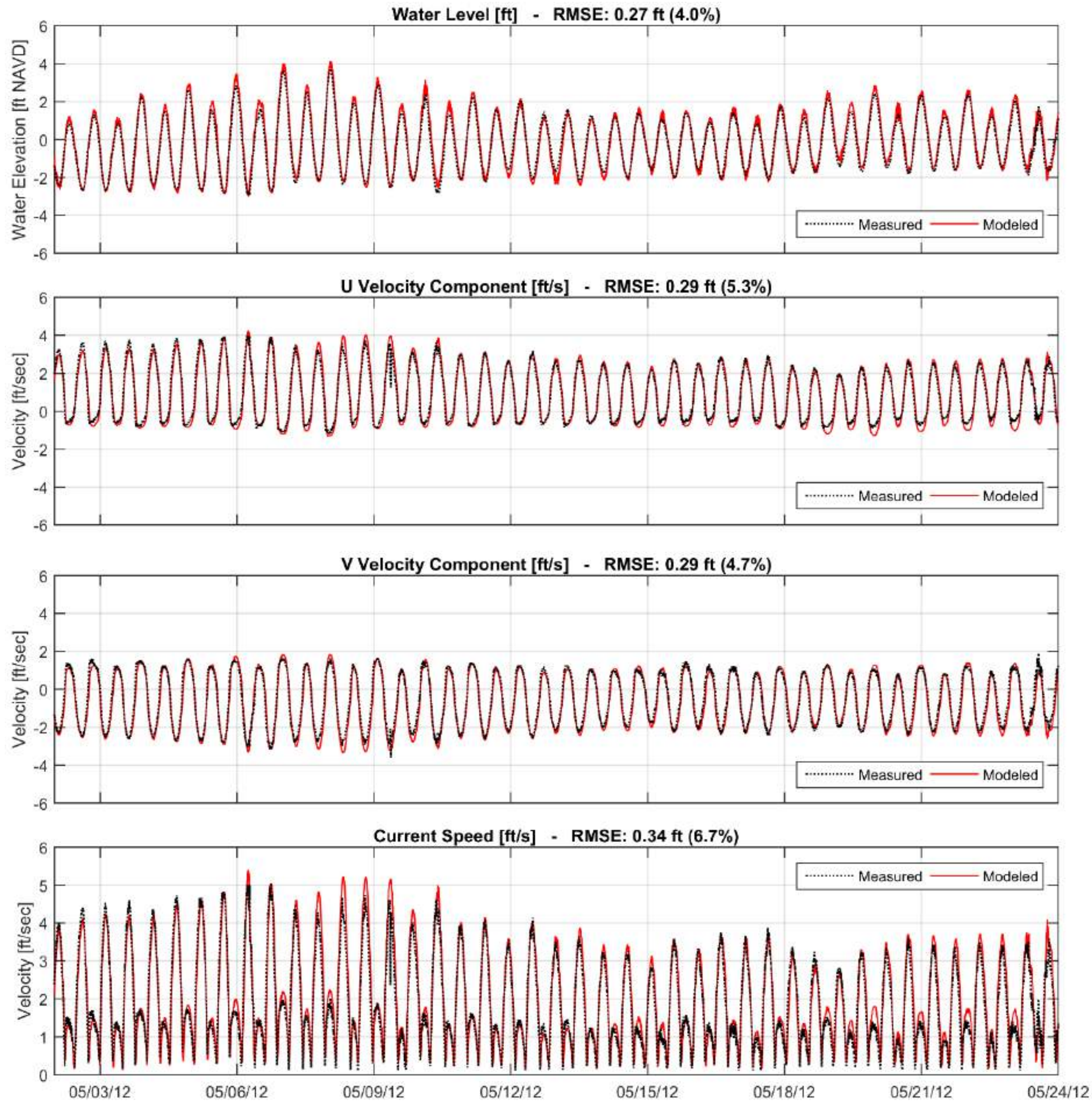
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-05



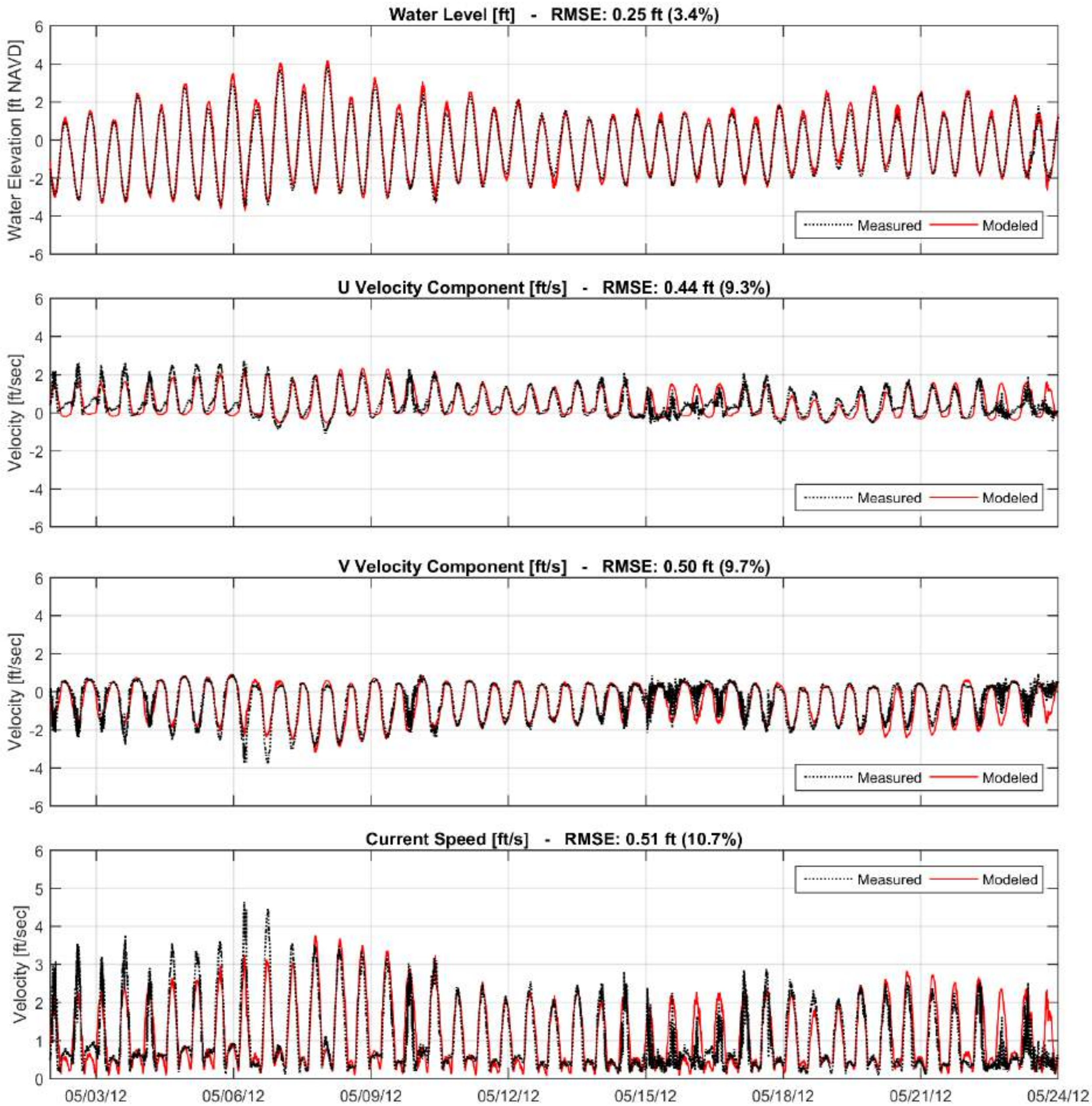
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-06



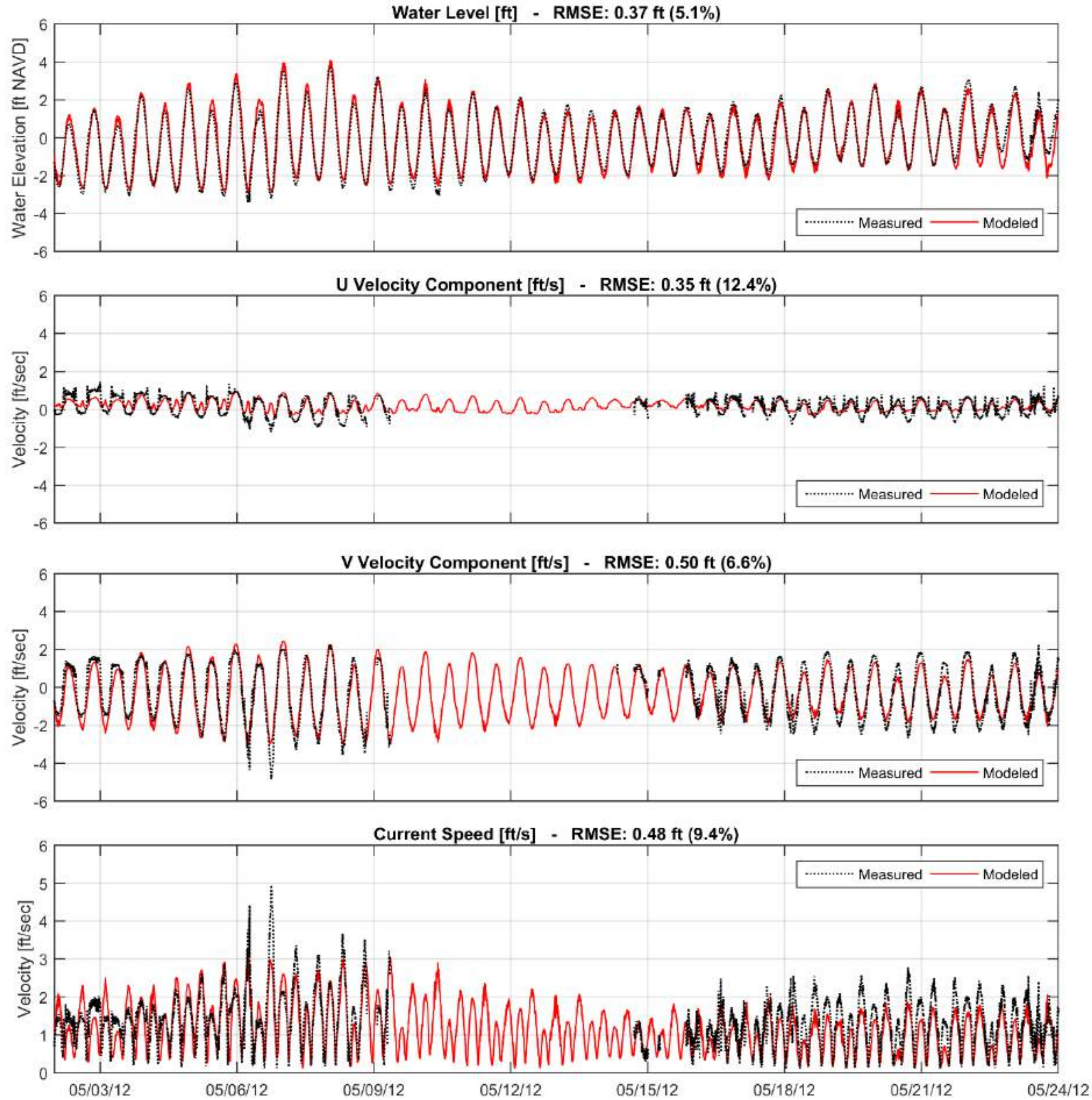
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-08



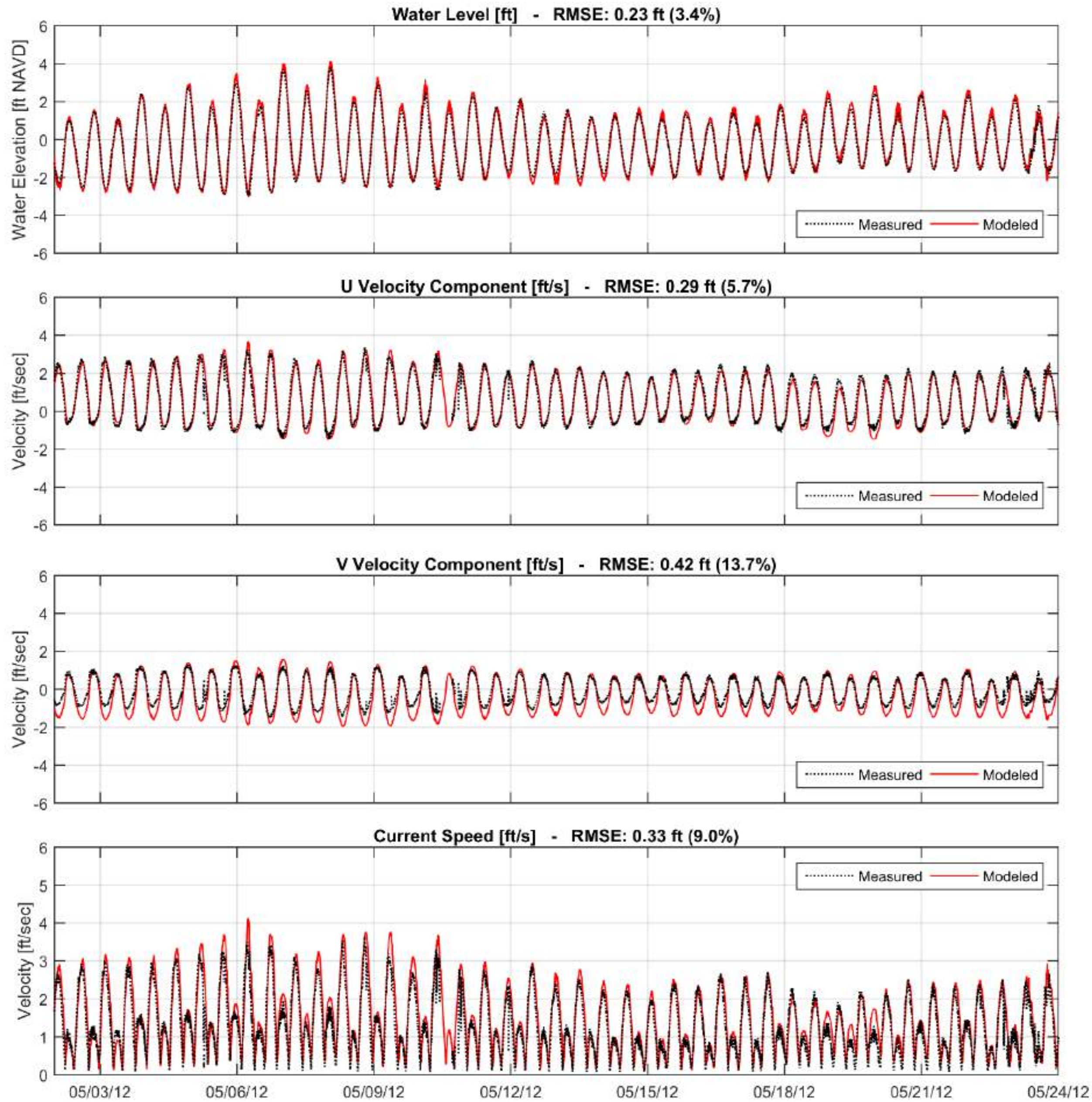
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-90



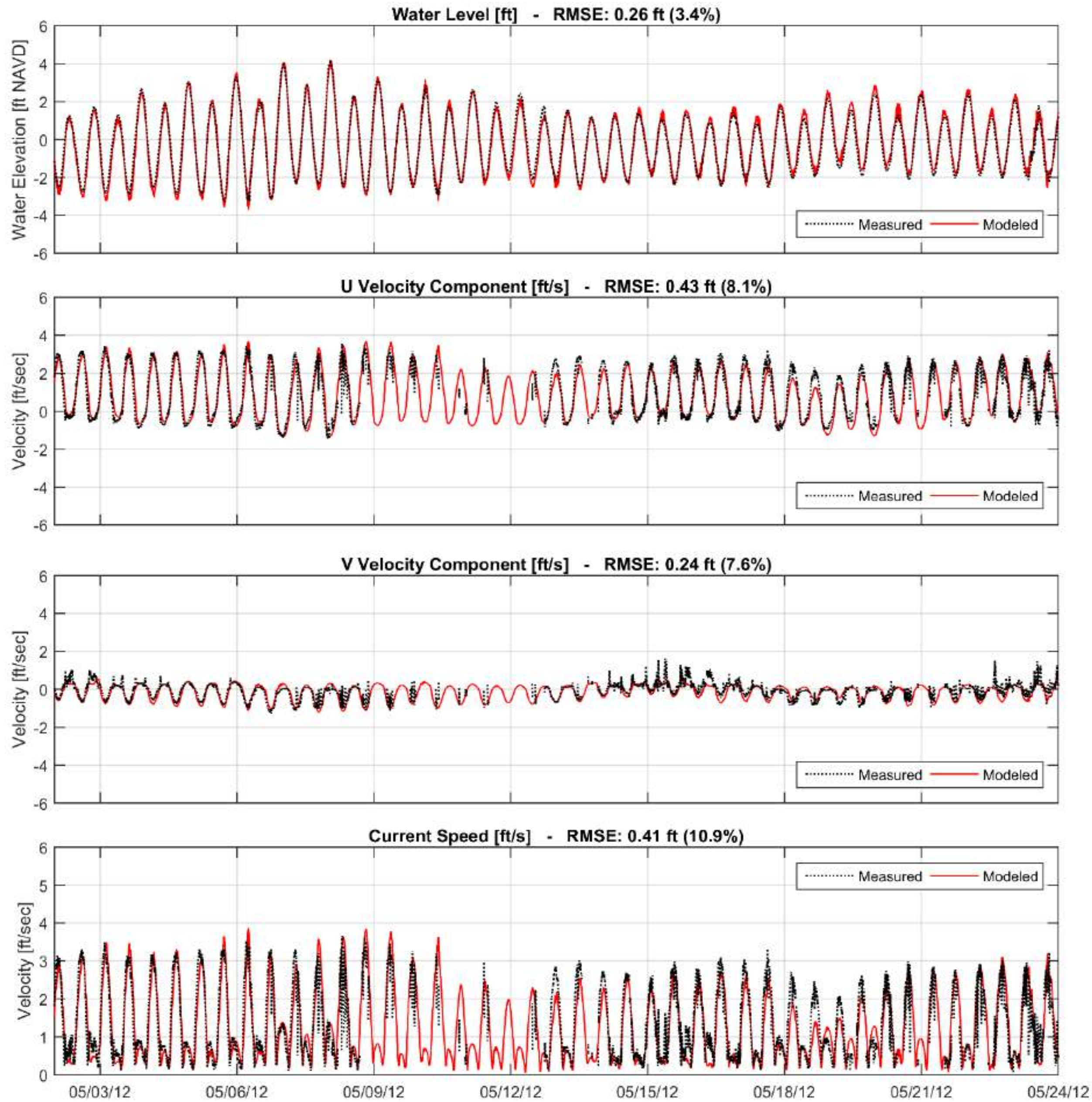
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-91



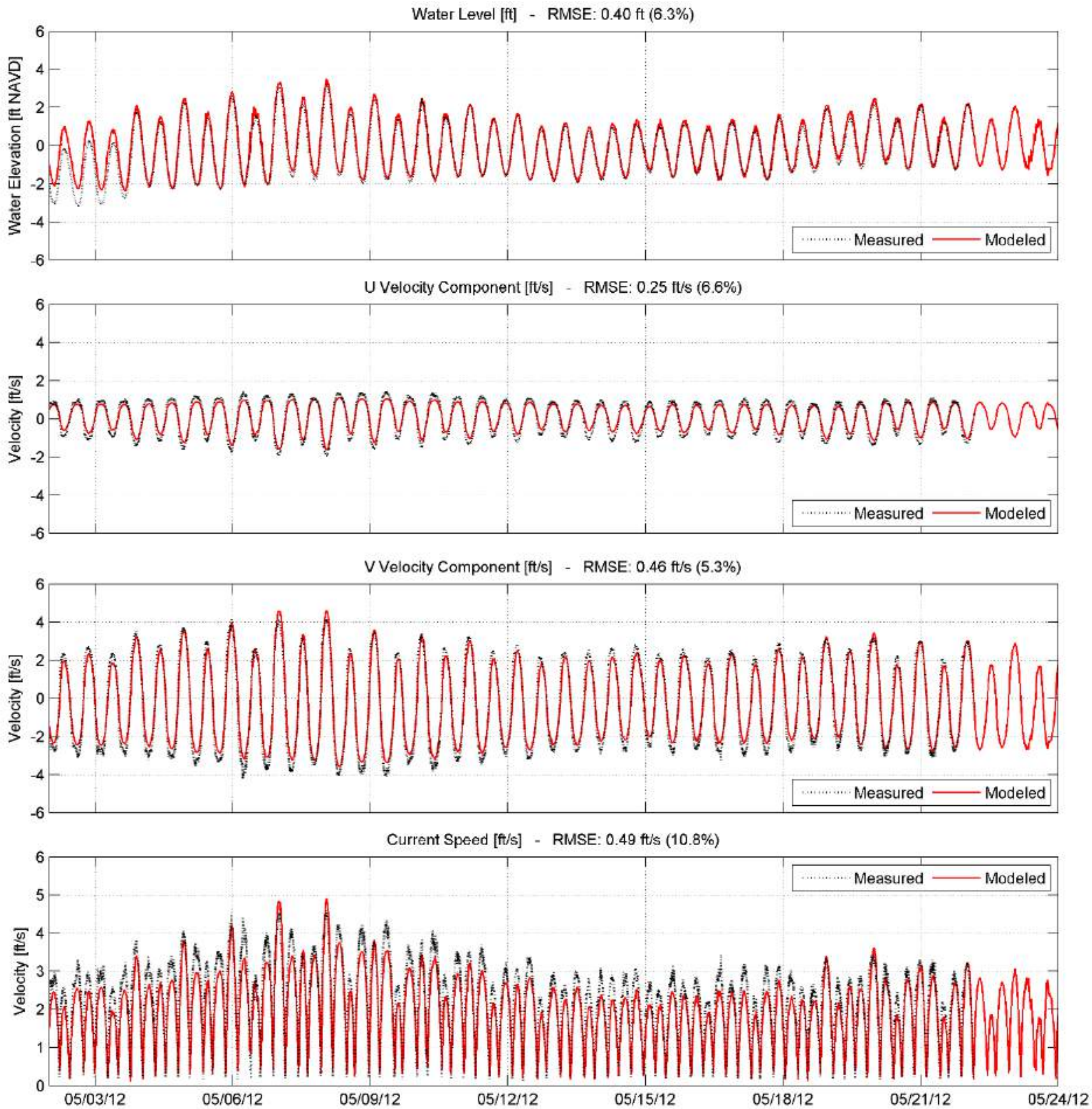
Regional Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-92



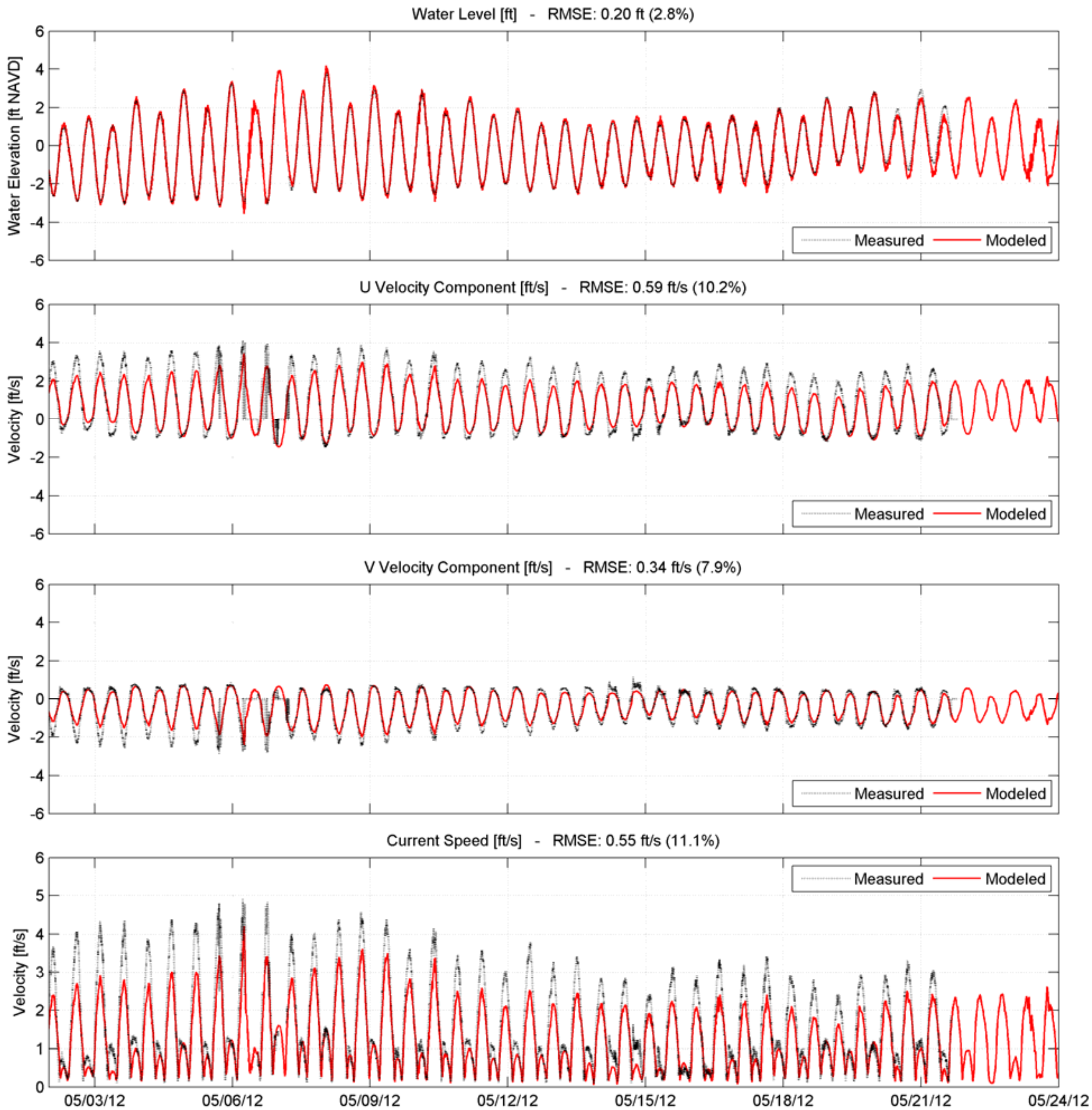
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

