TOWN OF DUCK NORTH CAROLINA
EROSION & SHORELINE MANAGEMENT
DESIGN REPORT

SUBMITTED TO:

TOWN OF DUCK

SUBMITTED BY:

COASTAL PLANNING & ENGINEERING OF NORTH CAROLINA, INC.

MAY 2015

TOWN OF DUCK NORTH CAROLINA
EROSION & SHORELINE MANAGEMENT
DESIGN REPORT

EXECUTIVE SUMMARY

The Town of Duck is located on the Outer Banks of North Carolina roughly 27 miles south-southeast of the North Carolina and Virginia border. The Town extends along 5.9 miles of Atlantic Ocean shoreline from the Dare County and Currituck County line south to the Town of Southern Shores.

The Town of Duck is focused on a long-term shoreline management program that will serve to sustain the beaches that support a significant portion of their local economy and maintains the tax base of the Town. In order to accomplish these stated goals, the Town is taking steps to maintain its oceanfront beach and dune to a configuration that 1) provides a reasonable level of storm damage reduction to public and private development, 2) mitigates long-term erosion that could threaten public and private development, recreational opportunities, and biological resources, and 3) maintains a healthy beach that supports valuable shorebird and sea turtle nesting habitat.

The existing shoreline management initiatives within the Town of Duck are limited to beach bulldozing or scraping, sand fencing, dune vegetation, and truck haul to build and/or repair dunes. The Town does not allow the use of temporary sandbags to protect threatened structures. Essentially all of the shoreline management efforts are presently carried out by individuals or groups of individual property owners. In an effort to develop a shoreline management plan for the Town, long-term erosion rates and storm impacts were analyzed to identify parts of the shoreline where structures are vulnerable to the effects of chronic erosion and episodic storm events.

Storm induced beach change modeling was used to identify project extents and evaluate various beach fill design cross-sections. Modeling identified a 1.7 mile section of oceanfront shoreline that has the potential to realize the greatest benefit from a shoreline protection project. Within this project area, beach fill design options were evaluated on their ability to mitigate design storm impacts to structures fronting the beach. Designs tested include beach fill cross-sections with berms of varying width as well as beach fill cross-sections that include both a berm and dune, varying both the width and elevation of the dune and the width of the berm. A total of 65 design cross-sections were tested.

The beach fill options were designed in accordance with National Research Council Recommendations. This consists of a two-section design composed of the design section, which is the cross-section required to meet project objectives, and advanced fill, which is the sacrificial portion of the fill required to protect the design section from anticipated sediment losses. Beach fill design options were evaluated using the results of the storm induced beach change modeling while advanced fill requirements were defined using background erosion rates and modeled diffusion losses.

Two primary beach design options were selected from the initial 65 designs considered. These two options were evaluated based on the assumption that the full beach fill design section will be in place when the design storm impacts the project area. This was accomplished by placing advanced fill in front of the design to compensate for 5 years of background erosion and diffusion losses. Option 1 included a 20-foot wide dune at elevation +20.0 feet NAVD fronted by a 60-foot wide berm at elevation +6.0 feet NAVD. Option 2 had a 20-foot wide dune at elevation +20.0 feet NAVD fronted by an 80-foot wide berm at elevation +6.0 feet NAVD. Both options would have a main fill section covering 7,970 feet of shoreline beginning on the north at profile station D-10, which is located near 140 Skimmer Way, and ending on the south near station D-19 which is located at the south property line of 137 Spindrift Lane. Five hundred (500) foot tapers would be on the north and south ends of the fill to provide a gradual merger of the project shoreline with the existing shoreline.

Long-term erosion threats and storm impacts were analyzed for both options to evaluate project performance. Implementation of either option would eliminate the long-term erosion treat. Storm damage risks associated with the beach fill options were evaluated using storm induced beach change modeling.

Following the preliminary evaluation of the two beach fill options, the recommended beach fill option, designated as Option 3, was developed. Option 3 is a variant of Option 2, i.e., the design includes a 20-foot wide dune at elevation +20.0 feet NAVD, but would be fronted by a variable width berm at elevation +6.0 feet NAVD. The width of the berm along the project shoreline was based on the results of a one-year simulation of shoreline response of Option 2 using the computer program GENESIS. In essence, GENESIS smoothed the Option 2 shoreline resulting in an alignment that would closely follow the existing shoreline alignment. Based on the one-year simulation, the average shoreline advance for Option 3 would be 67 feet. Option 3 requires 835,000 cubic yards of design fill and 234,000 cubic yards of advanced fill for a total fill volume of 1,069,000 cubic yards. Option 3 would reduce the number of structures at risk to storm damage by 90% within the fill area, decreasing from 79 to 8.

Borrow areas to construct and maintain the project are located in Federal waters approximately 4.5 to 16 miles southeast of the project area.

TOWN OF DUCK NORTH CAROLINA
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Table of Contents

1 INTRODUCTION 1
2 AUTHORIZATION 1
3 PROJECT GOALS AND OBJECTIVES 2
4 PROJECT LOCATION 2
5 PROJECT AREA DESCRIPTION 3
5.1 Geologic History 3
6 PROJECT DATA 5
6.1 Oceanographic Data 5
6.1.1 Wave Data 5
6.1.2 Water Level Data 7
6.2 Meteorological Data 7
6.3 Geophysical Data 7
6.4 Geotechnical Data 14
7 BORROW AREAS 15
7.1 Borrow Area Design 16
8 PHYSICAL CHARACTERISTICS OF THE PROJECT AREA 20
8.1 Tides 20
8.2 Winds 20
8.3 Waves 25
8.4 Surge 32
9 RELATIVE SEA LEVEL RISE 36
9.1 Relative Sea Level Rise 36
9.2 Global Effects 39
9.3 Local Effects 39
9.4 Sea Level Rise Projections 40
10 SHORELINE CHANGES 40
11 ACTIVE PROFILE HEIGHT 44
12 VOLUME CHANGES 47
12.1 Surface Based Volume Change 47
12.2 Profile Based Volume Change 49
12.3 Shoreline Based Volume Change 50
13 SEDIMENT BUDGET 52
13.1 Relative Sea Level Rise Losses 52

13.2 Longshore Transport ... 53
13.3 Even-Odd Analysis ... 57

14 MODELING ... 57
14.1 SBEACH ... 57
14.2 GENESIS ... 63

15 PLANFORM DESIGN ... 66
15.1 Long-Term Erosion Threat ... 66
15.2 Storm Damage Risk ... 67
15.3 Project Extents ... 72

16 CROSS-SECTION DESIGN ... 72

17 BEACH FILL DESIGN OPTIONS ... 74
17.1 Background Erosion Losses ... 74
17.2 Diffusion Losses ... 76
17.3 Taper Design ... 78
17.4 Development of Beach Fill Design Options ... 79
17.4.1 Option 1 ... 81
17.4.2 Option 2 ... 81

18 PROJECT PERFORMANCE ... 82
18.1 Long-Term Erosion Threat ... 82
18.2 Storm Damage Risk ... 82
18.2.1 Option 1 ... 83
18.2.2 Option 2 ... 83
18.2.3 Comparison – Option 1 versus Option 2 ... 84
18.2.4 Option 3 – Recommended Option ... 84

19 CONSTRUCTION ... 85

20 SUMMARY & RECOMMENDATIONS ... 88

21 REFERENCES ... 93

List of Figures

Figure 1. Project Location Map 3
Figure 2. Geologic Map 4
Figure 3. Data Location Map 6
Figure 4. Transect Location Map 8
Figure 5. Borrow Area Location Map 16
Figure 6. Borrow Area A Preliminary Cuts 18
Figure 7. Borrow Area C Preliminary Cuts 19
Figure 8. Monthly Wind Speed Statistics 21
Figure 9. Monthly Wind Direction Statistics 22
Figure 10. Directional Wind Statistics 23
Figure 11. Extreme Wind Statistics 24
Figure 12. Monthly Wave Height Statistics 26
Figure 13. Monthly Wave Period Statistics 27
Figure 14. Monthly Wave Direction Statistics 28
Figure 15. Directional Wave Height Statistics 29
Figure 16. Directional Wave Period Statistics 30
Figure 17. Extreme Wave Height Statistics 31
Figure 18. Extreme Wave Period Statistics 32
Figure 19. Monthly Surge Statistics 33
Figure 20. Extreme Surge Statistics 35
Figure 21. Independent Sea Level Rise Analysis 37
Figure 22. NOAA Sea Level Rise Analysis 38
Figure 23. Shoreline Change 43
Figure 24. Active Profile Examples 46
Figure 25. LIDAR Profile Comparison 48
Figure 26. Impact of Sea Level Rise on Shoreline Recession 53
Figure 27. Longshore Transport Rate - Relative to North End Longshore Transport 55
Figure 28. Longshore Transport Rate - Relative to South End Longshore Transport 56
Figure 29. Typical SBEACH Profiles 60
Figure 30. Storm Damage Risk 68
Figure 31. Project Area Shoreline Change Rate Comparison 75
Figure 32. Optional Beach Fill Design Cross-Sections 79
Figure 33. Beach Fill Plan View 80
Figure 34. Selected Option Storm Impacts 90

List of Tables

Table 1. Wave Gauges 5
Table 2. Profile Survey Baseline and Azimuth..12
Table 3. Summary of Available Survey..13
Table 4. Native Beach Geotechnical Data 15
Table 5. Borrow Area Sediment Characteristics..17
Table 6. Tidal Datums..20
Table 7. Monthly Wind Statistics 21
Table 8. Directional Wind Statistics 22
Table 9. Extreme Wind Events 23
Table 10. Extreme Wind Statistics..24
Table 11. Hurricane Wind Statistics 25
Table 12. Monthly Wave Statistics..26
Table 13. Directional Wave Statistics..28
Table 14. Extreme Wave Events..30
Table 15. Extreme Wave Statistics 31
Table 16. Monthly Surge Statistics..33
Table 17. Extreme Surge Events..34
Table 18. Extreme Surge Statistics 34
Table 19. FEMA Storm Stage Elevations..35
Table 20. Projected Relative Sea Level Rise During 30 Year Project Life 40
Table 21. Shoreline Change 42
Table 22. Depth of Closure Calculations..44
Table 23. Profile Based Volume Change Associated with Hurricane Sandy 50
Table 24. Shoreline Based Volume Change 51
Table 25. SBEACH Model Parameters 58
Table 26. Typical SBEACH Grid 59
Table 27. Extreme Event Comparison 62
Table 28. Storm Event Characterization..63
Table 29. GENESIS Model Parameters..65
Table 30. Long-Term Erosion Threat – Existing Conditions 67
Table 31. Storm Damage Risk – Existing Conditions 71
Table 32. Design Storm Risk..72
Table 33. Cross-Section Comparison 73
Table 34. GENESIS Modeled Diffusion Losses..77
Table 35. Diffusion Model Comparison 77
Table 36. Taper Benefit Analysis 78
Table 37. Optional Design Summary..80
Table 38. Option 1 Design Characteristics 81
Table 39. Option 2 Design Characteristics 82

Table 40. Option 1 - Storm Damage Risk Reduction 83
Table 41. Option 2 Storm - Damage Risk Reduction 83
Table 42. Option 3 Design Characteristics 85
Table 43. Option 3 - Storm Damage Risk Reduction 85
Table 44. Project Cost Estimate – Option....88

List of Appendices

Appendix A – LIDAR Shorelines
Appendix B – GENESIS Calibration and Verification

1 INTRODUCTION

This engineering report documents the process employed to develop a shoreline protection project for the Town of Duck, North Carolina.

2 AUTHORIZATION

On September 14, 2011, the Town of Kill Devil Hills held an interagency meeting in Washington, North Carolina with representatives from various State and Federal agencies including the North Carolina Division of Coastal Management (DCM), United States Army Corps of Engineers (USACE), United States Fish and Wildlife Service (USFWS), and National Marine Fisheries Service (NMFS). The purpose of the meeting was to present the scope of a proposed locally sponsored project and develop an agreed upon permitting approach and scope of necessary environmental documentation. One outcome of the meeting was the decision to develop a "Project Information Document" that would provide the USACE with a summary of the relevant existing environmental documentation and biological data that pertains to the proposed Kill Devil Hills Shoreline Protection Project. The information provided within the document was used to assist the USACE in determining the necessary permitting requirements. Following the submittal of the document, the USACE responded that due to the likelihood of determining a Finding of No Significant Impact (FONSI), an Environmental Assessment (EA) would be the recommended approach regarding the required environmental documentation.

Soon after the 2011 interagency meeting, two other beach towns in Dare County expressed interest in pursuing their own shoreline protection projects in light of continued erosion on their respective shorelines. Another interagency meeting was held on June 19, 2013 with representatives from many of the same agencies to discuss proposed permitting and environmental documentation approaches for the towns of Duck, Kitty Hawk, and Kill Devil Hills. Because potential borrow areas under consideration for the three nourishment projects are located in Federal waters, the Bureau of Ocean Energy Management (BOEM) will act as a co-lead agency along with the USACE. During an interagency meeting on July 19, 2013, representatives from the USFWS and the NMFS agreed that while individual EAs could be drafted for each of the three proposed projects, a batched Essential Fish Habitat (EFH) assessment and a batched Biological Assessment (BA) could be submitted to satisfy consultation requirements with the NMFS and USFWS for all three beach towns.

The proposed dredging of Outer Continental Shelf (OCS) borrow areas falls outside the jurisdiction of several existing Biological Opinions (BO). The 1995/1997 South Atlantic Regional Biological Opinion (SARBO) does not apply because 1) the USACE does not have regulatory jurisdiction over OCS borrow areas and 2) the project is not being funded or undertaken by the USACE. The USACE has re-initiated consultation with the USFWS and NMFS to include new species, actions, and geographic areas in the SARBO. The presently proposed dredging activities would be covered under this re-initiated SARBO, since both the USACE and BOEM would be party to it. However, the SARBO may not be completed in time to be applicable to the Duck project; therefore, the BOEM will need its own "stand-alone" BO and Incidental Take Statement (ITS) to authorize any potential protected species interactions occurring in Federal waters.

In May 2013, Coastal Planning & Engineering of North Carolina, Inc. (CPE-NC) completed an *Erosion and Shoreline Management Feasibility Study* (CPE, 2013) which evaluated potential management options for the oceanfront shoreline along the Town of Duck. The recommended option was a large scale beach fill project. Since the completion of the Feasibility Study, the Town of Duck has authorized a larger effort to design and permit the recommended plan.

3 PROJECT GOALS AND OBJECTIVES

The Town of Duck is focused on a long-term shoreline management program that will serve to sustain the beaches that support a significant portion of their local economy, maintains the tax base of the Town, retains existing recreational resources, and protects existing natural resources. In order to accomplish these stated goals, the Town is taking steps to maintain its oceanfront beach and dune to a configuration that provides a reasonable level of storm damage reduction to public and private development and mitigates long-term erosion impacts.

The objective of this engineering report is to develop a design for a locally funded beach nourishment project for the Town of Duck. This engineering report documents the design development and advanced fill requirements defined to meet these goals while fulfilling the Town's objective of maximizing the benefits of a beach fill project that meets the Town's stated budget goals.

4 PROJECT LOCATION

The Town of Duck is located on the Outer Banks of North Carolina roughly 27 miles south-southeast of the North Carolina and Virginia border. The Town extends along 5.9 miles of Atlantic Ocean shoreline from the Dare County and Currituck County line south to the Town of Southern Shores. The USACE Field Research Facility (FRF) is located within the Town limits, approximately 2.3 miles north of the southern limit and 3.6 miles south of the northern limit. A location map is provided in Figure 1. This location map highlights the proposed nourishment project along a 1.7 mile section of the Town's oceanfront shoreline and the two proposed OCS borrow areas located in Federal waters offshore of Dare County.

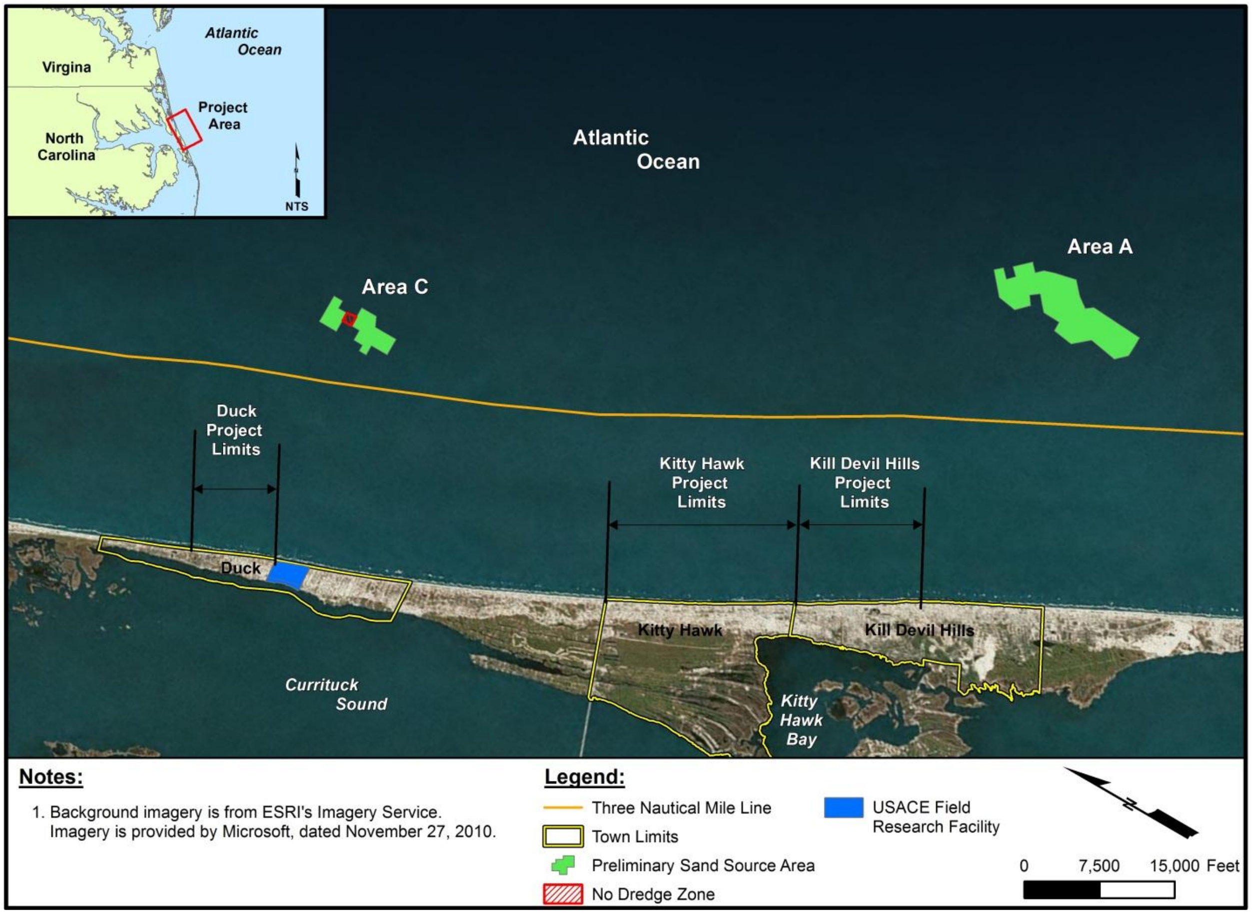


Figure 1. Project Location Map

5 PROJECT AREA DESCRIPTION

The Town of Duck encompasses 5.5 square miles and is oriented in a north-northwest/south-southeast direction. The natural habitats follow a profile typical of a coastal barrier island system, transitioning from open-ocean to island shoreline, dune, overwash and mud flat, salt marsh, and marine sound. The Project Area, as shown in Figure 1, is defined as the boundary where direct effects will occur and is inclusive of the area of nourishment along the shoreline and the OCS borrow areas.

5.1 Geologic History

The geomorphology of the North Carolina coastal environment can be geographically divided into northern and southern zones by the paleotopographic high referred to as the Cape Lookout High (see Figure 2). The region north of Cape Lookout lies within a structural basin known as the Albemarle embayment and consists of a 300 foot thick quaternary stratigraphic record (Mallinson et al, 2005). The northern zone has been shaped by multiple cycles of deposition and erosion related to global sea level cycles during the Pleistocene epoch. Sea level rise during the present geological epoch (Holocene) has resulted in non-uniform deposition of coastal sediments over the eroded Pleistocene embayments. The modern North Carolina barrier island system is therefore superimposed upon multiple irregular, partially preserved, and highly dissected geological strata and consists of sediments ranging from peat and mud to unconsolidated or semi-unconsolidated sands, gravel, and shell beds.

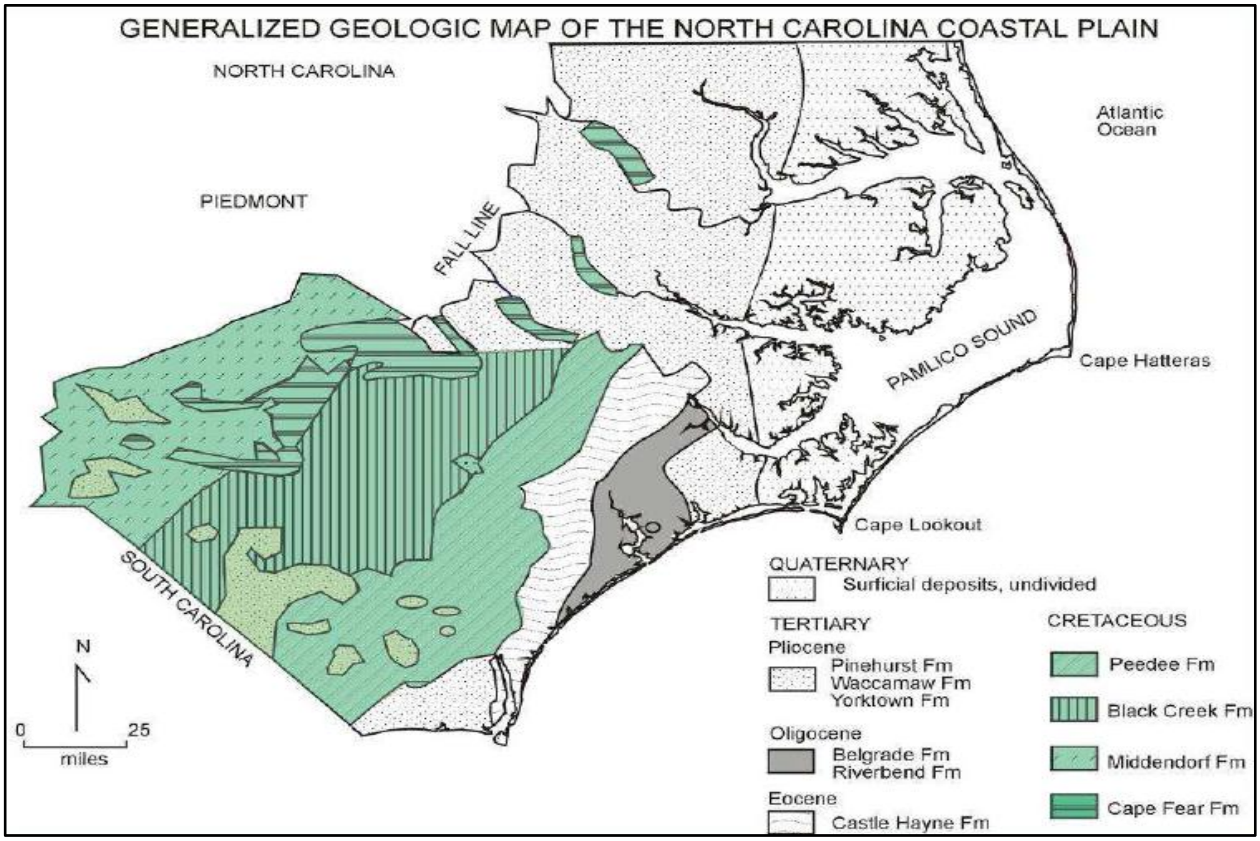


Figure 2. Geologic Map

The development of the slope and sandbars that characterize the beach and nearshore is highly influenced by this underlying geological framework (McNinch, 2004). The influence of this framework is even greater in areas with limited sand supply, such as North Carolina, where sediments for beach development are derived from the erosion and transport of sediments from adjacent beaches or the inner continental shelf (Thieler et al, 2014). Some of the characterizing features of the coastal zone of North Carolina's Outer Banks include the development of shore-oblique sandbars adjacent to large gravel outcrops that are surface exposures of the underlying geologic strata and identical redevelopment or sustained maintenance of large-scale sandbar morphology and position before and after very energetic conditions and close spatial alignment between the location of outcrops/shore-oblique bars and shoreline erosional hotspots (McNinch, 2004).

Along with the many variables that can affect a coastline's morphology, regional sediment composition, sediment size, and sediment shape can play a major role. The coastal zone of North Carolina's Outer Banks is characterized by a vertical and horizontal heterogeneity of lithology and grain-size and a minimum volume of sand, ranging from 0 to 5 feet thick (McNinch, 2004). Barrier islands in North Carolina, such as the Outer Banks and the beachfront of the Town of Duck, are primarily composed of unconsolidated fine- to medium-sized quartz and shell (calcium carbonate) material that is in a constant state of flux due to wind, waves, currents, and storms.

6 PROJECT DATA

Project data sets include oceanographic, meteorological, geophysical, and geotechnical. Where applicable, the location of the measuring devices (wave gauges, tide gauges, etc.) are referenced to the North Carolina state Plane Coordinate System. Details regarding each data set are discussed in the following sections.

6.1 Oceanographic Data

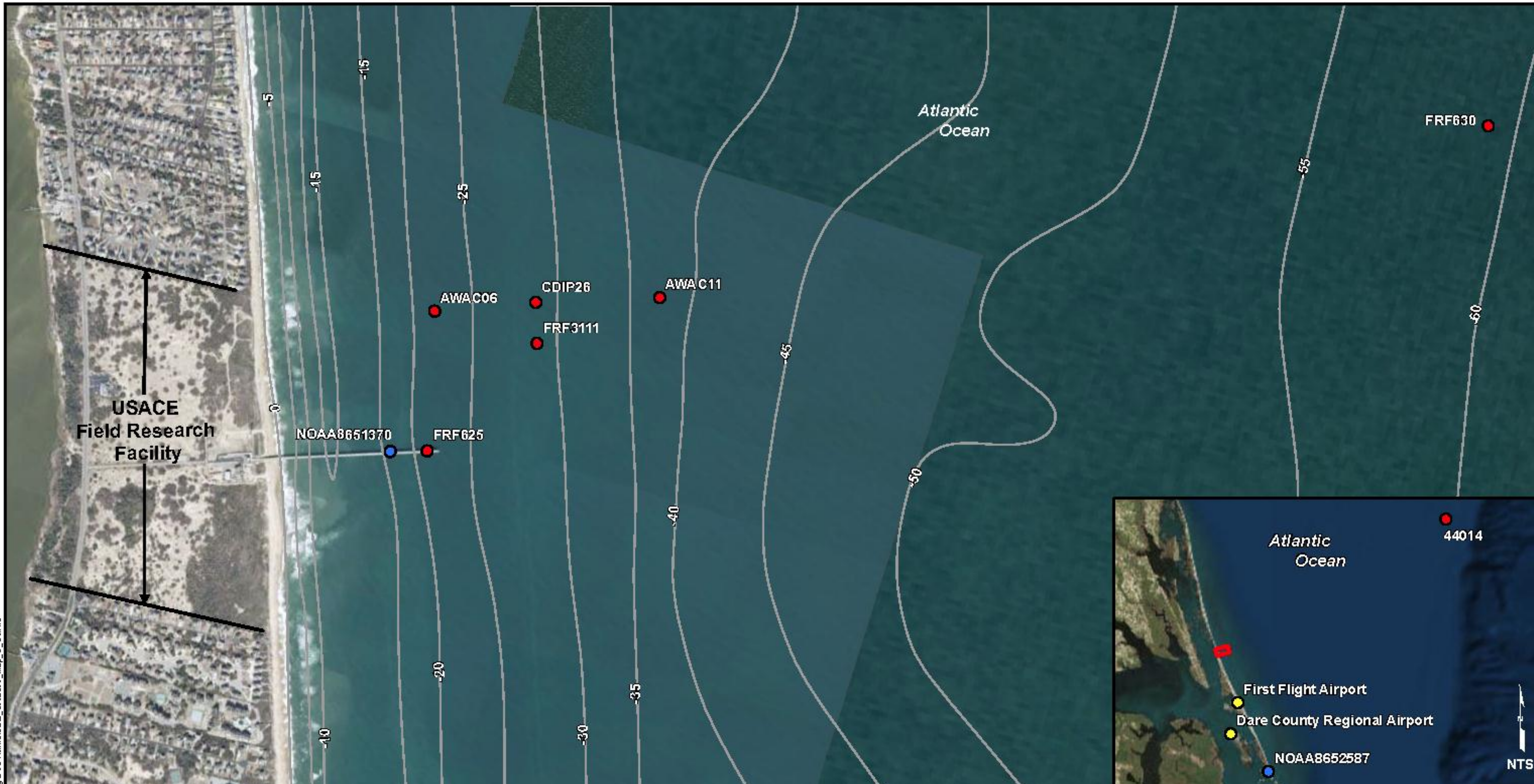
Although oceanographic data was not collected specifically for this project, comprehensive data sets are available from the USACE and NOAA. Recorded measurements include wave and water level data.

6.1.1 Wave Data

Wave data in the immediate vicinity of the USACE FRF has been collected since 1980 using several gauges. These gauges vary from a bottom mounted Acoustic Doppler Current Profiler (ADCP) located roughly 1,200 feet (0.2 miles) from shore in less than 10 feet of water to a surface wave buoy located roughly 30,000 feet (5.7 miles) offshore in 160 feet of water. A list of the various wave gauges and their period of operation is provided in Table 1, while the location of each gauge is shown in Figure 3.

Table 1. Wave Gauges

Wave Gauge	Location (ft,NAD83) Easting	Location (ft,NAD83) Northing	Depth (ft,NAVD)	Record Length
AWAC 05	2959491.1	902629.0	16.4	2008-present
AWAC 06	2959976.8	902827.1	19.7	2008-present
AWAC 08	2960899.0	903221.6	26.2	2008-present
AWAC 11	2962066.9	903624.2	36.1	2008-present
FRF625	2960322.6	901486.7	25.0	1980-present
FRF3111	2961043.9	902827.8	26.2	1987-present
44014	3222020.0	1066980.5	155.8	1990-present
CDIP26	2960909.1	903209.9	26.0	1980-1990
FRF630	2969396.8	907708.8	57.0	1996-present



Notes:

1. 2012 background imagery is from NC OneMap Imagery Service.
2. Contours developed using a compilation of NOAA bathymetric charts, the September 5, 2013 FRF bathymetric survey conducted by the USACE and the September 10-16, 2013 profile survey conducted by CPE-NC .
3. Contour elevations referenced in feet NAVD.

Legend:

Monitoring Station

- Wave Data
- Meteorological Data
- Water Level Data
- Contours

Feet
0 500 1,000

1 inch = 1,000 feet

TITLE:

Data Location Map
Dare County, North Carolina

Coastal Planning & Engineering of North Carolina, Inc.
4038 Masonboro Loop Road
Wilmington, NC 28409
Ph. (910) 791-9494
Fax (910) 791-4129

Figure 3. Data Location Map

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6.1.2 Water Level Data

Water level data has been collected at the Duck Pier since 1978. The NOAA tide gauge (Station 8651370) is located on the offshore end of the USACE FRF pier (Easting = 2959975.4, Northing = 901370.2, feet NAD83). Monthly mean and hourly water level data have been collected since June 1978, while verified high and low water levels have been recorded since November 1979. Six-minute data has been collected since October 1995. The location of this tide gauge is shown in Figure 3.

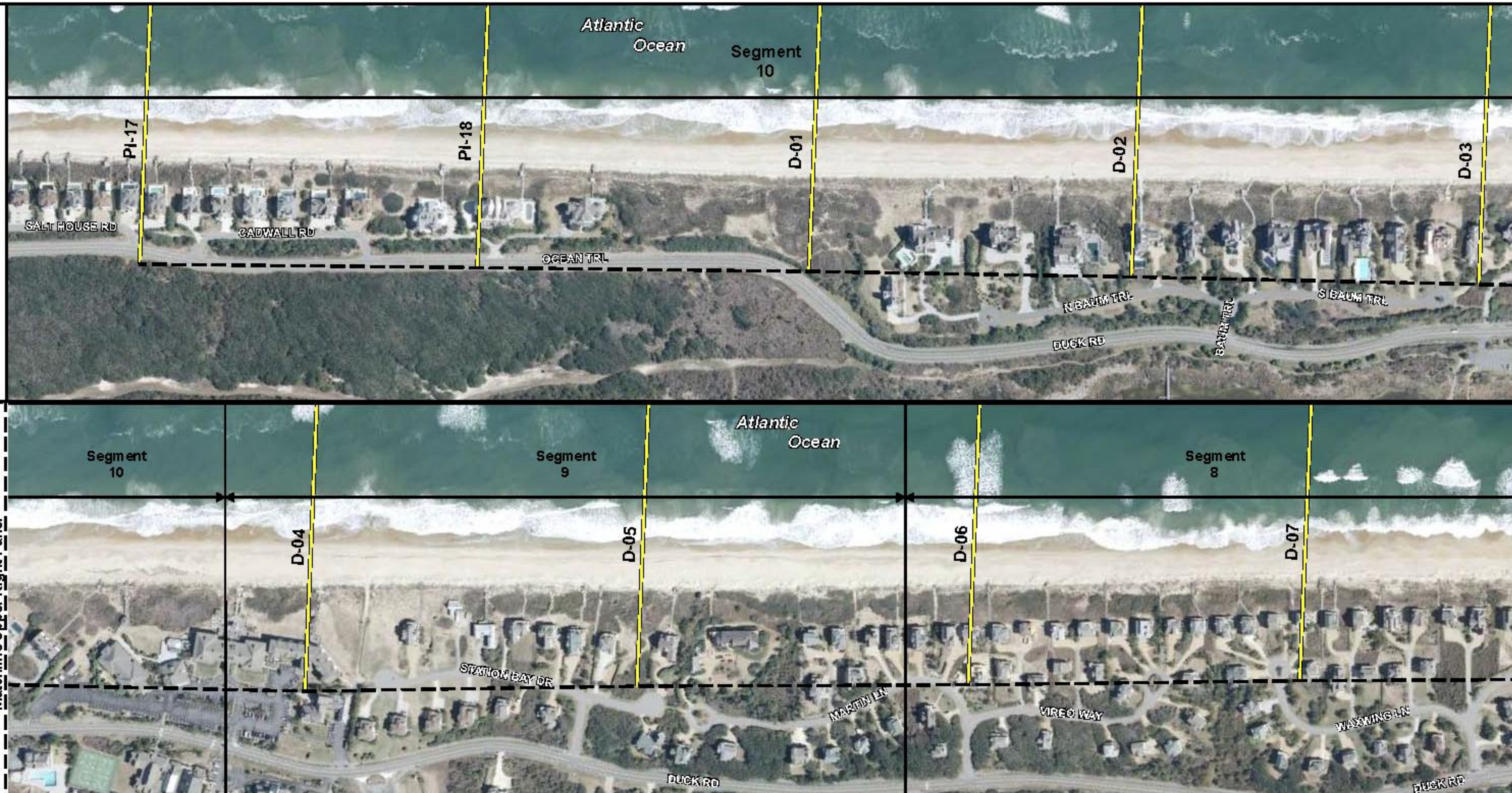
Water level data has also been collected at Oregon Inlet since 1996. The NOAA tide gauge (Station 8652587) is located near the bay entrance at the Oregon Inlet Marina (Easting = 3023389.9, Northing = 762070.2, feet NAD83), roughly 29 miles south of the USACE FRF pier. Monthly mean water level data has been collected since April 1994, while hourly water level data has been recorded since January 1996. Six minute data has been collected since January 2001. The location of this tide gauge is shown in Figure 3.

6.2 Meteorological Data

Meteorological data, such as wind velocity and barometric pressure, has been collected at the NOAA tide gauge located at the USACE FRF, referenced in Section 6.1.2, since June 1991. Although data has been collected since June 1991, data is only available after May 1996 and a gap in this data exists between February and July 2003. Additional meteorological data were collected at the nearby Dare County Regional (Easting = 2976918.6, Northing = 805071.3, feet NAD83) and First Flight (Easting = 2984193.1, Northing = 842072.8, feet NAD83) Airports. Data was collected at the Dare County Regional Airport between September 1985 and December 2004, while data has been collected at the First Flight Airport since May 2004. The locations of the NOAA tide gauge and the Dare County Regional and First Flight Airports are shown in Figure 3.

6.3 Geophysical Data

To clearly define existing conditions and better analyze vulnerability, a beach profile survey was conducted along the Town's shoreline. This survey consists of a total of 34 profiles with a spacing of roughly 1,000 feet. In addition, two profiles were surveyed both north and south of the Town limits to evaluate adjacent trends that might impact future project formulation should these areas be included in a proposed plan. Therefore, a total of 38 profiles, encompassing 35,000 feet of shoreline, were surveyed September 2013 as part of this project. Survey data was collected along transects detailed in Table 2, which are referenced to the North Carolina State Plane coordinate system in feet NAD83 with a profile azimuth in degrees referenced to true north. Transects listed in Table 2 are shown graphically in Figure 4. The complete survey report was provided to the Town in November 2013.



Notes:

1. 2012 Background imagery is from NC OneMap Imagery Service.

Legend:

Transects

Baseline

0 150 300 Feet

1 inch = 300 feet

TITLE:

Transect Location Map
Duck, North Carolina
Sheet 1

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4038 Masonboro Loop Road
Wilmington, NC 28409
Ph. (910) 791-9494
Fax (910) 791-4129

Figure 4. Transect Location Map

Notes:

1. 2012 Background imagery is from NC OneMap Imagery Service.

Legend:

Transects

Baseline

Feet
0 150 300
1 inch = 300 feet

TITLE:

Transect Location Map
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Wilmington, NC 28409
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Fax (910) 791-4129

Figure 4. Transect Location Map

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Notes:

1. 2012 Background imagery is from NC OneMap Imagery Service.

Legend:

Transects

Baseline

TITLE:

Transect Location Map
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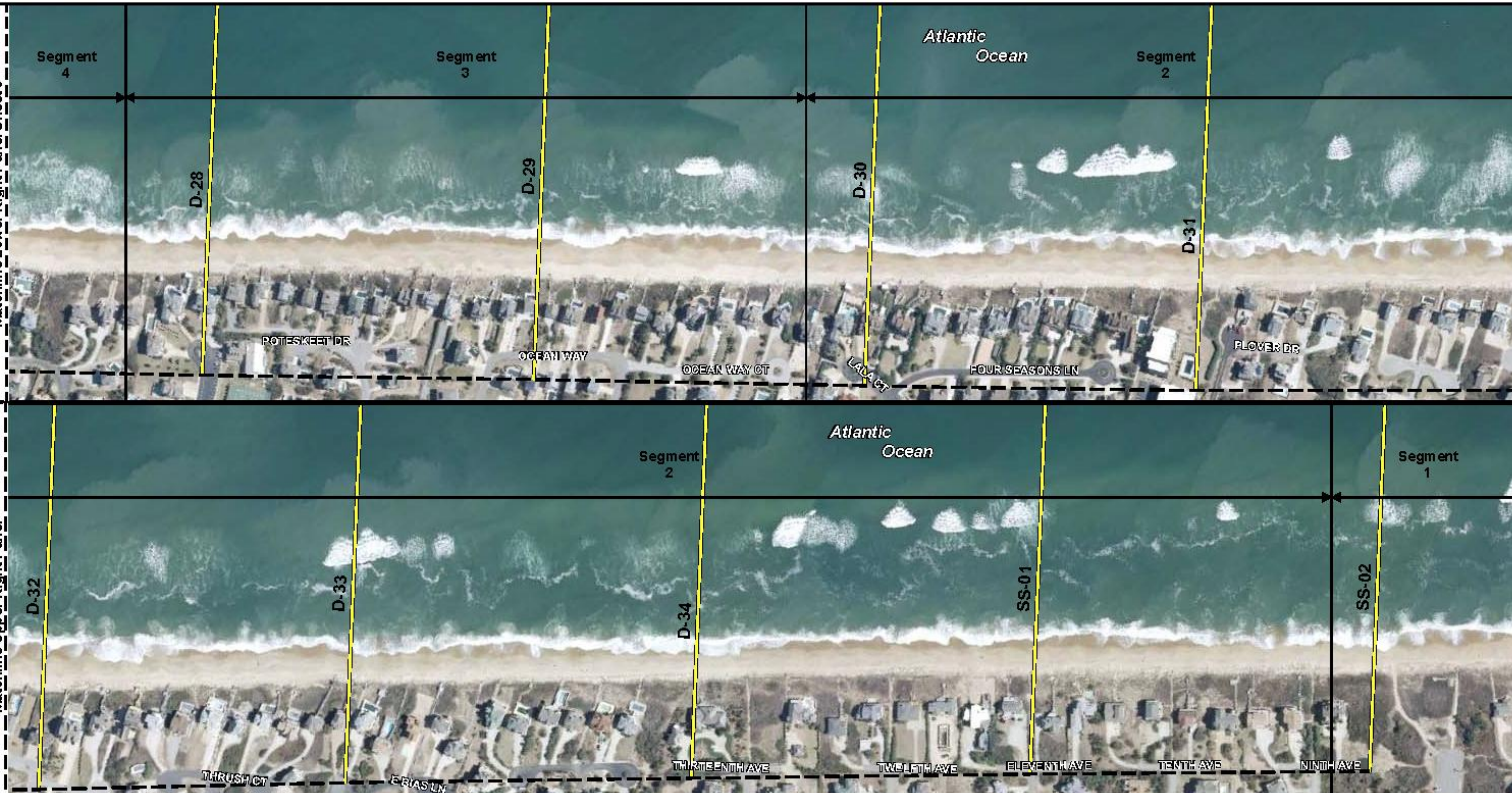
Fax (910) 791-4129

Figure 4. Transect Location Map



Notes:

1. 2012 Background imagery is from NC OneMap Imagery Service.

Legend:

Transects

Baseline

Feet
0 150 300
1 inch = 300 feet

TITLE:

Transect Location Map
Duck, North Carolina
Sheet 4

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4038 Masonboro Loop Road
Wilmington, NC 28409
Ph. (910) 791-9494
Fax (910) 791-4129

Figure 4. Transect Location Map

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Table 2. Profile Survey Baseline and Azimuth

Profile	Easting	Northing	Azimuth
PI-17	2950657.3	920098.9	70
PI-18	2951026.0	919175.4	70
D-01	2951387.5	918267.7	70
D-02	2951733.8	917384.4	70
D-03	2952103.0	916429.4	70
D-04	2952464.0	915495.3	70
D-05	2952849.3	914598.0	70
D-06	2953224.4	913696.9	70
D-07	2953607.3	912798.8	70
D-08	2953983.0	911897.9	70
D-09	2954356.7	910994.8	70
D-10	2954759.1	910066.7	70
D-11	2955158.1	909133.1	70
D-12	2955461.4	908412.5	70
D-13	2955874.3	907478.4	70
D-14	2956252.1	906578.3	70
D-15	2956628.6	905677.8	70
D-16	2956978.7	904767.7	70
D-17	2957333.7	903863.9	70
D-18	2957718.8	902886.5	70
D-19	2957932.5	902331.0	70
D-20	2958139.7	901760.7	70
D-21	2958472.1	900958.7	70
D-22	2958754.0	900228.8	70
D-23	2958992.7	899515.6	70
D-24	2959267.2	898739.8	70
D-25	2959601.7	897824.3	70
D-26	2959928.6	896902.3	70
D-27	2960250.6	895981.9	70
D-28	2960604.1	895073.0	70
D-29	2960963.6	894166.2	70
D-30	2961317.7	893257.6	70
D-31	2961676.7	892350.7	70
D-32	2962078.1	891379.4	70
D-33	2962439.4	890553.2	70
D-34	2962839.6	889616.1	70
SS-01	2963230.4	888697.7	70
SS-02	2963619.0	887775.8	70

The profile surveys extended landward until a structure was encountered or to a range 50 feet beyond the landward toe of dune, whichever is more seaward. Elevation measurements were also taken seaward along the profile to the -30 feet NAVD contour. Upland data collection includes all grade breaks and changes in topography to provide a representative description of the conditions at the time of the work. The maximum spacing between data records along individual profiles is 25 feet. The upland survey extends into wading depths sufficiently to overlap the offshore portion a minimum of 50 feet.

Additional geophysical surveys within the project area include those conducted by the United States Geological Survey (USGS), USACE, and the Town of Duck. The dates, techniques, and extents of all geophysical data sets within the project area are summarized in Table 3.

Table 3. Summary of Available Survey

Entity	Technique	Dates	Extents of Data
USACE	Topographic/ Bathymetric	Multiple surveys per year from 1981 to present	Between D-16 and D-26 Top of dune to -25' NAVD
USGS	LIDAR	1996-1999, 2001, 2004, 2005, 2008, 2009, 2012	North beyond PI-07 to south beyond SS-04 Landward toe of dune to MHW
	LIDAR	2010	D-07 to south beyond SS-04 Landward toe of dune to MHW
	LIDAR	2011	North beyond PI-07 to south beyond SS-04 Top of dune to MHW
	CLARIS	2012	North beyond PI-07 to south beyond SS-04 Top of dune to +5' NAVD
Town of Duck	Topographic/ Bathymetric	2011	PI-07 to 5,000' south of SS-04 Top of dune to -25' NAVD
	Topographic/ Bathymetric	2013	C-04 to SS-04 Landward toe of dune to -25' NAVD

The USACE has conducted numerous topographic and bathymetric surveys at the FRF since 1981. Surveys were conducted throughout each calendar year during both winter and summer months. The profile surveys extended alongshore between transects D-16 and D-26, approximately 3,600 feet north and south of the FRF pier. The cross-shore extent of the surveys was between the top of the dune and the -25 feet NAVD seaward contour.

LIDAR surveys were conducted by the USGS between 1996 and 2012. LIDAR is a remote sensing technology that uses light detection to map an area. It provides the most comprehensive data set for topography; however, the lack of water clarity restricts LIDAR from providing subaqueous data. The LIDAR data sets extended alongshore beyond the project area extents, except the 2010 data set that did not extend north of D-07. The cross-shore extent of the surveys was between the Mean High Water (MHW) shoreline and either the top or the landward toe of the dune.

In addition to LIDAR surveys, the USGS conducted a CLARIS survey in 2012. CLARIS stands for Coastal LIDAR and Radar Imaging System. Data was collected using an optical system mounted on a truck that traversed the subaerial portion of the beach. Surveys were limited by instrument line of sight. Thus, cross-shore data collection was confined between the approximate +5 feet NAVD contour and the top of the dune. The CLARIS survey extended alongshore between transects PI-07 (10,000 feet north of PI-17) and SS-04 (7,000 feet south of SS-02).

Profile surveys were collected by the Town of Duck in November 2011 and September 2013. As discussed previously, the 2013 topographic and bathymetric survey extended alongshore between transects PI-17 and SS-02. The 2011 bathymetric survey extended alongshore from PI-07 (10,000 feet north of PI-17) to 5,000 feet south of SS-04 (7,000 feet south of SS-02). The cross-shore extent of the 2013 survey was between the top of the dune and the -30 feet NAVD contour. Considering that the 2011 survey consisted of only a bathymetric component, the survey extended from the shoreline seaward to the -40 feet NAVD contour.

The Erosion and Shoreline Management Feasibility Study (CPE, 2013) used profile survey data that was collected before Hurricane Sandy. However, as discussed above, the profile survey consisted of only bathymetric data extending from the shoreline seaward to the -40 feet NAVD contour. In an effort to create a continuous pre-storm profile, profiles extracted from LIDAR data were added to the nearshore bathymetric data. The profile segment landward of the dune was created using 2009 LIDAR data, the profile segment between the dune and the shoreline was extracted from 2011 LIDAR data, while the profile segment seaward of the shoreline consisted of the nearshore bathymetric data.

Since the Erosion and Shoreline Management Feasibility Study (CPE, 2013), the beach profile naming convention and cross-shore location of the baseline have been changed, while the transect location and azimuth have remained the same. In the Feasibility Study, profiles were labeled according to their distance from the southern boundary of the USACE FRF property. Profiles to the north of this point were labeled as positive distances, while profiles to the south were labeled as negative distances. For this report, profiles were labeled using the initials of the locality controlling the adjacent beach interest and a number. The Feasibility Study (CPE, 2013) also broke the shoreline into segments that exhibited similar shoreline change trends. The location of the various segments relative to the survey transects is shown in Figure 4.

6.4 Geotechnical Data

Taking material from an offshore borrow area and placing it onto the beach has the potential to alter the physical characteristics of the native beach. To minimize the risk of such alterations, projects are designed to use similar sediment with regards to sorting, mean grain size, median grain size, and sediment composition. Furthermore, the North Carolina State Sediment Criteria Rule (15A NCAC 07H .0312) sets standards for borrow material aimed at preventing the disposal of incompatible material on the native beach. The rule limits the amount of material by weight in a borrow area with a diameter equal to or greater than 4.76 and less than 76 millimeters (gravel), between 4.76 and 2.0 millimeters (granular), and less than 0.0625 millimeters (fines) to no more than 5% above that which exists on the native beach. Additionally, the rule requires the proportion of calcium carbonate in borrowed material not to exceed 15% above that of the native beach.

Based on the State Sediment Criteria, sampling of the native material are required from a minimum of five transects regardless of the total project length. At least six samples are to be taken between the Mean Low Water (MLW) line and the dune and six samples are to be taken between the MLW line and the depth of closure. One sample is also required at the MLW line for thirteen samples per transect. The rule also sets forth guidelines to ensure the sediment characteristics of material placed on the recipient beach are compatible with the native sediment. Essentially, the rule states the following:

- The average percentage by weight of fine-grained sediment (less than 0.0625 millimeters) in a borrow site shall not exceed the average percentage by weight of fine-grained sediment of the recipient beach characterization plus five percent.

- The average percentage by weight of granular sediment (greater than or equal to 2 millimeters and less than 4.76 millimeters) in a borrow site shall not exceed the average percentage by weight of coarse-grained sediment of the recipient beach characterization plus five percent.

- The average percentage by weight of gravel (greater than or equal to 4.76 millimeters) in a borrow site shall not exceed the average percentage by weight of gravel-sized sediment of the recipient beach characterization plus five percent.

- The average percentage by weight of calcium carbonate in a borrow site shall not exceed the average percentage by weight of calcium carbonate of the recipient beach characterization plus fifteen percent.

In September 2013, CPE-NC collected samples of the Town of Duck native beach material along five transects (D-03, D-08, D-13, D-18, and D-24), with thirteen samples collected along each profile. Sampling began at the dune and extended seaward to the -20 feet NAVD contour. In keeping with the North Carolina Coastal Resources Commission (CRC) standards, sample distribution along profiles included six samples landward and six samples seaward of the MLW line and one additional sample at the MLW line. Mechanical sieve analyses were conducted on each sample and a composite grain size was calculated for each profile. A composite sample for each profile was prepared by mixing equal parts of samples from each sample location along the profile. The composite sample generated for each profile was analyzed for calcium carbonate content using an acid digestion process. Results of the native beach geotechnical analysis are summarized in Table 4.

Table 4. Native Beach Geotechnical Data

Parameter	Native Characteristic
Mean Grain Size (mm)	0.34
Sorting (Phi)	1.37
Wet Munsell Value	5
Dry Munsell Value	6
Carbonate Content Percentage	2.03%
Percent Fine (<0.0625mm)	0.96%
Percent Sand (0.0625mm - 2.00mm)	92.43%
Percent Granular (2.00mm - 4.76mm)	4.83%
Percent Gravel (4.76mm - 76mm)	1.77%

7 BORROW AREAS

Four offshore areas were investigated for use as potential sand sources for this project; one of the areas is within State waters, two are within Federal waters, and one straddles the State and Federal water border. The primary investigation areas, shown in Figure 5, include areas A, B, C, and S1-4. Because the sediments in these offshore areas are not part of the active littoral system, the sediment may be different from the beach sediment in terms of size and composition. Borrow area compatibility requirements are detailed in Section 6.4. The borrow area design is summarized below.