SIO Gauge – SIO-01



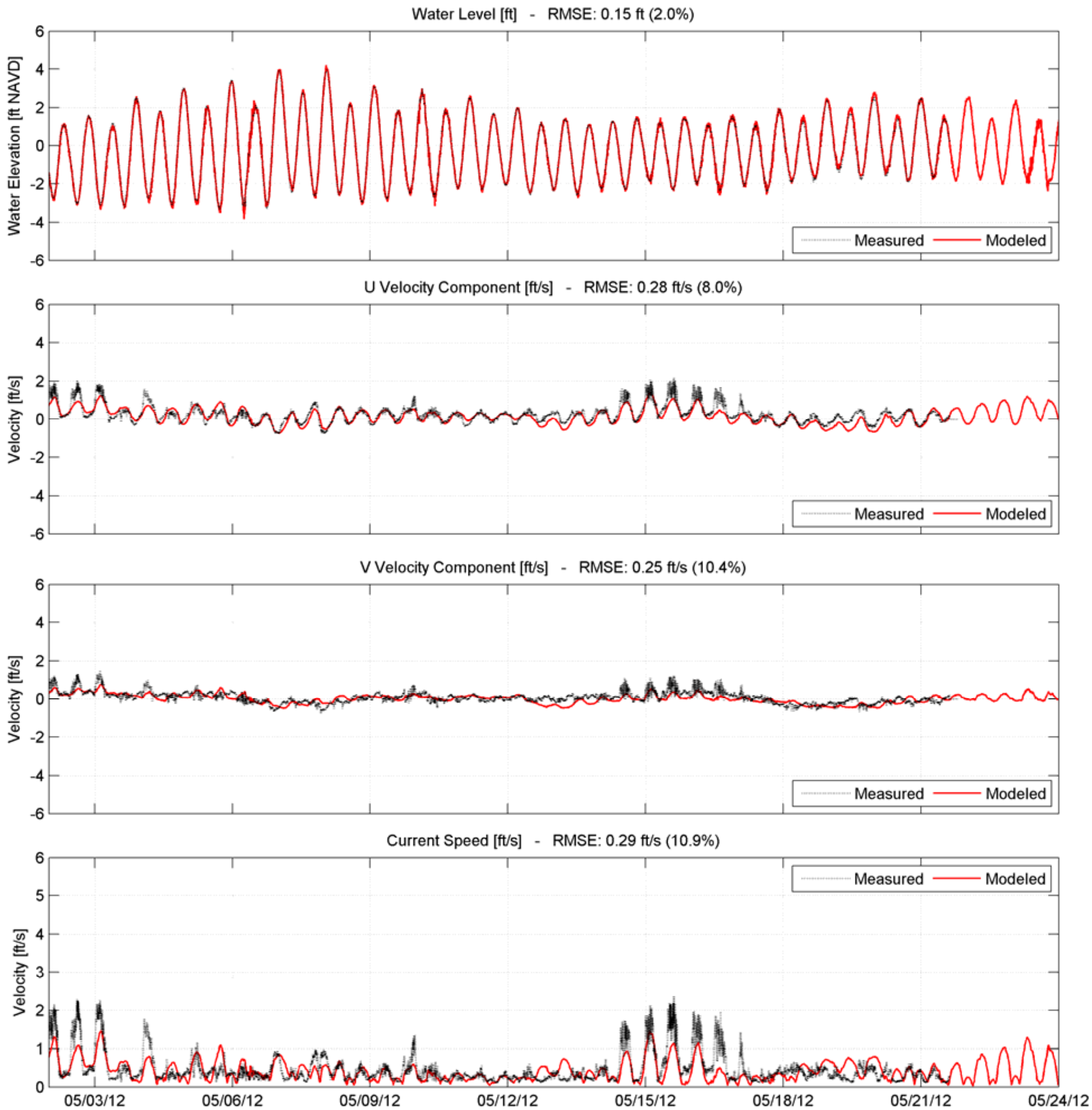
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

SIO Gauge – SIO-02



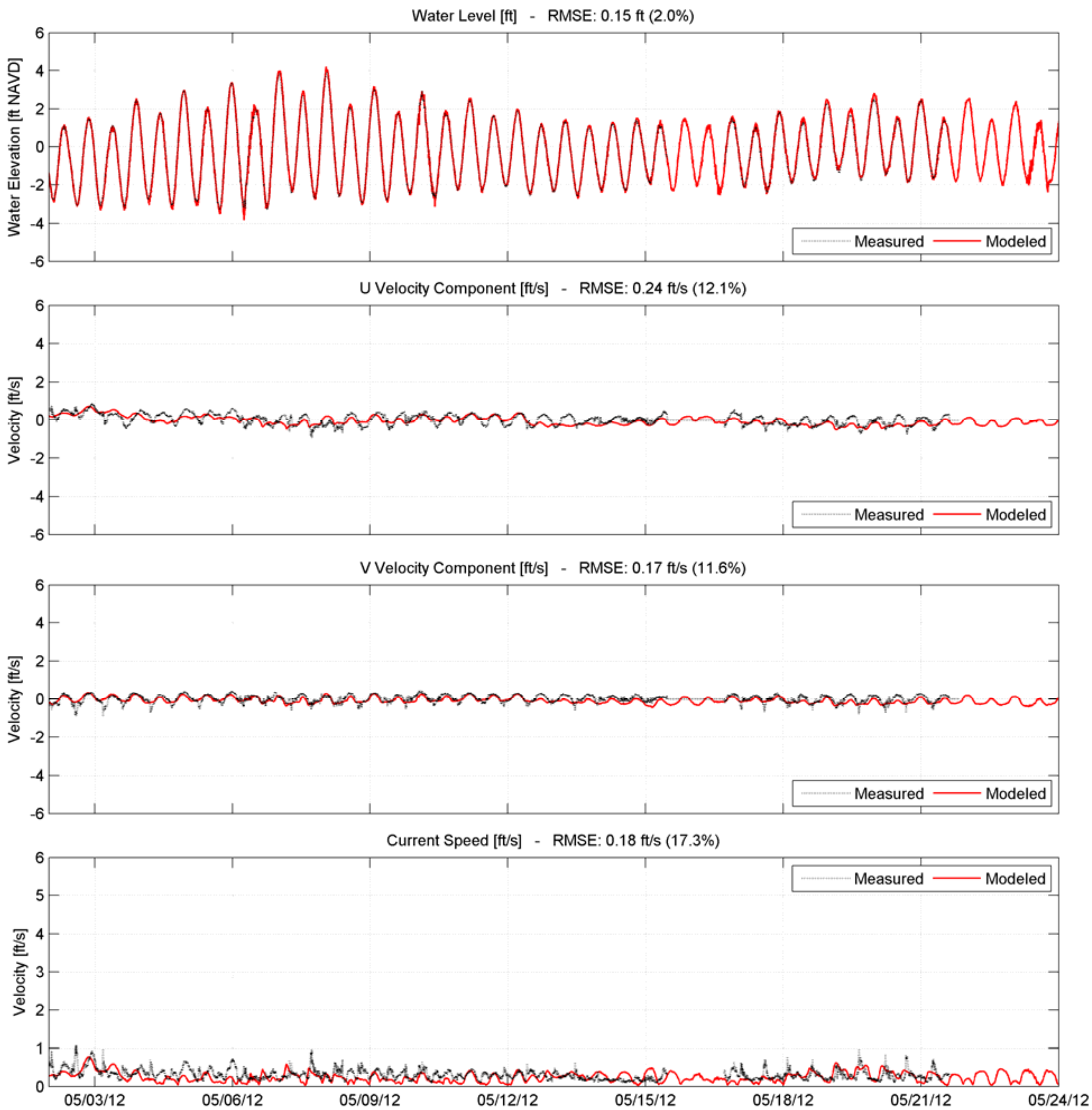
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

SIO Gauge – SIO-03



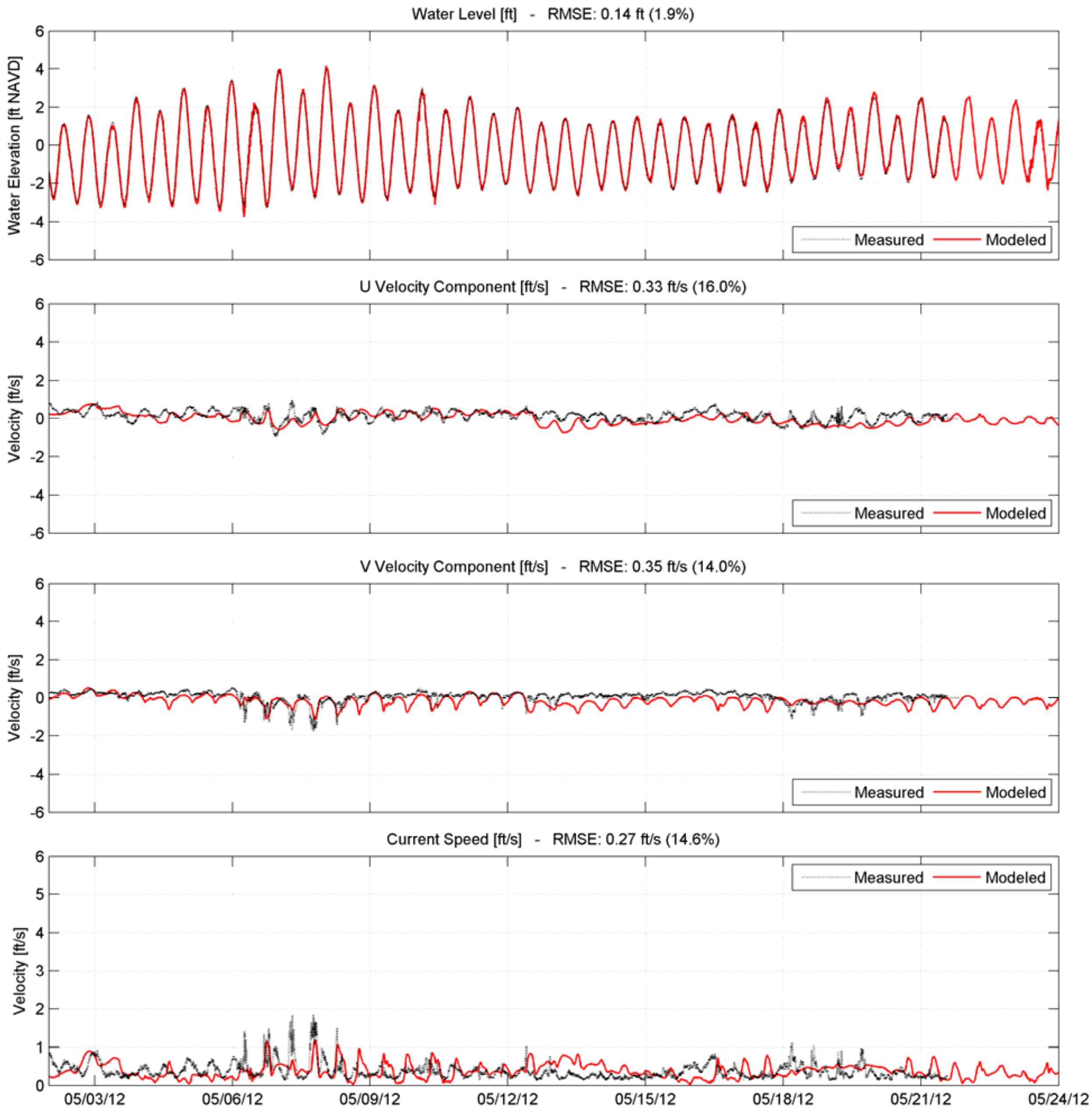
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

SIO Gauge – SIO-04



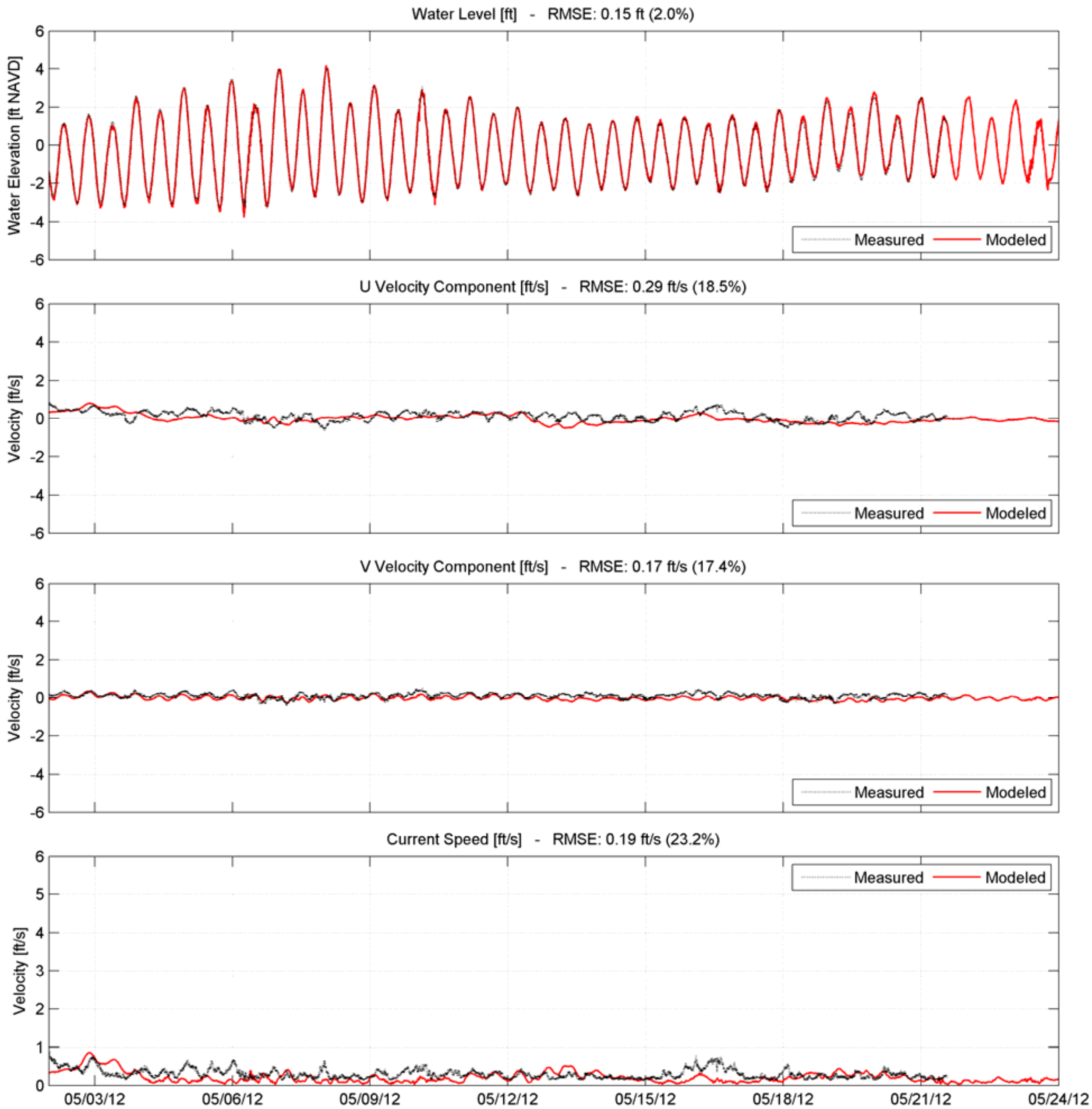
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

SIO Gauge – SIO-05



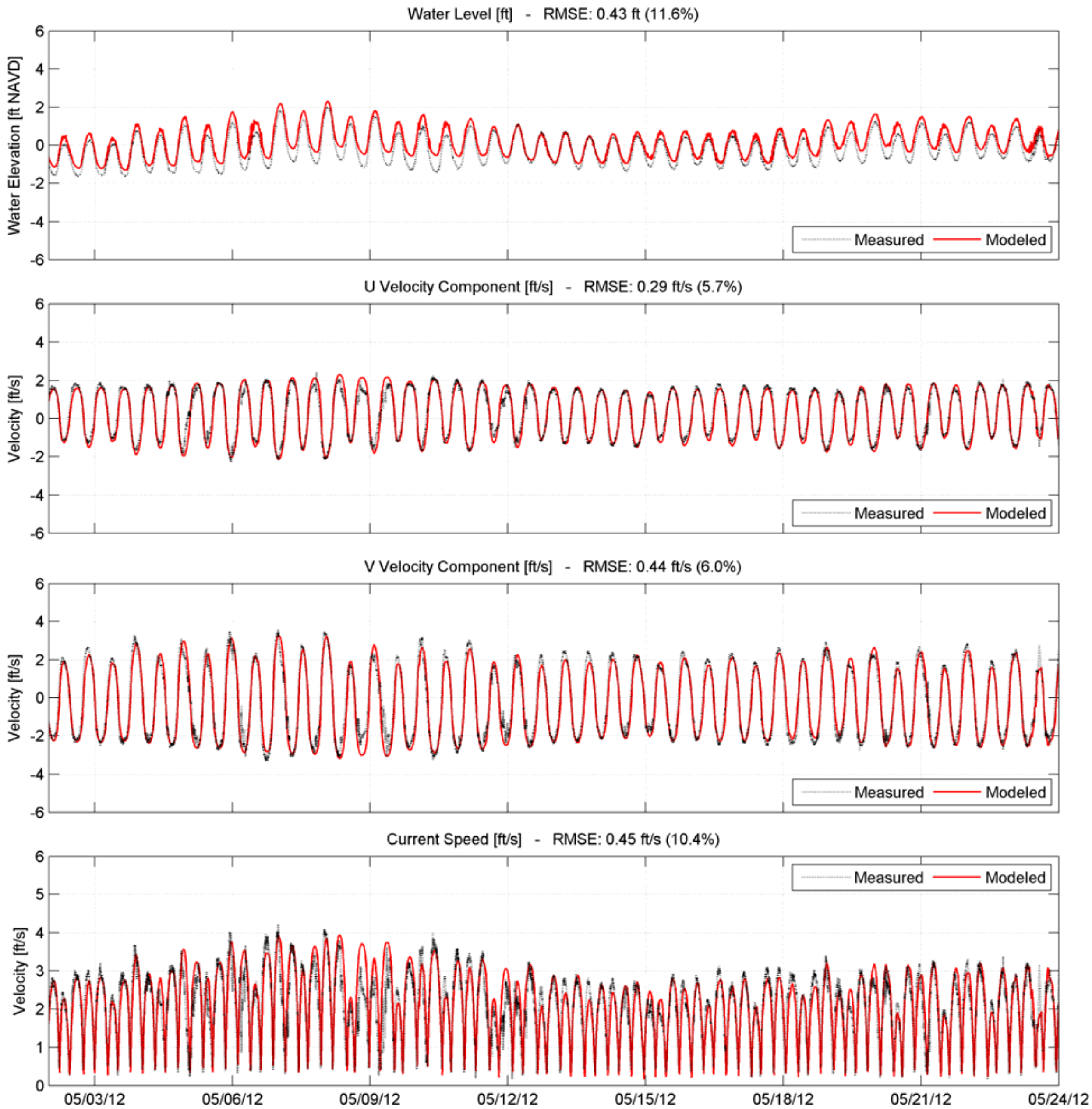
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

SIO Gauge – SIO-06



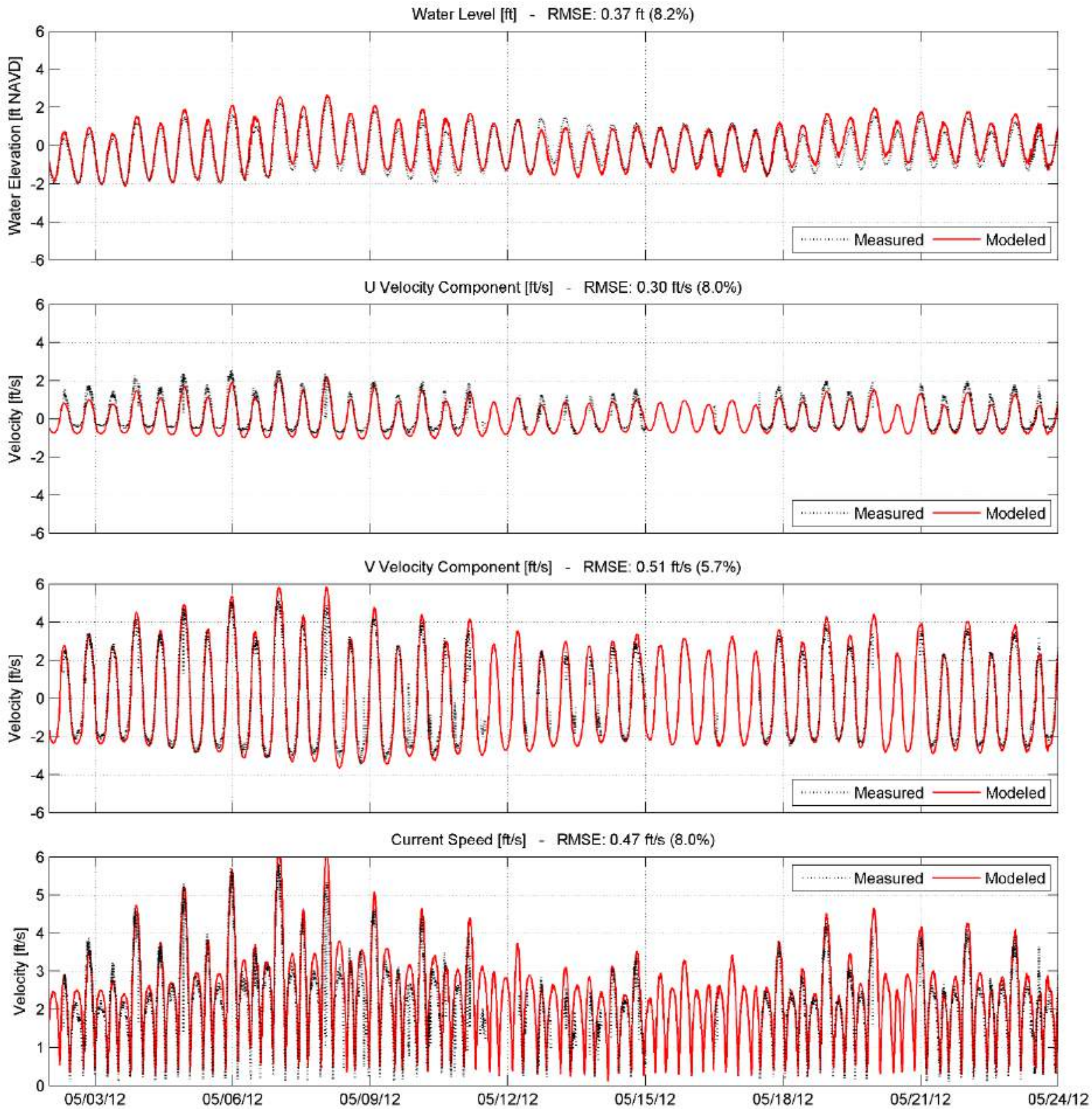
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-00



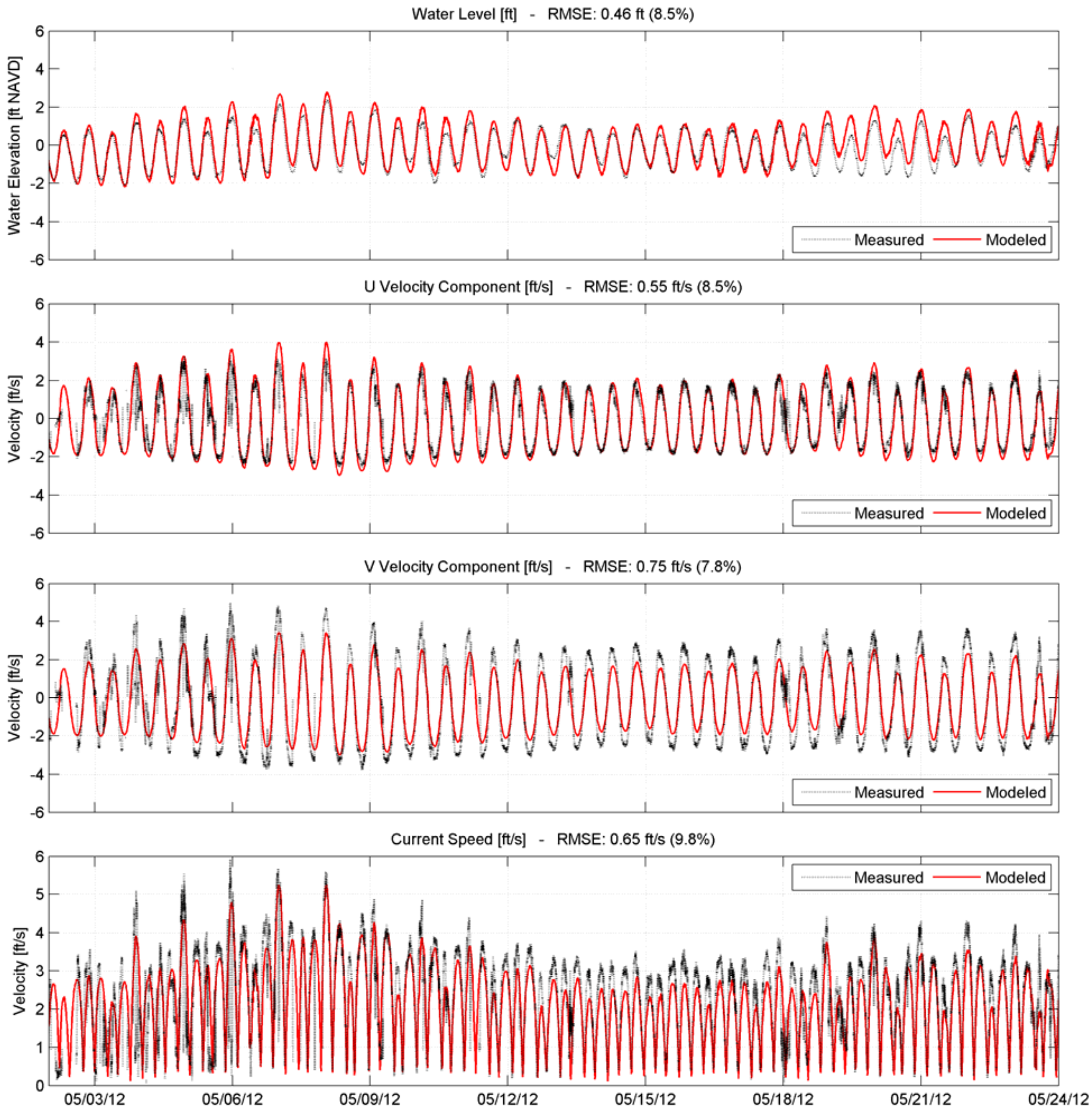
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-01



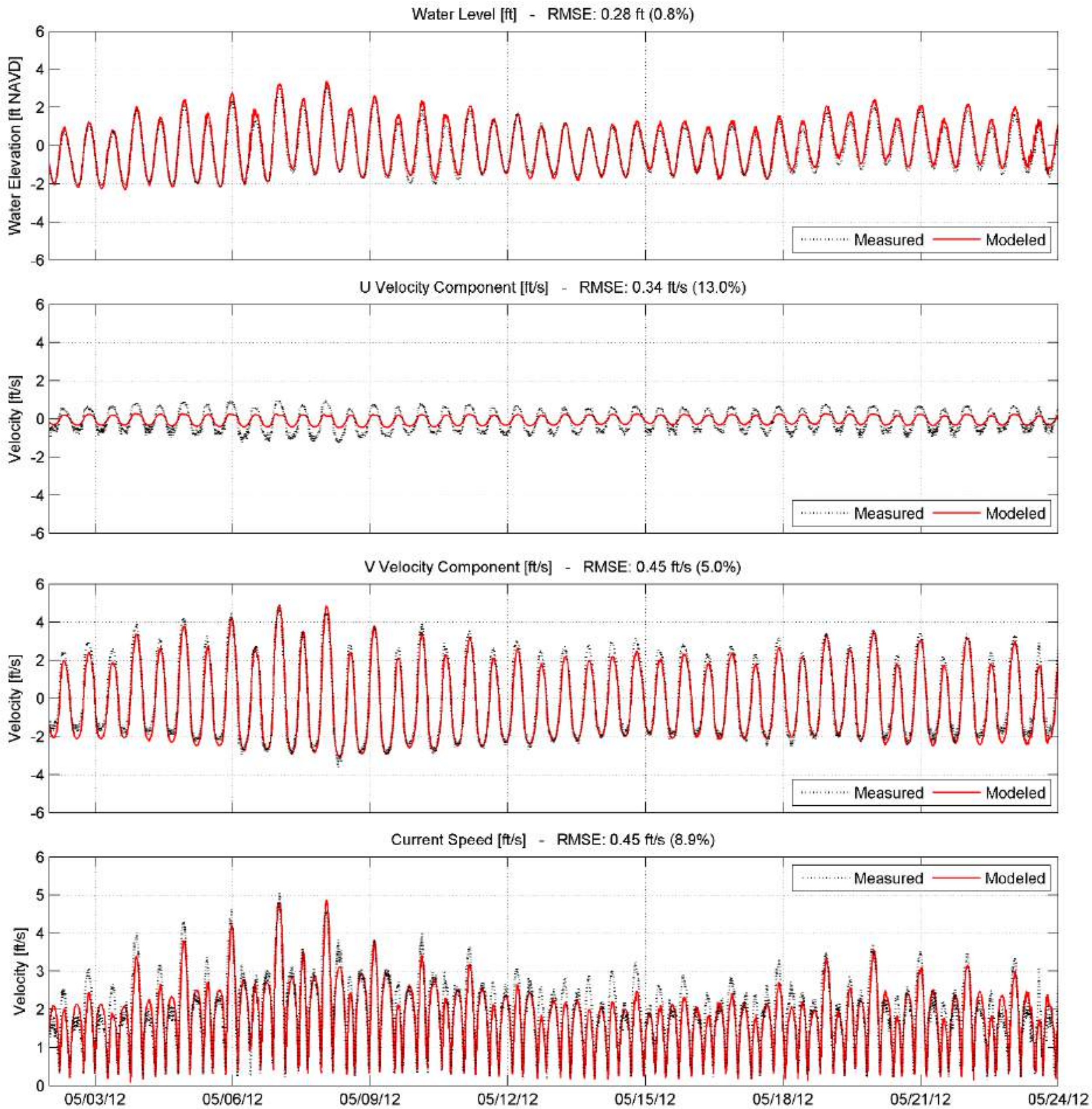
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-02



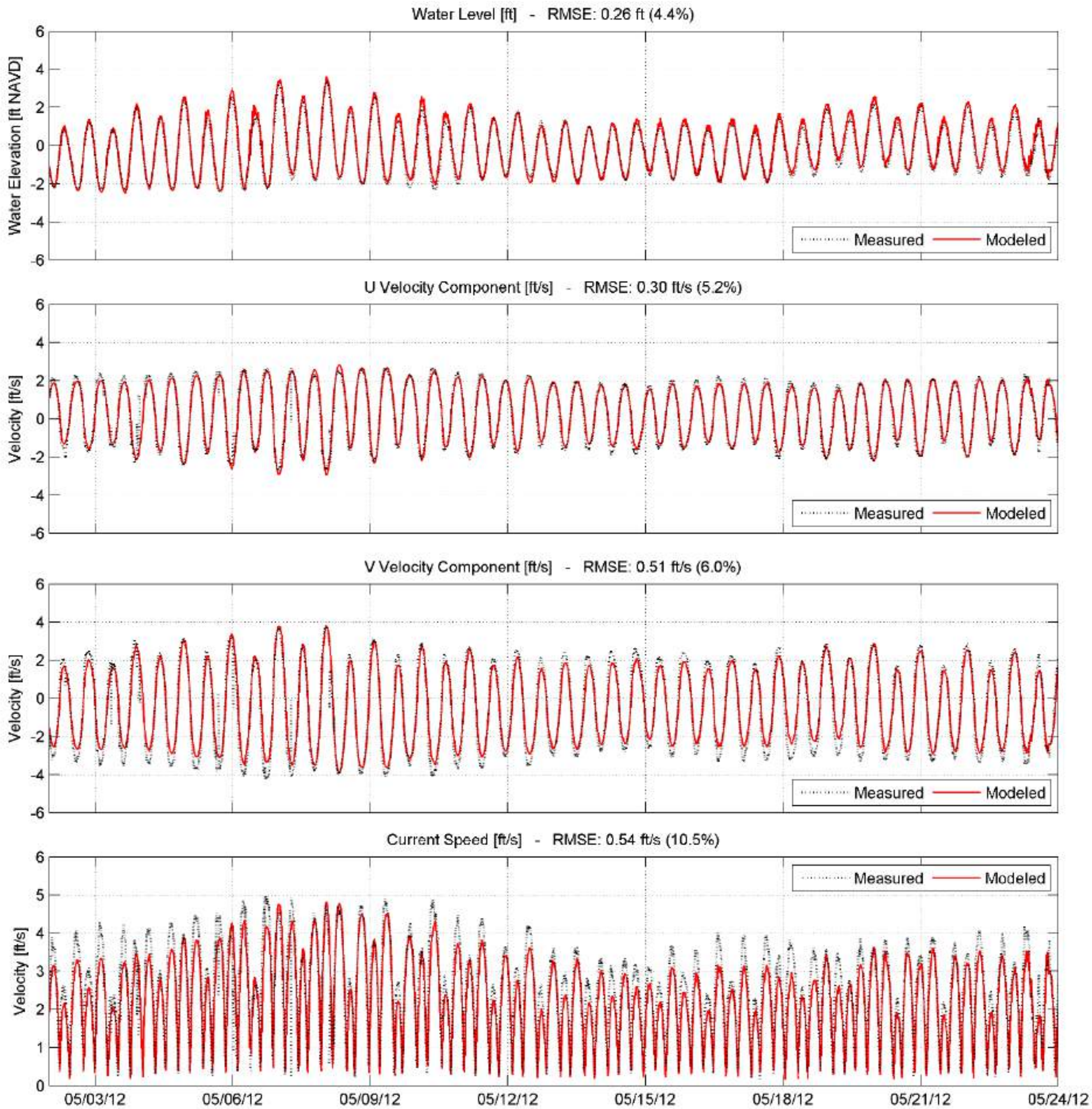
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-03



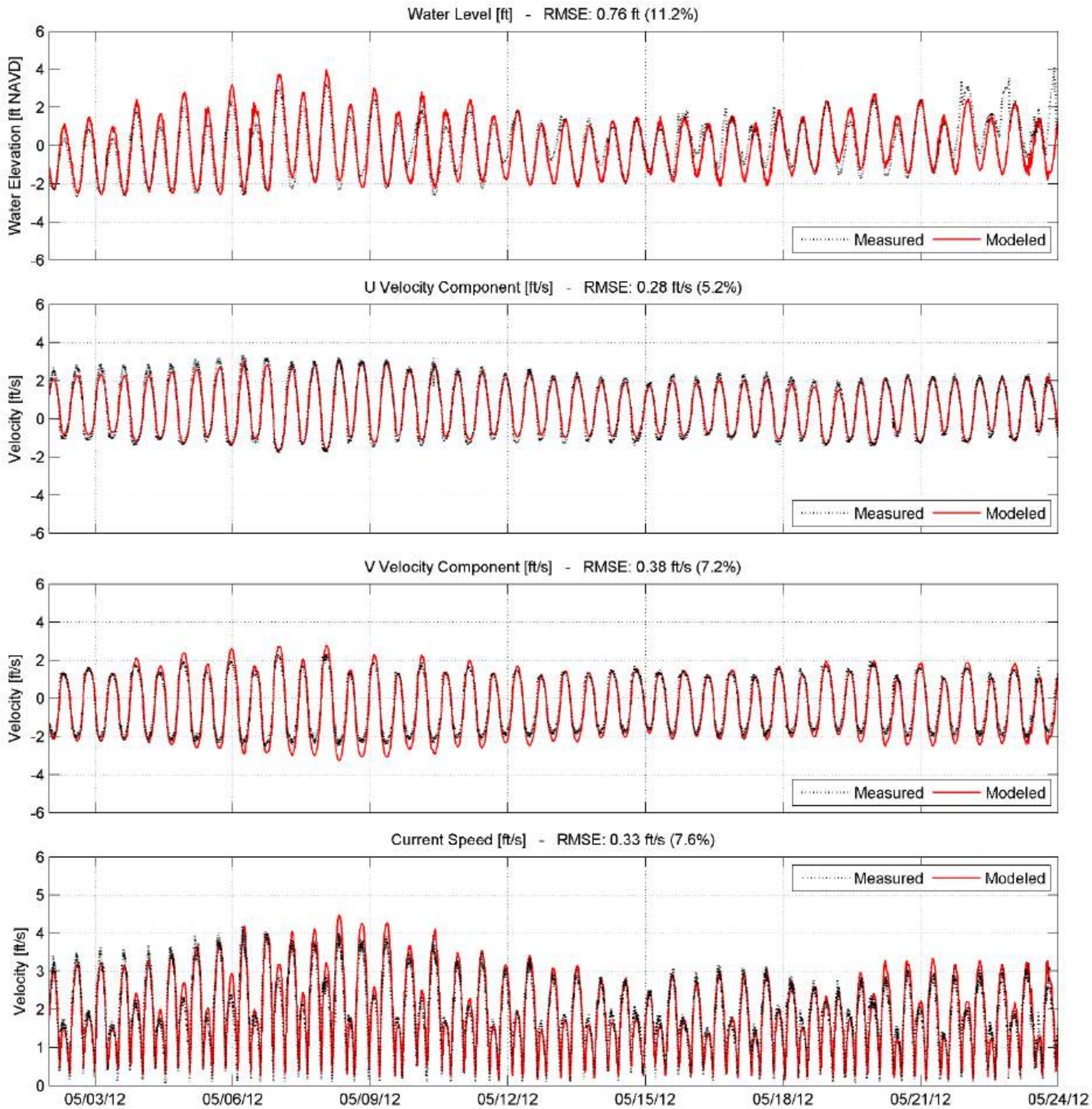
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-04



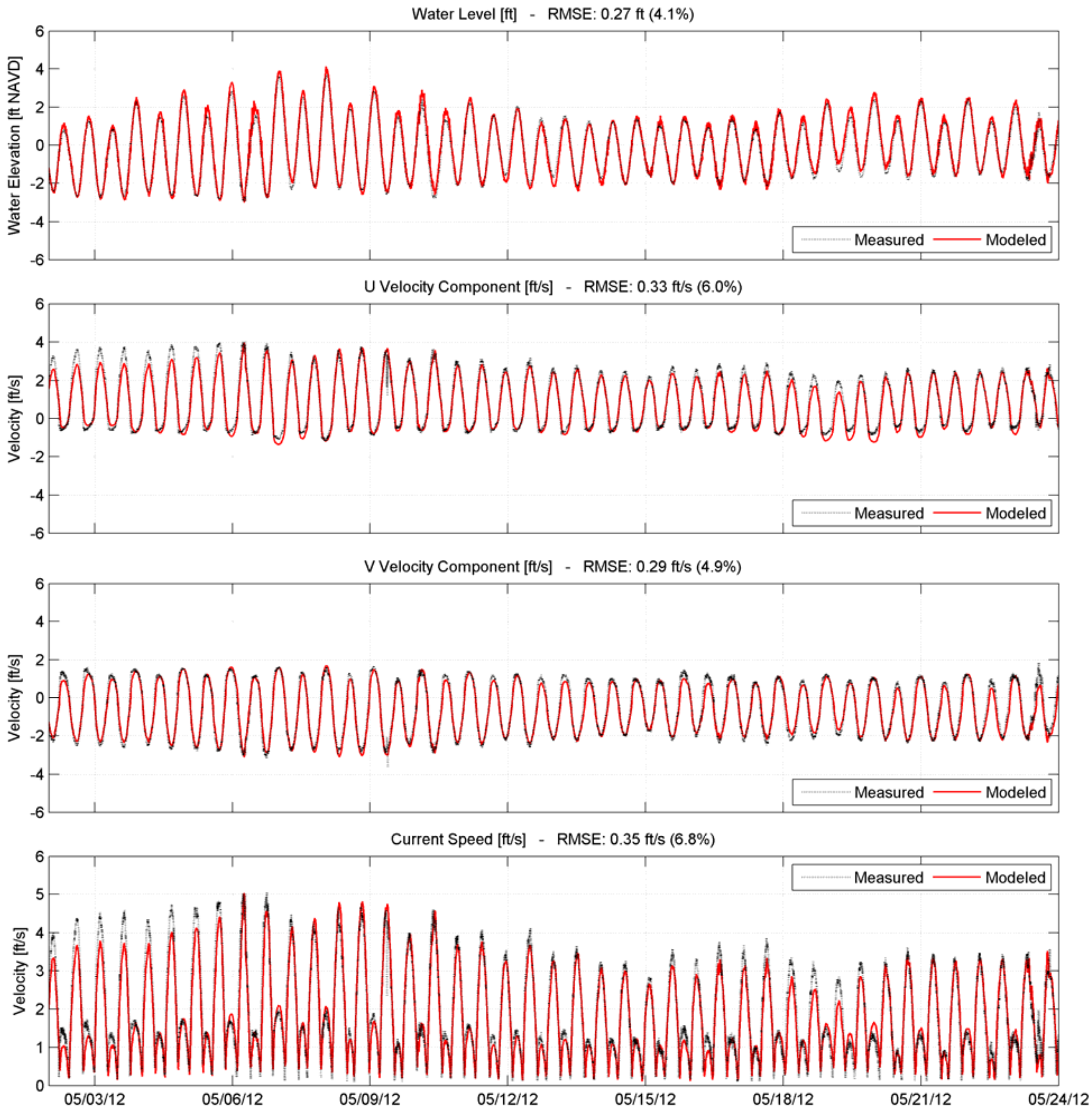
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-05



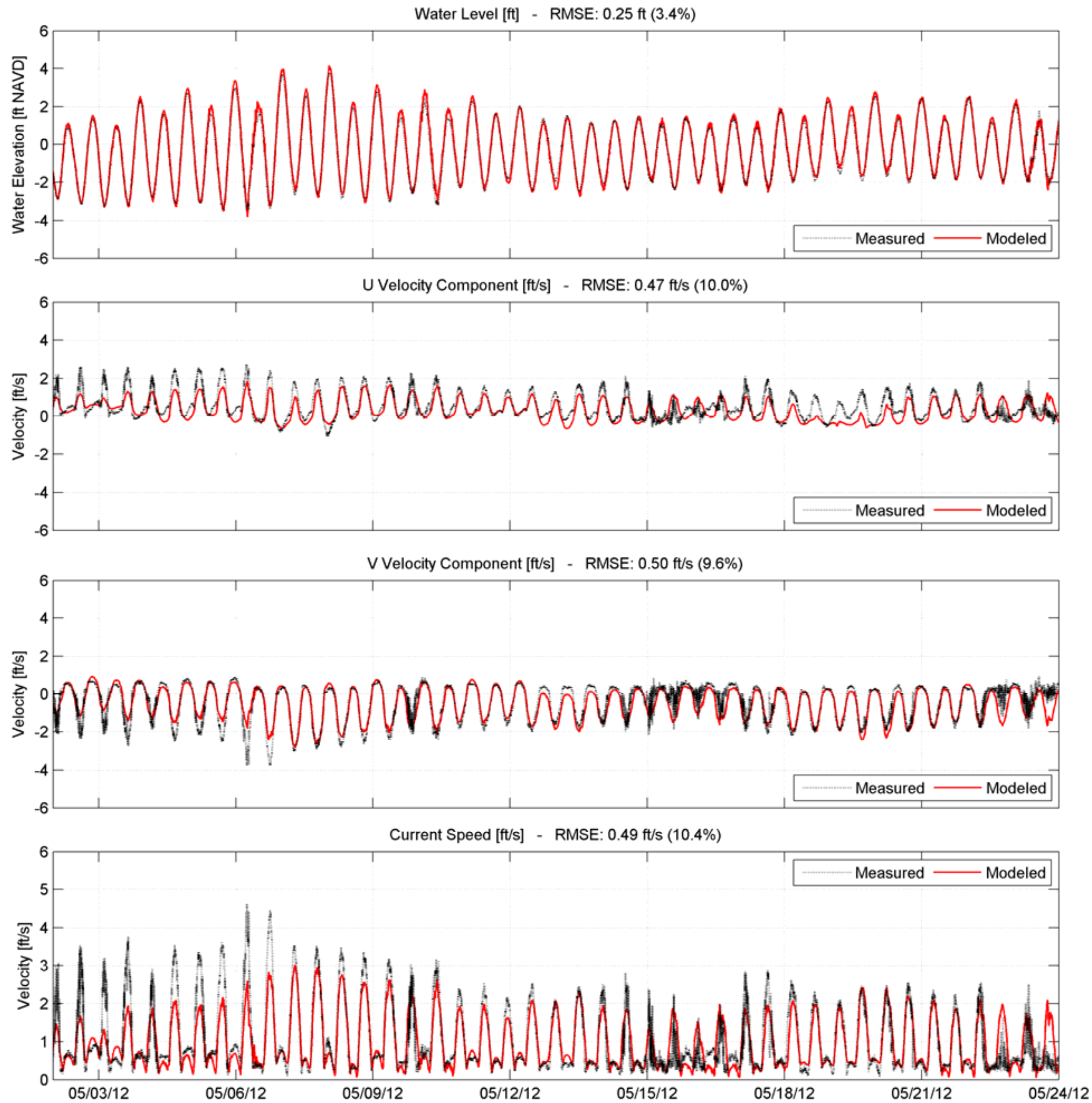
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-06



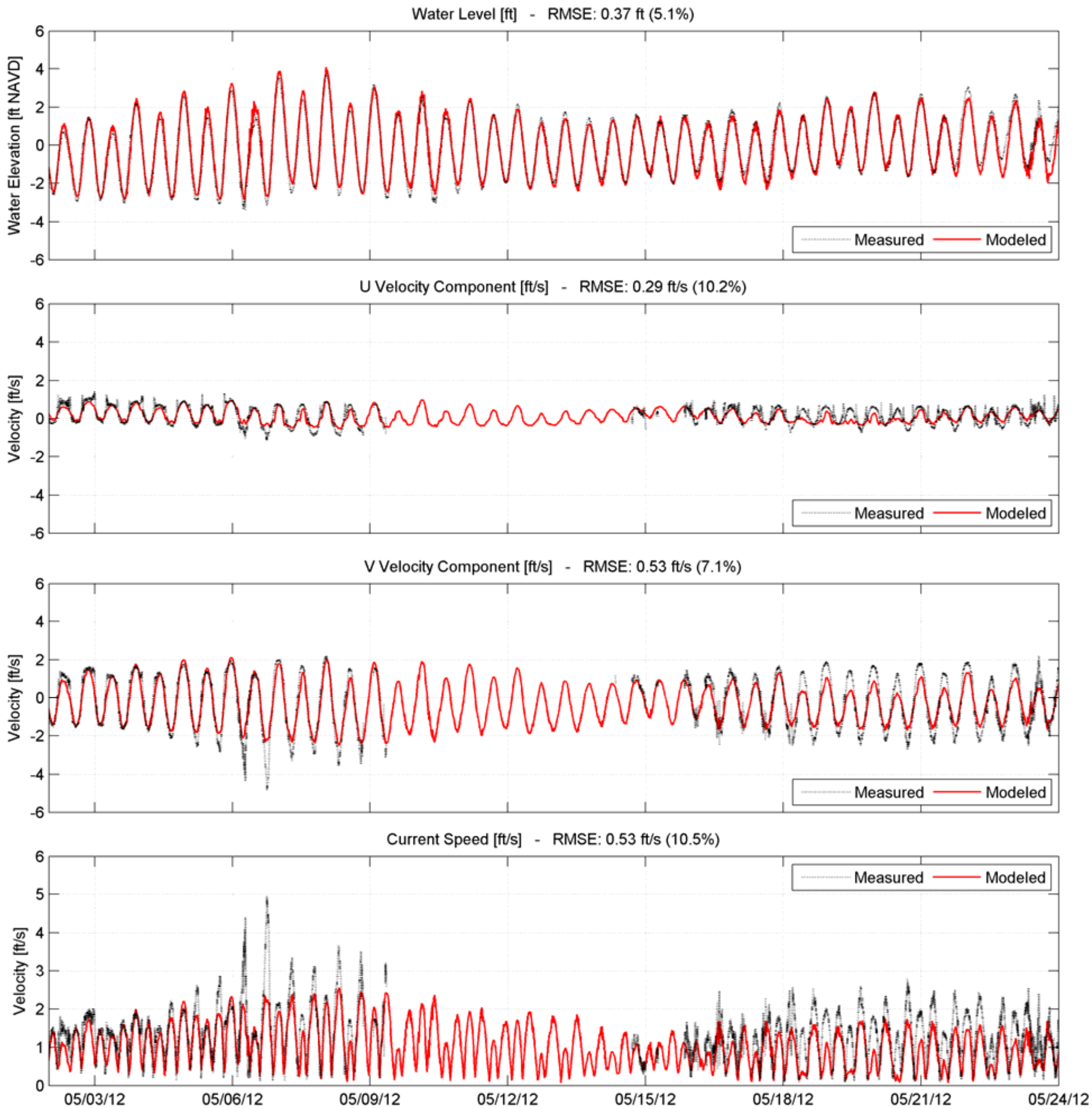
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-08