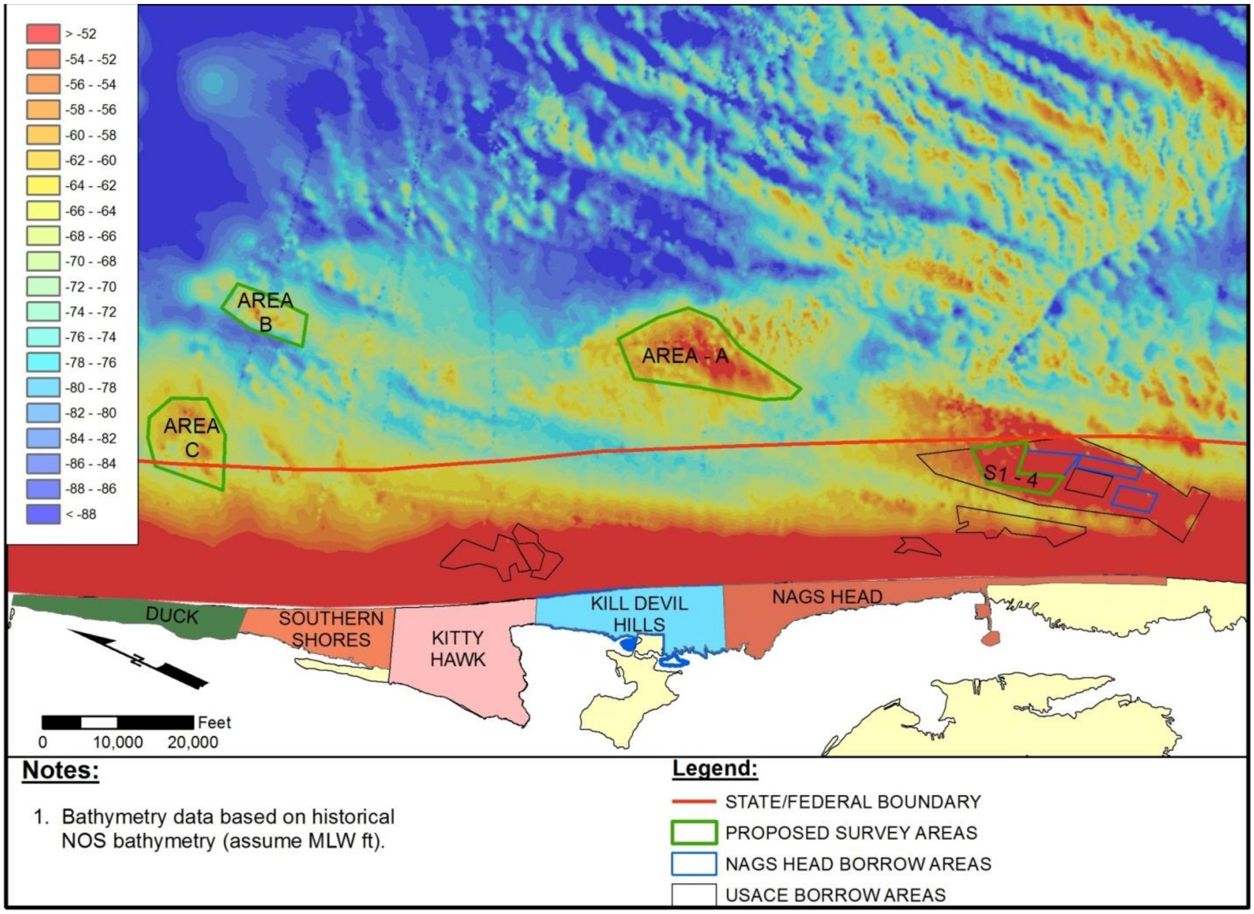

Figure 5. Borrow Area Location Map

7.1 Borrow Area Design

Using material for beach nourishment that differs significantly from the existing beach material can affect project performance. In order to identify and characterize the sand source material, CPE-NC used the systematic marine sand search approach developed by Finkl, Khalil, and Andrews (1997), Finkl, Andrews, and Benedet (2003), Finkl, Benedet, and Andrews (2005), and Finkl and Khalil (2005). The investigation was divided into three sequential phases, which included a comprehensive review of the project area (recipient beach) and sediment resources offshore of the project area; a reconnaissance level geotechnical (washbores) and geophysical (sub-bottom profiler, sidescan sonar, bathymetry, and magnetometer) survey; and design level geotechnical (vibracores) and geophysical (sub-bottom profiler, sidescan sonar, bathymetry, and magnetometer) investigations and borrow area design. These investigations were conducted to evaluate the four target areas and ultimately delineate the borrow area.

S1-4 was the only potential borrow area located entirely within State waters. S1-4 lies within a much larger area originally explored by the USACE as part of the Hurricane Protection and Beach Erosion Control Project for Dare County Beaches. This larger area, referred to as S1, was determined to have high quality material and portions of it were utilized during the Nags Head Beach Nourishment Project in 2011. Existing data associated with both the Federal Dare County Beaches Project and the locally constructed Town of Nags Head Beach Nourishment Project suggests that sufficient quantities of sand exist within S1. A reconnaissance washbore survey conducted September 2013 confirmed that the quality of the material within S1-4 warranted further investigations. During the June 2014 geophysical survey, additional data collected (sub-bottom profile, sidescan, magnetometer, and bathymetric) further suggested that the material was of good quality. However, during the July/August 2014 preliminary geotechnical (vibracore) investigations, CPE-NC geologists determined that a sufficient volume of quality material existed within area A and further investigation of area S1-4 was not necessary.

Potential areas of beach compatible sand located further offshore than area S1-4 but closer to the project location were identified. These areas are located more than 3 miles offshore and are therefore within Federal waters managed by the BOEM. These areas, referred to as areas A, B, and C, were investigated by CPE-NC geologists in 2013 and 2014. A reconnaissance washbore survey was conducted in September 2013 and confirmed that the quality of the material within areas A and B warranted further investigations. Based on the results of the washbore survey and the similar morpho-sedimentary characteristics of area C, this third area was also targeted for further investigation. During the June 2014 geophysical survey, additional geophysical data (sub-bottom profile, sidescan, magnetometer, bathymetric) were collected and further suggested that the material within these areas warranted vibracore investigations. During preliminary geotechnical (vibracore) investigations conducted in July/August 2014, CPE-NC geologists determined that the material contained in area B did not appear to be of as high a quality and in sufficient volume to warrant design level surveys. However, areas within A and C were identified as sufficient and additional vibracores were collected to support borrow area design.

The October 2014 cultural resource and design survey resulted in final delineation of borrow areas A and C. The location of borrow areas A and C are shown in Figure 5. Preliminary borrow area design cuts are shown in Figure 6 and Figure 7, while the borrow area sediment characteristics are provided in Table 5. Sand compatibility analyses, as discussed in Section 6.4, have determined that the borrow material meets North Carolina CRC compatibility requirements.

Table 5. Borrow Area Sediment Characteristics

Parameter	Native Beach	Borrow Area A	Borrow Area C
Mean Grain Size (mm)	0.34	0.36	0.27
Sorting (Phi)	1.37	0.9	1.09
Wet Munsell Value	5	5	5
Dry Munsell Value	6	6	6
Carbonate Content Percentage	2.03%	1%	7%
Percent Fine (<0.0625mm)	0.96%	0.83%	1.59%
Percent Sand (0.0625mm - 2.00mm)	92.43%	97.17%	95.31%
Percent Granular (2.00mm - 4.76mm)	4.83%	1.48%	2.05%
Percent Gravel (4.76mm - 76mm)	1.77%	0.52%	1.07%

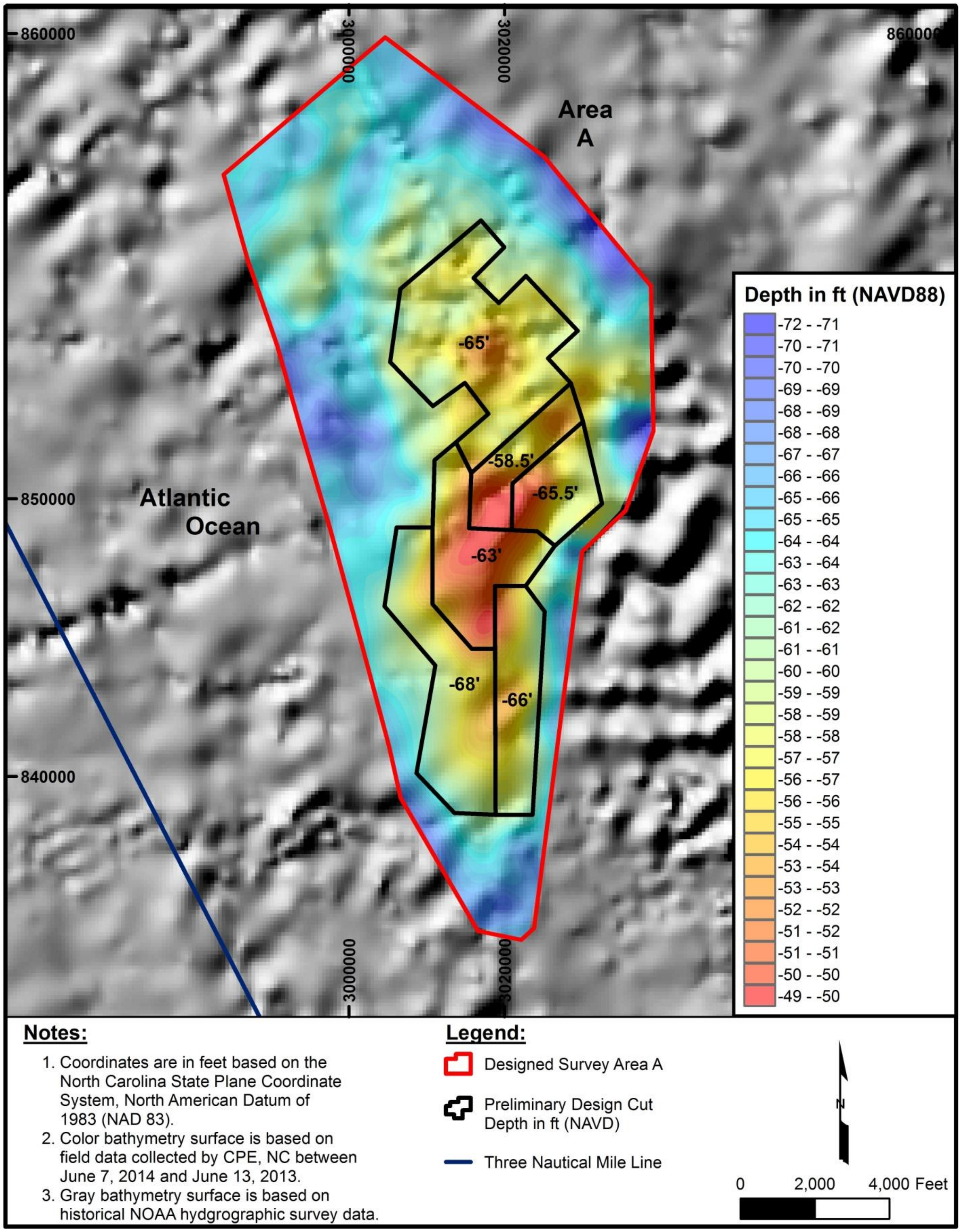

Figure 6. Borrow Area A Preliminary Cuts

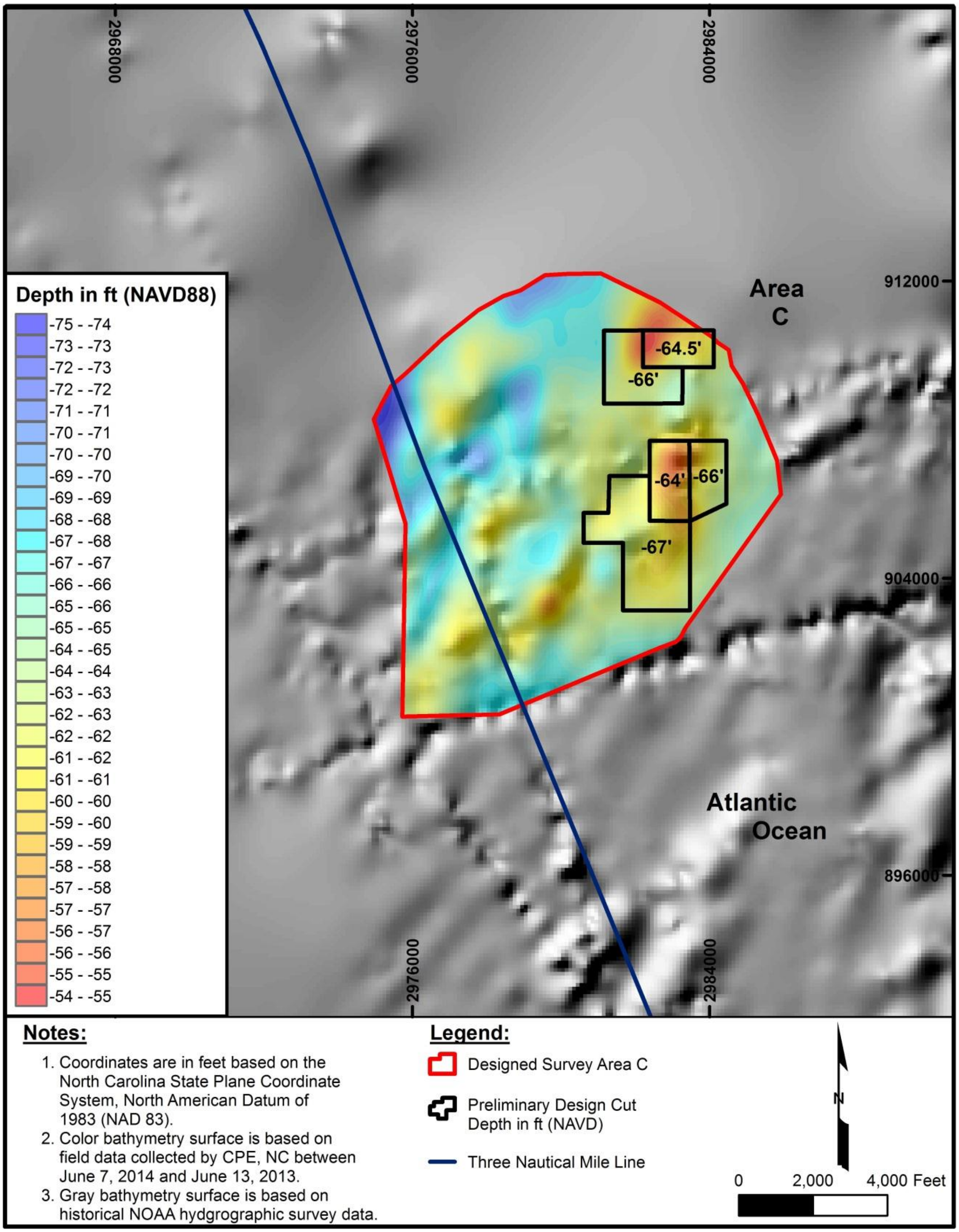

Figure 7. Borrow Area C Preliminary Cuts

8 PHYSICAL CHARACTERISTICS OF THE PROJECT AREA

The Town of Duck is subject to littoral processes typical of the barrier islands that line the North Carolina coast. The islands are exposed to varying winds, waves, and water levels. These physical characteristics that impact the project site are described in the following sections.

8.1 Tides

As discussed in Section 6.1.2, NOAA operates a tide gauge on the offshore end of the USACE FRF pier in Duck, North Carolina (NOAA 8651370). Tides at Duck are semi-diurnal with an average tidal range of approximately 3 feet. Tidal datums for the 1983-2001 epoch were accepted September 2011 and are shown in Table 6. The highest recorded water level of 5.63 feet NAVD was measured during Hurricane Isabel on September 18, 2003 at 16:06 GMT, while the lowest recorded water level of -4.85 feet NAVD was measured on March 16, 1980 at 17:54 GMT. The highest recorded astronomical tide of 2.76 feet NAVD was measured on October 16, 1993 at 12:24 GMT, while the lowest recorded astronomical tide of -3.17 feet NAVD was measured on February 8, 1997 at 06:24 GMT.

Table 6. Tidal Datums

Datum	Elevation (ft, NAVD)
Mean Higher High Water (MHHW)	1.50
Mean High Water (MHW)	1.18
Mean Sea Level (MSL)	-0.42
Mean Tide Level (MTL)	-0.43
Mean Low Water (MLW)	-2.05
Mean Lower Low Water (MLLW)	-2.19

8.2 Winds

Winds indirectly cause the littoral transport of sand by generating waves and currents. As discussed in Section 6.2, wind data near the project area were collected at the NOAA tide gauge on the offshore end of the USACE FRF pier and at the Dare County Regional and First Flight Airports. Even though a gap in the NOAA data was identified between February and July 2003, the NOAA data was collected closest to the project site and had the longest collection period using consistent techniques and was therefore used to characterize wind conditions at the project site.

The average measured wind speed is 13.1 miles per hour with a corresponding direction of 359° (N), while the maximum measured wind speed is 68.7 miles per hour. The strongest winds occur between August and October during hurricane season. With the exception of tropical storm events, the strongest winds under typical conditions occur in March and April, with the weakest winds occurring in July and August. The wind direction varies from the north during winter months to the south during summer months. The strongest winds come from the north-northeasterly direction band. Monthly wind statistics are provided in Table 7 and are shown pictorially in Figure 8 and Figure 9, while directional wind statistics are detailed in Table 8 and are shown graphically in Figure 10. The reported average wind direction is the monthly mean wind direction while the energetic wind direction is the monthly weighted average wind direction, with direction weighted using squared wind speed as it is proportional to wind energy.

Table 7. Monthly Wind Statistics

Month	Speed (mph) Average	Speed (mph) Maximum	Direction (deg) Average	Direction (deg) Energetic
January	14.2	58.8	301	337
February	14.2	48.3	329	351
March	14.4	51.9	359	357
April	14.4	47.4	215	325
May	13.3	59.7	170	34
June	11.7	40.9	191	205
July	10.6	42.1	207	222
August	10.7	63.3	115	60
September	13.1	68.7	71	46
October	13.3	59.1	5	22
November	13.5	51.7	324	358
December	13.7	48.8	306	346
Annual	**13.1**	**68.7**	**288**	**359**

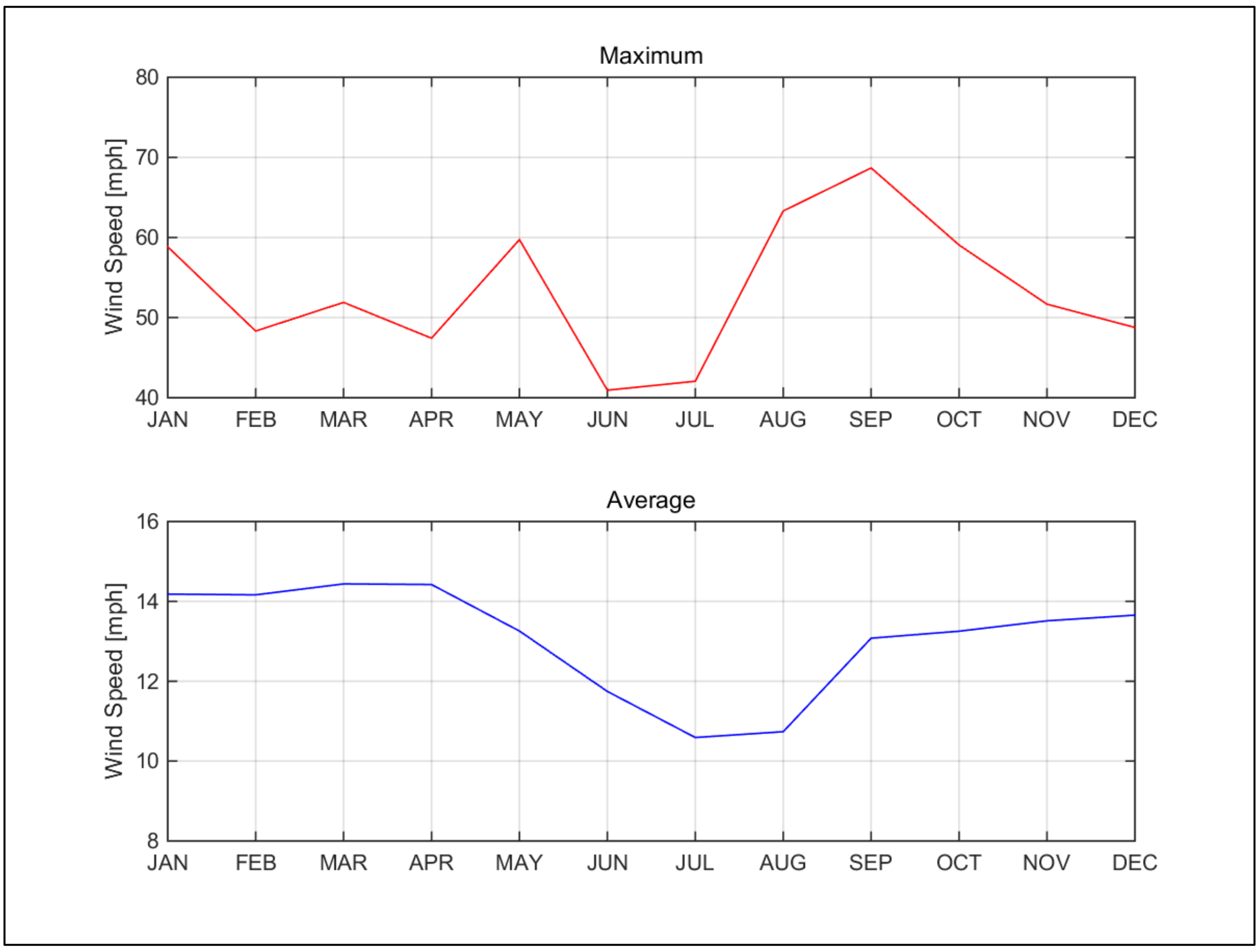

Figure 8. Monthly Wind Speed Statistics

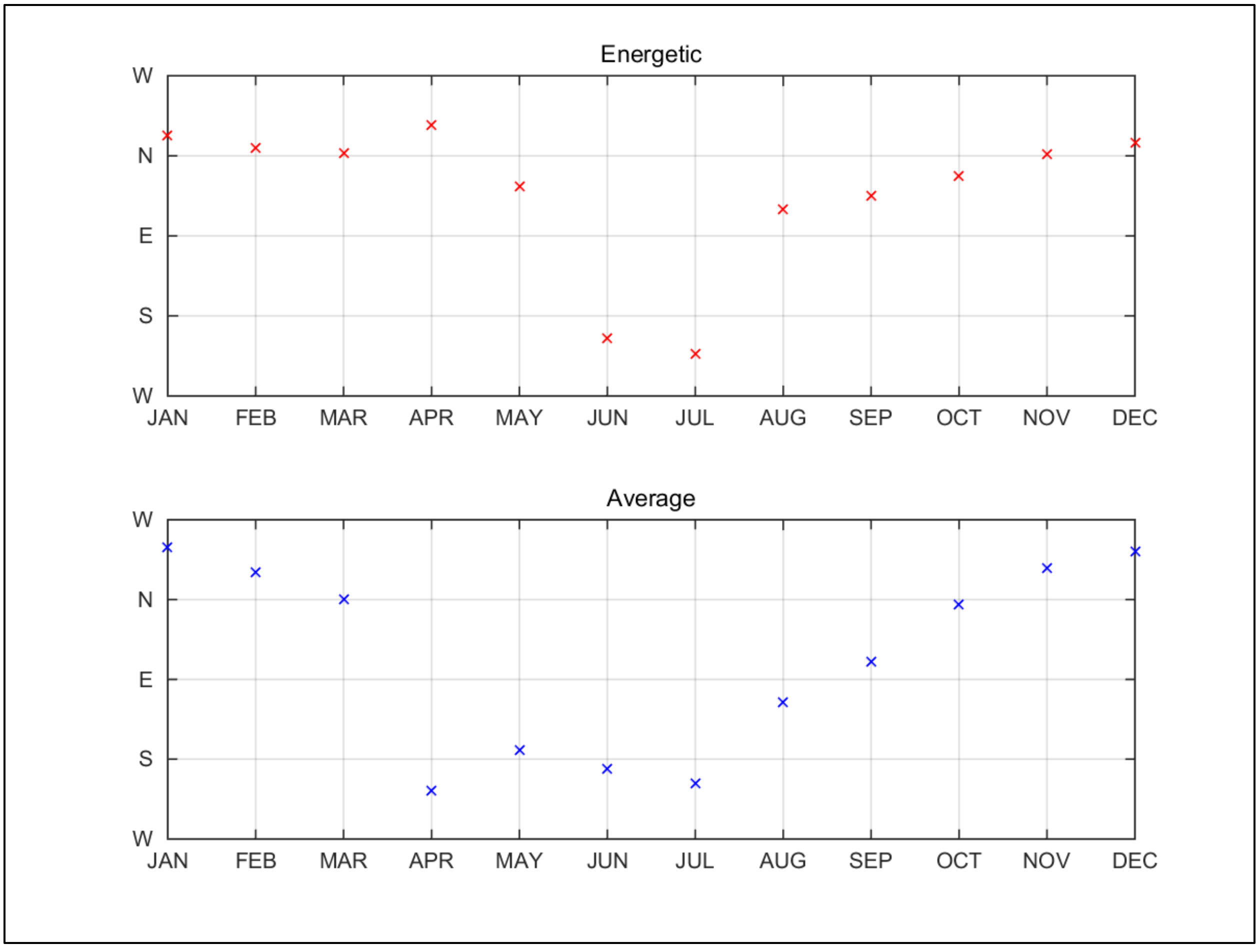

Figure 9. Monthly Wind Direction Statistics

Table 8. Directional Wind Statistics

Direction Bin	Percent Occurrence	Speed (mph) Average	Speed (mph) Maximum
0.0	6.9%	17.1	58.8
22.5	9.0%	17.1	59.7
45.0	9.3%	15.2	59.1
67.5	6.5%	12.7	56.4
90.0	4.6%	10.5	61.3
112.5	3.5%	9.1	61.3
135.0	4.0%	9.3	63.5
157.5	4.8%	10.1	57.0
180.0	5.4%	11.9	68.7
202.5	8.0%	12.9	46.5
225.0	10.3%	13.0	46.1
247.5	8.5%	12.1	51.4
270.0	5.4%	10.8	51.9
292.5	4.7%	11.4	40.9
315.0	5.1%	13.5	46.1
337.5	3.9%	15.8	49.2

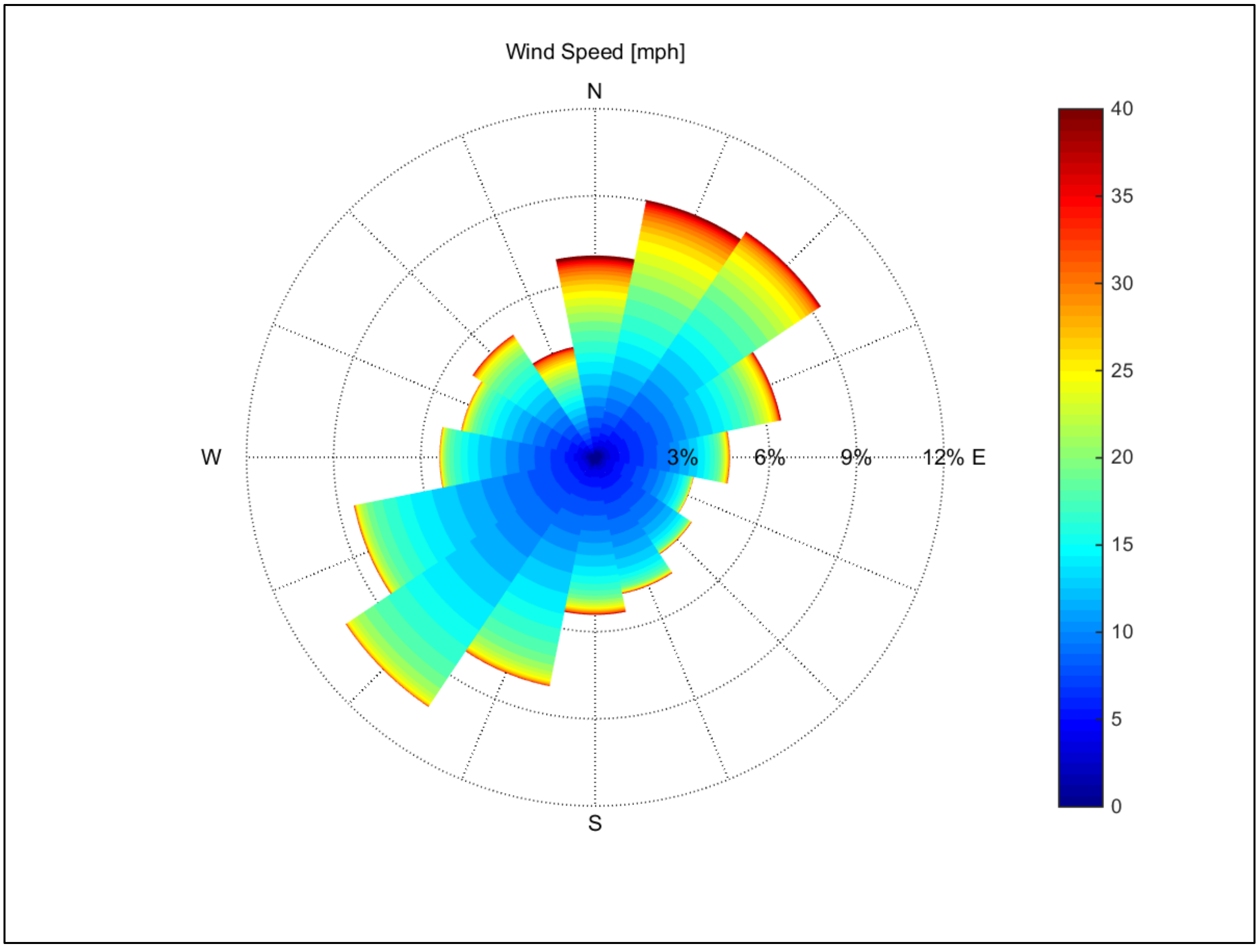

Figure 10. Directional Wind Statistics

Extreme wind events were extracted from the complete record to facilitate the projection of design storm conditions. Individual events were identified by noting the maximum wind speed measured during any five day period. After identifying individual events, extreme events used in the statistical analysis were further defined as events with a wind speed greater than 99.9% of the maximum wind speeds measured during all individual events. A summary of the top ten wind events is provided in Table 9.