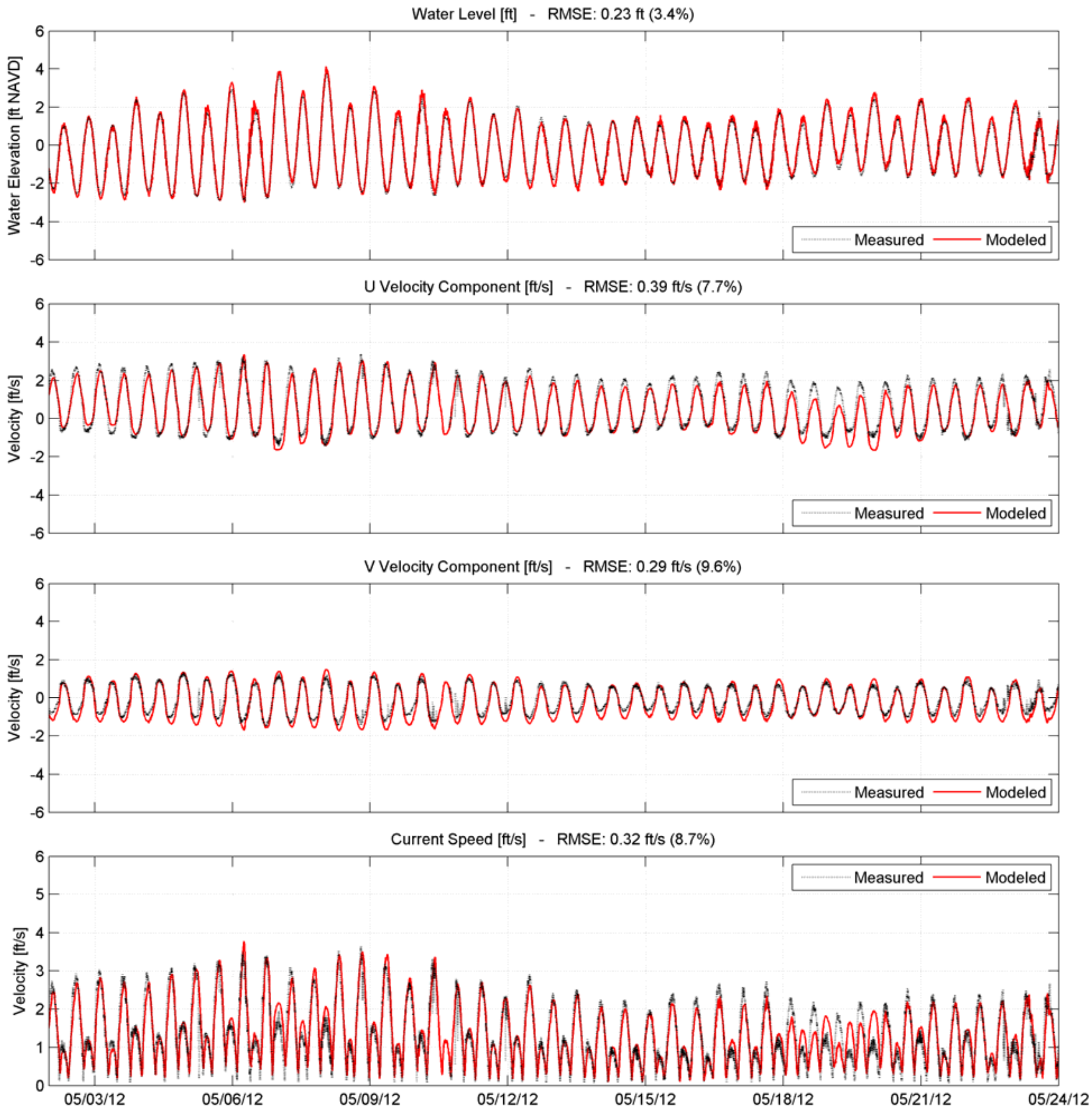
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-90



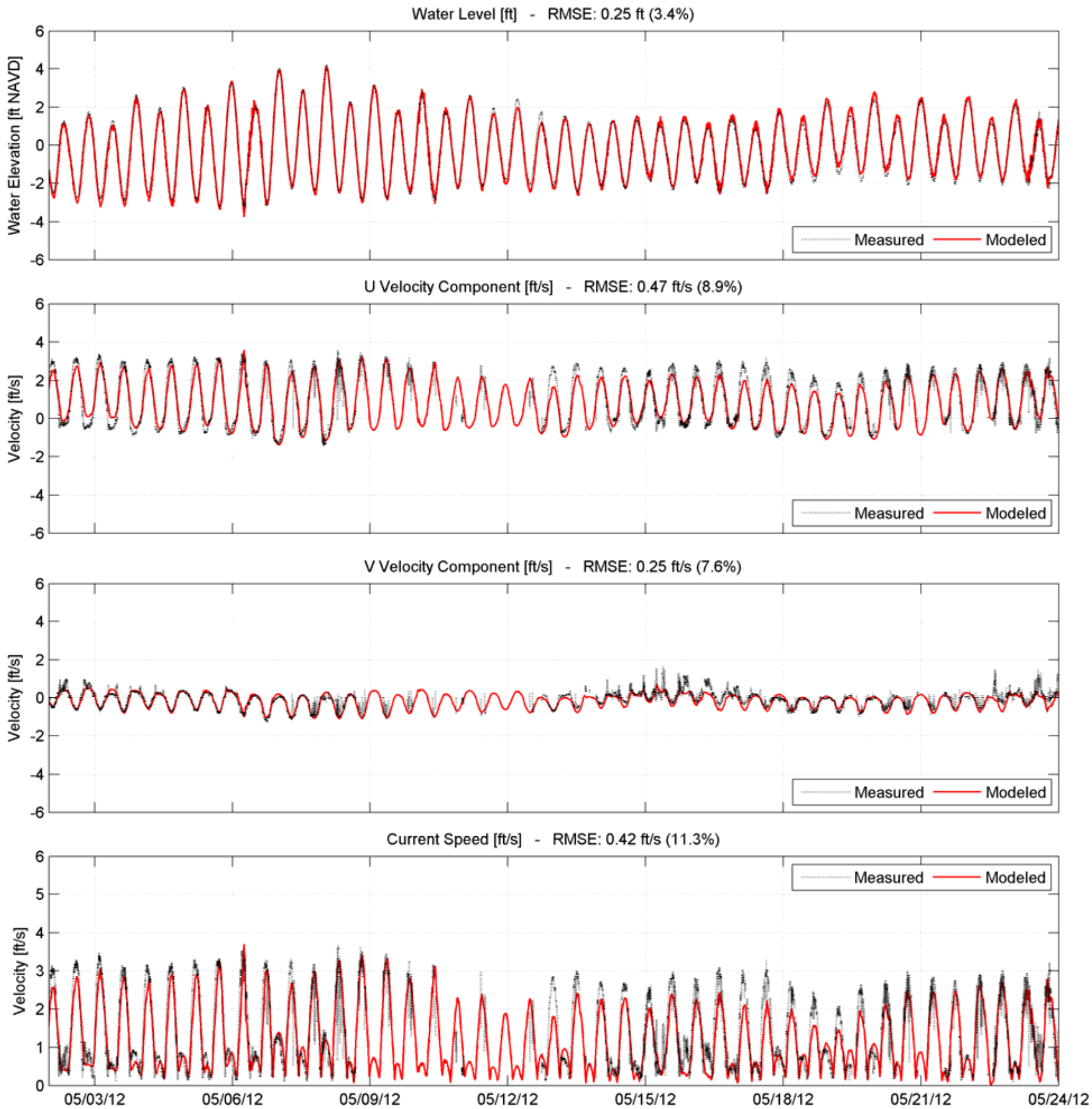
Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-91



Local Flow Model Calibration:
Comparison of Measured and Modeled Flow and Water Level Data

WHIO Gauge – WHOI-92



Appendix C

2016 Third Event Nourishment/Dredging Project Summary

COASTAL PLANNING & ENGINEERING OF NORTH CAROLINA, INC
4038 MASONBORO LOOP RD., WILMINGTON, NC 28449 910-791-9494 (PHONE)

July 13, 2016

Mickey Sugg
U.S. Army Corps of Engineers
69 Darlington Avenue
Wilmington, NC 28403

Subject: Environmental Considerations in Regards to the North Topsail Beach Shoreline Protection and Inlet Management Project, 3rd Event

Dear Mr. Sugg,

The Town of North Topsail Beach (Town) completed Phase 1 of the beach and inlet management plan in February 2013. Phase 1 included relocating the main bar channel of New River Inlet to a preferred position and alignment with deposition of the dredged material along approximately 7,730 feet of the Town's shoreline south of New River Inlet. The intent of the bar channel relocation was to induce a build-up of material on the south side of New River Inlet which would eventually result in accretion along the northern portion of the Town's shoreline. CPE-NC stated in the engineering report associated with the Final Environmental Impact Statement (EIS) that predictions of the actual time for the shoreline between stations 1140+00 and 1160+00 (area from between Buildings #5 and #6 of Topsail Reefs to the south shoulder of New River Inlet) to respond to the new channel cannot be made with a high degree of certainty; however, significant accretion should occur within 5 years with full recovery occurring within 15 years following the channel relocation (CPE-NC, 2009). These projections were based on the implementation of a channel maintenance program that would maintain the channel in the preferred location.

During the first two years following completion of Phase 1, fill placed along the northern portion of the project from baseline station 1130+00 north to the inlet experienced high rates of erosion. Essentially all of the fill material placed in this area eroded with most of the material being transported by natural processes to the north. A large portion of this naturally transported sand accreted on the southern shoreline of New River Inlet in the form of a sand spit. The erosion of the fill material placed seaward of the homes north of Topsail Reef during the Phase 1 project left these structures in imminent danger comparable to the conditions of the structures prior to the construction of the Phase 1 project. This prompted the Town of North Topsail Beach to construct a sand bag revetment to provide temporary erosion control along this section of shoreline.

In March 2015 CPE-NC provided the Town with a contingency report, which provided recommendations for modifications to the existing long-term inlet management strategy associated with the Town's long-term beach and inlet management program. One of the four recommendations described in the Contingency Plan was the modification of the channel alignment for the 2nd channel realignment event scheduled for the 2016/2017 environmental dredging window to improve project performance.

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In July 2015, the Town authorized CPE-NC to conduct a numerical modeling study using the Delft3D morphological model to evaluate alternative channel alignments for the proposed 2016-2017 project. The Delft3D model was used as an engineering tool to evaluate relative differences in response of a system (beach and inlet) to channel modification. Model simulations of alternate channel alignments were used to evaluate model-indicated volumetric changes along the adjacent shorelines, changes in the ocean bar channel (channel orientation, shoal volumes, channel depths, etc.), volumetric changes on the ebb tide delta, and sediment transport patterns. The model results were also used to assess relative differences in flow patterns from one option to another, differences in the significant wave heights that would impact the shoreline, and potential changes in the volume of water that would pass through various channels within the system.

As was discussed in a letter sent to you in February 2016, the Town is gearing up for the 3rd Event of the Shoreline Protection and Inlet Management Plan. Based on annual monitoring of the channel realignment project and the results of the Delft3D modeling analysis, the Town is proposing to modify the permitted channel design for the upcoming project. The proposed alternative channel alignment consists of a channel that is pivoted approximately 17 degrees to the east compared to the original channel dredged in 2012/2013. Figure 1 shows both the originally permitted and constructed channel dredged in 2012/2013 as well as the proposed modified channel. The width and depth of the modified channel are the same as the originally permitted channel, 500 ft. wide and -18 ft. NAVD88, respectively. Based on the April 2015 bathymetry, approximately 722,000 cy would need to be excavated to construct this channel.

Geotechnical Evaluations

Vibracores

The sediment to be dredged from the channel was characterized according to the methodology described in North Carolina's technical standards for beach fill projects (15A NCAC 07H .0312, section 2). In past efforts to characterize the material within the channel, vibracores had been collected in 2003, 2006, and 2008 in the vicinity of New River Inlet Ebb Shoal. CPE-NC reviewed the available data collected by both CPE-NC and the USACE and determined a minimum of 10 additional vibracores would need to be collected and analyzed within the proposed modified channel to comply with the state technical standards. A total of fourteen vibracores were taken in June 2016. The locations of these and past vibracores taken in the vicinity of the proposed realigned channel are shown in Figure 1.

Section (3) (a) of Rule 15A NCAC 07H.0312 requires that sediment completely confined to permitted dredge depth of a sediment deposition basin within an inlet shoal system has an average percentage by weight of fine-grained (less than 0.0625 millimeters) sediment less than 10%. An evaluation of vibracore data collected in 2003, 2006, 2008, and 2016 show the sediment in the proposed channel maintenance areas meet these criteria. Composite data for the channel dredged in 2012/2013 had an average composite percent by weight of fine-grained (less than 0.0625 millimeters) material of 1.53% based on vibracores collected in 2003, 2006, and 2008. Furthermore, vibracores obtained in 2016 show that the proposed channel has an average

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composite percent by weight of fine-grained (less than 0.0625 millimeters) material of 1.02% The composite percent fine grained material for the existing beach sampled along the east end of North Topsail Beach is 1.72%. Therefore, this compatibility analysis verifies that the borrow area material meets the allowable limits defined by the Technical Standards.

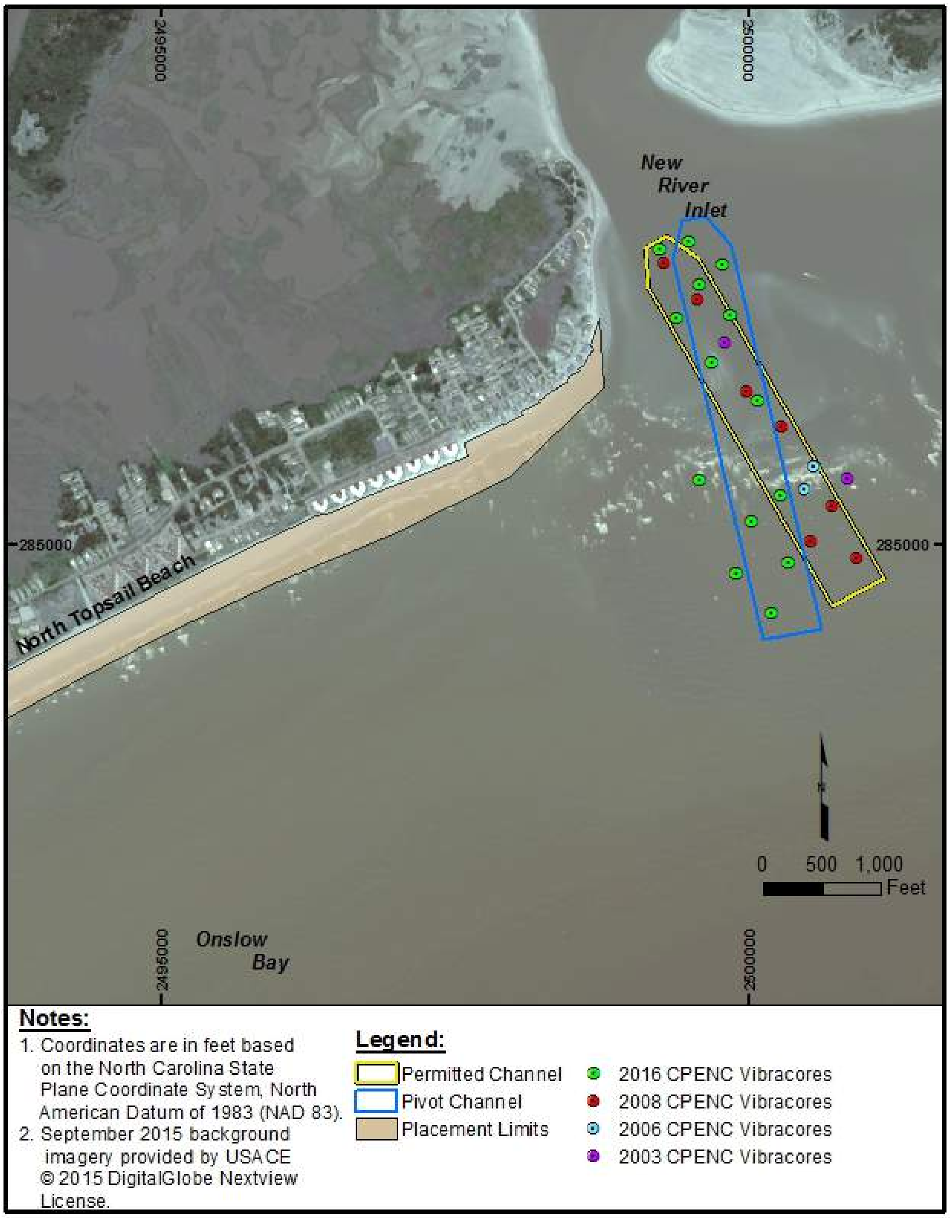

Figure 1. Location of original permitted channel and the newly proposed alterative channel alignment. Locations of vibracores taken in 2003, 2006, 2008, and 2016 are also provided. Placement limits for beach nourishment are displayed for area immediately adjacent to the inlet.

Cultural Resources

To identify submerged cultural resources within or in the vicinity of the original ocean bar channel, magnetometer and sidescan sonar surveys were conducted in 2005 and 2007 by Tidewater Atlantic Resources, Inc. (TAR). The original channel alignment was designed to avoid potentially significant magnetic anomalies identified during these surveys. The newly proposed pivot channel design will also avoid the magnetic anomalies identified in the 2005 and 2007 surveys. A portion of the southwest corner of the proposed realigned channel falls outside of the areas previously surveyed by TAR. Therefore, additional surveys will be conducted to cover that portion of the channel. Likewise additional areas will be surveyed to ensure adequate buffers are maintained around potentially significant anomalies in proximity (within 500 ft.) to the proposed channel. These surveys are scheduled for July 2016.

Project Footprint and Quantification of Impacts to Ecological Habitats

The ecological habitats present within the project area (Figure 2) have been delineated and described previously in the FEIS. The types of habitats present within the footprint of the proposed alternative channel and sand placement area are the same as those within the permitted channel footprint. These habitats include the intertidal flats and shoals, subtidal soft bottom and oceanfront dry beach (Figure 3). Because these habitats are located within the active construction footprint they will be directly and indirectly impacted by dredging and sand placement. Although not within the project footprint, the inlet dry beach will also be impacted by the alternative channel alignment. The acreages of each habitat impacted by the proposed activities are described below.

The alternative channel alignment is not anticipated to result in any additional or novel impacts to biological resources compared to the previously permitted channel for several reasons. First, as discussed below, the footprint of the alternative channel alignment overlaps much of the previously permitted channel. Secondly, those non-overlapping portions of the alternative channel contain the same habitat types encompassed by the permitted channel. Finally, because the timing and method of construction of the proposed 3rd event will be the same as the 1st event (containing the originally permitted channel orientation), the mechanisms, magnitude and timing of impacts to these habitats will be the same as was discussed in the FEIS.

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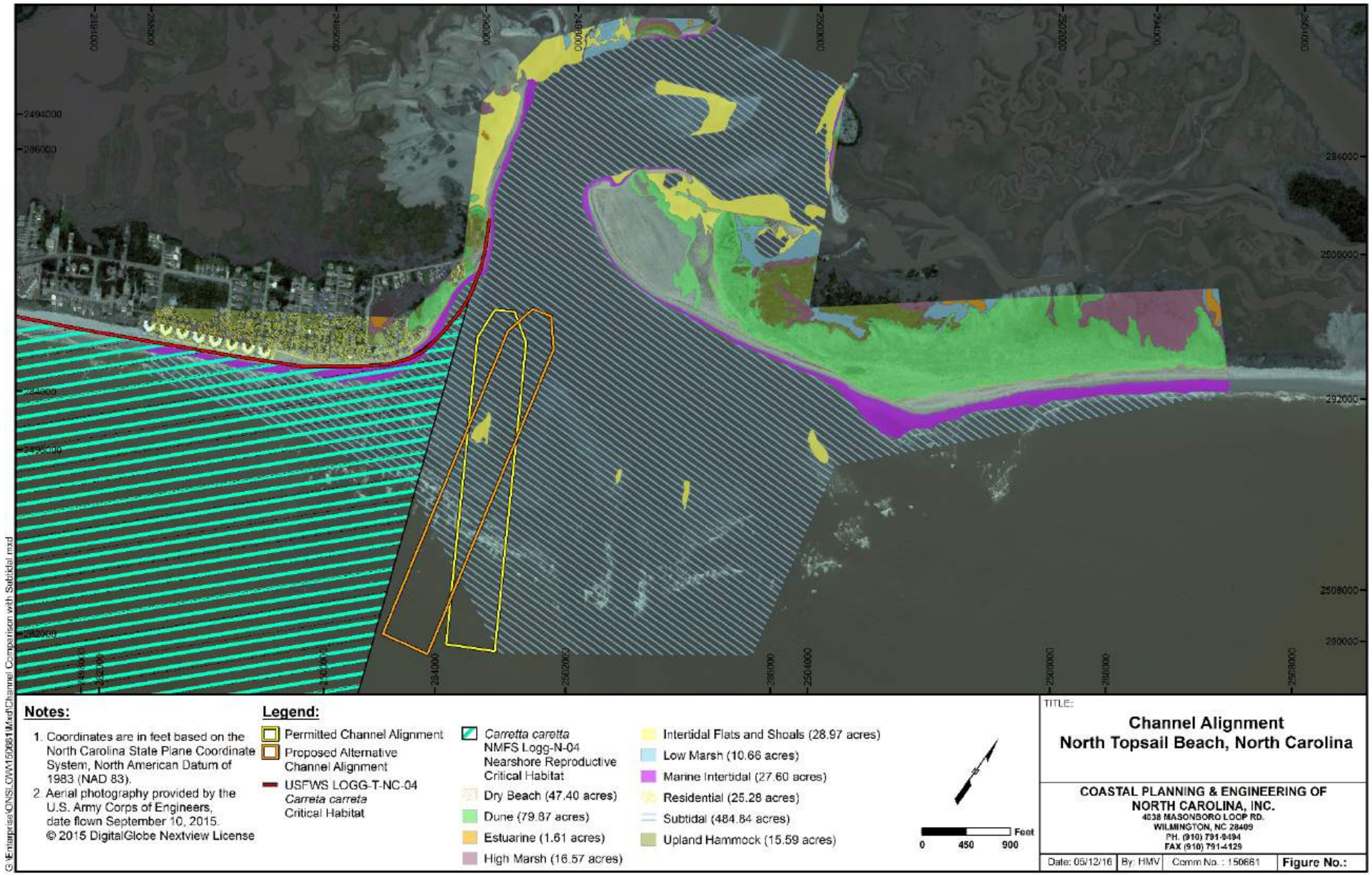

Figure 2. Biological habitats identified within the permit area. Critical habitat units LOGG-T-03 (terrestrial) and LOGG-N-04 (nearshore reproductive) are also shown.

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Intertidal Shoals and Subtidal Softbottom

Intertidal flats and shoals and the subtidal softbottom are the only two habitats within the footprint of the new channel alignment. The proposed alternative channel alignment encompasses a total area of 40.8 acres (ac), consisting of 40.0 ac of subtidal soft bottom and 0.8 ac of intertidal shoal, as of conditions observed on September 10, 2015 (Figure 2; Table 1).

The alternative channel alignment overlaps the original channel alignment by 18.7 ac; this area is composed of 17.9 ac of subtidal soft bottom and 0.8 ac of intertidal flats and shoals.

Table 1. Acreages of habitats within the permitted channel and proposed alternative channel.

Habitat	Alternative Channel (ac)	Overlap with Permitted Channel (ac)
Subtidal Soft bottom	40	17.9
Intertidal Flats/Shoals	0.8	0.8
Total (ac)	40.8	18.7

The 0.8 acre of intertidal shoals within the new channel alignment falls within the section that overlaps the originally permitted channel; therefore impacts to this area remain the same as was discussed in section 5.3.2.3 of the FEIS. These include the complete removal of sediment down to -18 ft. NAVD88, which will result in direct and immediate mortality of the infaunal community within the dredging footprint, and associated indirect impacts to shorebirds, fish, and macroinvertebrates that forage upon the benthic infaunal community.

There are 40.0 ac of subtidal soft bottom within the alternative channel, 17.9 ac of which overlap the original channel. Therefore, a total of 22.1 ac of new subtidal soft bottom habitat will be impacted; however, impacts will be the same as discussed in the FEIS. These impacts are similar to those discussed above for the intertidal shoal, and include the complete removal of sediment down to -18 ft. NAVD88, which will directly, but temporarily, impact the benthic infaunal community via mortality. Complete removal of this infaunal community will indirectly impact fish and macroinvertebrates via temporarily reducing the infaunal prey base until recovery occurs. Other direct impacts will include a temporary increase in turbidity and suspended sediments within the water column.

Oceanfront Dry Beach

The beach nourishment and channel alignment is predicted to positively impact the oceanfront and inlet dry beaches by expansion of these habitats with quality sand. Sand placement will create additional dry beach, while the proposed new channel alignment is anticipated to encourage further accretion along the northeastern end of North Topsail Beach. The mechanisms and repercussions of these impacts to other natural resources will be the same as discussed in section 5.3.3.2 of the FEIS. When nourished with quality, compatible sand, the additional dry beach habitat will provide nesting habitat for sea turtles, and foraging and roosting habitat for shorebirds and waterbirds.

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The anticipated accretion and stabilization of the ocean front dry beach along the north end of North Topsail Beach may reduce the occurrence of dynamic and ephemeral habitats like overwash and dune blow-outs that are primary habitats for piping plovers and seabeach amaranth. However, the presence of dense residential areas and sandbag walls largely eliminate these ephemeral habitats from this area; therefore the increased beach acreage may ultimately serve to benefit species such as piping plovers and sea beach amaranth by providing habitat that is otherwise non-existent.

Tidal Prism

A Delft3D numerical modeling study was conducted to evaluate relative differences in the response of the beach and inlet system to channel modification. The volume of water flowing through the New River Inlet throat as well as the volume of water flowing through Cedar Bush Cut (Cross-Sections A & B in Figure 3, respectively) during the ebb cycle was computed by averaging all of the ebb flows simulated for the period May 2, 2012 to May 31, 2012 for all channel alternatives. The volume of water flowing out of an inlet during the ebb phase of the tidal cycle is referred to as the tidal prism. The simulations of the proposed channel alignment indicated a 2.2 percent and 2.3 percent increase in the simulated tidal prism at transects A and B, respectively when compared to the simulated No Action alternative. These relatively small differences in flow volume for the channel alternative implies there would be essentially no measurable difference in impacts on the estuarine environment associated with a change in the tidal regime of New River Inlet.

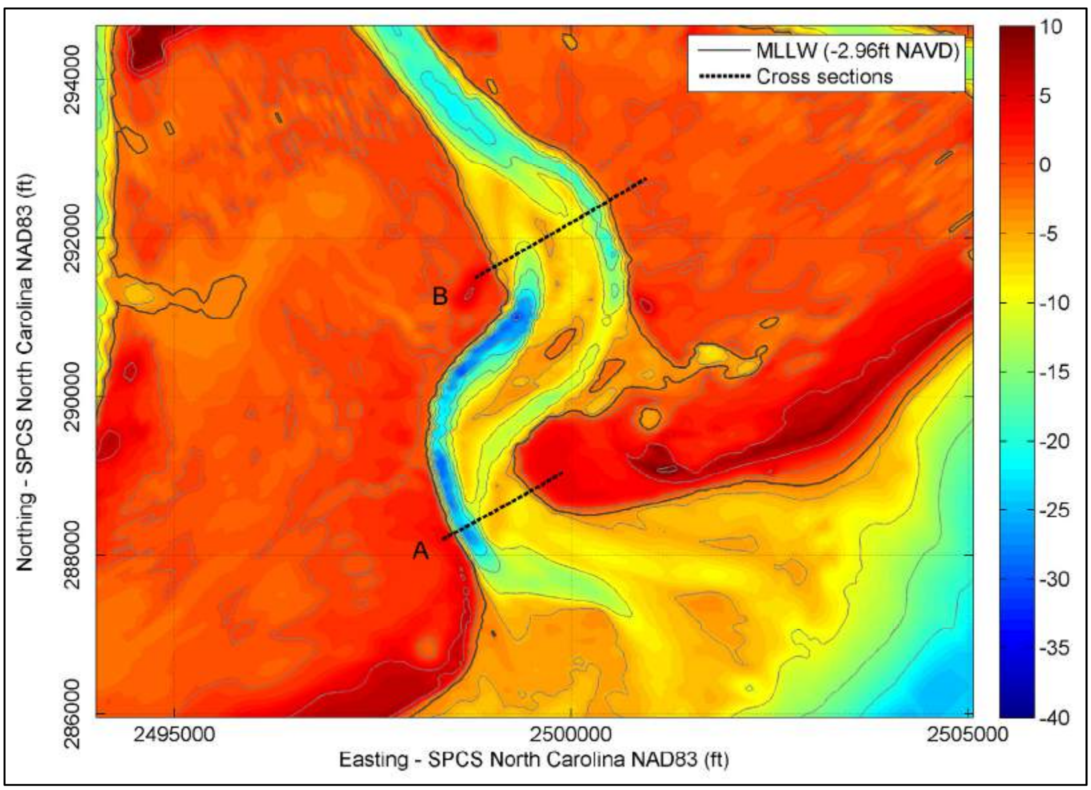

Figure 3. Cross-Sections used to compute model tidal prisms

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Overview of General Impacts to Biological Resources

As mentioned previously, no new or novel impacts are anticipated with the proposed modification owing to the overlap between the proposed alternative channel and the permitted channel, similarity of habitats within the two footprints and unchanged construction methodologies. As such, the impacts to biological resources associated with the affected habitats are the same as those discussed within section 5.2 of the FEIS. This information is incorporated by reference and summarized below. Likewise new or updated data and biological information is also provided.

Sedimentation and Turbidity

Dredging within the channel will cause an increase in suspension of sediment particles in the water column, thereby temporarily elevating turbidity levels. This, in turn can affect water quality and light levels within the water column. Resulting indirect impacts will include temporary dispersion or disorientation of fish within the area, and reduced photosynthesizing capabilities of phytoplankton within the immediate area. A hydraulic pipeline cutterhead dredge will be used to extract material from the channel. With this type of dredge, the excavated material is suctioned directly into the dredge head, and the re-suspension of sediment is localized to the area around the draghead. Thus, relative to other types of dredges, a pipeline cutterhead dredge suction limits the possibilities for re-suspension of sediment to the point of extraction from the seafloor. The low percent silt content of material contained within the channel will allow material to settle out of the water column relatively quickly; therefore, the increase in suspended sediments and associated turbidity is expected to be temporary.

Placement of sand on the oceanfront shoreline will also elevate turbidity levels within the nearshore marine environment. Studies of similar projects suggest elevated concentrations within the nearshore extend 1,310 – 1,640 ft. from the discharge pipe in the swash zone, and is generally reported to be short-term (Hitchcock *et al.* 1999; Anchor Environmental 2003; Roman-Sierra *et al.* 2011). The length and shape of the plume depends, in part, on the hydrodynamics within the water column as well as the sediment grain size. In sandy substrates typical of borrow sites, the grain size is larger and the extent of sediment suspension is therefore more restricted. The composition of sediment that will be taken from the channel and placed along the shoreline is considered compatible with North Topsail Beach native beach sand, per the State's Technical Standards for Beach Fill Projects (15A NCAC 07H .0312). The placed material will therefore have a low percentage of fine-grained sediment, thus promoting settling out of sediments suspended in the water column. Importantly, natural conditions also support fluctuating turbidity levels that can be quite high (9.7 to 35.2 Nephelometric Turbidity Units [2009 FEIS]) in the inlet and nearshore water column. Due to the combination of ambient turbidity conditions and the quality of fill material that will be used, project-related impacts to turbidity should be minimized and temporary.

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Infaunal Benthic Community

Dredging and sand placement results in the immediate mortality of the benthic infaunal communities within the borrow area as well as on the beach where sand placement occurs. Mechanisms of this impact include mechanical damage incurred during dredging, entrainment in the dredge pipe, burial and suffocation at the fill site, and crushing by heavy equipment as the material is shaped and graded on the beach. The reported recovery rates of these infaunal communities, as cited within the literature, vary and are dependent upon the taxa studied (Wooldridge et al. 2016). Recovery rates were documented to be within one year or less for amphipods (Jones et al. 2008; Leewis et al. 2012; Schlacher et al. 2012), mole crabs (*Emerita* spp.) (Hayden and Dolan 1974; Leewis et al. 2012; Peterson et al. 2014), bean clams (*Donax* spp.) (Leewis et al. 2012) and polychaetes (Leewis et al. 2012; Manning et al. 2014). Other studies cited in the paper report complete recovery within one year of certain other taxa (Burlas et al. 2001; Peterson et al., 2006, Jones et al. 2008; CZR Incorporated and CSE, Inc., 2013; CZR Incorporated and CSE, Inc., 2014).

There are a number of factors associated with construction practices that can influence recovery rates. In studies focusing on the intertidal macrofauna at beach fill sites, the reported recovery rates ranged from less than one month (Gorzelany and Nelson 1987), less than one year (Parr et al. 1978; Jutte et al 2002a and b), and to up to 2 years (Rakocinski et al. 1996). Factors contributing to the recovery rates, as cited in these studies, included the seasonality of construction and the similarity of sediments used as fill material to the native beach sediments. Populations of infaunal benthic invertebrates typically follow a cyclic, seasonal pattern of recruitment in the spring months and migration offshore during the winter months. Therefore, projects incorporating well-matched sediments and construction periods that avoid the spring recruitment pulse are associated with faster recovery rates. By contrast, springtime construction and a poor sediment match (too coarse, shelly, or fine) leads to longer recovery times.

The proposed project will take place within the recommended environmental dredging window (November 15 to April 30) and will therefore not coincide with the spring recruitment period for benthic infauna resulting in limited impacts to these organisms. Furthermore, the sediment obtained from the channel is also compatible with native sediments. Therefore, the dredge and sand placement is anticipated to result in immediate negative impacts to the benthic infaunal community; however, these impacts are expected to be temporary and short-term recovery is anticipated. Additionally, the inlet maintenance cycle time period will remain 4-years, as with the old channel alignment, which will allow sufficient recovery of the infaunal community both within the dredged channel area, and on the beach.

Shorebirds

A variety of waterbirds and shorebirds utilize the estuarine, beach, dune, and other habitats within the inlet complex. Pre- and post-construction bird monitoring surveys of the New River Inlet complex conducted in association with the 1st event identified numerous bird species that utilize these areas for foraging, roosting and breeding habitats throughout the year. The habitats within the inlet complex also vital resources as “stopover locations” for migratory bird species.

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The most-recent post-construction survey (Grant 2016) identified 40 waterbird species, with Laughing Gulls (*Larus atricilla*), Brown Pelicans (*Pelecanus occidentalis*), Herring Gulls (*Larus argentatus*), Ring-billed Gulls (*Larus delawarensis*), and Double-crested Cormorants (*Phalacrocorax auritus*) accounting for the majority of species observed (Grant 2014; 2015; 2016). Among 23 species of shorebirds observed, the most common species typically identified include the Dunlin (*Calidris alpina*), Sanderling (*Calidris alba*), Short-billed Dowitcher (*Limnodromus griseus*), Semipalmated Plover (*Charadrius semipalmatus*), and Black-bellied Plover (*Pluvialis squatarola*) (Grant 2016).

Impacts to shorebirds and waterbirds will include disturbance due to noise and activity associated with construction on the beach and in the channel. Birds may be flushed from the area, but will likely seek alternate foraging and roosting habitat on adjacent sections of beach on North Topsail Beach or Onslow Beach. Likewise, as mentioned above, the infaunal communities will be eradicated from any existing dry or intertidal beach upon placement of sediment on the beach. The resulting depressed infaunal abundances are expected to be temporary as the combination of winter time construction schedule and use of quality, compatible sediments should allow for short term recovery (one to two years) of the infaunal prey-base for shorebirds. Additionally, the amount of dry beach and intertidal habitat currently available for birds along the northeastern end of North Topsail Beach is extremely reduced; therefore the increased acreage of dry beach and intertidal habitats provided by nourishment would ultimately benefit shorebirds. The pre- and post-construction bird monitoring surveys demonstrate that construction did not deter birds from utilizing the area – total species and total number of individuals observed increased from pre- to post-contsruction. Species richness typically peaked during spring and fall migrations, and remained similar throughout all surveys.

Parameter	Pre-Construction	Post-Construction 1	Post-Construction 2	Post-Construction 3
Total Species	94	101	98	91
Total Abundance	39736	43357	45257	41352
Peak Shorebird Species Richness	20	19	20	19
Peak Waterbird Species Richness	17	24	25	25

Fish and Essential Fish Habitat (EFH)

The soft bottom community provides foraging habitat to fish. Disturbance to these areas can reduce food availability if the infaunal prey base is impacted, causing fish species to seek out alternate foraging habitat. Dredging and discharge of slurry at the fill site can also disperse fish from these construction sites via increases in turbidity, discussed above.

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Impacts to Threatened and Endangered Species

Terrestrial Critical Habitat for the Loggerhead Sea Turtle

Since the development of the 2009 Final EIS, critical habitat has been established for the loggerhead sea turtle (*Caretta caretta*). On July 10, 2014 the USFWS designated 1,102 km of the western Atlantic and Gulf of Mexico coastlines as terrestrial critical habitat for the Northwest Atlantic Ocean Distinct Population Segment (NWA DPS) of loggerhead sea turtles. Critical habitat has been designated on sandy beaches capable of supporting a high density of nests in North Carolina (Brunswick, Carteret, New Hanover, Onslow and Pender counties), as well as several counties within South Carolina, Georgia, and Florida (79 FR 39756). The closest terrestrial critical habitat unit to the permit area is LOGG-T-NC-03. This unit spans 21.8 miles of shoreline along Topsail Island in Onslow and Pender Counties, and extends from the New River Inlet to New Topsail Inlet. The unit includes lands from the Mean High Water (MHW) line to toe of secondary dune or developed structures. The local municipality portion is the North Topsail Beach Park, which is managed by the Town of North Topsail Beach. This unit reportedly has high density nesting by loggerheads, and contains all the physical or biological features (PBFs) and primary constituent elements (PCEs) considered essential to the conservation of this species. Detailed descriptions and maps may also be found in the USFWS final rule for critical habitat designation (79 FR 39756).

Sand placement will occur within the environmental dredging window and will therefore not coincide with any sea turtle nesting activity. Unit LOGG-T-NC-03 will be directly impacted by sand placement activity; namely via the anticipated increase in acreage of dry beach available for nesting. While the placement of new sand can potentially indirectly impact sea turtle nesting critical habitat via increasing compaction, alteration of color or geological composition, the use of highly compatible material will greatly minimize these impacts. The new channel alignment will indirectly impact the unit via the anticipated stabilization of dry beach and intertidal habitat along the northern end of North Topsail Beach. USFWS defines the PBFs of nesting habitat, summarized as having elements that support nutritional/physiological requirements such as space, food, water, light, air, shelter etc., sites for breeding, and habitats protected from disturbance or are representative of the historical, geographic, and ecological species distribution. The PCEs that support these PBFs can be summarized as suitable nesting beach that 1) has unimpeded access between beach and ocean, 2) sand that allows suitable nest construction, sand diffusion, and maintains appropriate temperatures, 3) sufficient darkness 4) natural coastal processes or artificially created or maintained habitat mimicking natural conditions. In the final rule designating critical habitat (79 FR 39756), the USFWS determined that habitat modification and loss occurs with beach stabilization activities that prevent the natural transfer and erosion and accretion of sediments along the ocean shoreline. However, USFWS acknowledges that when sand placement activities result in beach habitat that mimics the natural beach habitat conditions, impacts to sea turtle nesting are minimized. The proposed alternative channel alignment is therefore not expected to adversely modify critical habitat LOGG-T-NC-03 for the loggerhead sea turtle.

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Nearshore Reproductive Critical Habitat for the Loggerhead Sea Turtle

On July 10, 2014 the National Marine Fisheries Service (NMFS) designated marine critical habitat for the loggerhead sea turtle NWA DPS within the Atlantic Ocean and the Gulf of Mexico. The closest unit to the permit area is unit LOGG-N-04, which encompasses nearshore reproductive habitat. The unit extends from MHW line seaward 1.6 km, and extends from the New River Inlet to Rich Inlet (crossing New Topsail Inlet). The entire nearshore reproductive unit LOGG-N-04 encompasses 16,721 ac. Nearshore reproductive habitat includes hatchling swim frenzy and nesting female habitat. NMFS describes the PBF of nearshore reproductive habitat as a portion of the nearshore waters adjacent to nesting beaches that are used by hatchlings to egress to open-water environment as well as by nesting females to transit between beach and open water during the nesting season. The PCE that support this PBF are summarized as nearshore waters offshore the highest density nesting beaches, that are sufficiently free of obstructions, artificial lighting, or manmade structures that would hamper transit between beach and open water for hatchlings and nesting females.

Dredging will be performed during the cooler winter months within the environment dredging window, between November 16 and March 31. The seaward-most portion of the alternative channel lies approximately 80 ft. to the northeast of unit LOGG-N-04. Because the dredging activity will occur outside of the sea turtle nesting season, and the project footprint does not overlap the unit, no impacts to the Primary Biological Factors or Primary Constituent elements are anticipated. The project will not adversely modify critical habitat unit LOGG-N-04 for the loggerhead sea turtle.

Rufa Red Knot

The *rufa* red knot (*Calidris canutus rufa*) is one of the six subspecies of red knots and one of the three that resides in the Western Hemisphere. Subspecies *rufa* winters in northern Brazil, the greater Caribbean and along the U.S. coast from Texas to North Carolina. Due in part to substantial population declines in the 1990's and 2000's. On December 11, 2014, the USFWS published the final rule listing the *rufa* red knot as threatened under the ESA (79 FR 238). Although the Delaware Bay and coastal Virginia represent the largest stopover concentration of *rufa* red knots, coastal North Carolina does support the birds during their spring and fall migrations. Various surveys for *rufa* red knots have been performed throughout the state and data from these surveys are maintained by the North Carolina Wildlife Resources Commission (NCWRC). These surveys are performed at discrete times of year, as well as opportunistically to fulfill various permit requirements or research interests. Surveys are not performed systematically or monthly, therefore, it should be emphasized that lack of data in the NCWRC database does not imply absence of the species; rather, it implies only that no surveys were performed at that time (Sara Schweitzer, *pers. comm.*, March 26, 2014). To determine red knot presence in the vicinity of the permit area, data from the various surveys within the NCWRC database were summarized to determine total counts per month of *rufa* red knot observations along Topsail Island and Onslow Beach. Although statewide red knot surveys have been performed since 1985, data for these areas are only available from 2006 through 2013. Habitats surveyed include oceanfront beaches along barrier islands, dredge material islands and sand and

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inlet shoals. It should be noted that surveys for the *rufa* red knot in North Carolina are quite varied, inconsistent and were not conducted every month or in all years. Therefore, it cannot be determined if red knots were present at un-surveyed times or locations

As shown in the Figure 4, red knots were present during each month surveyed, which included January, February, May, June, November and December. The greatest number of red knot observations occurred during the month of May; however, this month also corresponds with the greatest number of survey events. It should be noted that the surveys from which these data are obtained are quite varied, and the methodologies, geographic extent, and effort for each survey type were not readily available, therefore the data could not be standardized for further comparison.

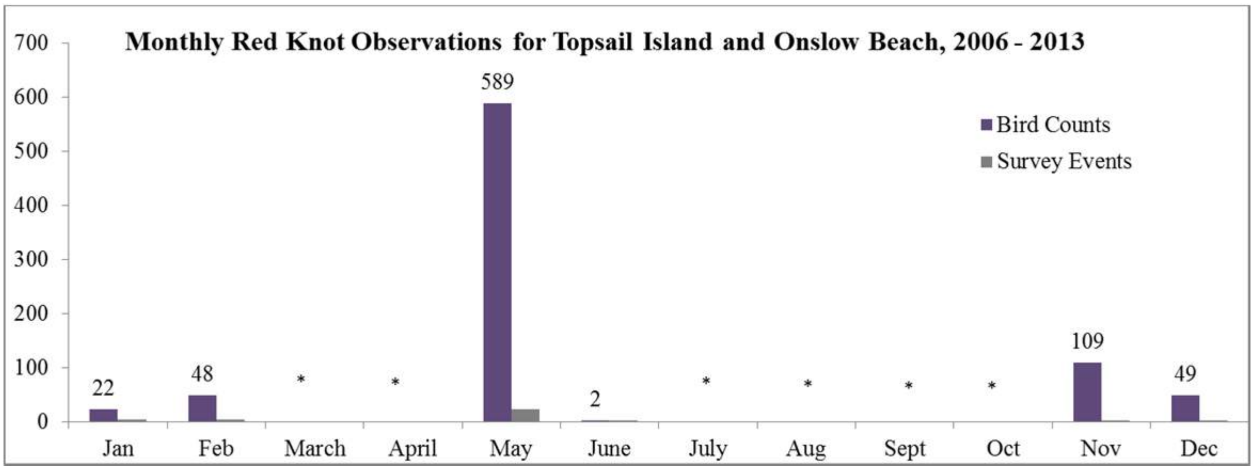

Figure 4. Summary of total red knot observations per month, identified within various surveys conducted aalong Topsail Island and Onslow Beach. Data obtained from NCWRC (Sara Schweitzer, pers. comm.)

It is unlikely that red knots will be present within the permit area when project operations are underway. The planned construction period of November 16 through March 31 is outside of the red knot spring and fall migration period and populations are not known to overwinter in NC. In the unlikely event a red knots are present during construction, it is likely these individuals will adjust to the disturbance and continue to rest and forage in adjacent surrounding habitat. This was demonstrated during the initial New River Inlet channel project, where the presence of other bird species (piping plovers) was recorded as dredging and beach-fill work was ongoing. Additionally, Grant (2016) observed 131 red knots in the New River Inlet area during the third-year post-construction surveys, as compared to 119 red knots observed during the pre-construction survey. It is therefore determined that the project is not likely to adversely affect the *rufa* red knot.

As discussed, we request that you consider the information provided herein to assess whether any additional impacts on T&E species, critical habitat, or biological resources are anticipated as a result of our proposed modification to the inlet channel. By virtue of information cited within, we feel that no additional effects beyond what was documented during our initial consultation with USFWS and NMFS would be incurred to these resources. We will continue to provide you any additional relevant information as it becomes available.

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Please feel free to contact me with any questions or comments regarding this information.

Very truly yours,

COASTAL PLANNING & ENGINEERING OF NORTH CAROLINA, INC.

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Project Manager

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Adam Priest, CPE-NC
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Appendix D

2017 Vulnerability and Optimization Study

COASTAL PLANNING & ENGINEERING OF NORTH CAROLINA, INC.

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January 3, 2017

Stuart Turille
Town of North Topsail Beach
2008 Loggerhead Court
North Topsail Beach, NC 28460

Subject: Findings of Shoreline Risk Assessment (Phases 2, 3 and 4) and Fill Optimization (Phase 1)

Dear Mr. Turille:

On October 14, 2016, you provided CPE-NC with notice to proceed on a Change Order to conduct a shoreline risk assessment of Phases 2, 3 and 4 as well as a fill optimization analysis for Phase 1. These analyses are aimed at determining the ideal beach placement and fill configuration for the 3rd Event Project proposed for 2017/2018. This letter report and appendices serve as the deliverable as described in the Change Order agreement between CPE-NC and the Town of North Topsail Beach. The findings include an introduction, a description of the methodology, results of the analysis, conclusions, and recommendations.

Introduction

In September 2016, CPE-NC concluded a numerical modeling study using the Delft3D morphological model to evaluate alternative channel alignments for the proposed 3rd Event project. In this study, model simulations of alternate channel alignments were used to evaluate model-indicated volumetric changes along the adjacent shorelines, changes in the ocean bar channel (channel orientation, shoal volumes, channel depths, etc.), volumetric changes on the ebb tide delta, and sediment transport patterns. The model results were also used to assess relative differences in flow patterns from one option to another, differences in the significant wave heights that would impact the shoreline, and potential changes in the volume of water that would pass through various channels within the system.

Based on the results of the numerical modeling study, the geomorphic analysis conducted by CPE-NC as part of the EIS (2009), and the monitoring conducted by CPE-NC since the Phase 1 project was completed (CPE-NC, 2014; CPE-NC, 2016a and CPE-NC, 2016b), CPE-NC recommended that if the Town were to proceed with realigning the ocean bar channel of New River Inlet in 2017/2018, Alternative 3 (Pivot Channel) would be the recommended channel alignment.

CPE-NC also recommended the Town proceed with obtaining permit modifications to allow for the construction of Alternative 3 for the next scheduled channel realignment project. In April 2016, the Town authorized CPE-NC to prepare environmental documentation and permit modification applications to allow for the modified channel alignment. The plan for which the Town is requesting a permit modification includes dredging of sand from the Pivot Channel and placement of that sand along portions of Phase 1 to renourish the beach and to place sand along portions of Phase 2 to initially construct that portion of the shoreline protection project. Based on feedback from property owners in other sections of the Town that have not yet had the design beach constructed, Town staff asked CPE-NC to assess the risk of properties in all areas where a shore protection project had not yet been constructed.

The Town completed construction of Phase 1 of its Shore Protection Project in March 2013 (1st event) and Phase 5 (2nd event) in June 2016. The Phase 1 beach fill covered the shoreline from station 1163+00 south to station 1090+00. Most of the fill placed north of station 1140+00 was lost within 2 years following placement. A third recommendation of the September 2016 CPE-NC numerical modeling analysis was to run model simulations to identify an optimal beach fill for the north end for the next channel realignment project.

Shoreline Risk (Vulnerability) Assessment:

Methodology: Storm induced shoreline changes were computed using the USACE beach profile storm response model known as SBEACH during the feasibility phase of the Town of North Topsail Beach's Shoreline Protection Project. The results of this analysis are described in the Engineering Report (CPE-NC, 2009). As part of that analysis, CPE-NC generated storm frequency curves that were used to develop impact distances for storms with return intervals of 5, 10, 15, 20, and 25 years applicable to various sections of North Topsail Beach. The term "return interval" refers to an estimate of the frequency where the statistical likelihood of a storm event with certain characteristics may occur. For example, a storm event with a 25-yr return interval (25-yr storm event) means there is the statistical probability that the storm has a 1-in-25 (or 4%) chance of occurring in any given year. Similarly, a storm event with a 5-yr return interval (5-year storm event) has a 1-in-5 (or 20%) chance of occurring in any given year.

The storm impact distances are based on the distance from the position of the 2013 0-foot NAVD contour to the landward point where the post-storm profile is 0.5 foot lower than the existing profile. The analysis utilized available survey data collected along all three phases in 2013. Consideration was given to the possible changes in the dunes from the 2005/2006 profile data used in the SBEACH analysis and the condition of the dunes in 2013. The differences in the condition of the dunes at each of the profile locations between the survey datasets were compared and although changes had occurred the changes were not significant. The storm impact distances are given in Table 1.

Table 1. Return Intervals for Storm Impact Distances (CPE-NC, 2009)

Baseline Stations	Storm Impact Distances (feet)(1)				
	Return Intervals (years)				
	5	10	15	20	25
1035+00 to 1165+00	200	223	254	293	305
955+00 to 1035+00	192	221	255	274	280
905+00 to 955+00	187	232	298	316	320
855+00 to 905+00	180	216	253	268	275
785+00 to 855+00	173	199	208	220	230

(1)Storm Impact Distance = Distance from 0-ft NAVD on existing profile to the 0.5-foot erosion point on the post-storm profile.