Table 9. Extreme Wind Events

Event	Date	Speed (mph)	Direction (deg)
1	September 16, 1999	68.7	174
2	September 18, 2003	63.5	135
3	August 27, 2011	63.3	137
4	May 29, 2000	59.7	21
5	October 27, 1993	59.1	35
6	August 30, 1999	58.8	0
7	January 25, 2000	58.8	0
8	January 28, 1998	52.6	0
9	March 14, 1993	51.9	265
10	November 22, 2006	51.7	47

A wind speed extreme analysis, using extreme wind events identified above, was completed using Log-Linear, Fisher-Tippet Type 1, and Weibull (k=0.75, 1.00, 1.40, and 2.00) distributions to identify design storm conditions. The Log-Linear distribution was found to provide the best fit for wind speed and this distribution was used to estimate wind speeds for selected extreme events, where the extreme events are defined in terms of return periods. The wind speeds for the selected extreme wind events, calculated using the selected model, are detailed in Table 10 and are shown graphically in Figure 11.

Table 10. Extreme Wind Statistics

Return Period (yr)	Speed (mph)
0.25	37.4
0.5	42.3
1	47.3
2	52.3
5	58.8
10	63.8
20	68.8
25	70.4
50	75.4

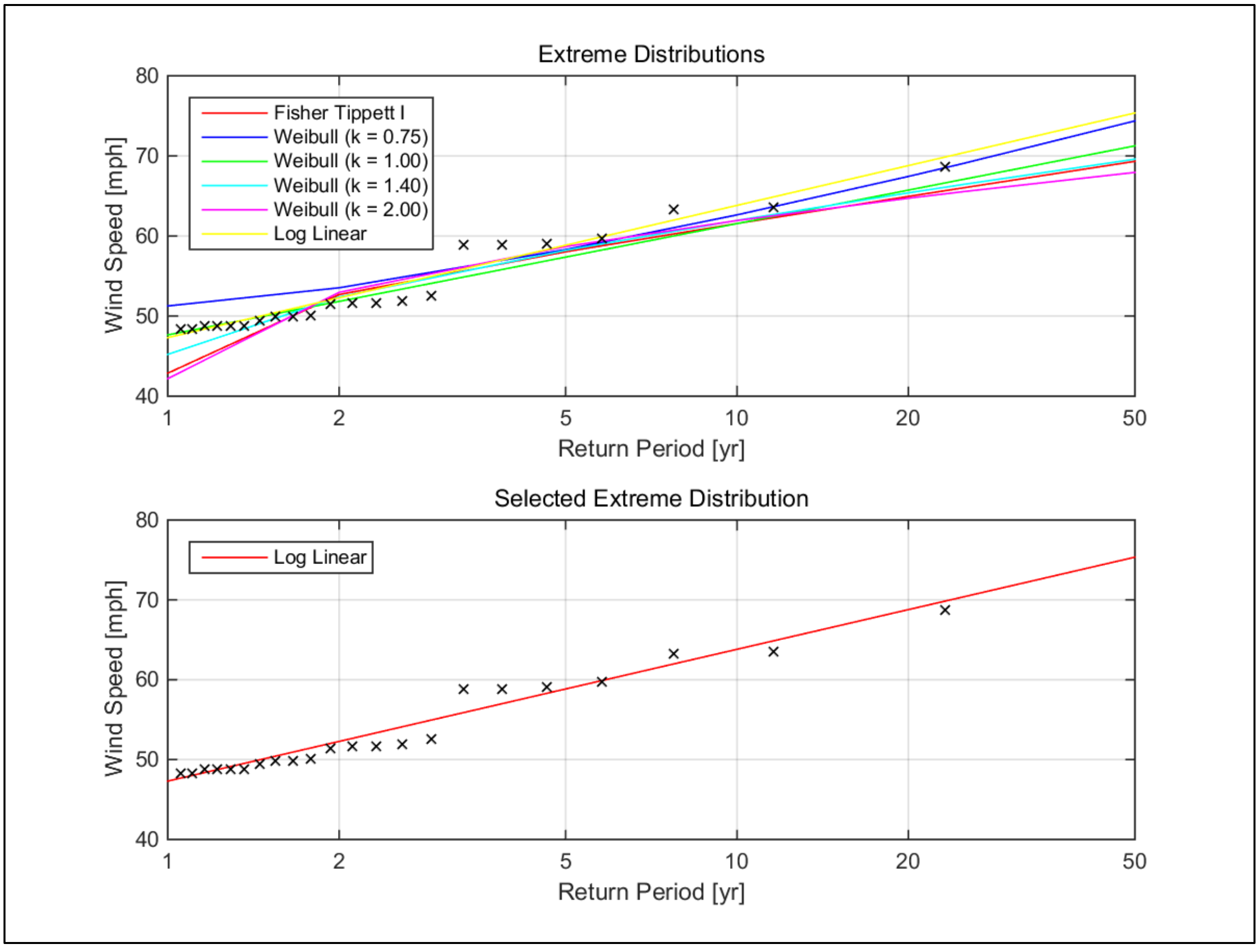

Figure 11. Extreme Wind Statistics

The extent of maximum hurricane wind speed is spatially limited along the storm's track which restricts the ability to measure maximum hurricane wind speed at any stationary location. However, hurricane winds can be estimated indirectly from sources such as ship's logs and reports using the Beaufort wind scale (Jarvinen et al., 1988) and from statistical distributions of hurricane climatological characteristics using physical models of the hurricane wind speed field (USACE, 1985). Jagger and Elsner (2006) estimated extreme hurricane winds for the Northeast Coast of the United States using data from the best-track hurricane database (HURDAT) record. Hurricane wind speed statistics (USACE, 1985) are detailed in Table 11.

Table 11. Hurricane Wind Statistics

Return Period (yr)	Speed (mph)
10	70
25	87
50	97
100	105
2000	134

8.3 Waves

Waves drive the littoral transport of sand via the nearshore energy flux in the longshore direction. As discussed in Section 6.1.1, wave data near the project area were collected at numerous locations offshore of the USACE FRF in Duck, North Carolina. Even though several measured data sets and USACE Wave Information Studies (WIS) and NOAA Wavewatch hindcast data sets are available for a longer duration, wave conditions at the project site were characterized using only data collected at the FRF630 wave gauge. The FRF630 wave gauge best describes wave conditions along the offshore boundary identified in the various model studies completed as part of this project.

The average measured wave height is 3.2 feet with a corresponding period and direction of 8.7 seconds and 74° (ENE), while the maximum measured wave height is 26.7 feet. The largest maximum wave heights occur between August and October during hurricane season. With the exception of tropical storm events, the average monthly wave heights are largest between November and March, with the smallest average waves occurring between June and August. The wave direction varies from the east-northeast during winter months to the east-southeast during summer months. The largest and longest waves under normal conditions come from the east-northeasterly direction band. Monthly wave statistics are provided in Table 12 and are shown pictorially in Figure 12 through Figure 14, while directional wave statistics are detailed in Table 13 and are shown graphically in Figure 15 and Figure 16. The average wave direction is the monthly mean wave direction while the energetic wave direction is the monthly weighted average wave direction, with direction weighted using squared wave height as it is proportional to wave energy.

Table 12. Monthly Wave Statistics

Month	Height (ft) Average	Height (ft) Maximum	Period (s) Average	Period (s) Maximum	Direction (deg) Average	Direction (deg) Energetic
January	3.2	17.5	8.5	20.0	76	61
February	3.7	14.4	8.5	22.2	73	69
March	3.7	16.8	9.0	18.2	80	72
April	3.3	15.2	8.6	17.4	81	70
May	3.0	16.8	8.4	18.2	88	76
June	2.4	11.3	8.4	18.2	96	85
July	2.2	8.6	8.4	20.0	99	84
August	2.6	23.4	8.7	20.0	95	85
September	3.8	26.7	9.4	20.0	87	82
October	3.6	20.7	8.8	18.2	80	75
November	3.7	16.9	8.8	18.2	74	70
December	3.3	15.0	8.6	20.0	75	65
Annual	**3.2**	**26.7**	**8.7**	**22.2**	**84**	**74**

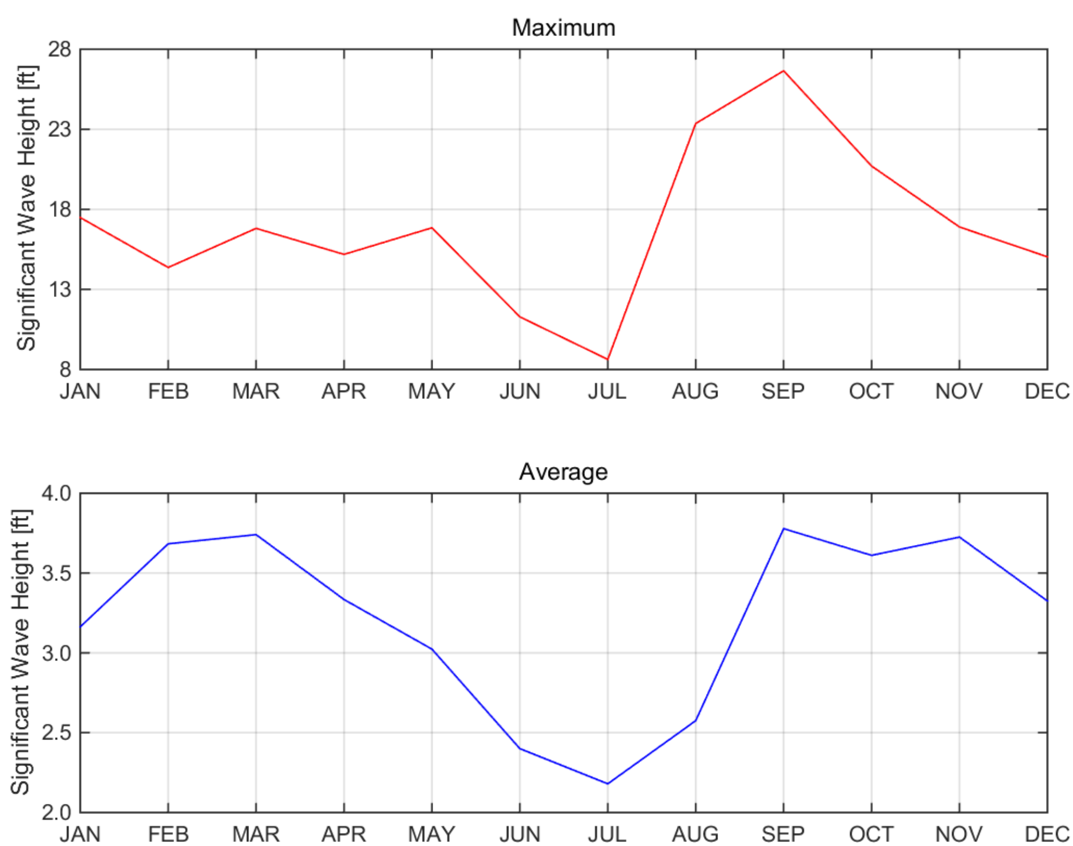

Figure 12. Monthly Wave Height Statistics

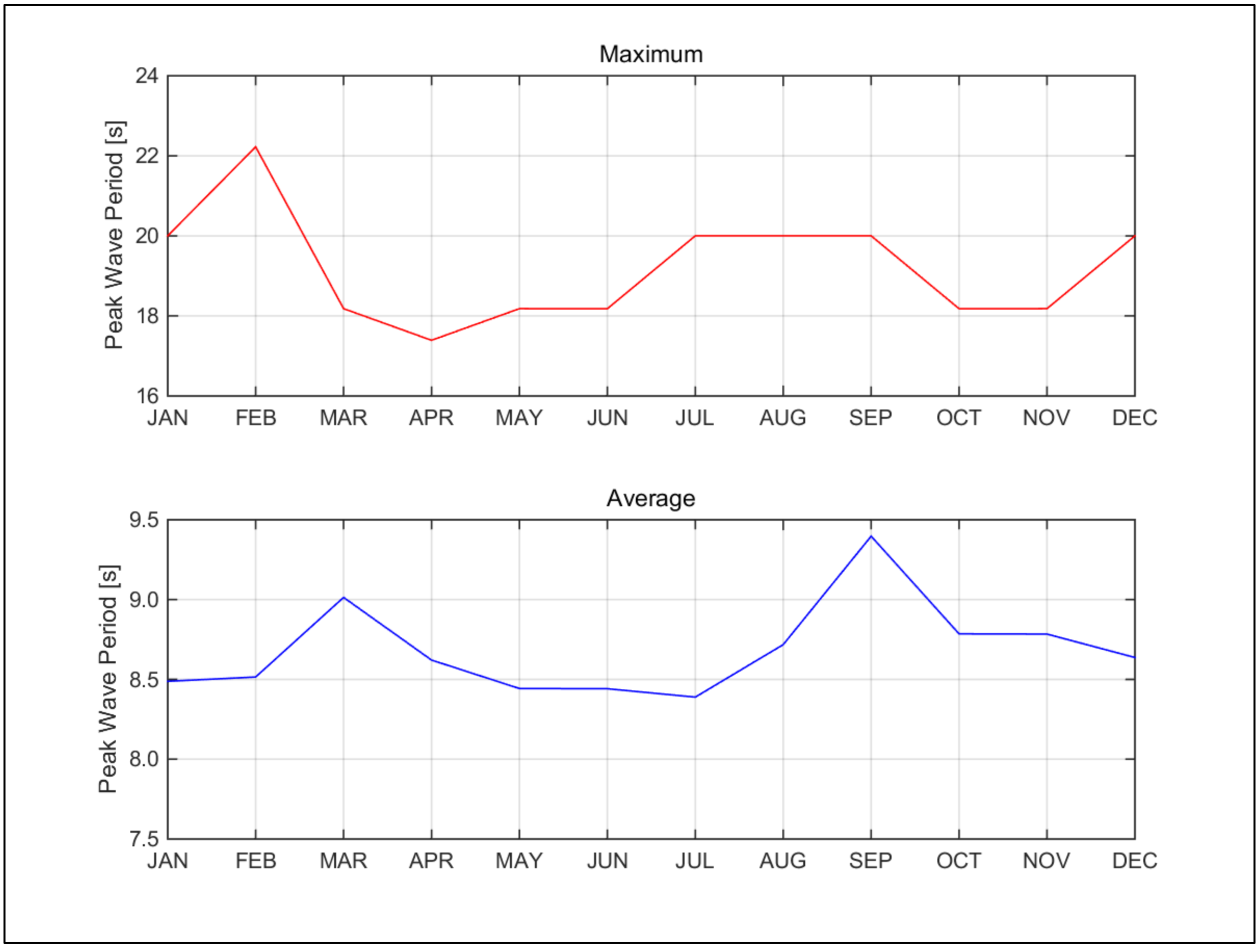

Figure 13. Monthly Wave Period Statistics

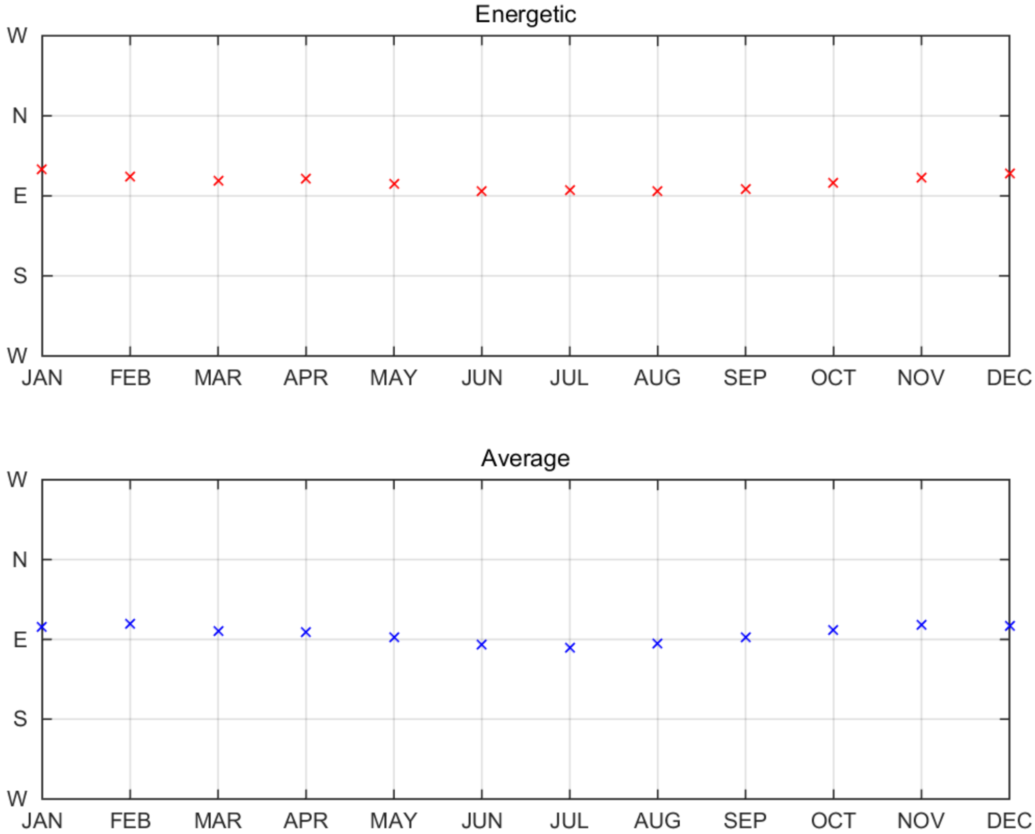

Figure 14. Monthly Wave Direction Statistics

Table 13. Directional Wave Statistics

Direction Bin	Percent Occurrence	Height (ft)		Period (s)	
		Average	Maximum	Average	Maximum
0.0	2.2%	3.3	9.9	4.4	6.8
22.5	6.5%	4.1	14.1	5.4	14.3
45.0	11.7%	4.2	23.4	6.7	22.2
67.5	18.4%	3.8	20.5	9.8	22.2
90.0	28.2%	2.9	20.7	10.1	20.0
112.5	24.8%	2.6	26.7	9.1	20.0
135.0	7.4%	2.5	19.1	6.9	20.0
157.5	0.4%	2.1	7.2	4.4	20.0
180.0	0.0%	1.9	4.1	4.6	17.4
202.5	0.0%	1.7	4.6	2.3	3.7
225.0	0.1%	1.7	3.9	2.4	13.3
247.5	0.0%	1.7	3.8	2.5	13.3
270.0	0.0%	2.6	5.6	2.9	4.0
292.5	0.0%	2.4	3.9	2.8	3.5
315.0	0.0%	2.4	3.9	3.0	4.2
337.5	0.1%	2.3	4.5	3.5	5.0

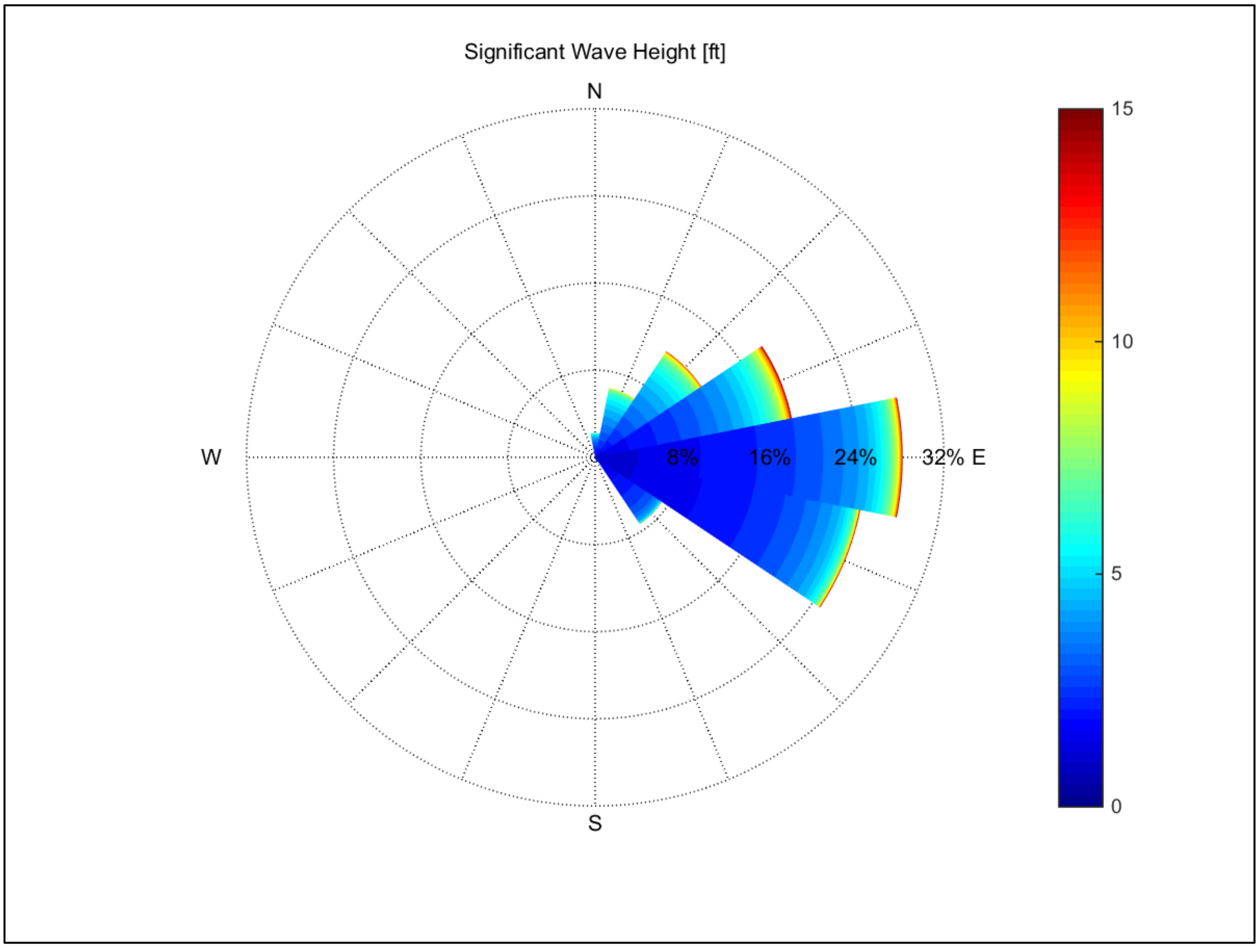

Figure 15. Directional Wave Height Statistics

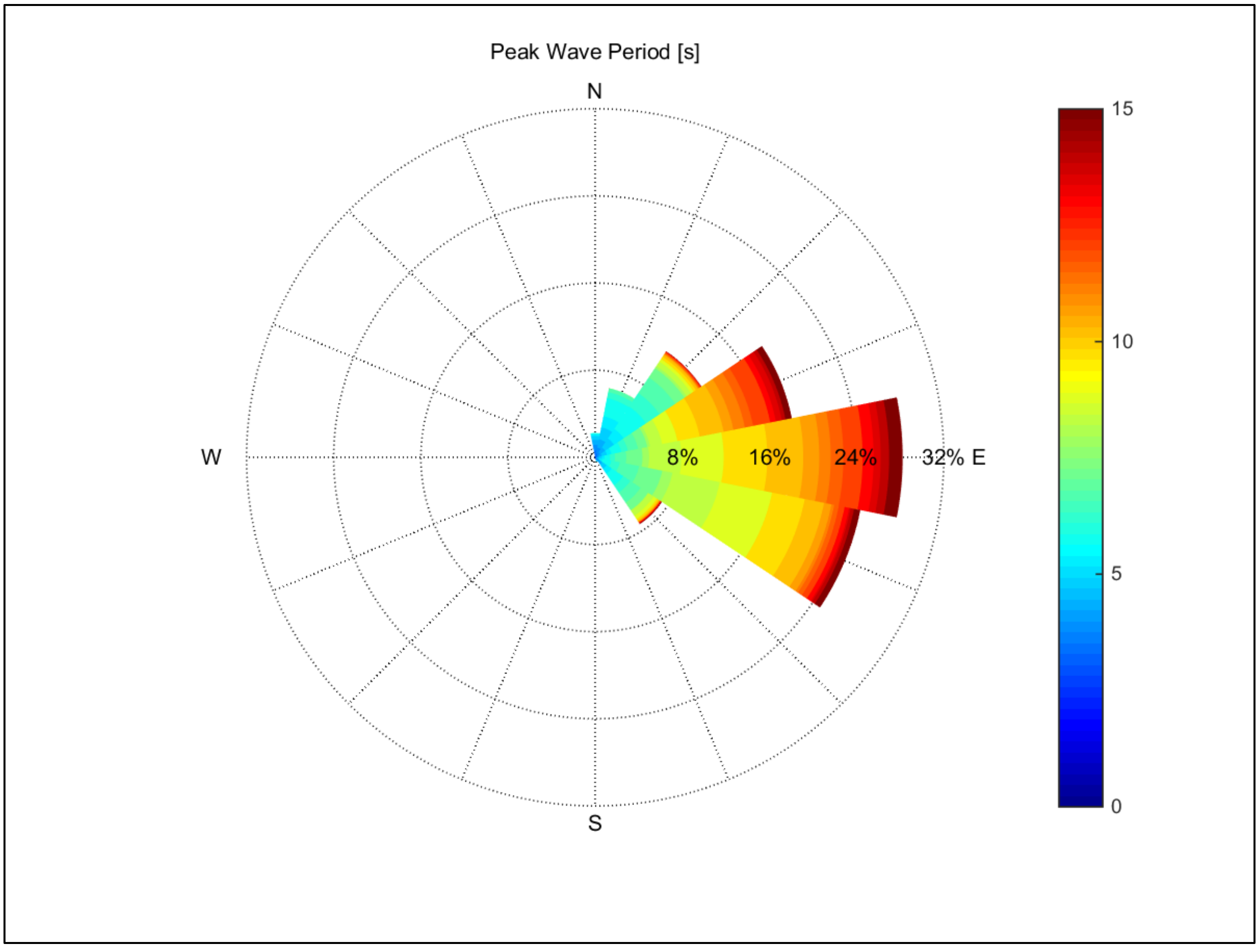

Figure 16. Directional Wave Period Statistics

Extreme wave events were extracted from the complete record to facilitate the projection of design storm conditions. Individual events were identified by noting the maximum wave height measured during any five day period. After identifying individual events, extreme events used in the statistical analysis were further defined as events with a wave height greater than 99.9% of the maximum wave heights measured during all individual events. A summary of the top ten wave events is provided in Table 14.