The potential for damage to buildings associated with the SBEACH results were evaluated by creating shoreline positions or "impact lines" based on the storm impact distances for the 5, 10, 15, 20, and 25 year return intervals. While the lowering of the profile by 0.5 foot would not necessarily result in damage to structures, the assumption associated with the SBEACH results is that the location of the impact line relative to structure provides an indication of whether the structure would be impacted by waves and storm surge associated with the storm. If the impact line remained seaward of a structure, the structure was considered not impacted. However, if the impact line is landward of the seaward face of the structure, the structure was considered impacted by the storm event. The impact lines were superimposed on existing aerial imagery to identify the location and number of structures as well as roads that could potentially be impacted by the selected storm events. SBEACH was run using a suite of 37 historic storms that affected the study area over the 107-year period from 1893 to 1999. Characteristics of the 37 historic storms were provided by the Wilmington District Corps of Engineers. Table 2 lists the maximum still water level (SWL), maximum wave height, and the maximum wave period.

Table 2. Summary of Extreme Wave Events (Tropical) Impacting North Topsail Beach

Event	Date	Max SWL (ft. MSL)	Max Wave Height (ft)	Max Wave Period (s)
1	October 3, 1893	5.3	23.0	12.9
2	October 20, 1910	8.5	23.0	13
3	September 18, 1928	7.0	23.0	11.8
4	October 2, 1929	3.8	23.0	12.6
5	September 12, 1930	4.2	16.8	8.7
6	September 5, 1935	6.6	23.0	17.4
7	August 2, 1944	4.2	15.4	8.4
8	October 19, 1944	3.9	23.0	12.7
9	September 24, 1947	3.1	23.0	12.7
10	September 27, 1953	2.7	23.0	12.2
11	October 15, 1954	13.9	20.5	9.7
12	August 12, 1955	3.1	23.0	11.1
13	August 17, 1955	6.0	11.4	7.2
14	September 19, 1955	3.6	23.0	12.5
15	September 27, 1956	4.2	23.0	11
16	September 11, 1960	16.6	23.0	10.6
17	June 11, 1966	3.0	21.1	9.8
18	June 12, 1968	4.8	21.8	10
19	October 19, 1968	3.4	11.0	7.1
20	August 27, 1971	3.7	17.4	8.9
21	June 21, 1972	3.5	23.0	12.8
22	September 5, 1979	3.9	12.1	14
23	August 20, 1981	5.9	12.8	9
24	June 19, 1982	3.7	17.1	11
25	September 12, 1984	7.1	23.0	14
26	September 27, 1985	3.6	23.0	19
27	November 23, 1985	4.5	12.5	11
28	September 22, 1989	6.3	18.0	15
29	June 6, 1995	3.7	16.1	12
30	June 19, 1996	2.5	12.3	9
31	July 12, 1996	2.7	23.0	14
32	September 6, 1996	5.1	23.0	15
33	October 8, 1996	5.3	17.4	11
34	August 26, 1998	5.7	23.0	18
35	August 30, 1999	3.6	23.0	16
36	September 16, 1999	11.0	23.0	15
37	October 18, 1999	2.8	23.0	14

The vulnerability of structures along the North Topsail Beach shoreline within the remaining phases was evaluated by identifying the number of structures and length of road that could be affected by the different impact lines. For the purpose of this assessment, the Phases are shown in Figure 1 and have been delineated as follows:

- Phase 2 - Station 1090+00 to Station 968+30 (South side of Ship Watch Villas to the south side of the Villa Capriani)
- Phase 3 - Station 968+30 to Station 900+00 (South side of the Villa Capriani to approximately 100 ft. north of the emergency access ramp at the Jeffreys Lot)
- Phase 4 - Station 900+00 to Station 763+00 (Approximately 100 ft. north of the emergency access ramp at the Jeffreys Lot to approximately 200 ft. north of NTB BA-28a)

Figure 1. Phase Location Map

Results and Discussion: The impacts for each storm were computed by totaling the number of structures where the impact lines either intersected with or were landward of the seaward face of the structure as delineated by the roof line. The evaluation utilized aerial imagery from October 2015 and parcel data from Onslow County updated in September 2016. Multiple residences within one structure, such as duplexes or triplexes, were counted separately. The Villa Capriani and St. Moritz condominium complexes were only counted as one structure. The associated impacts to Island Drive and New River Inlet Road were also quantified by measuring the linear footage of roadway where the impact lines crossed the centerline of the roadway. Table 3 summarizes the results of the analysis, listing the total number of structures impacted and linear feet of road impacted in each Phase under each of the storm simulations.

Table 3. Summary of Structures Impacted

Baseline Stations		5-yr Storm	10-yr Storm	15-yr Storm	20-yr Storm	25-yr Storm
		Number of Structures Impacts				
Phase 2	**968+30 to 1090+00**	23*	56	100	103	108
Phase 3	**900+00 to 968+30**	0	15	23	23	23
Phase 4	**763+00 to 900+00**	29	75**	106	110	112
		Island Drive / New River Inlet Road Impacts (feet)				
Phase 2	**968+30 to 1090+00**	0	400	2,300	4,100	4,800
Phase 3	**900+00 to 968+30**	0	0	200	1,400	1,600
Phase 4	**763+00 to 900+00**	0	0	0	100	200

* - Villa Capriani impacted, ** - St. Moritz impacted

The analysis indicates that a higher number of structures are at risk in Phases 2 and 4 than compared to Phase 3. This is primarily based on Phase 3 having the lowest number of structures. The results showed that the number of structures that would be impacted in Phases 2 and 4 were similar for all scenarios analyzed. However, the results show the 10-year storm impacts 400 linear feet of New River Inlet Road within Phase 2 in the vicinity of Marina Way and Bay Court. This 400 ft. stretch of road was the only one impacted by the 10-year storm. The 15-year storm impacts 2,300 ft. of New River Inlet Road within Phase 2 and 200 linear feet within Phase 3. No impacts were measured to roads in Phase 4 for the 15-year storm.

North End Fill Optimization

The North End Fill Optimization focused on assessing the optimal beach fill along Phase 1 to provide periodic nourishment for the Phase 1 project area during construction of the 3rd event. The 3rd event was assumed to be scheduled for the 2017/2018 dredge window. The original Phase 1 beach fill constructed in 2012/2013 covered the shoreline from station 1163+00 south to station 1090+00. Most of the fill placed north of station 1140+00 was lost within 2 years following placement. Numerical modeling, using the previously calibrated and verified Delft3D model, was used to determine the optimal dimensions of the periodic nourishment fill within the Phase 1 area.

In April 2016, a project to maintain navigation channels in New River and Cedar Bush Cut was completed through a partnership between the State of North Carolina, Onslow County, and the Town of North Topsail Beach. The project deposited material between baseline Stations 1152+00 and 1163+00. As a result of the disposal of the navigation channel maintenance material and by virtue of the DOA permit conditions, subsequent nourishment operations on the north end cannot include the area north of Station 1152+00 until the 2019/2020 environmental dredging window. However, a transition or taper section can extend north of Station 1152+00.

Methodology: The existing calibrated and verified Delft3D model was used to evaluate the performance of 6 options for the north end of North Topsail Beach. Details of the calibration and verification of the model are included in the *New River Inlet Channel Realignment Alternative Channel Modeling Study North Topsail Beach* (CPE-NC, 2016)[1]. Each option included the dredging of the recommended channel alternatives (Pivot Channel) as described in the modeling report (CPE-NC, 2016)[1]. The alternatives were each simulated for a four (4) year period. The options simulated in the model are generally described as:

(1) No beach fill
(2) 50 cy/lf beach fill extending from Station 1130+00 to 1150+00
(3) 100 cy/lf beach fill extending from Station 1130+00 to 1150+00
(4) 150 cy/lf beach fill extending from Station 1130+00 to 1150+00
(5) 50 cy/lf beach fill extending from Station 1090+00 to 1150+00
(6) 50 cy/lf beach fill extending from Station 1090+00 to 1130+00

The No beach fill option, Option 1, was used as a basis of evaluating the effectiveness of the various beach fill options. Options 2, 3, and 4 had beach fills extending from station 1130+00 to 1150+00, which covered the area starting just south of Topsail Reef north to New River Inlet. Beach fill evaluated included fill densities of 50 cy/lf, 100 cy/lf, and 150 cy/lf. The beach fill for Option 5 extended from baseline station 1090+00 north to 1150+00 with the main fill having a fill density of 50 cy/lf. The Option 6 beach fill also started at 1090+00 but terminated at station 1130+00 located just south of Building #8 of Topsail Reef. All the beach fill options had tapers on both the north and south ends.

Model simulations were run for a period of four years with volume changes computed for the area above the -6-foot NAVD88 contour, which provide a reasonable proxy for the behavior of the dry sand beach. A more detailed description of each option and the results obtained from the model simulations are provided below.

Results and Discussions:

Option 1: Option 1 was for comparative purposes and only included the rotated channel and no beach fill. Figure 2 shows the starting bathymetry for Option 1. Volume changes above the -6-foot NAVD88 contour within selected beach segments from station 1080+00 to 1160+00 are provided in Table 4. Plots of the volume changes measured every 200 feet from station 1080+00 to 1160+00 for each year of the model simulation for Option 1 are shown on Figure 3.

[1] Coastal Planning & Engineering of North Carolina, Inc (CPE-NC), 2016. New River Inlet Channel Realignment Alternative Channel Modeling Study, North Topsail Beach. Report Prepared for the Town of North Topsail Beach, NC. 84 pgs.

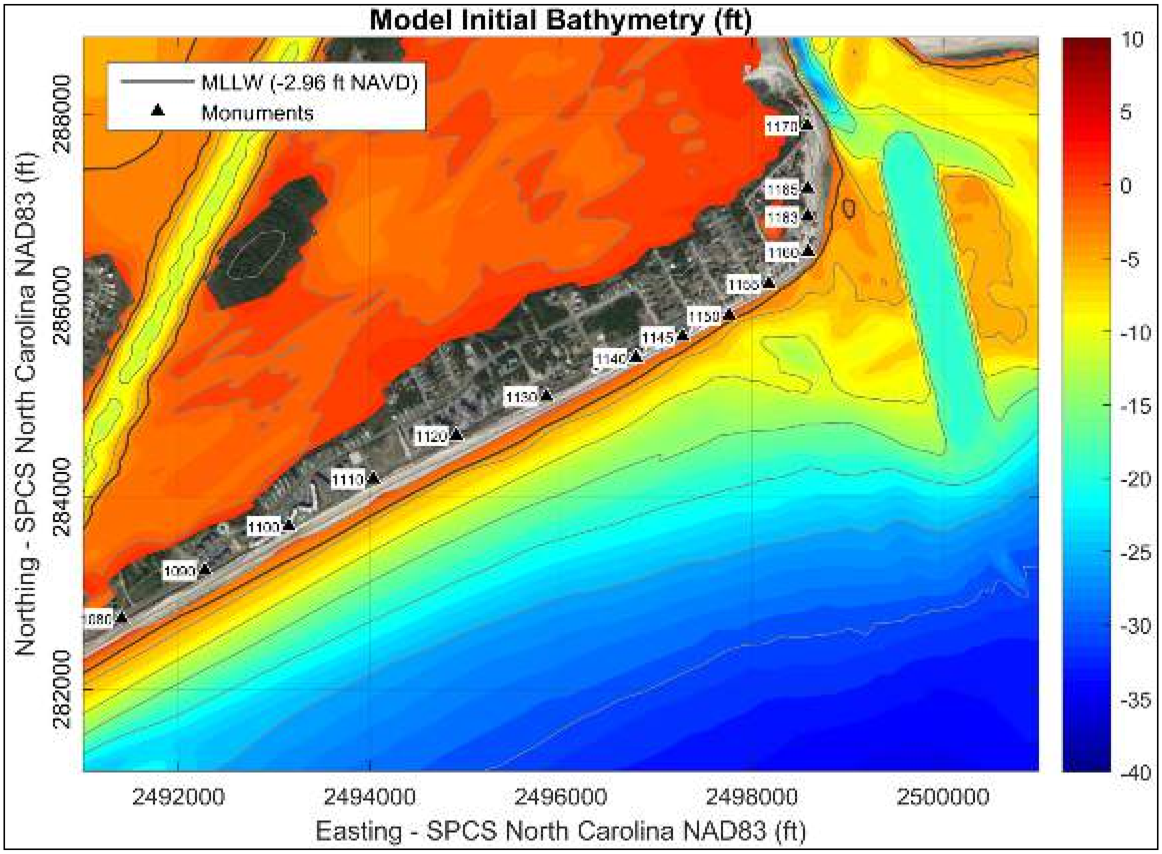

Figure 2. Map showing the starting bathymetry for Option 1. Depths are referenced to NAVD88.

Table 4. Option 1 Volume Changes: No Beach Fill – Rotated Channel

Stations	Year 0 initial fill vol. (cy)	Year 1	Year 2	Year 3	Year 4
1080-1120	0	-44,441	-102,176	-130,723	-149,139
1120-1130	0	-31,542	-48,413	-61,576	-66,176
1130-1140	0	-24,551	-30,145	-32,136	-34,972
1140-1150	0	9,443	12,533	22,951	33,733
1150-1160	0	6,521	34,034	52,704	54,578

Option 1 Volume Changes. The rotated channel induced positive volume changes over the 4-year simulation along the shoreline from about station 1140+00, which is between Buildings #5 and #6 of Topsail Reef, north to station 1158+00, located south of New River Inlet (Figure 3). For the area between stations 1130+00 and 1140+00, the shoreline lost approximately 24,600 cy during the first year of the simulation. Volume losses from this area moderated over the final 3 years of the simulation. Farther south, the shoreline between stations 1120+00 and 1130+00 experience rather persistent volume losses during the entire 4-year simulation. South of station 1120+00 to station 1080+00, the area lost a total of approximately 149,000 cubic yards with the maximum erosion of about 78 cy/lf over the 4-year simulation occurring near station 1122+00, which is near the St. Regis Resort.

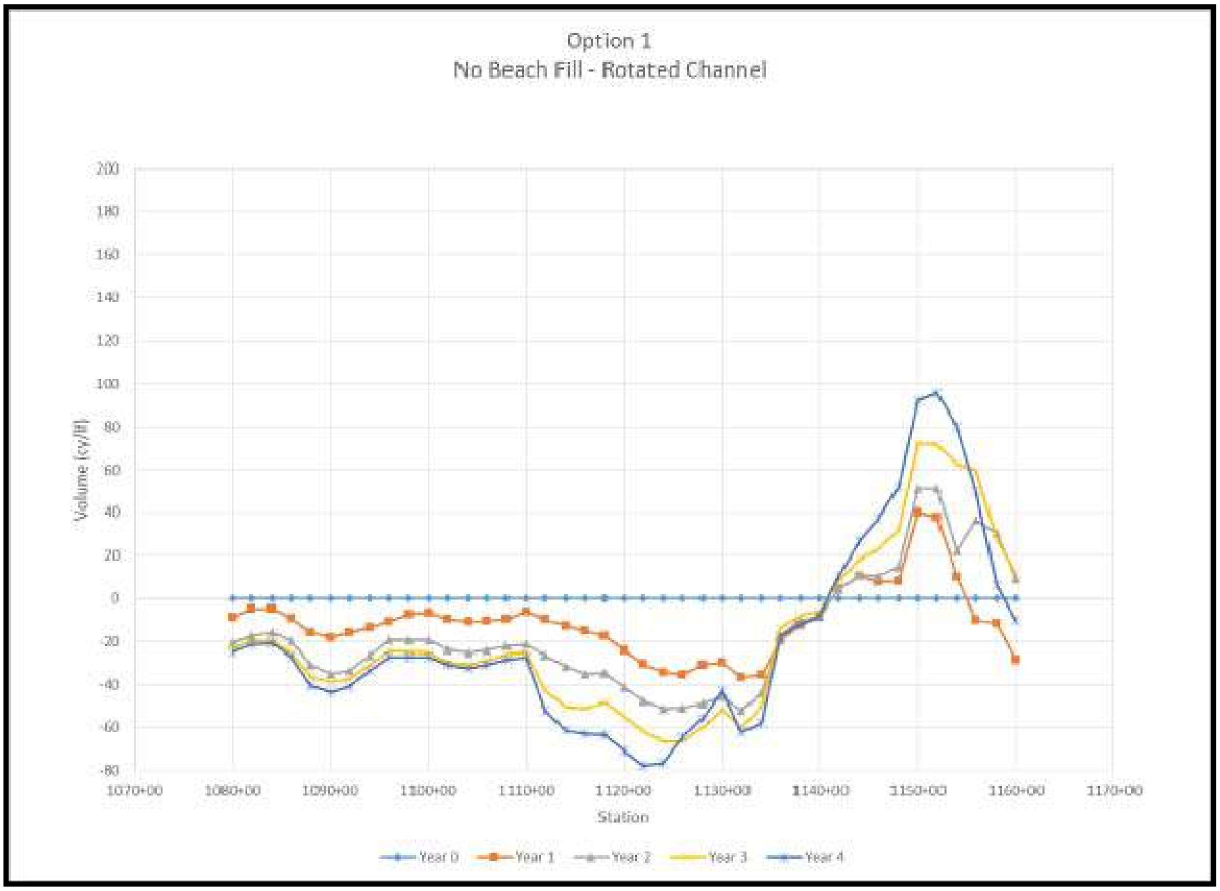

Figure 3. Simulated volume changes along North Topsail Beach – Option 1: Rotated Channel – No Beach Fill.

The volume losses that occurred from station 1130+00 to 1140+00 during the simulation would have removed most of the sand from in front of the sandbag revetment that protects the Topsail Reef Condominiums. In this regard, the Topsail Reef sandbag revetment begins near baseline station 1135+00 and extends to about baseline station 1148+00. With the completion of the Town's sandbag revetment in 2015, the sandbag revetment now fronts the entire shoreline from station 1135+00 to the south shoulder of New River Inlet (about station 1163+00).

The Delft3D model simulates the sandbag revetment as a non-erodible surface, i.e., the model does not allow material to be lost from behind the sandbag revetment. Therefore, once material is removed from in front of the revetment, the source of sand that would normally be transported to the south out of the area is lost. The loss of this sand source created a deficit in the supply of sand to the south which contributed to the higher rates of volume loss observed between 1120+00 and 1130+00.

Option 2:
Includes a beach fill with approximately 50 cy/lf between 1130+00 and 1150+00; a south taper between 1124+00 and 1130+00; and a north taper between 1150+00 and 1156+00. Figure 4 shows the starting bathymetry for Option 2. Even though the south taper did not initially extend beyond station 1124+00 and the north taper did not initially extend beyond station 1156+00, simulated volume changes between 1120+00 and 1130+00 as well as 1150+00 and 1160+00 were used to represent volume changes in the south and north taper sections of the fill, respectively. Fill volumes for Option 2 are as follows:

Fill Volumes for Option 2	
Main Fill	109,776 cy
South Taper	16,026 cy
North Taper	15,855 cy
Total	**141,656 cy**

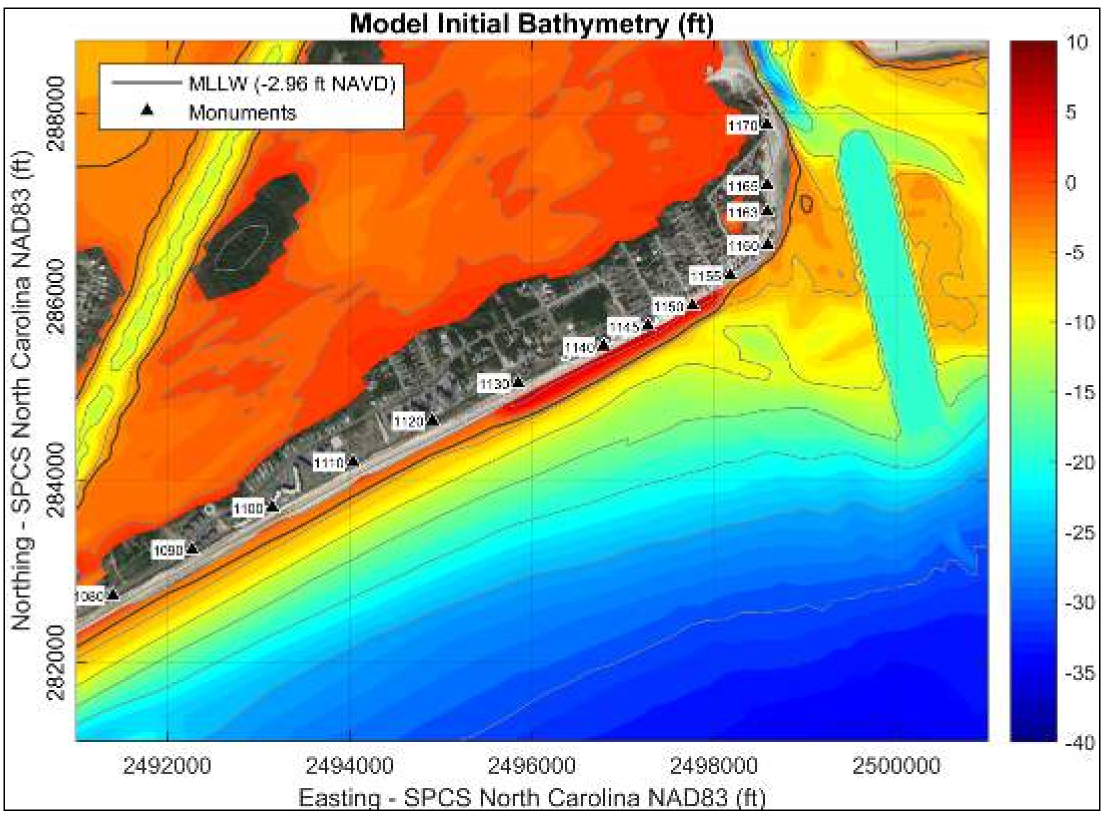

Figure 4. Map showing the starting bathymetry for Option 2. Depths are referenced to NAVD88.

Error! Reference source not found. provides a summary of volume of fill remaining in the beach fill area for 1,000-foot segments from station 1120+00 to 1160+00, which includes the two taper sections. Volume changes in the area from station 1080+00 to 1020+00, which did not receive any beach fill, are also provided in Table 5. Plots of the volume changes measured every 200 feet from station 1080+00 to 1160+00 for each year of the model simulation for Option 2 are shown on Figure 5.

Table 5. Option 2 Beach Fill Performance

Stations	Year 0 initial fill vol. (cy)	Fill Volume Remaining after Year (cy)			
		Year 1	Year 2	Year 3	Year 4
1080-1120(1)	*0*	*-19,266*	*-67,844*	*-95,940*	*-112,887*
1120-1130	6,026	-6,757	-25,299	-40,149	-48,385
1130-1140	65,039	-730	-15,185	-18,645	-24,755
1140-1150	54,736	25,170	23,707	31,625	44,020
1150-1160	15,855	23,762	43,847	60,727	63,231
Total(2)	**141,656**	**41,445**	**27,070**	**33,558**	**34,111**

(1) These are volume changes over the 4-year simulation since no fill was placed in this area.
(2) Totals are only for the volume of beach fill remaining between station 1120+00 and 1160+00, which includes the taper sections.

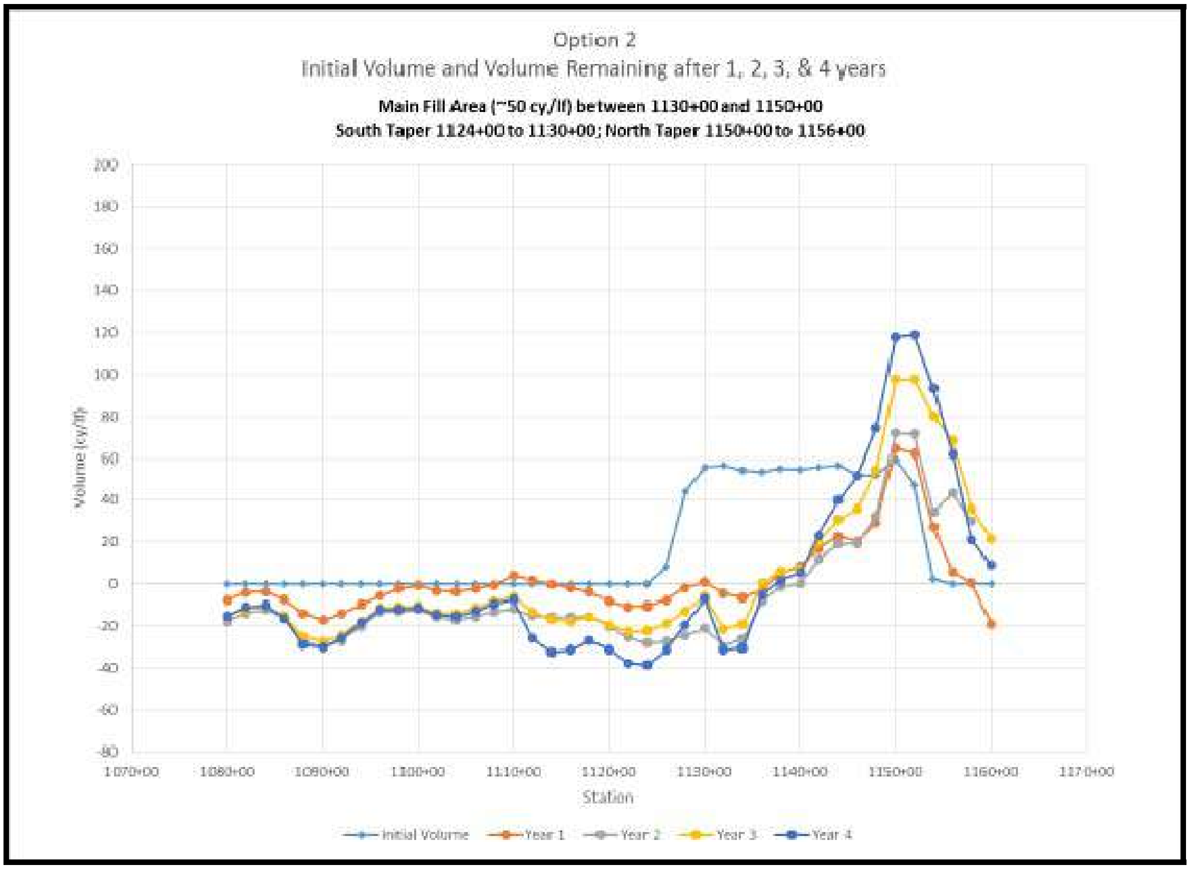

Figure 5. Simulated volume changes along North Topsail Beach for Beach Fill Option 2.