Table 14. Extreme Wave Events

Event	Date	Height (ft)	Period (s)	Direction (deg)
1	September 18, 2003	26.7	15.4	104
2	August 31, 1999	23.4	11.1	56
3	August 27, 2011	22.9	15.4	122
4	October 29, 2012	20.7	15.4	89
5	January 29, 1998	17.5	12.5	65
6	November 13, 2009	16.9	12.5	56
7	May 8, 2007	16.8	13.8	91
8	March 11, 2004	16.8	11.4	65
9	November 22, 2006	16.8	10.5	74
10	May 29, 2000	16.5	10.0	60

Wave height and period extreme analyses, using extreme wave events identified above, were completed using Log-Linear, Fisher-Tippet Type 1, and Weibull (k=0.75, 1.00, 1.40, and 2.00) distributions to identify design storm conditions. The Log-Linear distribution was found to provide the best fit for both wave height and wave period and these distributions were used to estimate wave heights and wave periods for selected extreme events, where the extreme events are defined in terms of return periods. The wave conditions for the selected extreme wave events, calculated using the selected models, are detailed in Table 15 and are shown graphically in Figure 17 and Figure 18.

Table 15. Extreme Wave Statistics

Return Period (yr)	Height (ft)	Period (s)
0.25	9.6	6.3
0.5	12.2	8.3
1	14.8	10.3
2	17.4	12.3
5	20.9	15.0
10	23.5	17.0
20	26.1	19.0
25	27.0	19.6
50	29.6	21.6

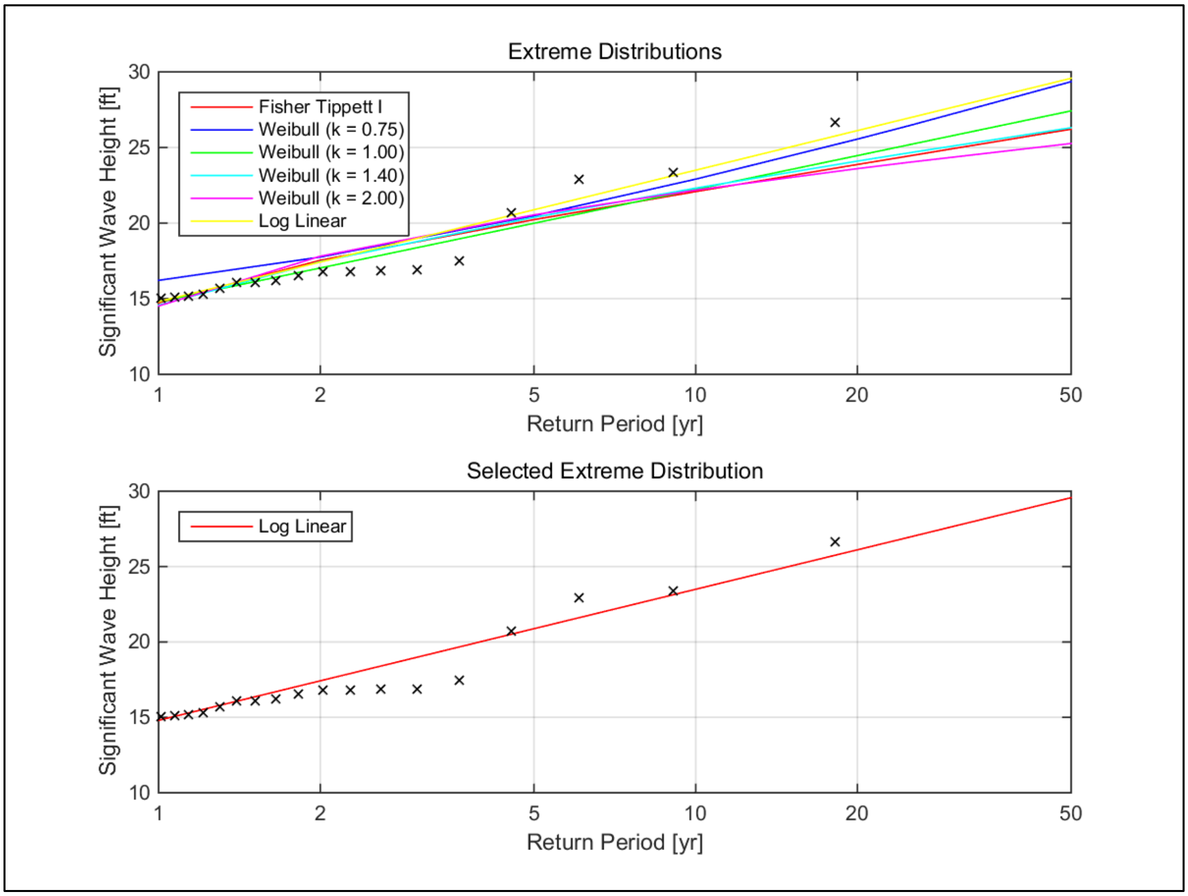

Figure 17. Extreme Wave Height Statistics

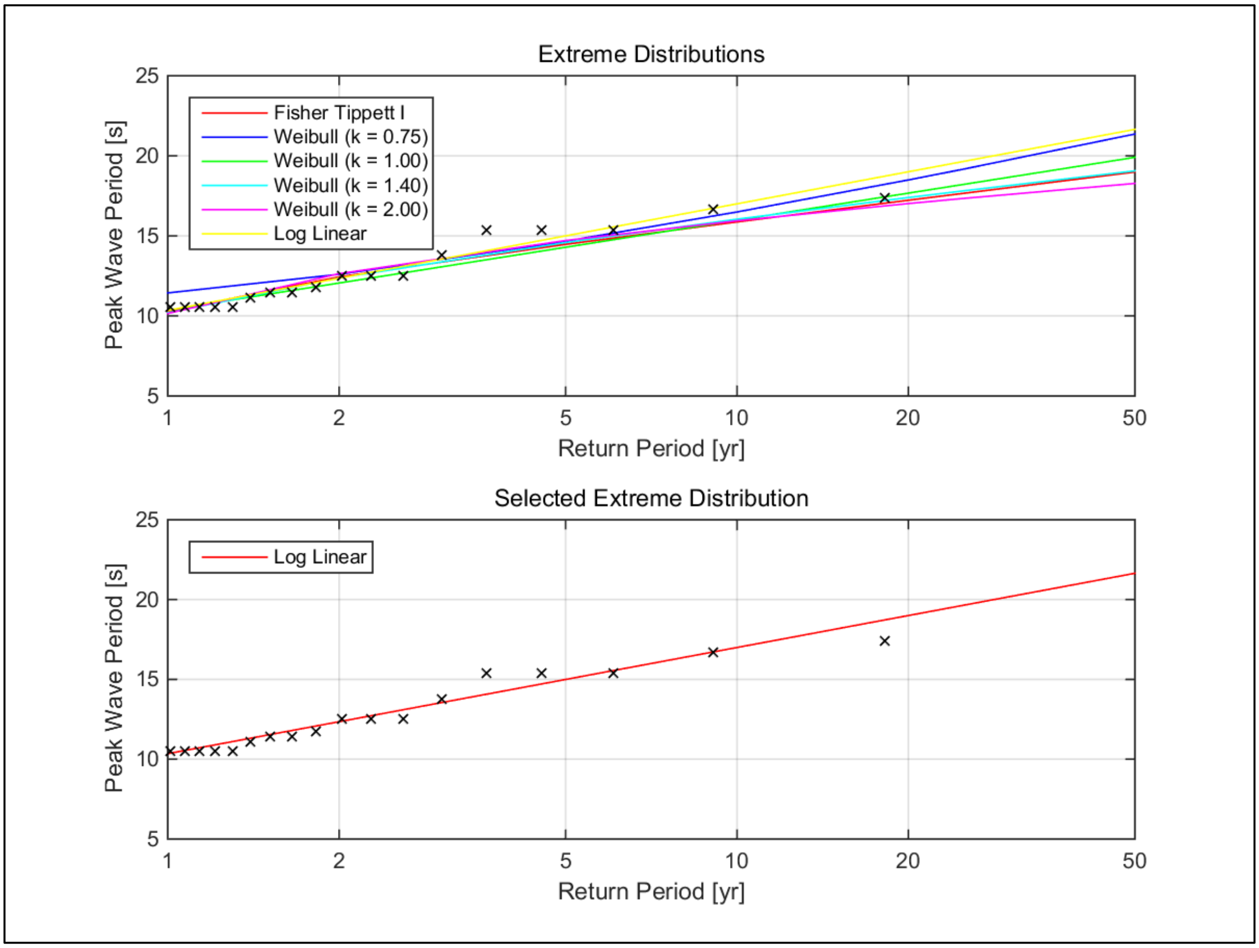

Figure 18. Extreme Wave Period Statistics

8.4 Surge

Storm surge is defined as the rise of the sea surface above its astronomical tide level due to storm forces. The elevation that the storm surge reaches is known as the storm stage. The increased elevation is attributable to a variety of factors, including waves, wind shear stress, and atmospheric pressure. Increased water depth will increase the potential for shoreline recession, long-term erosion, and overtopping from severe waves.

An estimate of storm surge is essential to the development of the design of a shoreline protection project as it is a major component in identifying the dune elevation required to offer the desired level of protection. As discussed in Section 6.1.2, NOAA operates a tide gauge on the offshore end of the USACE FRF pier. Storm surge was calculated from the recorded data by subtracting the predicted water level (astronomical tide level) from the verified water level (measured water level that includes tide and storm surge).

The maximum measured surge height is 4.4 feet. The largest surge events occur in September and October during hurricane season. With the exception of tropical storm events, the largest surge events under typical conditions occur in November, with the smallest surge events occurring in July. Monthly surge statistics are provided in Table 16 and are shown pictorially in Figure 19.

Table 16. Monthly Surge Statistics

Month	Maximum Surge (ft)
January	3.24
February	2.70
March	2.60
April	3.25
May	2.99
June	2.45
July	1.61
August	3.44
September	4.44
October	4.04
November	3.63
December	2.97
Annual	**4.44**

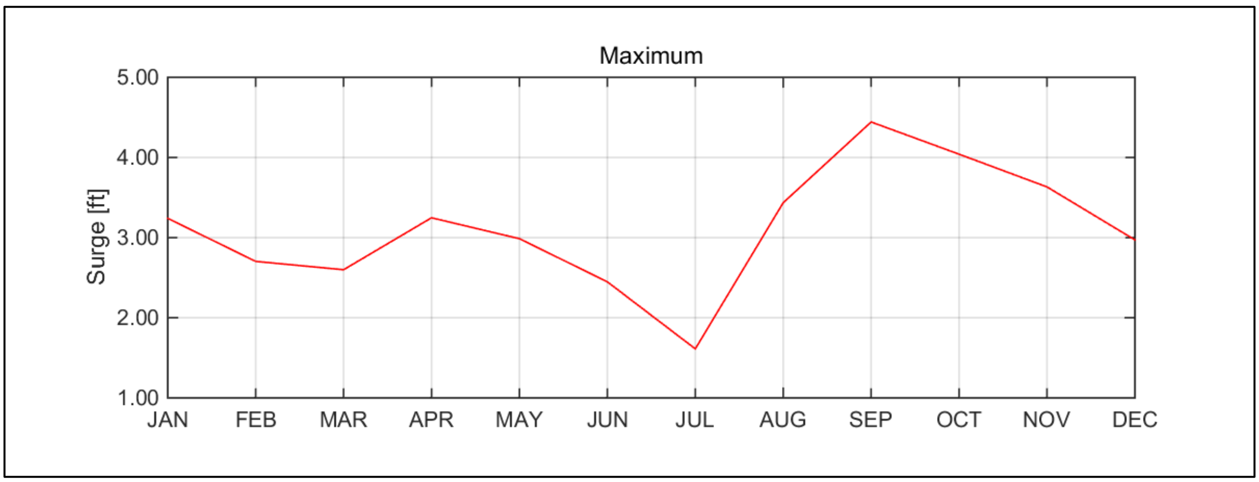

Figure 19. Monthly Surge Statistics

Extreme surge events were extracted from the complete record to facilitate the projection of design storm conditions. Individual events were identified by noting the maximum surge height measured during any five day period. After identifying individual events, extreme events used in the statistical analysis were further defined as events with a surge height greater than 99.9% of the maximum surge heights measured during all individual events. A summary of the top ten surge events is provided in Table 17.

Table 17. Extreme Surge Events

Event	Date	Surge (ft)
1	September 18, 2003	4.4
2	October 29, 2012	4.0
3	November 13, 2009	3.6
4	August 30, 1999	3.4
5	April 13, 1988	3.2
6	January 25, 2000	3.2
7	November 22, 2006	3.2
8	September 27, 1985	3.2
9	October 25, 1982	3.1
10	January 28, 1998	3.0

A surge height extreme analysis, using extreme surge events identified above, was completed using Log-Linear, Fisher-Tippet Type 1, and Weibull (k=0.75, 1.00, 1.40, and 2.00) distributions to identify design storm conditions. The Log-Linear distribution was found to provide the best fit for surge height and this distribution was used to estimate surge heights for selected extreme events, where the extreme events are defined in terms of return periods. The surge conditions for the selected extreme surge events, calculated using the selected model, are detailed in Table 18 and are shown graphically in Figure 20.

Table 18. Extreme Surge Statistics

Return Period (yr)	Surge (ft)
0.25	1.6
0.5	1.9
1	2.3
2	2.7
5	3.2
10	3.6
20	4.0
25	4.1
50	4.5

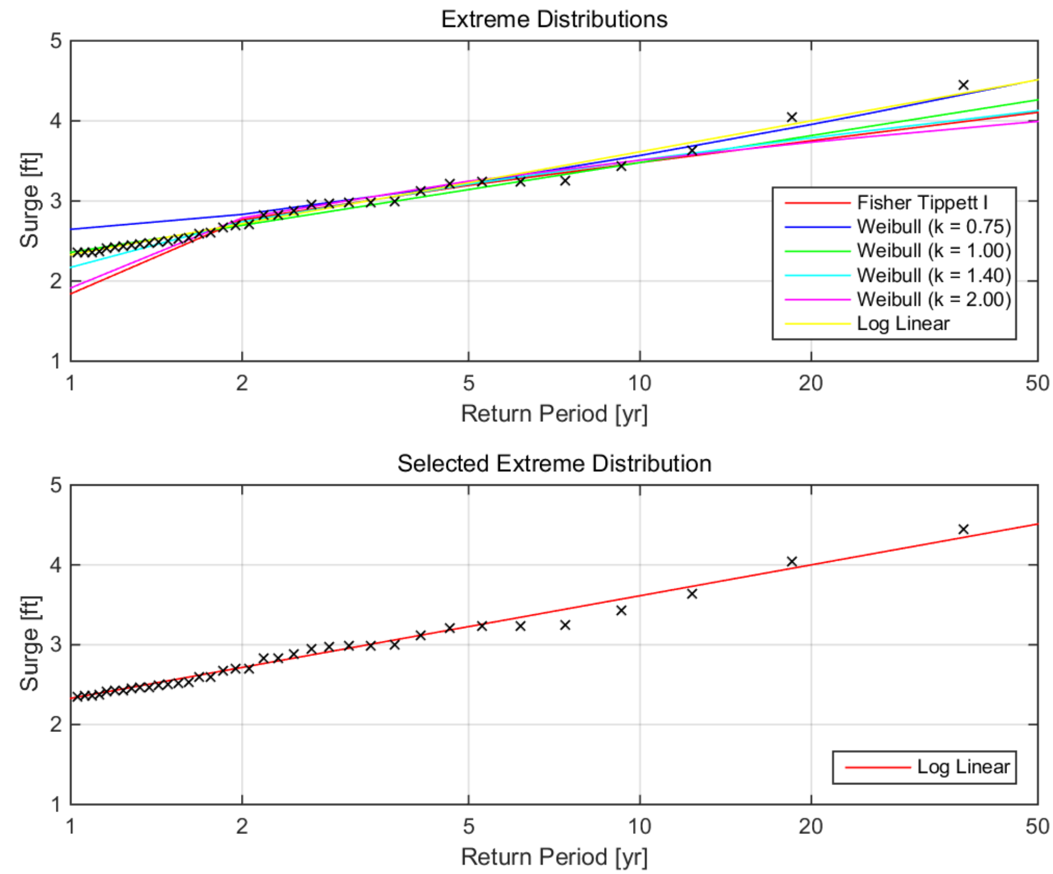

Figure 20. Extreme Surge Statistics

The Federal Emergency Management Agency (FEMA) completed a Flood Insurance Study (FIS) for Dare County as part of the North Carolina Floodplain Mapping Program (FEMA, 2006). According to the report, the dominant source of flooding in Dare County is storm surge generated in the Atlantic Ocean by tropical storms and hurricanes. Storm stage elevations identified in the study for the Town of Duck are shown in Table 19. These storm stage elevations reflect the stillwater elevations resulting from tide and wind setup but do not include contributions from wave action effects.

Table 19. FEMA Storm Stage Elevations

Return Period (yr)	Stage (ft, NAVD)
10	4.8
50	6.2
100	6.8
500	8.6

9 RELATIVE SEA LEVEL RISE

Relative sea level rise is a factor to be considered in any coastal project design. Relative sea level rise consists of two components:

1. Global Effects. Global effects are defined as eustatic sea level change or the global change in oceanic water level.

2. Local Effects. Local effects include the combination of vertical land movement and oceanographic effects.

Estimates of relative sea level rise and its global and local components appear in a number of sources including the North Carolina Coastal Hazards Science Panel (2015), NOAA (2013 and 2014), and the Intergovernmental Panel on Climate Change (2007 and 2013).

9.1 Relative Sea Level Rise

Tide records provide information on relative sea level rise as they measure sea level relative to local land elevations. These records include components that vary spatially and temporally, such as oceanographic processes, vertical land movement, and changes in global sea level (NOAA, 2010). Therefore, the trends derived from these data sets describe relative sea level change (Zervas, 2009).

The rate of relative sea level rise was calculated independently using verified high and low water level data collected at Duck between 1979 and 2014. The average rate of relative sea level rise was approximated by performing a first-order linear regression of the daily high/low water level data. This analysis suggests that between 1979 and 2014 the sea level increased at an average rate of approximately 0.01546 feet per year. Figure 21 shows the measured data and the results of the relative sea level rise analysis. This same analysis was completed using monthly and yearly mean sea levels over the same analysis period which suggests a relative sea level rise rate of 0.01522 feet per year and 0.01515 feet per year, respectively. These results differ from the value calculated using the daily high and low water level data by approximately 1.6% and 2.0%, respectively. Regardless of the method selected, the above analysis suggests that the sea level at Duck is rising at a relative rate of roughly 1.5 feet per century.

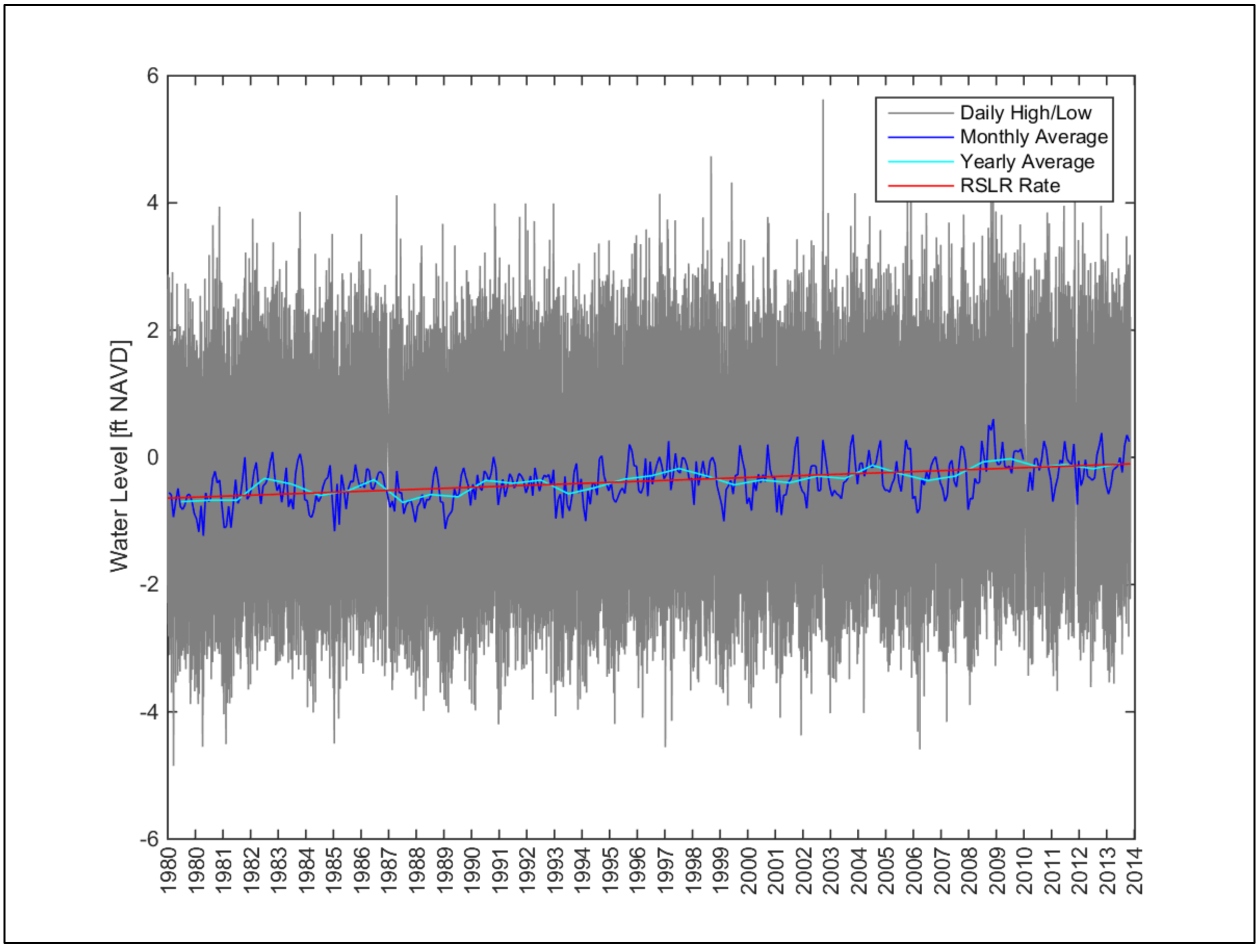

Figure 21. Independent Sea Level Rise Analysis

The independent relative sea level rise analysis discussed above is rudimentary in form and does not take into account regular seasonal fluctuations due to coastal ocean temperatures, salinities, winds, atmospheric pressures, and ocean currents. NOAA (2014) included the above mentioned factors in their analysis and estimated that the sea level at Duck is rising at a relative rate of 0.01499 feet per year with a 95% confidence interval of 0.002756 feet per year. These estimates were based on data collected at Duck between 1978 and 2013. Plots in Figure 22 show the monthly mean sea level and trend without seasonal fluctuations, the inter-annual variations without the seasonal fluctuations and linear sea level trend, and the average seasonal fluctuations. Therefore, regardless of the analysis technique, the sea level at Duck is rising at a relative rate of roughly 1.5 feet per century.

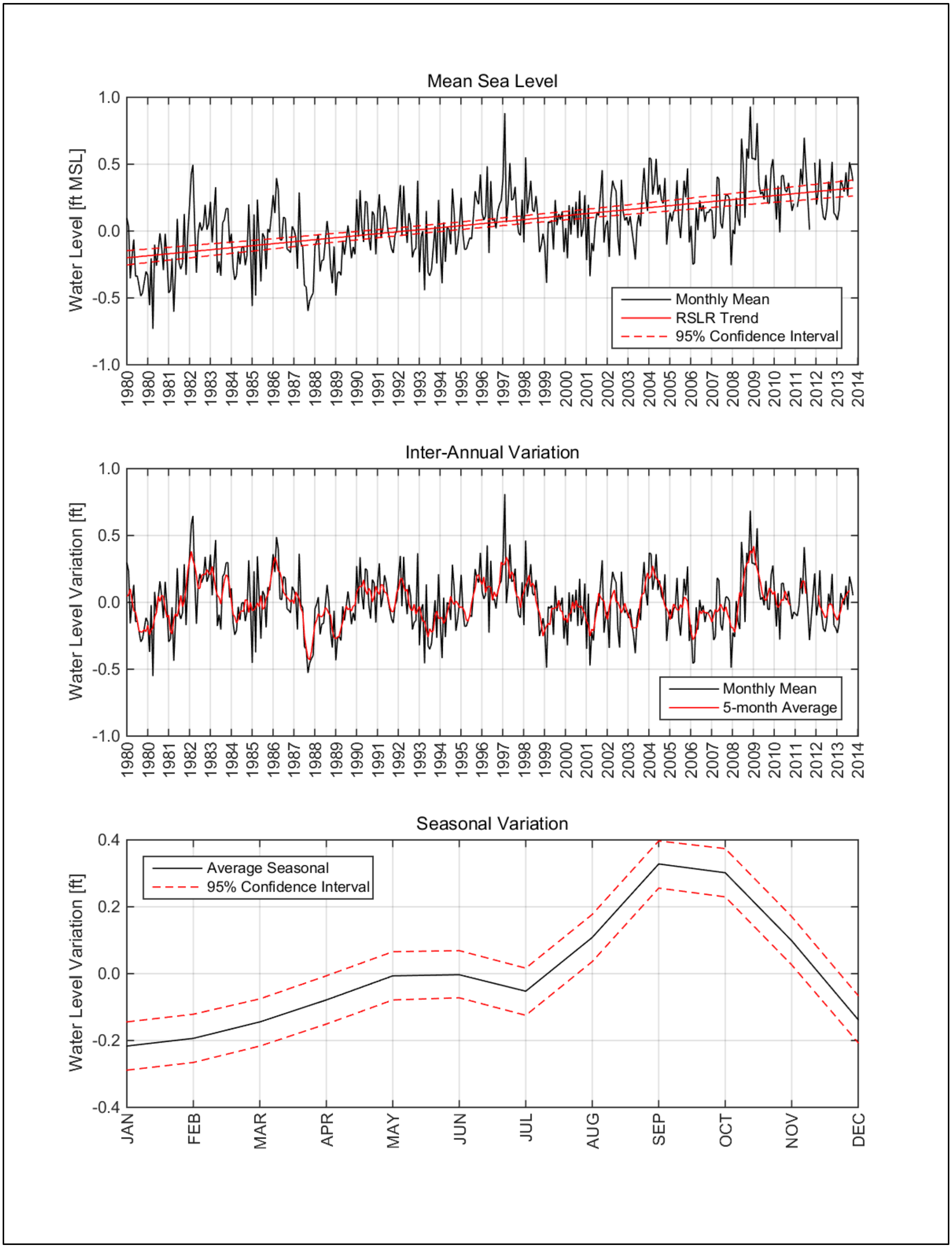

Figure 22. NOAA Sea Level Rise Analysis

9.2 Global Effects

Long term tide records are a primary source for estimating global or eustatic sea level rise (Church and White, 2011). Only records with long term high quality data collected along an open coast with relatively stable land motion are used in these analyses. Global sea level rise estimates are then made after the records have been adjusted for vertical global glacial isostatic adjustment (Douglass et al, 2001). The typically accepted rate for eustatic sea level rise during the past century is 0.005577 feet per year, or 0.56 feet per century (IPCC, 2007).

The eustatic sea level rise rate recommended by the USACE is as reported by the Intergovernmental Panel on Climate Change (IPCC). The IPCC recently updated their analysis of present and projected eustatic sea level rise (IPCC, 2013). According to the IPCC report, the global mean rate of eustatic sea level rise has increased from 0.005577 feet per year (0.56 feet per century or 1.7 mm/yr) between 1901 and 2010 to 0.01050 feet per year (1.05 feet per century or 3.2 mm/yr) between 1993 and 2010. The IPCC modeling indicates that thermal expansion, glacier melting, and land water storage explain 65% of the global mean sea level rise between 1901 and 1990 and 90% of the global mean sea level rise between 1971 and 2010, suggesting that the greater rate of sea level rise since 1993 is not a part of a natural oscillation.

The rate of eustatic sea level rise is expected to increase. The IPCC (2013) projects that sea level rise will be greater than previously reported, primarily due to improved modeling of land-ice contributions. All process based models employed in the IPCC study suggest that the rate of sea level rise between 2081 and 2100 is expected to be roughly twice the rate of sea level rise between 1986 and 2005. Moreover, sea level change is expected to observe a regional pattern with some places experiencing significant deviations from the global mean.

9.3 Local Effects

The local contribution to relative sea level rise is typically estimated by subtracting the global rate of mean sea level rise from the local rate of relative sea level rise (USACE, 2009). Using the generally accepted eustatic sea level rise rate of 0.005577 feet per year (1.7 mm/yr) and the NOAA reported relative sea level rise rate at Duck of 0.01499 feet per year (4.57 mm/yr) yields a 0.009416 feet per year (2.87 mm/yr) local contribution. However, considering that the IPCC recently reported an increase in the global rate of mean sea level rise between 1993 and 2010, the eustatic sea level rise rate employed in the previous calculation may be an underestimate. To account for the updated eustatic sea level rise rate, the relative sea level rise rate at Duck was calculated between 1993 and 2010; the relative sea level rise rate at Duck calculated using monthly mean tide data between 1993 and 2010 is roughly 0.01784 feet per year. Using the eustatic sea level rise rate during the same period (0.01050 feet per year), the resulting local contribution at Duck would be roughly 0.007341 feet per year. These calculated rates differ by roughly 0.002075 feet per year, which is greater than a 20% difference. However, for the remainder of this report it will be assumed that the local contribution to relative sea level rise is 0.0094 feet per year as referenced in the North Carolina 2015 Sea Level Rise Assessment (North Carolina Coastal Hazards Science Panel, 2015).

9.4 Sea Level Rise Projections

Over a 30 year project life, the rate of eustatic sea level rise is expected to increase. The North Carolina Coastal Hazards Science Panel (2015) investigated three different scenarios that were used to project the extent of relative sea level rise over the next 30 years. The baseline scenario assumes sea level will continue to rise at its current rate, while IPCC (2013) scenarios were used to project the extent of relative sea level rise when assuming eustatic sea level rise acceleration. The baseline scenario employs NOAA's (2014) calculated relative sea level rise rate of 0.01499 feet per year ± 0.002756 feet per year, which suggests relative sea level will increase 5.4 inches ± 1 inch in 30 years at Duck. Considering IPCC (2013) scenario RCP 2.6 combined with vertical land movement, relative sea level at Duck would increase 7.1 inches ± 2.3 inches over the next 30 years. Finally, combining vertical land movement and assuming IPCC (2013) scenario RCP 8.5, relative sea level would increase 8.1 inches ± 2.5 inches in 30 years at Duck. A summary of the projected increase in sea level and associated rate for all scenarios over the 30 year project life is provided in Table 20; the low acceleration allows the rate of sea level rise to be treated as a constant, so the average rate of sea level rise can be calculated by dividing the total sea level rise by the analysis period length. However, considering that periodic nourishment of the project is likely, the existing relative sea level rise rate of 0.015 feet per year (0.18 inches per year) will be assumed for all project calculations. The table below is provided to show that though the global mean sea level may be accelerating at a low rate, the total change over the course of 30 years is considerable, even when assuming no acceleration.

Table 20. Projected Relative Sea Level Rise During 30 Year Project Life

	Total Change (inches)			Annual Change (inches/year)		
Scenario	**Global**	**Local**	**Relative**	**Global**	**Local**	**Relative**
Baseline	3.6 ± 1.0	1.8	5.4 ± 1.0	0.12 ± 0.03	0.06	0.18 ± 0.03
RCP 2.6	5.3 ± 2.3	1.8	7.1 ± 2.3	0.18 ± 0.08	0.06	0.24 ± 0.08
RCP 8.5	6.3 ± 2.5	1.8	8.1 ± 2.5	0.21 ± 0.08	0.06	0.27 ± 0.08

10 SHORELINE CHANGES

A shoreline change analysis was completed to assess shoreline advance and recession along the project area. The shoreline is typically defined as a specified elevation contour. For this study, the shoreline was defined as the Mean High Water (MHW) contour, which represents the +1.2 feet NAVD elevation (as identified Section 8.1). Shoreline change is calculated by comparing shoreline position along shore perpendicular transects. Typically, shoreline change is then annualized to describe recession and advance rates. Regardless of whether total or annual shoreline changes are described, positive shoreline change denotes advance while negative shoreline change indicates recession.

CPE (2013) conducted a shoreline change analysis as part of the Erosion and Shoreline Management Feasibility Study for the Town of Duck. This analysis used LIDAR data collected by various Federal agencies including the USGS between 1996 and 2011 and a 2012 CLARIS survey obtained by the USACE FRF. Subsequently, a new LIDAR survey was conducted by the USGS following Hurricane Sandy, which affected the project area late October 2012. This newly acquired data was used to update the CPE (2013) shoreline change analysis.

Shoreline changes were evaluated using various LIDAR data sets described in Section 6.3. The specific LIDAR data used in the analysis presented in this report includes the following: October 1996, November 2011, and November 2012 (post-Sandy). The MHW position for each survey was identified along shore perpendicular transects spaced at 100 foot intervals along the study area. The transects were positioned from approximately 1 mile south to 1 mile north of the Town's incorporated limits. The southern and northern borders for the Town of Duck are at transects 109 and 415, respectively. The FRF pier is represented by transect 229. A plan view of the study area showing each LIDAR generated MHW shoreline is provided in Appendix C.

Shoreline change is defined as the distance between shoreline positions along the identified transects. Positive shoreline change is indicative of shoreline advance while negative shoreline change is indicative of shoreline retreat. Shoreline change is also provided in an annualized form by dividing the shoreline change by the time period (number of years) between survey events. The average shoreline change within the Town limits between 1996 and 2011 was -7 feet, which is equivalent to an average annual shoreline change of -0.4 feet per year. During Hurricane Sandy the shoreline receded an average of 17 feet (measured change between 2011 and 2012 LIDAR surveys). As a result of this event, the long-term (1996 to 2012) average annual shoreline change within the Town limits increased to -1.5 feet per year.

Shoreline changes are presented in Table 21. The area north of the FRF pier (D-01 to D-21) experienced greater shoreline recession between the 2011 and 2012 LIDAR surveys than the area south of the pier (D-22 to D-34), though the shoreline change trends along the study area were similar to those identified and discussed in the Feasibility Study (CPE, 2013). Total, Hurricane Sandy, and annual shoreline changes are shown graphically in Figure 23.

Table 21. Shoreline Change

Profile		Shoreline Change (ft)		Shoreline Change (ft/yr)	
From	**To**	**1996-2011**	**1996-2012**	**1996-2011**	**1996-2012**
D-01	D-02	25	8	1.7	0.5
D-02	D-03	35	-3	2.3	-0.2
D-03	D-04	31	-8	2.1	-0.5
D-04	D-05	-13	-58	-0.8	-3.6
D-05	D-06	-8	-66	-0.6	-4.1
D-06	D-07	-26	-42	-1.7	-2.6
D-07	D-08	10	-11	0.7	-0.7
D-08	D-09	36	7	2.4	0.4
D-09	D-10	47	3	3.1	0.2
D-10	D-11	53	18	3.5	1.1
D-11	D-12	41	14	2.8	0.8
D-12	D-13	6	-19	0.4	-1.2
D-13	D-14	-16	-45	-1.1	-2.8
D-14	D-15	-44	-76	-2.9	-4.7
D-15	D-16	-82	-102	-5.5	-6.4
D-16	D-17	-100	-135	-6.7	-8.4
D-17	D-18	-88	-112	-5.9	-7.0
D-18	D-19	-64	-76	-4.3	-4.7
D-19	D-20	-48	-60	-3.2	-3.8
D-20	D-21	-23	-30	-1.5	-1.9
D-21	D-22	-29	-29	-1.9	-1.8
D-22	D-23	13	18	0.8	1.1
D-23	D-24	13	18	0.8	1.1
D-24	D-25	6	5	0.4	0.3
D-25	D-26	-31	-20	-2.1	-1.2
D-26	D-27	-31	-34	-2.1	-2.1
D-27	D-28	-8	-6	-0.6	-0.4
D-28	D-29	30	25	2.0	1.6
D-29	D-30	30	30	2.0	1.8
D-30	D-31	21	22	1.4	1.4
D-31	D-32	-14	-2	-0.9	-0.1
D-32	D-33	-19	-17	-1.3	-1.1
D-33	D-34	-2	-13	-0.2	-0.8
D-01	**D-34**	**-7**	**-24**	**-0.4**	**-1.5**

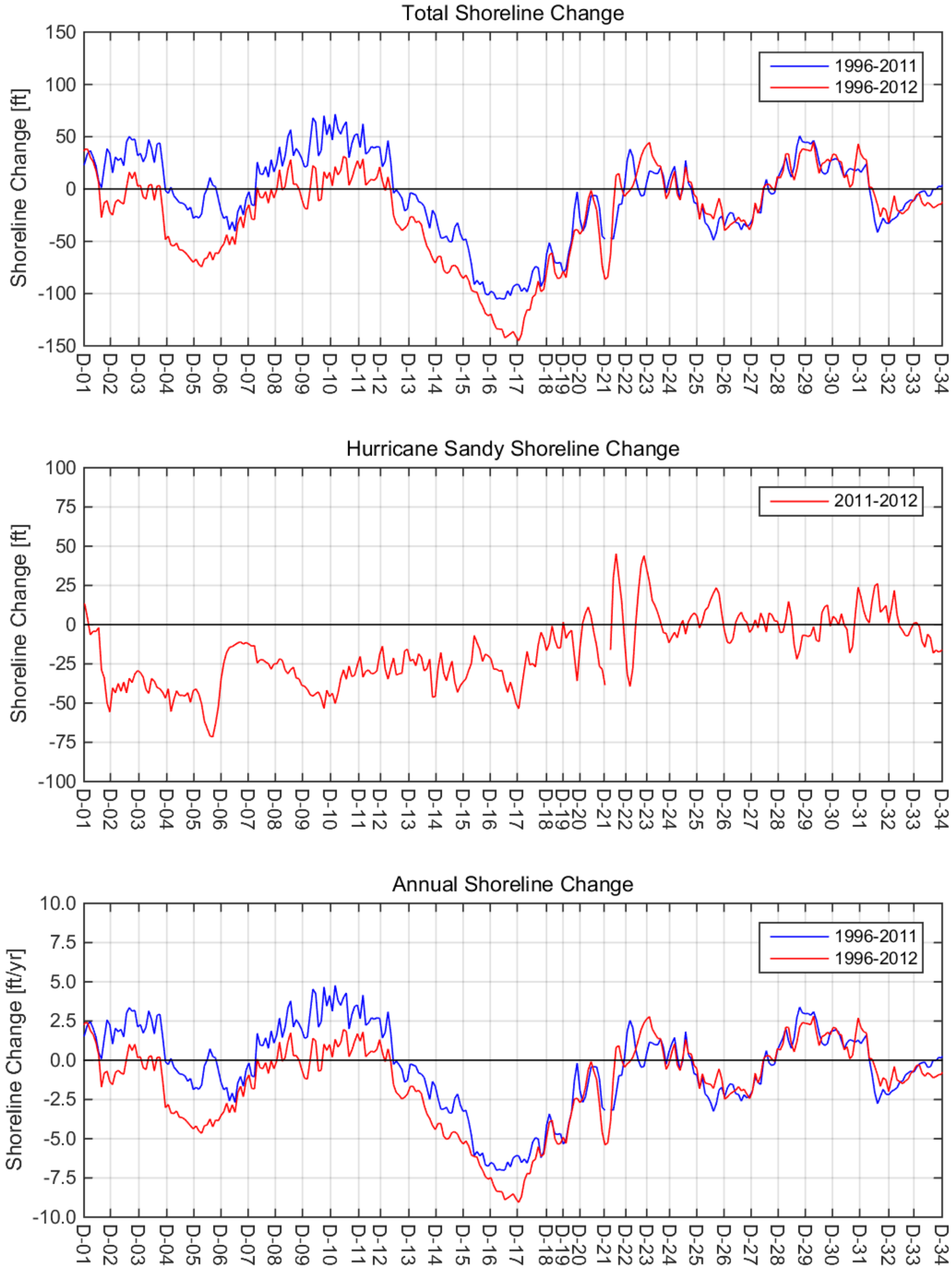

Figure 23. Shoreline Change

11 ACTIVE PROFILE HEIGHT

The active profile is defined as the part of the beach profile where sediment motion occurs. Typically, the active profile extends from the berm crest to the depth of closure, where the depth of closure is defined as the depth at which there is no net cross-shore movement of sediment. Therefore, the active profile height is the difference between the upper and lower limits of the active profile. Multiplying the active profile height by shoreline change (and alongshore distance between profiles) is a method used to estimate volume changes from shoreline changes.

Hallermeier (1978) and Birkemeier (1985) developed empirical equations to estimate the depth of closure based on the characteristics of an extreme wave event. Both of their equations suggest that the depth of closure can be calculated using the maximum significant wave height (and associated period) that is exceeded 12 consecutive hours within the record employed. The maximum significant wave height, and associated period, exceeded 12 consecutive hours were identified for each yearly wave record using data collected at the FRF630 wave gauge. Annual events and calculated closure depths are summarized in Table 22. Considering event variability, an average depth of closure is typically used for descriptive purposes. Using an average wave height of 12.8 feet with an associated period of 12.0 seconds yields closure depths of 26.8 feet and 20.3 feet for Hallermeier and Birkemeier, respectively.

Table 22. Depth of Closure Calculations

	Event Date		Height	Period	Depth of Closure (ft)	
Year	**Month**	**Day**	**(ft)**	**(s)**	**Hallermeier**	**Birkemeier**
1997	Oct	20	8.6	11.1	18.4	14.0
1998	Jan	29	11.8	14.3	25.5	19.5
1999	Aug	31	15.8	10.5	31.2	23.6
2000	May	30	12.5	10.0	25.1	19.0
2001	Mar	21	11.0	11.1	23.0	17.5
2002	Sep	11	8.1	7.1	15.8	11.9
2003	Sep	18	18.7	15.4	39.6	30.1
2004	Mar	11	12.7	12.5	26.7	20.3
2005	Apr	16	13.6	13.8	28.9	22.0
2006	Apr	30	11.2	12.5	23.9	18.2
2007	May	07	15.3	15.4	32.7	25.0
2008	Sep	25	12.9	11.4	26.7	20.3
2009	Nov	12	13.6	9.8	26.9	20.4
2010	Nov	12	13.5	15.4	29.1	22.2
2011	Aug	27	14.1	15.4	30.4	23.2
2012	Oct	28	17.3	13.3	35.9	27.3
2013	Mar	07	10.6	10.5	22.1	16.8
2014	Feb	13	8.6	7.1	16.5	12.4
	Average		**12.8**	**12.0**	**26.8**	**20.3**

Profile inspection is another method that can be used to estimate the upper and lower limits of the active profile. As discussed in Section 6.3, profile data was collected along the Duck shoreline before (November 2011) and after (September 2013) Hurricane Sandy. Comparison of these profile surveys suggests that the profiles close at an elevation between -20 and -30 feet

NAVD, with a majority of the profiles closing around -25 feet NAVD. Also of note was the presence of a berm platform found between elevations of +4 and +8 feet NAVD. Example profiles that show the upper (berm platform) and lower (depth of closure) limits of the active profile are provided in Figure 24.

Generally, it is best to define the limits of the active profile using profile data. However, limits of the active profile can change depending on when the profiles were surveyed. If the profiles were surveyed immediately after a storm, the berm may not be as apparent or may be misrepresented due to erosion occurring along the dune face. Also, comparison of profiles collected after a long period of calm wave activity may result in the appearance of profile closure at a shallower depth. Considering the limits of the active profile are to be defined to identify the extent of the profile experiencing sediment movement, it is best to compare as many profile surveys as possible. Comparing multiple surveys along the same transect assists with identifying both the upper and lower limits of the active profile. Often times this data may not be available, but results of similar analyses may be available in reports or be accepted values used in specific regions. If regional or historic data is available, this should be considered and only contested if sufficient data is available and the rationality for use of different values is justified and can be supported.

Considering the limited profile data collected along the entirety of the Town's shoreline, the depth of closure was defined using Hallermeier's (1978) and Birkemeier's (1985) equations. In an effort to remain conservative yet provide a reasonable estimate of the depth of closure, Hallermeier's (1978) and Birkemeier's (1985) values were averaged to define the depth of closure for the project area. This approach was employed even though Birkemeier's (1985) equation was developed using profile data collected at the USACE FRF, as it suggested a shallower depth of closure than that calculated using Hallermeier's (1978) equation. Applying the average closure depth of 23.6 feet at Mean Sea Level (-0.4 feet NAVD) was used to define a -24 feet NAVD depth of closure elevation. Therefore, for this study, the active profile extends from -24 to +6 feet NAVD.

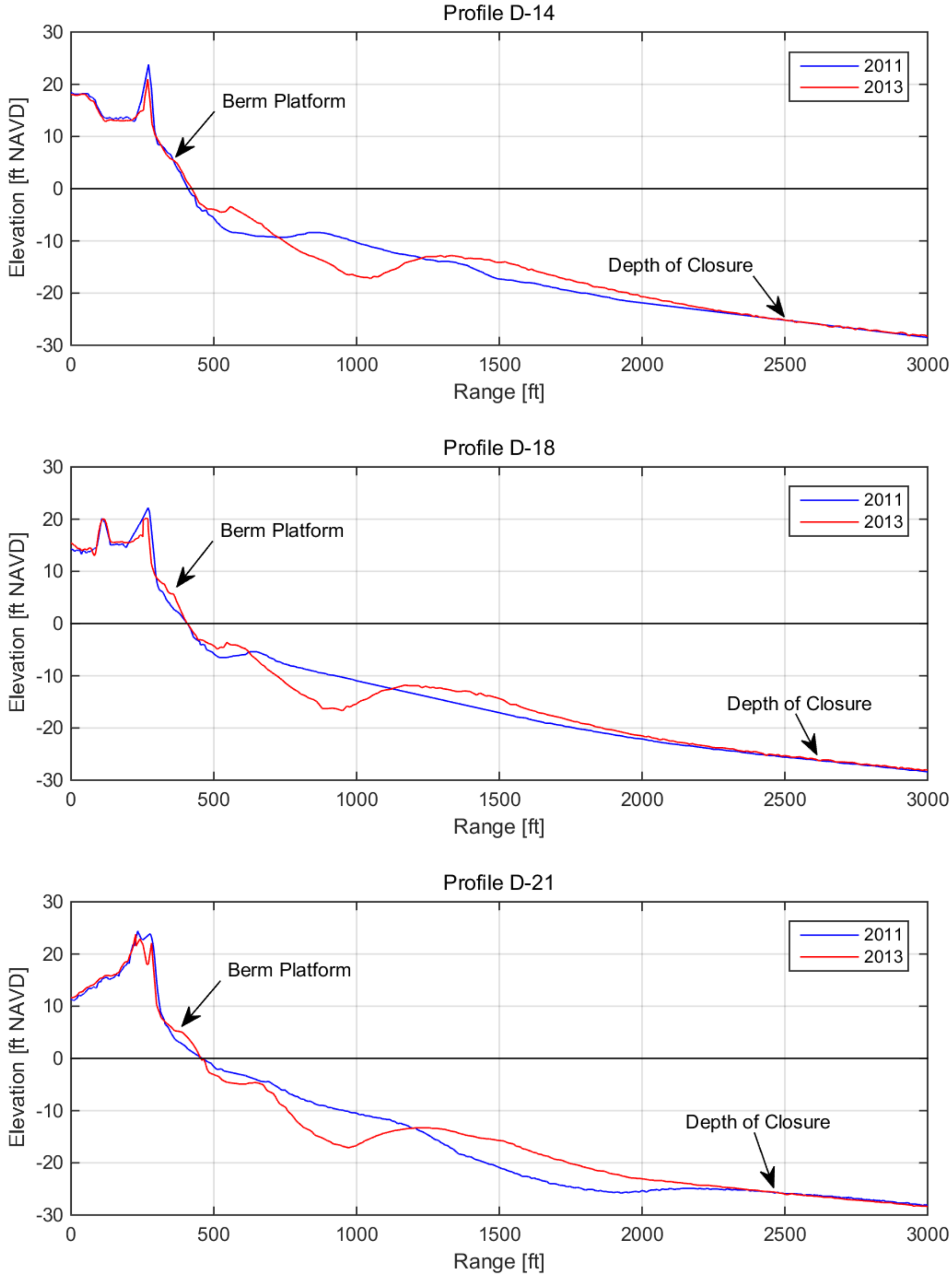

Figure 24. Active Profile Examples

12 VOLUME CHANGES

A volume change analysis was completed to assess erosion and accretion within the project area. Volume change can be calculated by comparing surfaces, profiles, and shorelines. Typically, the calculated volume change is then annualized to describe erosion and accretion rates. Regardless of the volume change calculation technique, negative volume changes denote erosion while positive volume changes indicate accretion.

12.1 Surface Based Volume Change

Surface based volume change is calculated by multiplying the elevation change between surfaces by the surface area. This is accomplished by gridding a specified area and assigning elevations to each grid point. The elevation change at each grid point is then multiplied by the area that the grid point represents to estimate the volume change for the representative grid point. The volume change between surfaces is the summation of the volume changes calculated at each grid point within a desired area. Surface based volume change estimates assume that the elevation of each grid point represents the elevation of the entire area associated with that grid point. Though surface comparisons may be capable of providing the best volume change estimate, accurate quality controlled data is required as measurement error can quickly compound and result in poor estimates. Moreover, surface surveys are typically completed using LIDAR survey techniques, which usually limits the survey to a subaerial extent which miss valuable descriptions of changes occurring below the water surface.

Post Hurricane Sandy LIDAR data was inspected to determine whether it is suitable for use in volume change estimates. When filtering data to remove obstructions above the surface (structures, vegetation, etc), which result in signal returns that misrepresent the bare earth elevation, ground-truthing is essential to ensure that the data is appropriate and meets accuracy requirements for the intended use. Profile comparison suggests that some of the dune features were not well represented in the post-Sandy LIDAR surface. This is shown in Figure 25, where successive profiles extracted from the LIDAR data were compared with profiles collected during the September 2013 profile survey and profiles extracted from the 2011 LIDAR/profile survey. Considering the potential for surface misrepresentation along the shoreline, particularly in the dune area which would likely skew calculations, volume change was not estimated using the surface based method.