Option 2 Volume Changes. The south taper (1120+00 to 1130+00) and the area south of 1120+00 to station 1080+00 lost material during the entire 4-year simulation. Losses in the south taper moderated during the 4th year of the simulation. The main fill between stations 1130+00 and 1140+00 lost 100% of the fill volume during the 1st year and continued to experience erosion into the pre-nourished beach during the remainder of the simulation. The main fill between station 1140+00 and 1150+00 retained about 43.3% of the initial fill volume at the end of the 2nd year of the simulation and then began to gain material over the last two years of the simulation. At the end of the 4-year simulation, the volume of material in this section was equal to about 80.4% of the initial fill volume. The north taper area accumulated sediment during the entire 4-year simulation. The response in the north taper section was similar to Option 1; however, the volume accumulation under Option 2 was about 8,600 cy more than Option 1. While the accumulation in the north taper is primarily attributed to the impacts of the rotated channel, the higher volume of accumulation under Option 2 was probably due to the fill material spreading north.

For the area that did not receive beach fill under Option 2 (1080+00 to 1120+00), volume losses during the first two years of the simulation were about 34,300 cy less than Option 1. This may be indicative of fill material spreading south out of the placement area. After year 2 of the simulation for Option 2, the area behaved in a similar manner to Option 1, i.e., the southward spreading of the fill material had apparently ceased.

Option 3:
Includes a beach fill with approximately 100 cy/lf between 1130+00 and 1150+00; a south taper between 1124+00 and 1130+00; and a north taper between 1150+00 and 1156+00. Figure 6 shows the starting bathymetry for Option 3. Even though the south taper did not initially extend beyond station 1124+00 and the north taper did not initially extend beyond station 1156+00, simulated volume changes between 1120+00 and 130+00 as well as 1150+00 and 1160+00 were used to represent volume changes in the south and north taper sections of the fill, respectively. Fill volumes for Option 3 are as follows:

Fill Volumes for Option 3	
Main Fill	210,265 cy
South Taper	34,038 cy
North Taper	31,223 cy
Total	**275,526 cy**

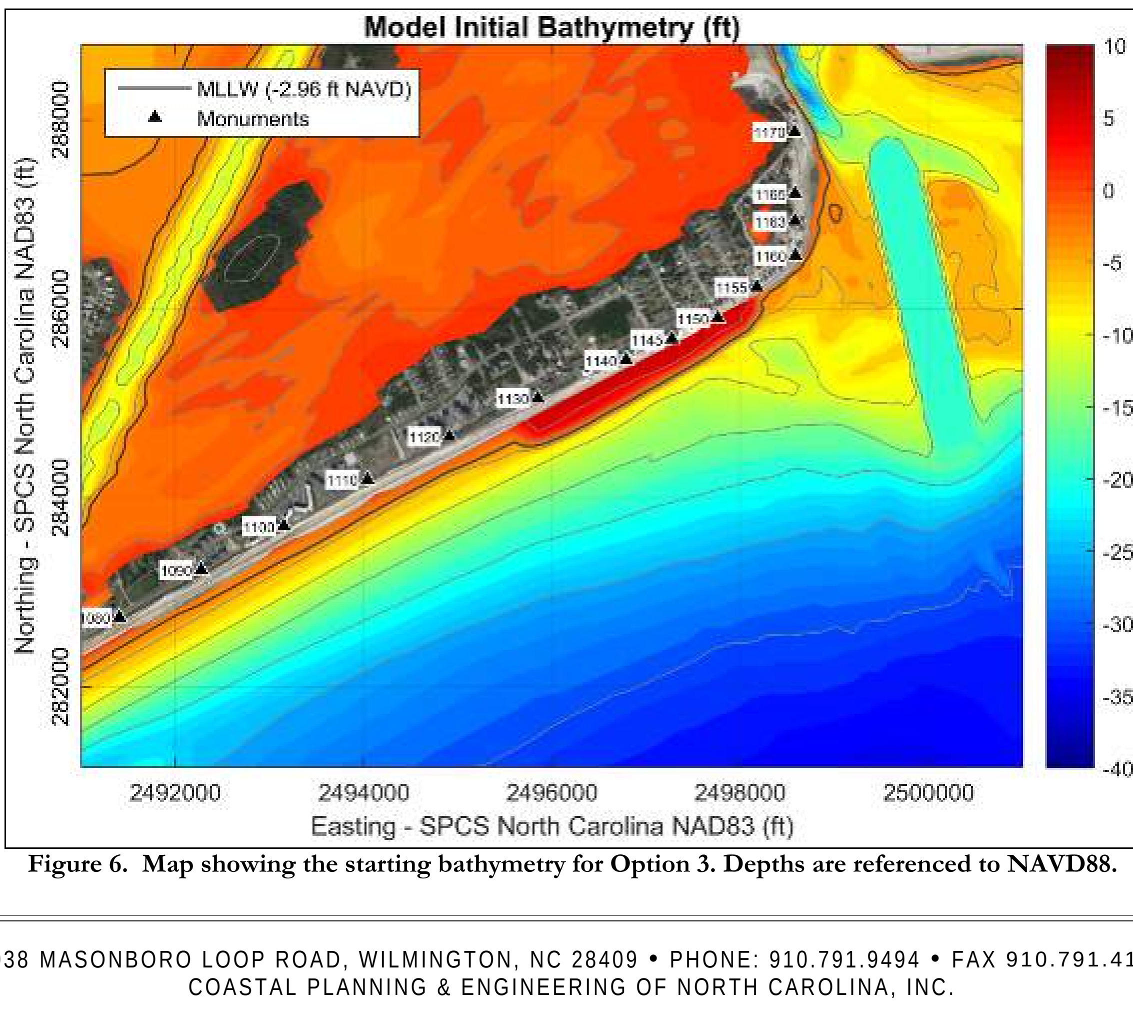

Figure 6. Map showing the starting bathymetry for Option 3. Depths are referenced to NAVD88.

Table 6 provides a summary of volume of fill remaining in the beach fill area for 1,000-foot segments from station 1120+00 to 1160+00, which includes the two taper sections, for Option 3. Volume changes in the area from station 1080+00 to 1020+00, which did not receive any beach fill, are also provided in Table 6.

Table 6. Option 3 Beach Fill Performance

Stations	Year 0 initial fill vol. (cy)	Fill Volume Remaining after Year (cy)			
		Year 1	Year 2	Year 3	Year 4
1080-1120(1)	*0*	*3,523*	*-34,020*	*-59,682*	*-75,135*
1120-1130	34,038	14,527	-3,754	-17,887	-29,193
1130-1140	105,388	29,021	2,607	-6,689	-13,238
1140-1150	104,877	44,540	33,022	38,648	50,185
1150-1160	31,223	38,044	52,355	68,489	71,794
Totals(2)	**126,132**	**84,230**	**82,561**	**79,547**	**126,132**

(1) These are volume changes over the 4-year simulation since no fill was placed in this area.
(2) Totals are only for the volume of beach fill remaining between station 1120+00 and 1160+00, which includes the taper sections.

Option 3 Volume Changes. As was the case with Option 2, the south taper (1120+00 to 1130+00) lost material during the entire 4-year simulation (Table 6). The area from 1080+00 to 1120+00 that did not receive beach fill (1080+00-1120+00) gained material during the first year of the simulation but then began to lose material at a steady rate over the remaining three years of the simulation. At the end of the 4-year simulation, volume losses in the area from 1080+00 to 1120+00 were about 37,800 cy less than Option 2. This was due to a higher degree of spreading of the material from the larger fill associated with Option 3.

The main fill between stations 1130+00 and 1140+00 lost 100% of the fill volume by the end of the 3rd year of the simulation and continued to experience erosion into the pre-nourished beach during the 4th year of the simulation. The main fill between station 1140+00 and 1150+00 retained about 31.5% of the initial fill volume at the end of the 2nd year of the simulation and then began to gain material over the last two years of the simulation. At the end of the 4-year simulation, the volume of material in this section was equal to about 47.9% of the initial fill volume. The north taper area accumulated sediment during the entire 4-year simulation. The response in the north taper section was similar to Option 1; however, the volume accumulation under Option 3 was about 17,200 cy more than Option 1. While the accumulation in the north taper is primarily attributed to the impacts of the rotated channel, the higher volume of accumulation under Option 3 was probably due to the fill material spreading north.

Volume changes along the north end of North Topsail Beach from station 1080+00 to 1160+00 for Option 3 are shown on Figure 7.

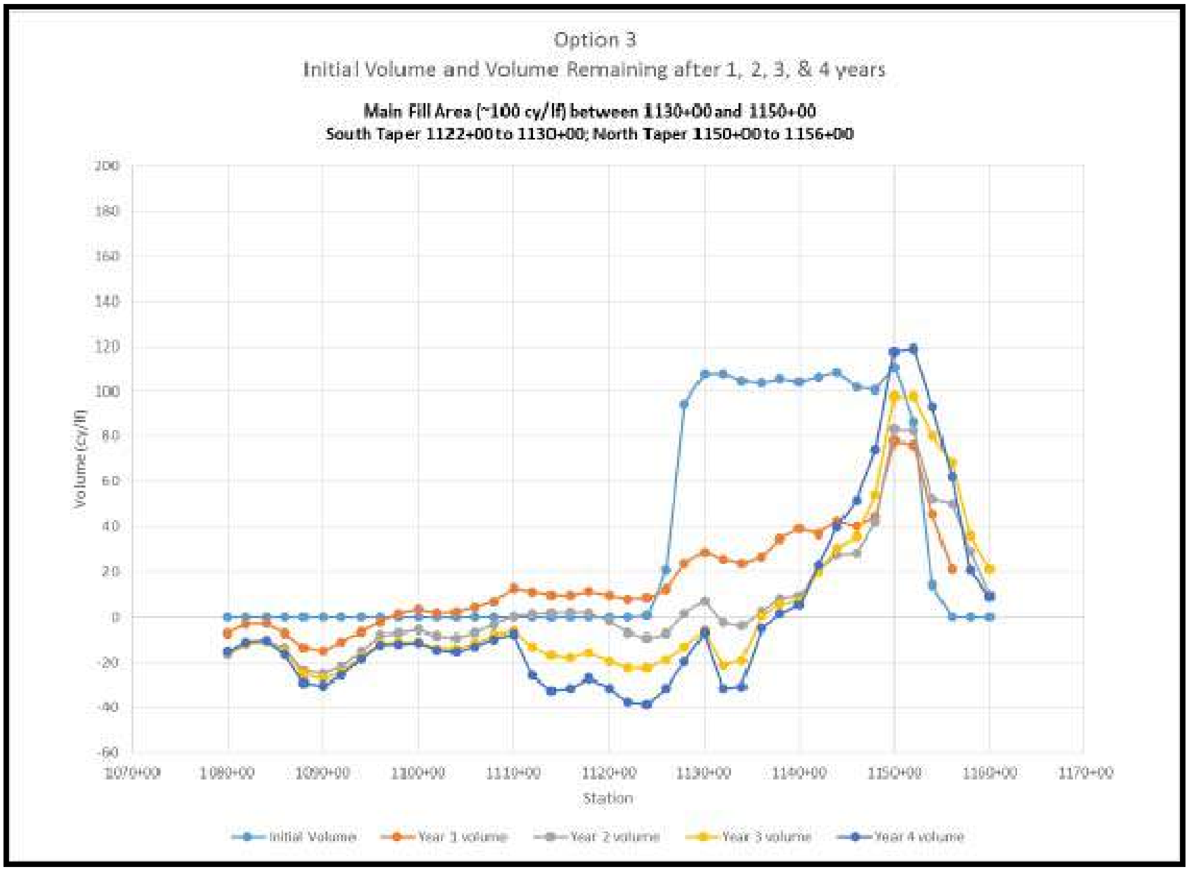

Figure 7. Simulated volume changes along North Topsail Beach for Beach Fill Option 3.

Option 4: Includes a beach fill with approximately 150 cy/lf between 1130+00 and 1150+00; a south taper between 1124+00 and 1130+00; and a north taper between 1150+00 and 1156+00. Figure 8 shows the starting bathymetry for Option 4. Even though the south taper did not initially extend beyond station 1124+00 and the north taper did not initially extend beyond station 1156+00, simulated volume changes between 1120+00 and 1130+00 as well as 1150+00 and 1160+00 were used to represent volume changes in the south and north taper sections of the fill, respectively. Fill volumes for Option 4 are as follows:

Fill Volumes for Option 4	
Main Fill	310,743 cy
South Taper	62,702 cy
North Taper	51,647 cy
Total	**425,093 cy**

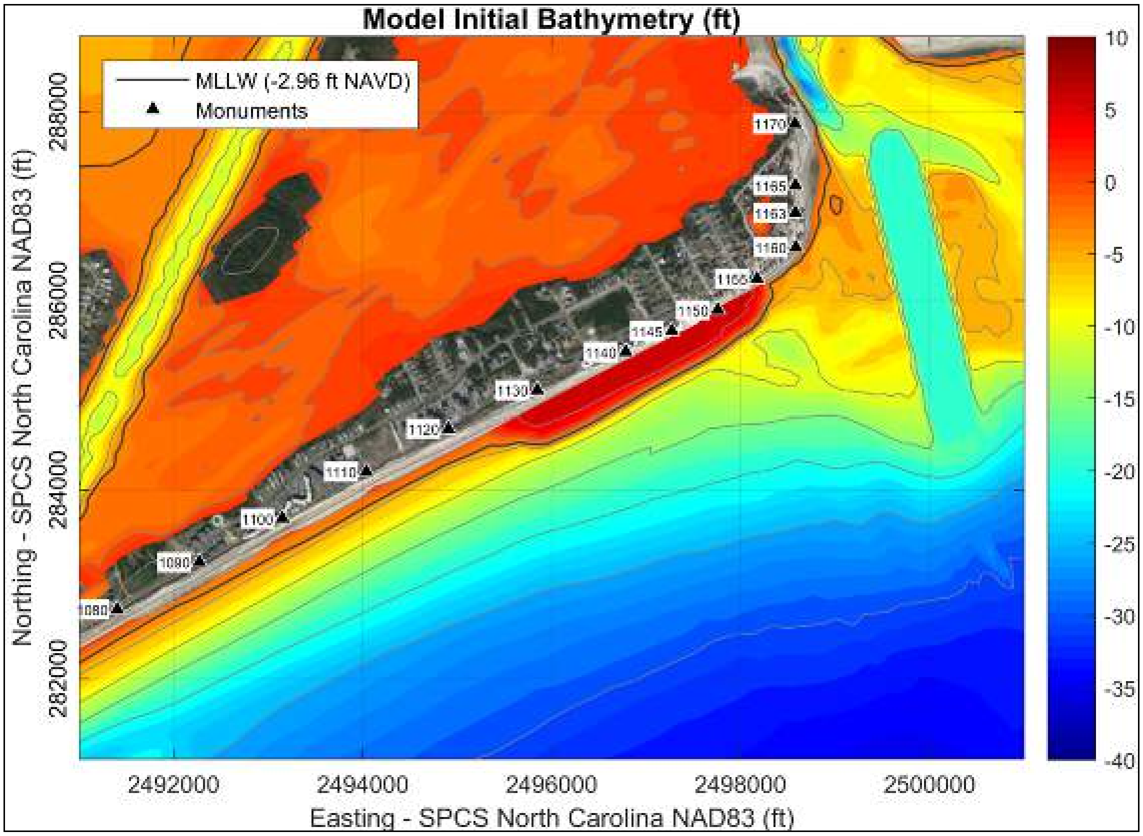

Figure 8. Map showing the starting bathymetry for Option 4. Depths are referenced to NAVD88.

Table 7 provides a summary of volume of fill remaining in the beach fill area for 1,000-foot segments from station 1120+00 to 1160+00, which includes the two taper sections, for Option 3. Volume changes in the area from station 1080+00 to 1020+00, which did not receive any beach fill, are also provided in Table 7.

Table 7. Option 4 Beach Fill Performance

Stations	Year 0 initial fill vol. (cy)	Fill Volume Remaining after Year (cy)			
		Year 1	Year 2	Year 3	Year 4
1080-1120(1)	*0*	*25,812*	*611*	*-21,557*	*-37,223*
1120-1130	62,702	39,530	20,957	6,536	-5,421
1130-1140	154,963	59,929	27,373	11,940	1,195
1140-1150	155,780	72,656	46,282	47,494	60,280
1150-1160	51,647	52,845	63,107	76,735	79,517
Totals(2)	**425,093**	**224,960**	**157,719**	**142,705**	**135,572**

(1) These are volume changes over the 4-year simulation since no fill was placed in this area.
(2) Totals are only for the volume of beach fill remaining between station 1120+00 and 1160+00, which includes the taper sections.

Option 4 Volume Changes. The relatively large volume of fill associated with Option 4 resulted in the retention of some of the initial fill material in each of the four 1000-foot segments during the first 3 years of the simulation. Volume changes along the north end of North Topsail Beach from station 1080+00 to 1160+00 for Option 4 are shown on Figure 9. By the end of Year 4, the south transition had lost over 100% of the initial fill volume while the main fill section between 1130+00 and 1140+00 retained less than 1% of the initial fill volume. The section of the main fill between 1140+00 and 1150+00 did retained about 30% of the initial fill volume at the end of Year 2 of the simulation but then began to accumulate material over the last two years resulting in a total retention of about 39% of the original fill volume. Of the 425,100 cy initially placed, the beach fill area lost almost 290,000 cy or 69.4% of the initial fill volume.

The significantly larger fill volume compared to Options 2 and 3 resulted in substantial spreading of the material to the south and north out of the main fill area. In the area from 1080+00 to 1120+00 that did not received beach fill, volume losses over the 4-year simulation totaled only 37,200 cy compared to losses under Option 1 that amounted to 149,100 cy.

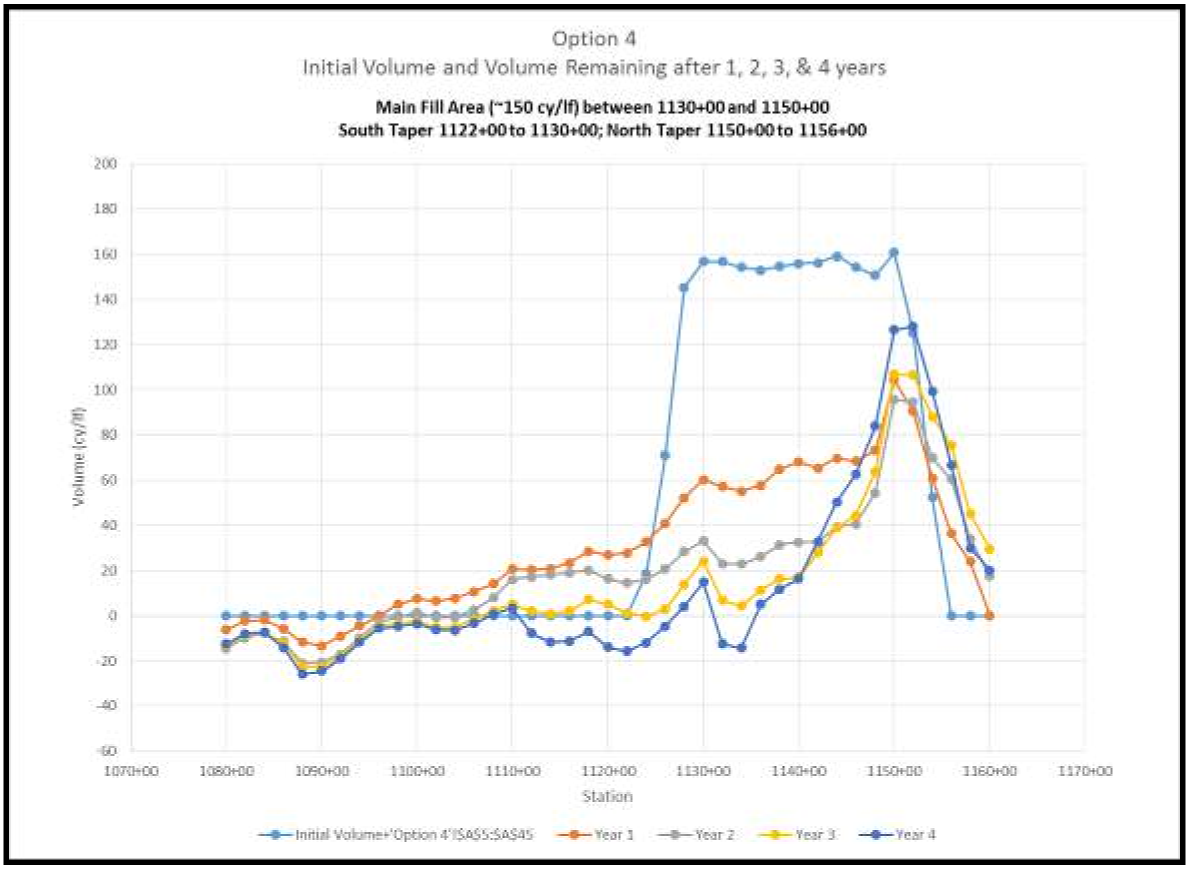

Figure 9. Simulated volume changes along North Topsail Beach for Beach Fill Option 4.

Option 5: Includes a beach fill with approximately 50 cy/lf between 1090+00 and 1150+00; a south taper between 1084+00 and 1090+00; and a north taper between 1150+00 and 1156+00. Figure 10 shows the starting bathymetry for Option 5. As with the other options, the north taper is represented by changes between stations 1150+00 and 1160+00. The south taper is represented by changes between 1080+00 and 1090+00.

Fill Volumes for Option 5	
Main Fill	322,206 cy
South Taper	12,193cy
North Taper	15,855 cy
Total	**360,254 cy**

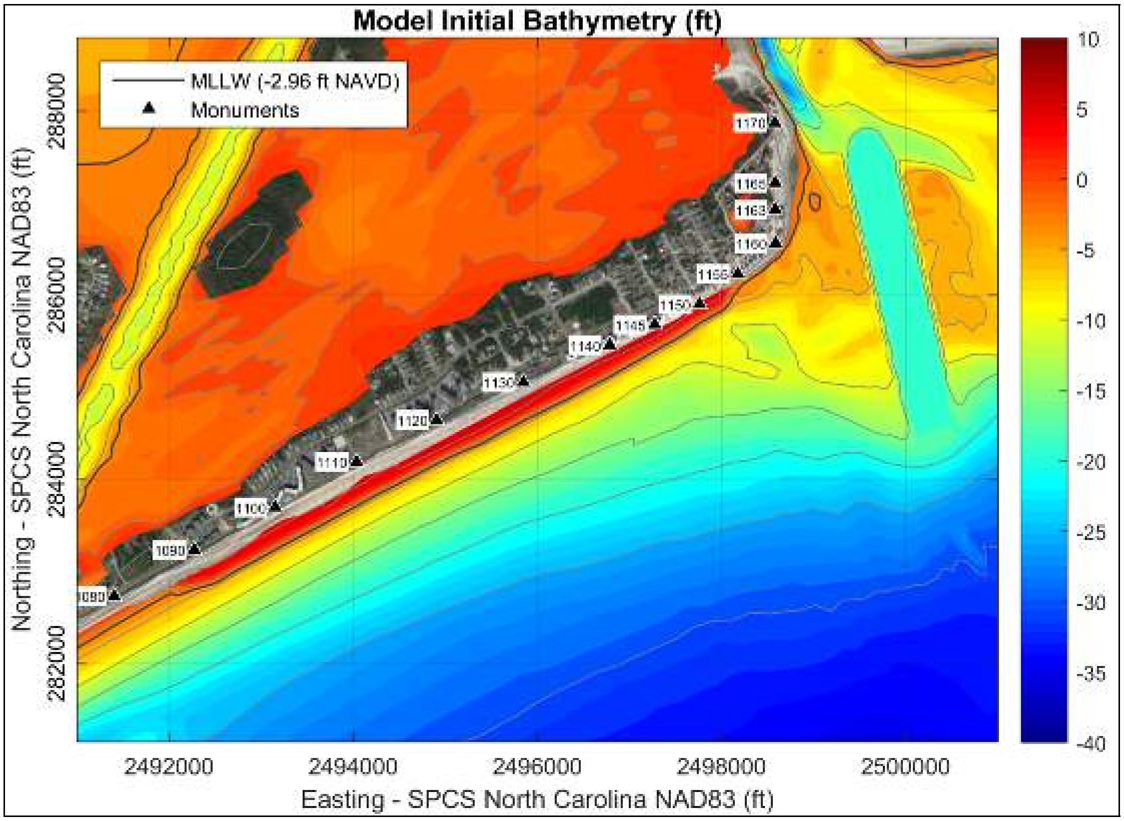

Figure 10. Map showing the starting bathymetry for Option 5. Depths are referenced to NAVD88.