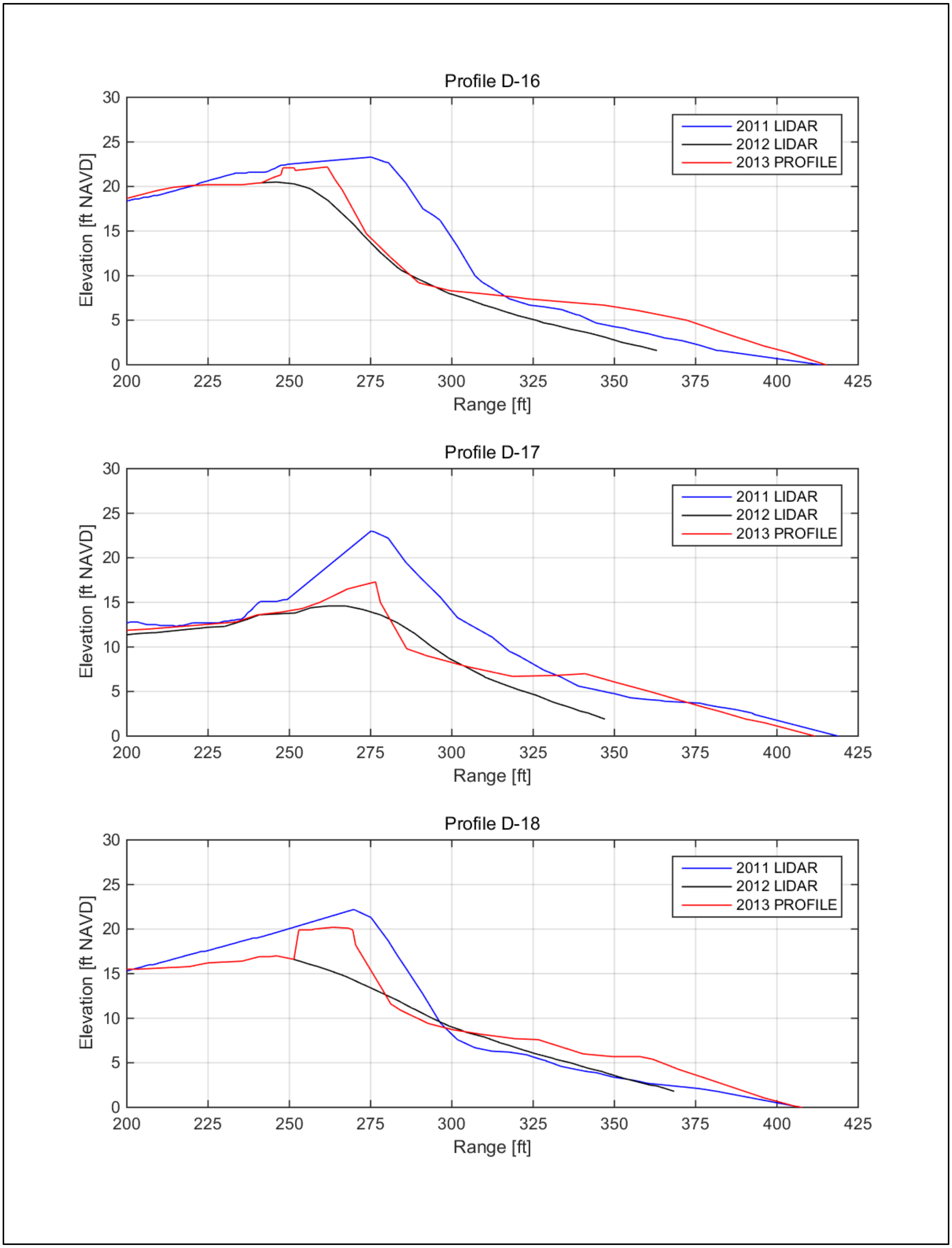

Figure 25. LIDAR Profile Comparison

12.2 Profile Based Volume Change

Profile based volume change is calculated by multiplying the area change between profiles collected along the same transect by the alongshore extent of the descriptive profile. Typically, multiple profiles are surveyed alongshore to facilitate volume change estimates between profiles using the average end area method. Volume change between profiles is calculated by averaging the change in area of the bounding profiles and multiplying this by the effective distance between the profiles.

Unfortunately, for the immediate project area, there is a paucity of survey data that covers the entire active portion of the beach profiles. However, by utilizing a compilation of data obtained prior to Hurricane Sandy and the September 2013 profile survey obtained by the Town, a rough estimate was made of profile volume changes that could be attributed primarily to Hurricane Sandy. As previously discussed, the pre-Sandy survey data compilation consisted of 2009 LIDAR data landward of the dune, 2011 LIDAR data from the dune to the shoreline, and November 2011 bathymetric data from the shoreline seaward to the -40 feet NAVD contour.

The results of the pre- and post-Sandy profile comparisons are provided in Table 23 in which the volume changes are reported between the landward limit of the survey data seaward to the specified contour shown along the top of the table. For example, the volume change between profiles D-17 and D-18 indicated a loss of 10,100 cubic yards between the +6.0 feet NAVD contour and the landward limit of the profile whereas for the area landward of the -6.0 feet NAVD contour, the profile actually gained 4,900 cubic yards. In general, this response was typical of most of the profiles in the study area in which material was eroded from the upper portion of the profile and deposited immediately offshore in the form of a sand bar. When the volume change computations were extended seaward of the -6.0 feet NAVD contour, most profiles experienced an overall loss of material.

Of particular note was the volume of sediment lost from the dune (above +12 feet NAVD). This is of special importance as the material lost from the dune does not generally return naturally while some of the material eroded from the upper portion of the profile is moved back on shore following the passage of the storm event. Therefore, post-storm recovery of the dune and upper portions of the profile would have to be accomplished through artificial placement of material.

Although material was lost from the upper part of the profile, most of the displaced material remained within the system. This is best explained by looking at the volume change above the -18 feet NAVD contour.

Table 23 shows that there was a net gain of 32,800 cubic yards between the pre-Sandy and post-Sandy Surveys; however, there is significant variability in losses and gains from profile to profile. The apparent gain calculated along the entire Town is of the same order of magnitude as the accuracy of the survey data, suggesting that there was no significant volumetric gain or loss associated with Hurricane Sandy. However, as stated above, material was lost from the dune and this area of the profile does not generally return naturally suggesting that artificial placement of material is necessary to bring back protective features that the dune previously provided.

Table 23. Profile Based Volume Change Associated with Hurricane Sandy

Profile		Volume (cy)						
From	To	-18.0	-12.0	-6.0	+1.2	+6.0	+12.0	+18.0
D-01	D-02	17,900	800	18,900	8,900	2,300	-900	-300
D-02	D-03	6,100	3,400	19,800	6,700	-500	-1,900	-500
D-03	D-04	-18,000	-23,600	1,200	-4,200	-6,200	-3,600	-2,100
D-04	D-05	-18,800	-33,900	-9,300	-7,900	-6,100	-3,500	-2,300
D-05	D-06	-3,900	-17,200	-2,200	-2,100	0	-700	-400
D-06	D-07	30,700	2,600	5,300	2,900	500	-1,300	-800
D-07	D-08	38,000	11,900	8,400	3,100	-1,600	-3,600	-2,800
D-08	D-09	33,900	12,900	12,700	1,300	-1,900	-4,000	-2,400
D-09	D-10	38,000	6,500	10,600	1,100	-1,000	-3,600	-1,100
D-10	D-11	17,800	-12,400	-3,400	-5,500	-300	-3,800	-1,700
D-11	D-12	9,000	-10,200	-4,100	-5,800	-1,500	-3,100	-1,500
D-12	D-13	13,500	-2,600	4,100	-5,300	-2,900	-4,800	-3,300
D-13	D-14	-5,600	-5,000	5,200	-6,300	-3,200	-4,400	-3,500
D-14	D-15	-9,100	-13,000	5,800	-5,700	-3,100	-2,700	-1,600
D-15	D-16	-200	-15,900	2,500	-9,100	-6,400	-5,400	-2,500
D-16	D-17	-9,300	-23,500	-5,500	-14,500	-13,400	-10,000	-3,700
D-17	D-18	-15,000	-25,100	4,900	-3,500	-10,100	-8,000	-2,600
D-18	D-19	-2,300	-5,300	10,700	5,300	-2,400	-3,500	-2,000
D-19	D-20	-14,500	-11,800	500	-3,700	-6,800	-7,100	-4,200
D-20	D-21	-43,200	-33,000	-8,600	-5,000	-11,600	-10,400	-6,600
D-21	D-22	-45,300	-45,600	-2,300	1,100	-7,300	-5,800	-4,000
D-22	D-23	-13,800	-27,800	8,700	1,200	-3,200	-2,900	-2,200
D-23	D-24	25,800	3,600	13,200	5,100	-2,800	-2,300	-2,700
D-24	D-25	26,900	10,200	9,000	4,100	-3,000	-1,700	-2,800
D-25	D-26	12,000	10,200	9,100	9,200	3,700	800	-1,500
D-26	D-27	8,300	4,500	8,400	10,900	3,700	100	-1,500
D-27	D-28	20,600	5,600	11,100	8,600	900	-300	-1,600
D-28	D-29	9,800	-8,100	1,100	4,800	-200	-900	-1,900
D-29	D-30	-1,600	-16,000	-3,300	2,000	-2,100	-2,300	-3,000
D-30	D-31	-15,000	-15,800	-1,500	1,600	-2,100	-2,000	-2,300
D-31	D-32	-24,100	-18,000	-1,200	-200	-1,500	-2,100	-1,300
D-32	D-33	-14,000	-15,600	1,600	-3,200	-2,700	-3,100	-1,900
D-33	D-34	-21,800	-23,700	-7,000	-11,400	-3,500	-4,600	-3,700
D-01	**D-34**	**32,800**	**-330,900**	**124,400**	**-15,500**	**-96,300**	**-113,400**	**-76,300**
D-10	**D-19**	**-1,200**	**-113,000**	**20,200**	**-50,400**	**-43,300**	**-45,700**	**-22,400**

12.3 Shoreline Based Volume Change

Shoreline based volume change is calculated by multiplying shoreline change by the active profile height and the alongshore extent of the descriptive profile. Similar to profile based volume change calculations, the area calculated by multiplying the shoreline change by the active profile height can be used to estimate volume change between profiles. Volume change between profiles is calculated using the same method as discussed in Section 12.2 above except the area of the bounding profiles is calculated using shoreline changes and the active profile height instead of the profile based area. Shoreline based volume change estimates assume that the entire profile translates uniformly between the upper and lower limits of the active profile and that the profile and volume does not change outside the limits of the active profile.

Long term volume changes were calculated using LIDAR data sets that extend along the Town's shoreline. As detailed in Section 10, shoreline changes were calculated using the 1996, 2011, and 2012 LIDAR data. Volume changes were calculated using an active profile height of 30 feet, as discussed in Section 11, and shoreline change data presented in Table 21. The total volume change within Town limits between 1996 and 2011 was -228,700 cubic yards, which is equivalent to an annual erosion rate of 15,560 cubic yards per year. Using the 2011 and 2012 LIDAR surveys and shoreline based volume change calculation methods, roughly 586,500 cubic yards of sediment eroded during Hurricane Sandy. As a result of this event, the annual erosion rate within the Town limits increased to 51,000 cubic yards per year. Volumetric changes calculated using LIDAR generated shorelines are presented in Table 24.

Table 24. Shoreline Based Volume Change

Profile		Volume Change (cy)		Volume Change (cy/yr)	
From	To	1996-2011	1996-2012	1996-2011	1996-2012
D-01	D-02	26,400	8,400	1,790	530
D-02	D-03	39,800	-3,400	2,620	-230
D-03	D-04	34,500	-8,900	2,340	-560
D-04	D-05	-14,100	-62,800	-870	-3,900
D-05	D-06	-8,700	-71,500	-650	-4,440
D-06	D-07	-28,200	-45,500	-1,840	-2,820
D-07	D-08	10,800	-11,900	760	-760
D-08	D-09	39,100	7,600	2,600	430
D-09	D-10	52,700	3,400	3,480	220
D-10	D-11	59,700	20,300	3,940	1,240
D-11	D-12	35,600	12,200	2,430	690
D-12	D-13	6,800	-21,500	450	-1,360
D-13	D-14	-17,300	-48,800	-1,190	-3,030
D-14	D-15	-47,700	-82,400	-3,140	-5,090
D-15	D-16	-88,900	-110,500	-5,960	-6,940
D-16	D-17	-107,900	-145,600	-7,230	-9,060
D-17	D-18	-102,700	-130,700	-6,890	-8,170
D-18	D-19	-42,300	-50,300	-2,840	-3,110
D-19	D-20	-32,400	-40,500	-2,160	-2,560
D-20	D-21	-22,200	-28,900	-1,450	-1,830
D-21	D-22	-25,200	-25,200	-1,650	-1,570
D-22	D-23	10,900	15,000	670	920
D-23	D-24	11,900	16,500	730	1,010
D-24	D-25	6,500	5,400	430	320
D-25	D-26	-33,700	-21,700	-2,280	-1,300
D-26	D-27	-33,600	-36,800	-2,280	-2,280
D-27	D-28	-8,700	-6,500	-650	-430
D-28	D-29	32,500	27,100	2,170	1,730
D-29	D-30	32,500	32,500	2,170	1,950
D-30	D-31	22,800	23,800	1,520	1,520
D-31	D-32	-16,300	-2,300	-1,050	-120
D-32	D-33	-19,000	-17,000	-1,300	-1,100
D-33	D-34	-2,300	-14,700	-230	-900
D-01	**D-34**	**-228,700**	**-815,200**	**-15,560**	**-51,000**

13 SEDIMENT BUDGET

Accurate description of sediment movement assists in developing projects that better meet objectives and performance requirements. A sediment budget was developed for the Town of Duck to describe the movement of beach sediment into, out of, and within the project area. This allows project performance to be modeled, thus facilitating an improved assessment of volumetric requirements.

A sediment budget was developed by decomposing the volumetric change into component parts. In an effort to create tools to assess project performance over the course of a nourishment interval, long term changes were used to better describe average conditions. Though surface and profile data are available, either their spatial or temporal coverage is not sufficient to estimate long term volume change over the entire active profile. Therefore, LIDAR based shoreline changes (Section 10) were used to estimate long term volume changes (Section 12.3) and were selected for use in developing the sediment budget.

The volume change calculation method was investigated to evaluate volume change components included in the estimate. Typical sediment budget components include longshore transport, relative sea level rise, overwash, aeolian transport, and loss of fines. Considering the upper limit of the active profile in the volume change calculations was defined as the berm crest and the dune is relatively stable (no landward migration), the shoreline based volume change calculation does not include a significant overwash component. Similarly, considering the berm is the upper limit of the active profile, which is the upper limit of the active swash zone during average conditions, it is expected that the volume change calculation does not include a significant aeolian transport component. Moreover, the geotechnical investigation indicates that the sediment fine fraction is small (0.96%), which suggests that silt loss does not need to be included as a sand only approximation is sufficient. Therefore, the shoreline based volume change estimates can be decomposed into components associated with relative sea level rise and longshore transport.

The following sections discuss the separation of relative sea level rise losses from the volume change estimate and the development of a longshore transport curve.

13.1 Relative Sea Level Rise Losses

A portion of the shoreline recession and calculated volume change is due to the effects of relative sea level rise. Along a sandy coast shoreline recession and volume change due to relative sea level rise does not result in a net volume change in the cross-shore profile but simply a redistribution of the sediment across the profile (Figure 26). The shoreline based volume change must therefore be reduced to account for relative sea level rise prior to calculating the volume change used to develop the sand longshore transport rate.

Bruun (1962) showed that beach profiles should adjust to increased water elevation with a recession of the shoreline and a deposition of sand in the offshore area. Bruun's rule is based on the following relationships given a profile in equilibrium: 1) there is a shoreward displacement of the beach profile as the upper beach is eroded; 2) the material eroded from the upper beach is equal in volume to the material deposited on the nearshore bottom; and 3) the rise in the nearshore bottom as a result of this deposition is equal to the rise in sea level, thus maintaining a constant water depth along the active profile.

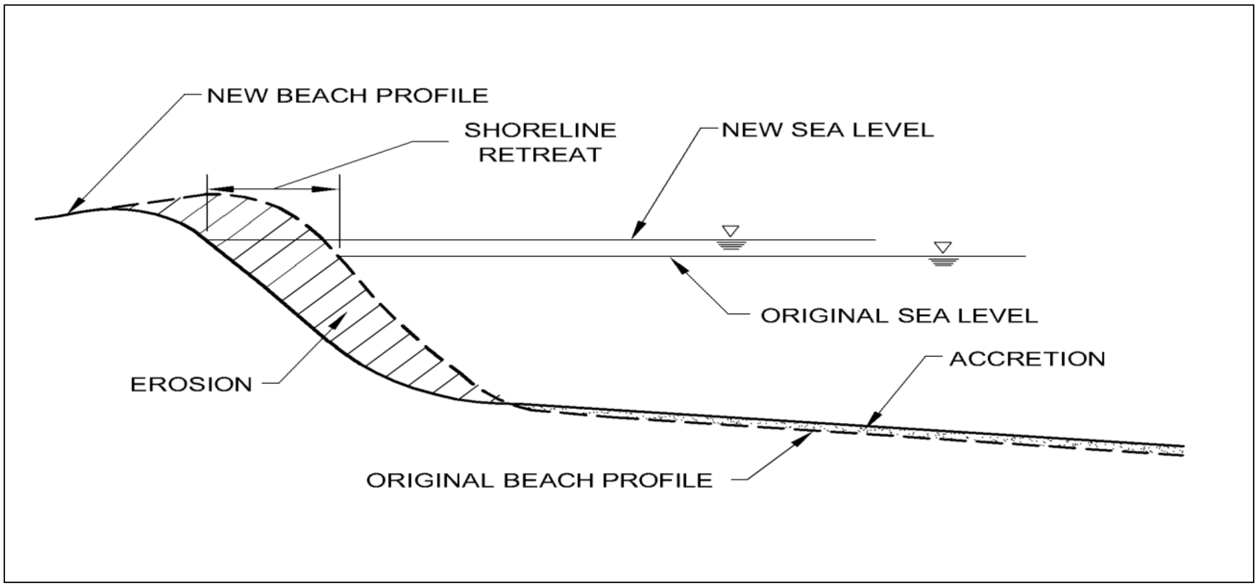

Figure 26. Impact of Sea Level Rise on Shoreline Recession

Shoreline change and volume change required to maintain an equilibrium profile given the current rate relative sea level rise was calculated using Bruun's (1962) rule. Based on the 2011 and 2013 beach profile surveys, the average cross-shore distance from the berm crest (+6 feet NAVD seaward contour) to the depth of closure (-24 feet NAVD seaward contour) is roughly 2,000 feet. Given these equilibrium profile characteristics and the current rate of relative sea level rise at Duck of 0.015 feet per year, theory suggests that the shoreline must recede at an annual rate of 1 foot per year to maintain an equilibrium profile, which equates to an erosion rate of roughly 1.1 cubic yards per foot per year. Considering the 30,850 foot stretch of shoreline within the Town limits, this analysis suggests roughly 34,000 cubic yards of sand are removed from the beach face each year to combat the effects of relative sea level rise.

13.2 Longshore Transport

This section discusses the longshore transport rate along the Duck shoreline. An annualized sediment budget was developed using shoreline changes, active profile heights, and relative sea level rise rates.

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

$$LT_{out} = V_{total} - V_{RSLR} + LT_{in}$$

LT_{out} = Longshore transport out of the cell

V_{total} = Volume change calculated based on shoreline change
V_{RSLR} = Volume change associated with relative sea level rise
LT_{in} = Longshore transport into the cell from an adjacent cell

Longshore transport was estimated by integrating volume change between cells in a longshore direction. Cell limits were defined as the shore perpendicular transects used in the shoreline change analysis (Section 10). The volume change within each 100 foot shore perpendicular cell was reduced to account for relative sea level rise so that a volume change associated with longshore transport could be calculated. If the resulting volume change was greater than zero (accretion), then the net sediment transport was into the cell. On the other hand, if the resulting volume change was less than zero (erosion), then the net sediment transport was out of the cell.

A starting point for the longshore transport integration must be identified to characterize the transport direction and magnitude. An area of zero net sediment transport (a nodal point) is the typical point at which to start such a summation because it is generally easier to identify an area of no net sediment transport than estimate the longshore transport at a given point. Alternatively, known transport at a particular location can also be used as a boundary condition to tune or establish the longshore transport rate along the study area. Considering the lack of a nodal point or known transport rate along any portion of the study area, only relative transport could be calculated.

Relative transport was calculated using LIDAR based shoreline changes discussed in Section 10 and shown in Figure 23. Relative transport curves were developed for the following two scenarios: 1) total relative transport which includes the effects of relative sea level rise and 2) longshore relative transport which removes losses associated with relative sea level rise. Both of these curves have a similar shape, but their slopes differ as relative sea level rise results in additional losses. The relative transport curves shown in Figure 27 assume a fixed transport rate at the northern (D-1) Town limit, with positive relative transport rates suggesting an increase in southerly transport (or decrease in northerly transport) and negative rates indicating a decrease in southerly transport (or increase in northerly transport). The same curves are presented in Figure 28, but assume a fixed transport rate at the southern (D-34) Town limit.

The slope of the longshore transport curve indicates whether erosion or accretion is occurring and the severity of this erosion or accretion. A positively sloping curve (increasing southerly transport or decreasing northerly transport) shows areas of erosion while a negatively sloping curve (decreasing southerly transport or increasing northerly transport) shows areas of accretion. Areas of higher erosion (or accretion) result in a steeper longshore transport curve, while stable areas result in a flatter longshore transport curve. Therefore, the longshore transport curve shown in Figure 27 suggests that the greatest erosion is occurring between D-12 and D-20. Also of note is that when including the effects of relative sea level rise the general trend is increasing relative transport, suggesting erosion. However, when removing the effects of relative sea level rise, the curve flattens suggesting a more stable shoreline that even exhibits an accretional trend during the 1996-2011 analysis period. This implies that losses associated with relative sea level rise may be a significant contributing factor of erosion along the northern (between D-1 and D-11) and southern (D-21 and D-34) Town limits.