Table 8 provides a summary of volume of fill remaining in the beach fill area for 1,000-foot segments from station 1120+00 to 1160+00, which includes the two taper sections, for Option 3. Volume changes in the area from station 1080+00 to 1020+00, which did not receive any beach fill, are also provided in Table 8.

Table 8. Option 5 Beach Fill Performance

Stations	Year 0 initial fill vol. (cy)	Fill Volume Remaining after Year (cy)			
		Year 1	Year 2	Year 3	Year 4
1080-1090	12,193	14,867	488	-3,688	-6,020
1090-1120	167,357	94,512	41,037	-6,127	-28,056
1120-1130	54,855	21,711	-159	-10,944	-20,915
1130-1140	55,259	16,684	-40	-5,133	-10,221
1140-1150	54,736	31,133	29,898	38,726	50,129
1150-1160	15,855	27,653	48,765	65,436	68,999
Totals	**360,254**	**206,560**	**119,948**	**78,272**	**53,916**

Option 5 Volume Changes. The distribution of the main fill material over the longer area from 1090+00 to 1150+00 compared to the short beach fill associated with Options 2, 3, and 4 improved the overall performance of the beach fill within the same 1,000-foot segments used in the evaluation of the shorter beach fill under Option 2. Option 2 also included a main fill density of 50 cy/lf. For example, the main fill area between 1130+00 and 1140+00 under Option 2 lost all of the fill volume within the first year while under Option 5, 100% loss of the fill did not occur until the end of Year 2 of the simulation. For the main fill area between 1140+00 and 1150+00, the volume of fill

retained during the first two years under Option 5 was about 10% greater than Option 2. In addition, volume accumulations between 1140+00 and 1150+00 during Years 3 and 4 was again about 10% greater under Option 5 compared to Option 2.

The fill segment between 1090+00 and 1120+00 retained 24.5% of the initial fill volume through Year 2 with essentially all the fill material lost by the end of Year 3.

Volume changes along the north end of North Topsail Beach from station 1080+00 to 1160+00 for Option 5 are shown on Figure 11.

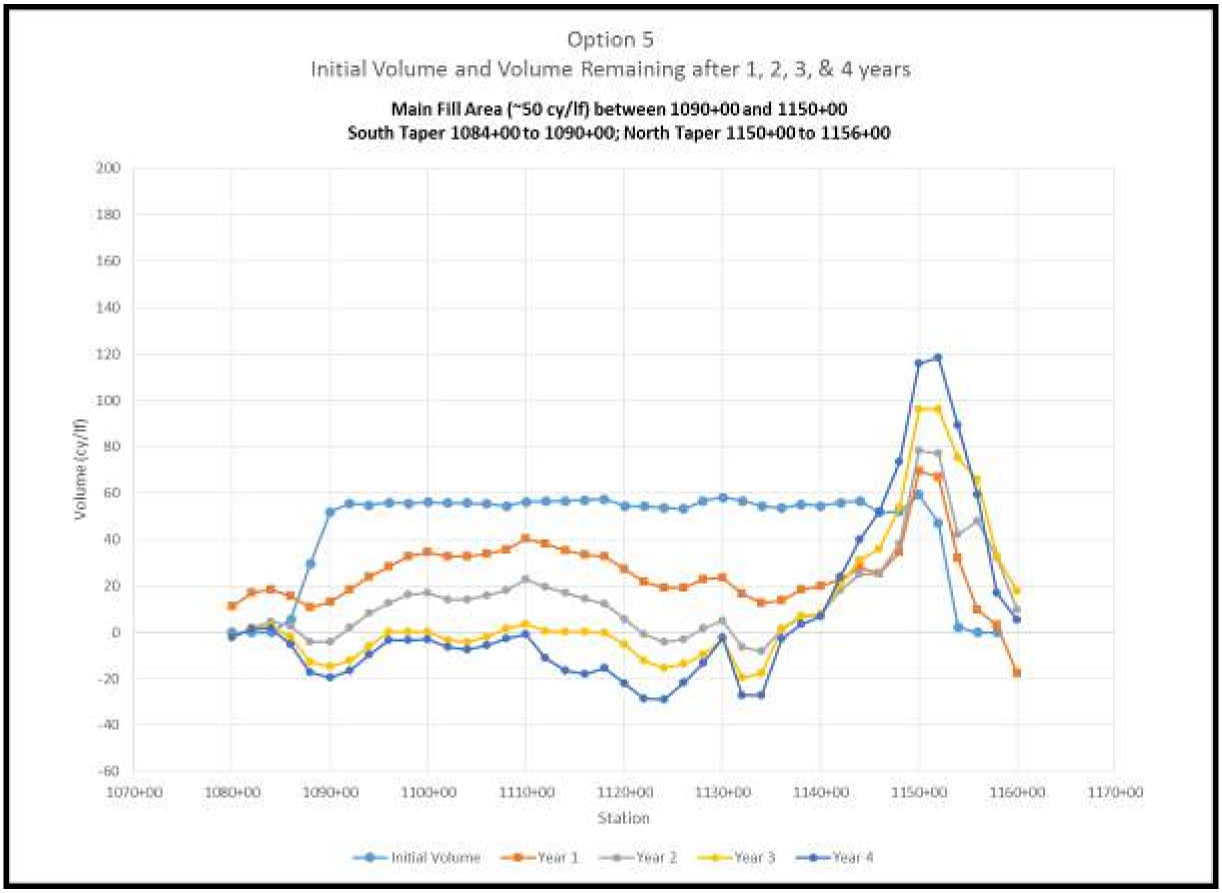

Figure 11. Simulated volume changes along North Topsail Beach for Beach Fill Option 5.

Option 6. Includes a beach fill with approximately 50 cy/lf between 1090+00 and 1130+00; a south taper between 1084+00 & 1090+00; and a north taper between 1130+00 and 1136+00. Figure 12 shows the starting bathymetry for Option 6. The southern taper is represented by changes between 1080+00 and 1090+00 while changes between 1130+00 and 1140+00 were used to represent changes in the northern taper.

Fill Volumes for Option 6	
Main Fill	221,322 cy
South Taper	12,193cy
North Taper	8,660 cy
Total	**242,175 cy**

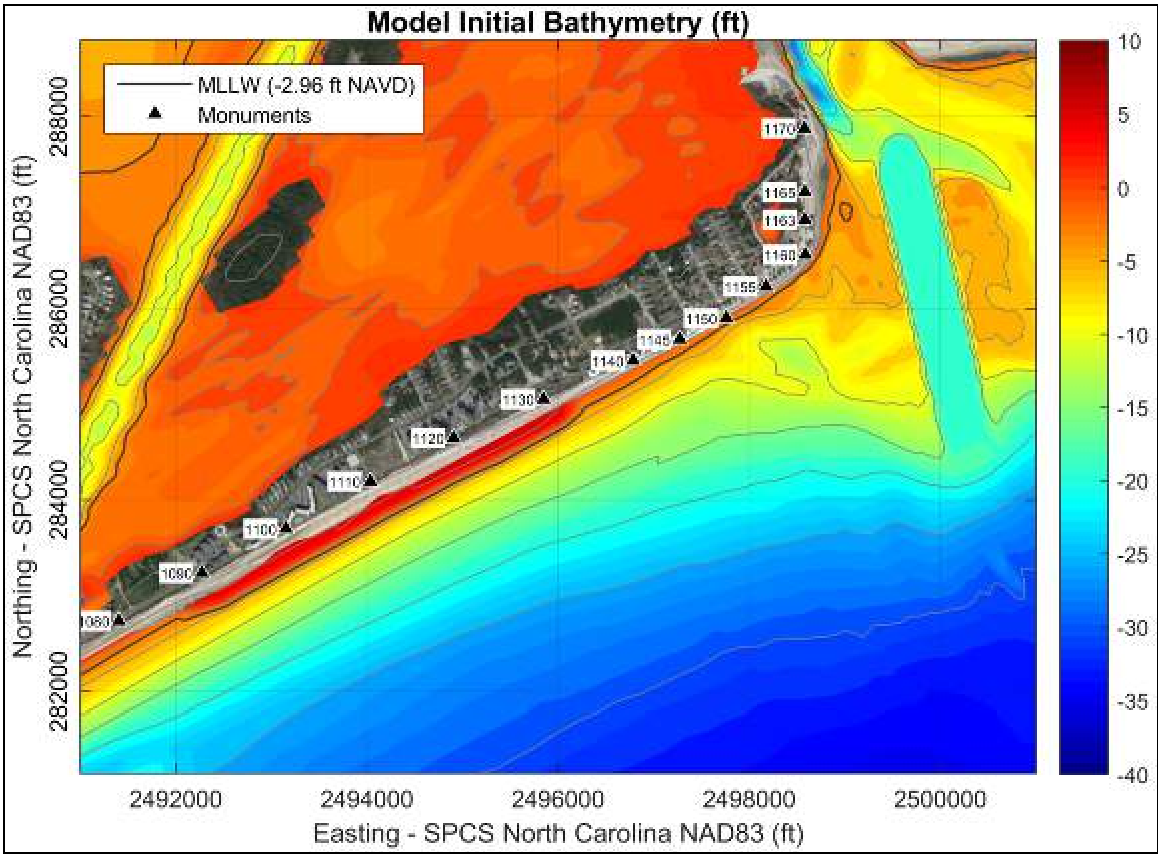

Figure 12. Map showing the starting bathymetry for Option 5. Depths are referenced to NAVD88.

Table 9 provides a summary of volume of fill remaining in the beach fill area for 1,000-foot segments from station 1120+00 to 1160+00, which includes the two taper sections, for Option 3. Volume changes in the area from station 1080+00 to 1020+00, which did not receive any beach fill, are also provided in Table 9.

Table 9. Option 6 Beach Fill Performance

Stations	Year 0 initial fill vol. (cy)	Fill Volume Remaining after Year (cy)			
		Year 1	Year 2	Year 3	Year 4
1080-1090	12,193	14,061	538	-4,961	-7,044
1090-1120	167,357	77,643	18,681	-28,835	-47,414
1120-1130	53,965	3,863	-16,870	-26,066	-35,853
1130-1140	8,660	-4,345	-13,196	-13,493	-18,092
1140-1150(1)	0	17,416	20,406	30,699	42,190
1150-1160(1)	0	12,313	40,283	57,788	61,956
Totals(2)	**242,175**	**91,222**	**-10,848**	**-73,355**	**-108,402**

(1) These are volume changes over the 4-year simulation since no fill was placed in this area.
(2) Totals are only for the volume of beach fill remaining between station 1080+00 and 1140+00, which includes the taper sections.

Option 6 Volume Changes. Option 6 was run to determine if there is any benefit associated with not placing beach fill in front of Topsail Reef. The results for Option 6 are directly comparable to Option 5 in the areas south of station 1130+00 since both options included the same beach fill template.

The south taper performed in a similar manner for both Option 5 with Option 5 losing 18,200 cy over the 4-year simulation and Option 6 losing 19,200 cy. This small difference is within the accuracy of the model. The major difference in fill performance between Options 5 and 6 occurred between 1090+00 and 1020+00 with losses under Option 5 and Option 6 totaling about 195,400 cy and 214,800 cy over the 4-year simulation, respectively. The improved performance of the beach fill from 1080+00 to 1030+00 under Option 5 compared to Option 6 indicates there is some benefit for extending the fill beyond 1130+00 as this apparently reduces volume losses from the north end of the fill associated with dispersion of the fill to the north by wave action.

As was the case with all the beach fill options evaluated as well as Option 1, the no beach fill option, the model indicated material could have accumulated on the profile north of 1140+00 to about 1158+00 due to the changes in sediment transport patterns associated with the rotated bar channel. However, this potential change in behavior should be verified before any beach nourishment options are considered for the area.

Volume changes along the north end of North Topsail Beach from station 1080+00 to 1160+00 for Option 6 are shown on Figure 13.

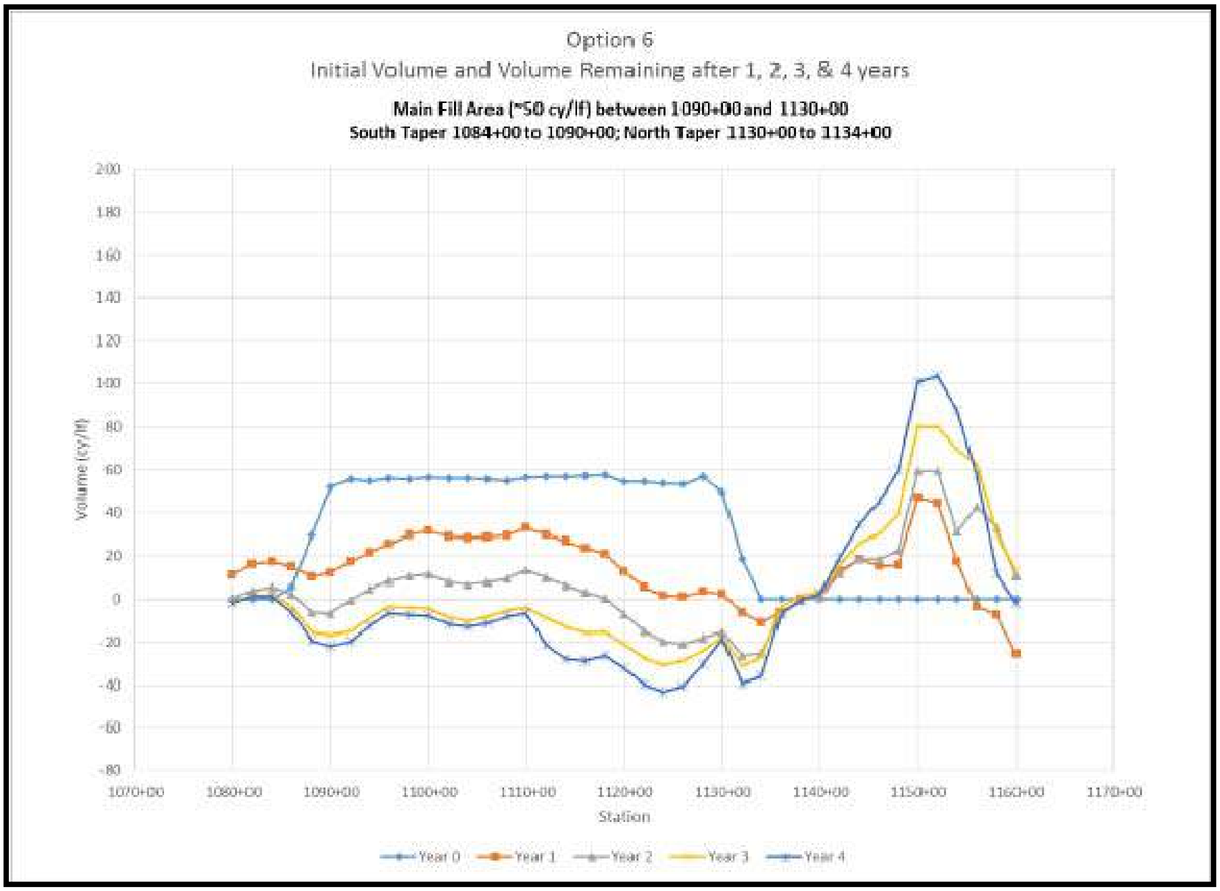

Figure 13. Simulated volume changes along North Topsail Beach for Beach Fill Option 6.

Conclusions: According to the simulations, the beach fill options that only placed material between 1130+00 and 1150+00 (Options 2, 3, and 4) performed poorly. In the case of Options 2 and 3, most of the fill south of station 1140+00 was lost in two years, whereas Option 4 lost essentially all the fill south of 1140+00 in three years.

Option 5, which included a main beach fill with 50 cy/lf from station 1090+00 to 1150+00 provided some degree of long-term erosion mitigation, but by Year 3 of the simulation, the beach fill south of 1140+00 was lost. Terminating the 50 cy/lf fill at station 1130+00 (Option 6) south of Topsail Reef resulted in loss of the main fill south of station 1130+00 in about 2 years.

Based on the results of these model simulations, the most optimal of the beach fill scenarios for the third event include a taper on the north end between station 1140+00 and 1150+00, a main fill consisting of 50 cy/lf south to station 1090+00, and a south taper between 1090+00 and 1080+00. Based on the monitoring of the Phase 1 beach fill, which indicated significant southward spreading of the material, placement of material between stations 1080+00 to 1060+00 should be limited to enhancing the size of the dune in this area to a crest width of at least 25 feet at an elevation of +14.0 feet NAVD88. The beach fill south of station 1060+00 should include a 1,000-foot taper, a main fill with a fill density of 50 cy/lf extending as far south as the volume of material to be removed in repositioning the New River Inlet bar channel will allow.

All the Options evaluated using the Delft3D model indicated some sediment accumulation could occur north of 1140+00. This accumulation was apparently due to the impacts of the rotated channel. However, until this tendency is verified following the next channel relocation project, no beach fill is recommended north of station 1150+00. Protection of the area north of Topsail Reef should rely on the sandbag revetment which will require the Town to implement a program to provide routine maintenance of the revetment until conditions improve or until the Town can implement a terminal groin project (presently in the planning stage).

Recommended Beach Fill Design

The recommended Beach Fill for the proposed ocean bar channel realignment project proposed to be constructed in winter 2017/2018 was developed based on the following observations, conclusions and assumptions:

- The recommendation assumes a hardened structure (terminal groin or Jetties) will not be constructed on North Topsail Beach for at least 3 years.
- The recommendation assumes the realignment of the ocean bar channel will produce ~ 625,000 cy of sand.
- The vulnerability analysis conducted for Phases 2, 3 and 4 concluded that Phase 2 is the most vulnerable phase with respect to the number of structures and linear feet of road.
- The current Dept. of Army permit will not allow a full fill section to be constructed north of Station 1152+00 until 2020; however, sand can be placed north of station 1152+00 as part of a taper.
- Observations of sand placed during navigation maintenance operations conducted by the USACE and the Town of North Topsail/Onslow Count, as well as the initial channel realignment project suggest sand placed on the extreme north end of North Topsail Beach will be short lived under the existing ebb shoal configuration.
- The Town of North Topsail Beach wishes to balance the following priorities equally:
  - Avoid losing oceanfront structures and road on the north end of the Town;
  - Maintain the "Beach Fill Performance" section of the Phase 1 project between stations 1090+00 and 1130+00; and
  - Construct as much of the remaining original project as possible in the most vulnerable phase.
- The recommendation assumes the sand bag revetments along the north end will be maintained by the Town as needed to continue to provide erosion protection.
- The recommendation assumes the Town will render the geotextile containment tubes ineffective by May 1, 2017.

The recommended Beach Fill for the proposed ocean bar channel realignment project proposed to be constructed in winter 2017/2018 includes the placement of sand between station 1154+00 (Port Drive) to station 1012+00 (approximate south boundary of 1148 New River Inlet Rd.) (Figure 14).

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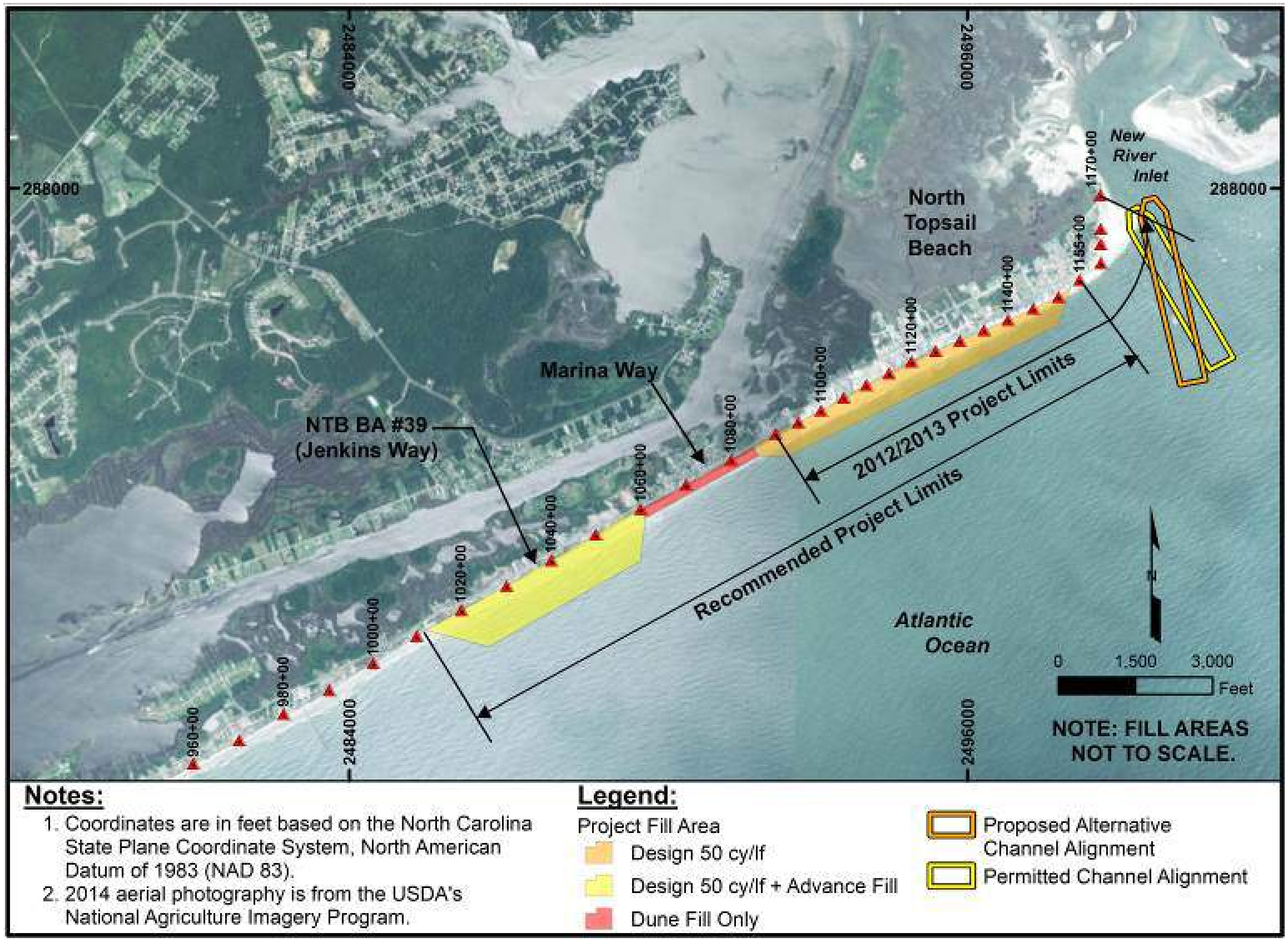

Figure 14. Recommended Beach Fill for the proposed ocean bar channel realignment project.

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The beach fill would include the following components:

- A northern taper from station 1154+00 (Port Drive) to station 1148+00 (northern boundary of the Topsail Reefs) – Approximately 16,000 cy;
- A 50 cy/lf beach fill section between station 1148+00 (northern boundary of the Topsail Reefs) to station 1090+00 (southern boundary of Ship Watch Villas) – Approximately 321,000 cy;
- A 500 ft. taper from station 1090+00 (southern boundary of Ship Watch Villas) to 1085+00 (approximately 300 ft. north of Barton Bay Ct.) – Approximately 15,000 cy;
- Between station 1090+00 (southern boundary of Ship Watch Villas) and 1060+00 (approximate southern boundary of 1521 New River Inlet Rd.) fill will be placed in the dune to achieve the design dune dimensions of 25 ft. crest width at elevation +14.0 ft. NAVD88 – Approximately 10,000 cy;
- A 500 ft. taper from station 1060+00 (approximate southern boundary of 1521 New River Inlet Rd.) to 1055+00 (approximate northern boundary of 1504 New River Inlet Rd.) – Approximately 11,000 cy;
- The original beach fill design +4 years of advanced fill will be placed between (1055+00 approximate northern boundary of 1504 New River Inlet Rd.) and 1022+00 (approximate northern boundary of 1208 New River Inlet Rd.) – Approximately 204,000 cy; and
- A 1000 ft. taper from station 1022+00 (approximate northern boundary of 1208 New River Inlet Rd.) to 1012+00 (approximate south boundary of 1148 New River Inlet Rd.) – Approximately 47,000 cy.

The description listed herein is the recommended plan at the time this letter is submitted to the Town. A number of factors may impact the extent of the project constructed including actual construction cost and the availability of sand in the permitted channel.

Cost Estimates

Based on this recommended beach fill plan, CPE-NC has updated the cost estimates included in the 2015/2016 FEMA Maintenance Plan. Table 10. shows the updated estimated construction cost for the recommended beach fill.

Table 10. Construction Estimate – Phase 1 Maintenance and Initial Construction of Phase 2

Item	Unit	Quantity	Unit Cost	Total Cost
Renourish Phase 1 and Construct Phase 2 using Inlet Material				
Mob & Demob	Job	1	$2,078,000	$2,078,000
Dredging – Renourish Phase 1 and Construct portions of Phase 2 (Sta. 1150+00 to Sta. 1000+00)	CY	625,000	$6.73	$4,206,000
Subtotal				$6284000
Contingencies (20%)				$1257000
Total Constr. Cost				**$7,541,000**
E&D(1)				$120,000
S&I(2)				$135,000
Total				**$7,796,000**

(1)Engineering and Design (i.e., Plans and Specifications)
(2)Supervision and Inspection (Construction oversight)

Please feel free to contact me with any questions or comments regarding these deliverables.

Very truly yours,

COASTAL PLANNING & ENGINEERING OF NORTH CAROLINA, INC.

Tom Jarrett, P.E.
Senior Vice President

cc: Carin Faulkner, Town of North Topsail Beach
Ken Willson, CPE-NC