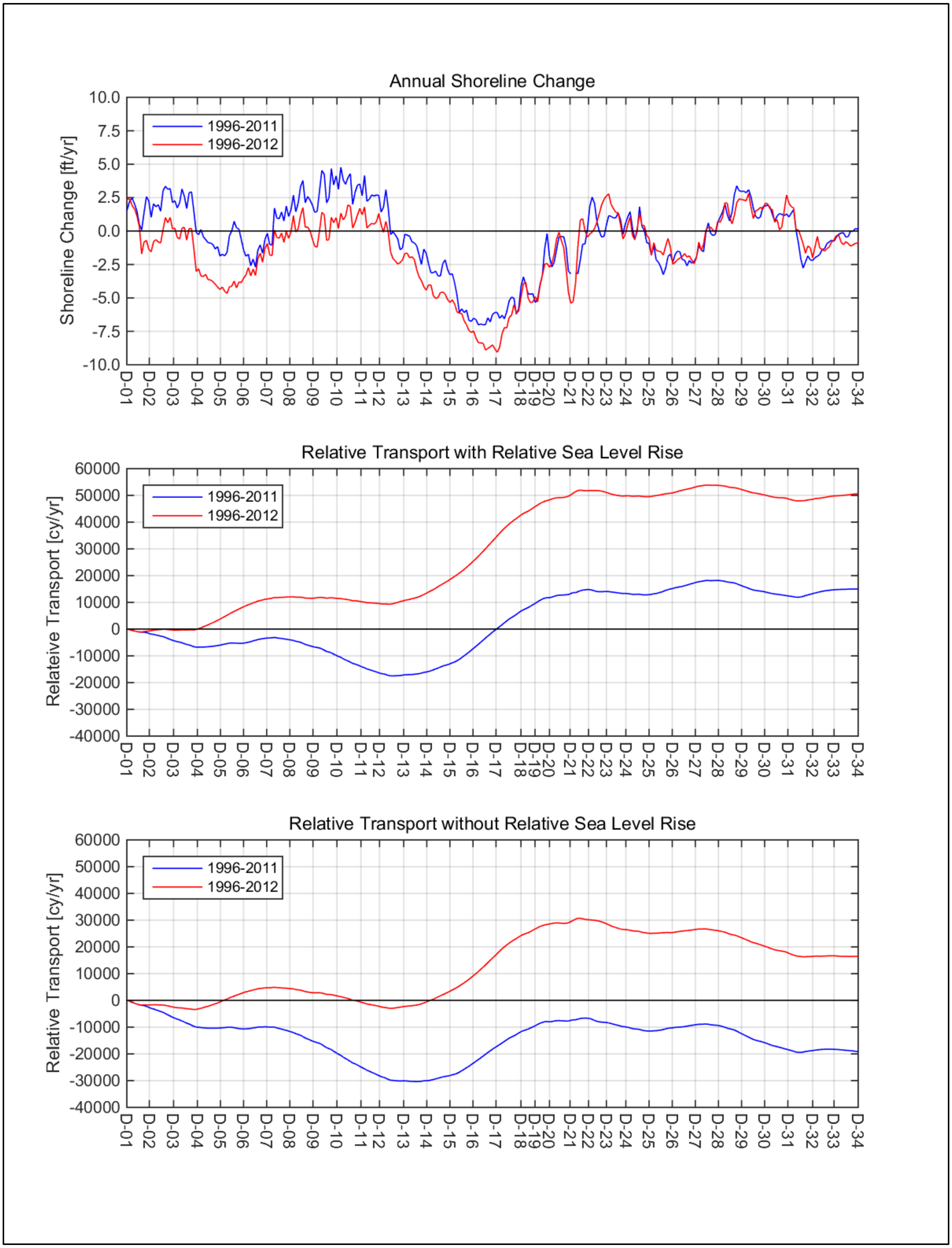

Figure 27. Longshore Transport Rate - Relative to North End Longshore Transport

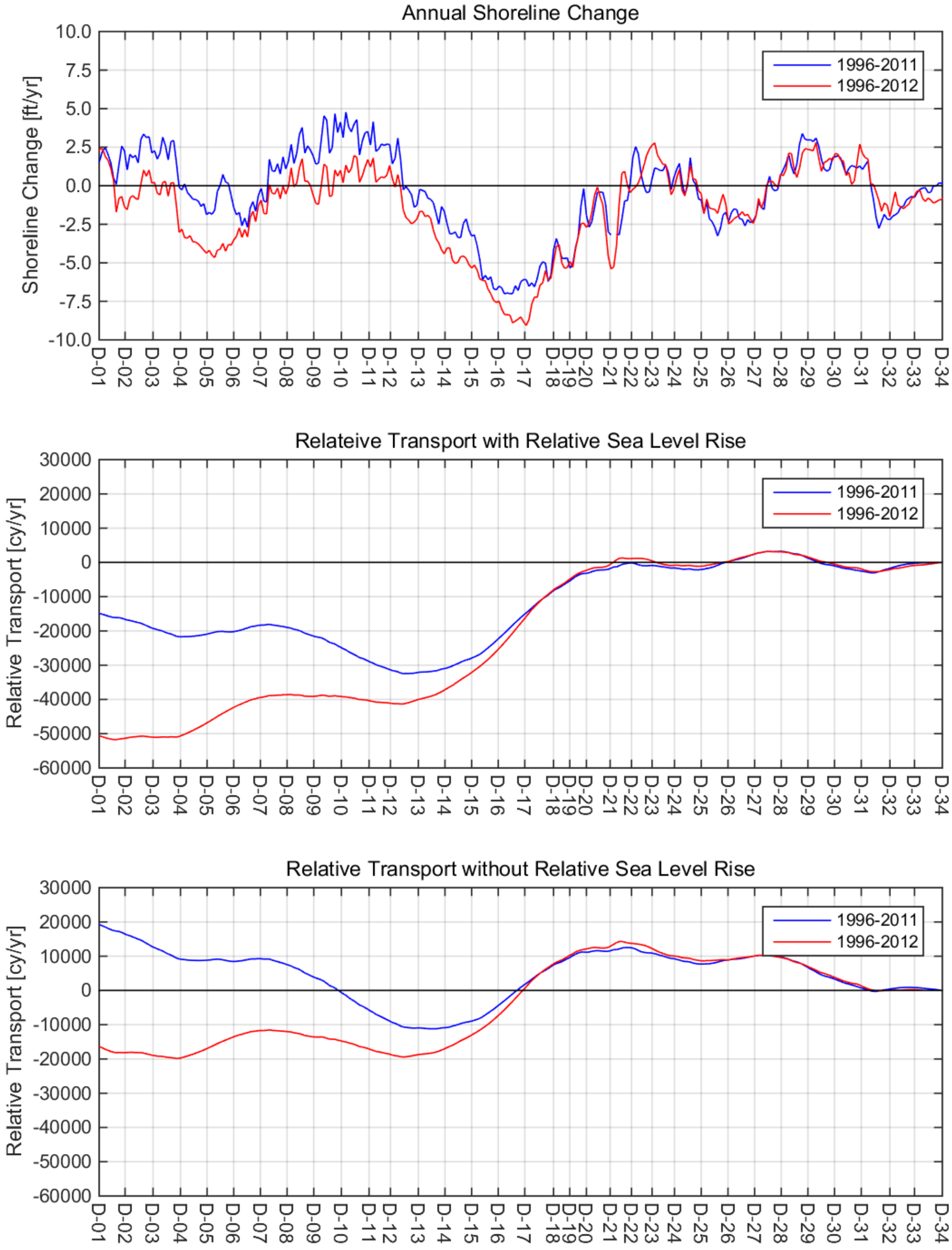

Figure 28. Longshore Transport Rate - Relative to South End Longshore Transport

13.3 Even-Odd Analysis

CPE (2013) conducted an even-odd analysis of the USACE FRF pier as part of the Erosion and Shoreline Management Feasibility Study for the Town of Duck. The analysis showed that the pier does create a longshore transport barrier with impacts that extend 400 to 1,000 feet alongshore. However, impacts are contained within the FRF property limits that extend roughly 1,600 feet north and 1,700 feet south of the pier. The interpolated impact limits are not interpreted as finite but provide a reasonable boundary estimate. The analysis also suggests sediment transport in the cross-shore direction is altered by the pier. Graphical representation of the even function, which generally shows the cross-shore shoreline change rate, implies that shoreline close to the pier is sheltered by the structure which reduces the wave impacts around the pier and aids in decreasing erosion. Alleviating the erosion stress is considered a benefit, but this benefit is minor and was not valid for all time frames analyzed.

14 MODELING

Numerical models were used in this study to evaluate design needs and estimate project performance. Tools employed include the cross-shore model SBEACH and the longshore model GENESIS. Model descriptions, calibration and verification details, boundary conditions, and result analysis procedures for both models are detailed below.

14.1 SBEACH

Cross-shore performance evaluations utilize the Storm Induced Beach Change (SBEACH) model (Larson and Kraus, 1989). SBEACH is a two-dimensional model which simulates beach profile changes that result from varying storm waves and water levels. These profile changes include the formation and movement of morphological features such as longshore bars, troughs, berms, and dunes. SBEACH assumes that the simulated profile changes are produced only by cross-shore processes, while longshore sediment transport processes are neglected. This empirically based numerical model was formulated using both field data and the results of large-scale physical model tests. Input data required by SBEACH includes beach cross-sections, the median sediment grain size, several calibration parameters, and a temporally varying storm hydrograph (wave height, wave direction, wave period, and water surface elevation) and wind field (wind speed and direction). Simulated profile changes are driven by the cross-shore variation in wave height and wave setup calculated at discrete points along the profile from the offshore zone to the landward survey limit.

The following basic assumptions underlie the SBEACH model:

- Breaking waves and variations in water level are the major causes of sand transport and profile response.

- The median sediment grain diameter along the profile is reasonably uniform across shore.

- The shoreline is straight (ie. longshore effects are negligible during the term of simulation).

- Linear wave theory is applicable everywhere along the beach profile.

Considering SBEACH is an empirical model, the user must define beach and sediment transport parameters. To define these parameters, the model was calibrated using surveys collected at the nearby USACE FRF prior to and following Hurricane Isabel. The model was calibrated by adjusting the beach and sediment transport parameters until the post-storm beach profiles generated by SBEACH were similar to surveyed post-storm profiles. The calibrated model was then validated using surveys collected at the nearby USACE FRF prior to and following the 1991 Perfect Storm. Calibration and verification details are provided in the Duck Erosion and Shoreline Management Feasibility Study (CPE, 2013). The beach and sediment transport parameters used in all SBEACH production runs are summarized in Table 25.

Table 25. SBEACH Model Parameters

Parameter	Units	Value
Landward Surf Zone Depth	ft	1
Effective Grain Size	mm	0.59
Maximum Slope Prior to Avalanching	deg	45
Transport Rate Coefficient	m^4/N	2.50E-06
Overwash Transport Parameter	m^2/s	5.00E-03
Coefficient of Slope Dependent Term	-	2.50E-03
Transport Decay Coefficient Multiplier	-	0.5
Water Temperature	°C	20

During October 2012, the Town of Duck was impacted by Hurricane Sandy. The storm originated in the western Caribbean Sea and traversed northeastward across the Bahamas before emerging in the Atlantic Ocean. From there, the storm generally traveled on a northward path before turning westward making landfall near Brigantine, New Jersey on October 29, 2012. Although the storm passed offshore of Duck, the winds and waves impacted the area and caused significant erosion to the shoreline and damage to coastal structures. Considering profile survey data was collected along the Town of Duck before (November 2011) and after (September 2013) Hurricane Sandy, the effects of the storm could be used to validate the SBEACH model. Prior to completing production runs, the SBEACH model was further validated using the pre- and post-storm surveys and oceanographic and meteorological data collected during Hurricane Sandy at USACE wave gauge FRF630 and the Duck tide gauge.

Profiles modeled during production runs were delineated along all project transects. Modeled profiles were developed using a compilation of survey data sets to extend the profile from the landward extent of expected overwash offshore to a location beyond the depth of closure; extending the profiles beyond the September 2013 survey limits was necessary to ensure model stability. Surveys used to generate the profiles are provided below in the order in which they were compiled to generate the best description of the existing conditions:

- September 10-16, 2013 profile survey conducted by CPE

- September 5, 2013 FRF survey conducted by USACE

- November 2012 LIDAR survey conducted by USGS
- NOAA bathymetric charts

Typical profiles compiled using the above surveys to represent the existing conditions are shown in Figure 29. Considering that SBEACH profiles are gridded during the analysis, a variable grid was used to provide fine resolution to adequately detail topographic and nearshore features in the model simulation while less variable offshore bathymetry was gridded using a coarser resolution to improve model efficiency. An example of typical profile grid spacing and associated limits is provided in Table 26.

Table 26. Typical SBEACH Grid

Cell		Limits - Range (ft)		Limits - Elevation (ft, NAVD)	
Width (ft)	Number	Nearshore	Offshore	Nearshore	Offshore
5	500	-750	1,750	Subaerial	-15
10	200	1,750	3,750	-15	-25
20	100	3,750	5,750	-25	-30
50	50	5,750	8,250	-30	-40
100	50	8,250	13,250	-40	-50
200	20	13,250	17,250	-50	-60
500	20	17,250	27,250	-60	-70
1,000	10	27,250	37,250	-70	-80
2,000	10	37,250	57,250	-80	-90
5,000	5	57,250	82,250	-90	-100

Options were modeled by placing a design template on the existing profile. The subaerial portion of the profile within the limits of the design template took the form of the design, while the part of the profile landward of the design template remained unchanged. Adjustment of the subaqueous profile was not necessary if the design did not advance the shoreline. However, when adding a design that advanced the shoreline, the subaqueous part of the profile was modified by translating the profile between the shoreline and the depth of closure (identified in Section 11) seaward the distance that the shoreline advanced. Therefore, the part of the profile seaward of the depth of closure remained unchanged. A typical design profile overlaid on the existing profile is shown in Figure 29.

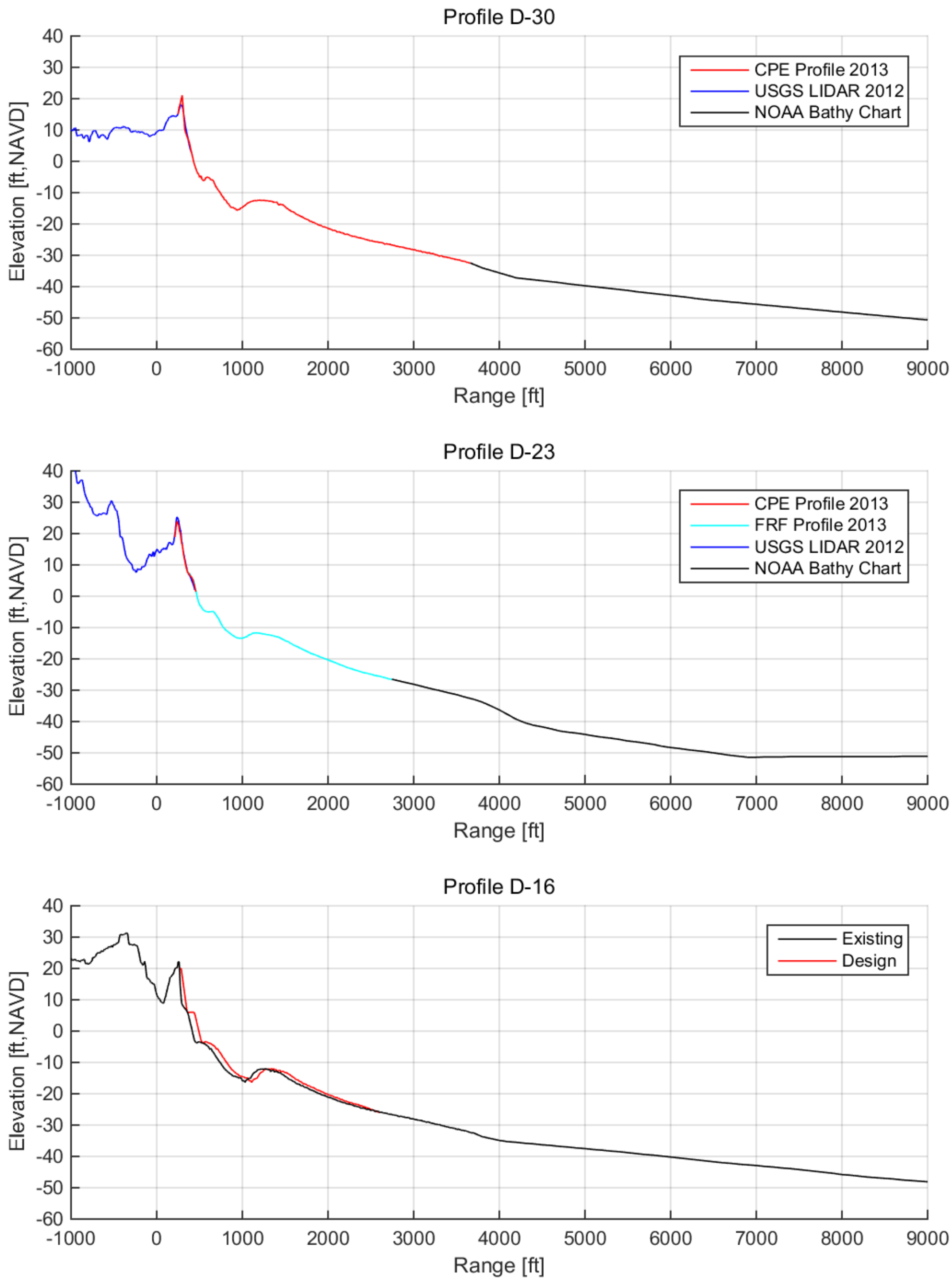

Figure 29. Typical SBEACH Profiles

Design storms were developed using oceanographic and meteorological data collected at Duck. Hourly wave, wind, and water level data were used as the model boundary conditions. Wave data used to create the storm hydrograph was collected at USACE wave gauge FRF630. Water surface elevation and meteorological data used to create the storm wind field and complete the storm hydrograph were collected at the Duck tide gauge. Considering the location of the FRF630 wave gauge, the water depth at the offshore boundary was set to 57 feet. Wave height randomization, using model default values, was included in the model. The model utilized a 1 minute time step for all simulations.

The SBEACH analyses completed for the Duck Feasibility Study (CPE, 2013) used water level, wind, and wave data from two historical storms, the Perfect Storm in 1991 and Hurricane Isabel in 2003, to develop the various storm events modeled in SBEACH. Data was obtained between October 23 and November 3, 1991 for the Perfect Storm while data was obtained between August 30 and September 22, 2003 for Hurricane Isabel. These historic storms were then scaled to the 1, 5, 10, 20, 25, and 50 year return period events. This approach was initially used to evaluate project performance, but considerable differences were noted between the Northeaster and Hurricane impacts. Review of the storms showed that storm length varied considerably and successive (back-to-back) storms were included in the analysis. Wave data was reviewed to identify whether this was an accurate representation of the events. Analysis indicated that back-to-back events such as this are not uncommon. SBEACH simulations were completed with a smaller storm following a larger storm and vice versa. Results were compared to determine the effect and whether both storms should be included. This assessment determined storm duration had less of an impact than the duration of the extreme storm components. Therefore, only the single major storm was used in the production runs.

Design storm characteristics were identified using the results of the extreme event analysis discussed in Section 8. The SBEACH model study completed as part of the Duck Feasibility Study (CPE, 2013) included storms that were scaled to extreme values published by the USACE (1985, 2011) and FEMA (2006). Though these values are appropriate for use when completing a Feasibility Study, a detailed investigation was deemed relevant to ensure appropriate storm selection for design modeling purposes. Reasons for updating the extreme values are detailed below:

- The USACE (2011) extreme wave data was calculated at a deeper offshore location (depth of 75 feet) while the SBEACH model boundary and storm wave history were based on the location and data collected at the USACE FRF630 gauge (depth 57 feet). Therefore, using USACE extreme wave data misrepresents the boundary conditions that drive the SBEACH model.

- The USACE (2011) extreme wave analysis covered the 1980-1999 time period. This is problematic considering the largest wave event occurred in 2003 (Hurricane Isabel) and 8 of the top 10 and 17 of the top 20 events occurred after 2000.

- The USACE (2011) extreme wave analysis was based on hindcast data rather than collected data. Collected wave data over a sufficient duration for extreme analyses were available. Moreover, this data was collected at FRF630 (used as the SBEACH offshore boundary and wave history), which made it ideal for use in driving the model.

- The FEMA (2006) storm stage extreme values were reported for the 10 and 50 year return periods. The 1, 5, 20, and 25 year extreme values were scaled using these results. There is no need to scale values as tide data to complete an extreme surge analysis was readily available at the Duck tide station. Using water level data collected at Duck facilitates a better understanding of storm surge events that improves the identification of storms that are suitable for design purposes.

- The USACE (1985) wind speed extreme values were reported for the 10, 25, and 50 year return periods. The 1, 5, and 20 year extreme values were scaled using these results. Though this data may provide a better estimate of extreme hurricane winds, it does not clearly identify how these wind speeds relate to measured extreme wave and surge events. There is no need to scale values as wind data to complete an extreme wind speed analysis was readily available at the Duck tide station and nearby airports. Using wind data collected at Duck facilitates a better understanding of local wind events that improves the identification of storms that are suitable for design purposes.

A comparison of design storm characteristics employed in the Duck Feasibility Study (CPE, 2013) and the values calculated for this study (as discussed in Section 8) is provided in Table 27. The extreme analysis completed for this study evaluated storm surge and not storm stage as used in the feasibility study; therefore, in an effort to maintain consistency, the storm surge values were converted to storm stage using a constant offset. Considering that both analyses calculated the 10-year return period event, this event was used to identify an appropriate offset (the 50-year event was not used as this was a projected value as the recording period was less than 50 years). The calculated offset was determined to be 1.2 feet NAVD, which is the mean high water (MHW) elevation. All interpolated values are shown as red in Table 27. Based on this reassessment, design storm characteristics used in this study are described using the results of the extreme analyses presented in Section 8.

Table 27. Extreme Event Comparison

Return Period	Wave Height (ft)		Wave Period (s)		Stage (ft, NAVD)		Wind Speed (mph)	
	Feasibility	Design	Feasibility	Design	Feasibility	Design	Feasibility	Design
1	17.6	14.8	9.9	10.3	4.0	3.5	50.2	65.6
5	21.2	20.9	12.9	15.0	4.2	4.4	53.9	78.6
10	22.7	23.5	14.2	17.0	4.8	4.8	64.0	82.2
20	24.3	26.1	15.5	19.0	5.7	5.2	78.5	85.3
25	24.8	27.0	16.0	19.6	5.8	5.3	81.0	86.3
50	26.3	29.6	17.3	21.6	6.2	5.7	91.0	89.0

An extensive analysis was completed to determine whether a scaled hurricane or northeaster should be used for design purposes. Review of historic data indicated that scaling all storm parameters to create synthetic storms results in conditions that do not represent natural occurrences (see Table 28). Using a method such as this may be appropriate for a quick study to investigate project feasibility but could ultimately result in an over- or underestimate of project need and performance. As a result, actual storm characteristics were reviewed to select storms that best matched extreme event characteristics. Considering that the storm hydrograph is the primary model driver, the top wave height and storm surge events were compared with the calculated return period descriptors (the Perfect Storm was also included as it was used in the SBEACH calibration/verification). Table 28 shows that the Perfect Storm best represents a 3-year event, Hurricane Sandy resembles a 5-year event, and Hurricane Isabel may best describe a 25-year event. Considering a goal of this project is to provide a reasonable level of storm damage reduction, Hurricane Isabel was adopted as the design storm.

Table 28. Storm Event Characterization

	Wave		Storm		Wind
Event	Height (ft)	Period (s)	Surge (ft)	Stage (ft, NAVD)	Speed (mph)
Perfect	17.5	12.5	3.0	4.0	52.6
Sandy	20.7	15.4	4.0	4.5	48.8
Isabel	26.7	15.4	4.4	5.6	63.5
1	14.8	10.3	2.3	3.5	65.6
5	20.9	15.0	3.2	4.4	78.6
10	23.5	17.0	3.6	4.8	82.2
20	26.1	19.0	4.0	5.2	85.3
25	27.0	19.6	4.1	5.3	86.3
50	29.6	21.6	4.5	5.7	89.0

SBEACH can be used to identify structures impacted during storm events. A 1 foot change in profile elevation is a reasonable threshold for estimating when structures become vulnerable to wave damage, including undermining and/or inundation (USACE, 1985). Therefore, a structure is considered damaged if any part of the structure is seaward of the landward most location where the profile is lowered by 1 foot. For this study, the landward most location where the profile is lowered by 1 foot is extracted from model results along profiles to identify *impact points*. These *impact points* are then connected to create an *impact line* that is used to identify structures damaged between profiles.

14.2 GENESIS

Longshore performance evaluations for the beach fill project utilize the Generalized Model for Simulating Shoreline Change (GENESIS) (Hanson and Kraus, 1991). GENESIS is a two-dimensional model which simulates shoreline changes that result from variations in the wave-driven longshore sediment transport. GENESIS assumes that the simulated shoreline changes are produced only by longshore processes, while cross-shore sediment transport processes are neglected. This empirically based numerical model was formulated using both field data and the results of large-scale physical model tests. Input data required by GENESIS includes shoreline locations, the median sediment grain size, several calibration parameters, and a temporally varying wave field (wave height, period, and direction). Simulated shoreline changes are driven by variations in the longshore transport rate calculated at discreet points alongshore using the USACE CERC equation with an additional term to account for longshore variations in the breaking wave height.

The following basic assumptions underlie the GENESIS model:

- Breaking waves and variations in longshore transport are the major causes of sand transport and shoreline response.
- The median sediment grain diameter along the shoreline is reasonably uniform along and across shore.
- The profile shape is constant with time (ie. cross-shore effects are negligible during the term of simulation).
- Shoreline change is directly proportional to volume change.
- The berm elevation and depth of closure are uniform alongshore.

The effects of the offshore bathymetry can be added to the model by providing an optional set of wave refraction coefficients and refracted wave angles. The wave refraction coefficients and refracted wave angles are usually determined using an external wave transformation model such as STWAVE (Smith, 2001), SWAN (Delft University of Technology, 2008), or another industry-standard wave transformation model. For this model study, wave transformation estimates along the study area utilize the Simulating Waves Nearshore (SWAN) model, which accounts for the shoaling, refraction, diffraction, wind growth, whitecapping, and bottom damping of spectral waves. SWAN has several advantages over other models as it includes most of the key processes that govern the transformation of nearshore and offshore waves and it can utilize curvilinear grids with non-uniform grid spacing to follow the orientation of shorelines and offshore contours. Inputs to the SWAN model include bathymetric grids, offshore wave conditions, wind velocities, water levels, and the following input parameters:

- Wave height to water depth ratio for depth-limited wave breaking.
- Secondary wave breaking coefficient.
- “Triad” coefficients for energy transfer from long waves to short waves in shallow water.
- Bottom friction coefficient.
- Diffraction coefficients, if desired.
- Whitecapping formulation.

Calibration of the SWAN model was based on wave, wind, and water level measurements collected by the USACE at the FRF during Hurricane Irene, which passed offshore August 26-29, 2011. Model calibration was performed by varying the values of the bottom friction coefficient. Given the spacing of the grid, activating diffraction was not necessary; the directional spreading associated with each wave case was sufficient to account for diffraction-like effects (Luijendijk, 2011). All other model parameters were set to their default values. Model gridding and calibration details are provided in Appendix D.

In every GENESIS simulation, the forcing of the model is given sequentially. To simulate shoreline changes between two specific dates, a time series of offshore waves between the same two dates must be provided at 1 to 6 hour intervals. However, it is not practical to simulate shoaling, refraction, breaking, and other processes at every time step. To resolve this problem, the offshore wave record was divided into a large, but reasonable number of wave cases that encompasses the observed variability in wave height, wave period, and wave direction. Shoaling, refraction, breaking, and other processes were then evaluated for each wave case along the depth of closure using an offshore to nearshore wave transformation function (Hanson and Kraus, 1991; Bonanata, et al, 2010) to provide the necessary GENESIS wave boundary conditions.

Considering GENESIS is an empirical model, the user must define beach and sediment transport parameters. To define these parameters, the model was calibrated using observed shoreline changes between the November 2005 and November 2012 LIDAR surveys. The model was calibrated by adjusting the sediment transport parameters until the modeled shoreline changes generated by GENESIS were similar to measured shoreline changes. The calibrated model was then validated using observed shoreline changes between the October 1999 and November 2005 LIDAR surveys. Shorelines for all simulations and comparisons were defined as the mean high water (MHW) shoreline (contour elevation of +1.2 feet NAVD). Calibration and verification details are provided in Appendix D of this report. The beach and sediment transport parameters used in all GENESIS production runs are summarized in Table 29.

Table 29. GENESIS Model Parameters

Parameter	Units	Value
Transport Rate Coefficient K1	-	2.0
Transport Rate Coefficient K2	-	0.0
Effective Grain Size	mm	0.59
Berm Elevation	ft, NAVD	+6.0
Closure Depth	ft, NAVD	-24.0

Shorelines modeled were delineated along transects spaced at 100 foot intervals. Modeled shorelines extended from a location roughly 30,000 feet north of D-1 to a location roughly 30,000 feet south of D-34. Existing conditions were modeled as the 2012 LIDAR shoreline extracted at model transects. Beach fill design options were modeled by advancing the existing shoreline to the design shoreline at model transects within the longshore limits of the design; the existing shoreline was used as the design shoreline outside the design limits.

Design storms were developed using oceanographic and meteorological data collected at Duck between November 2002 and November 2012. Wave data collected at USACE wave gauge FRF630 was input into the SWAN generated wave transformation transfer function to define the offshore boundary conditions along the model depth of closure. Meteorological data used to create the wind field was collected at the Duck tide gauge. The model utilized a 2 hour time step for all simulations.

15 PLANFORM DESIGN

The existing shoreline management initiatives within the Town of Duck are limited to beach bulldozing or scraping, sand fencing, dune vegetation, and truck haul to build and/or repair dunes. The Town does not allow the use of temporary sandbags to protect threatened structures. Essentially all of the shoreline management efforts are presently carried out by individuals or groups of individual property owners. In an effort to develop a shoreline management plan for the Town, long-term erosion rates and storm impacts were analyzed to identify parts of the shoreline where structures are vulnerable to the effects of chronic erosion and episodic storm events.

15.1 Long-Term Erosion Threat

Shoreline recession rates determined from the analysis of the LIDAR data sets were used to evaluate long-term erosion threats. The 1996 to 2011 analysis period was used in this evaluation as rates calculated using LiDAR data collected immediately after Hurricane Sandy may be highly influenced by the effects of the storm. As shown in Figure 23, recession rates varied along the shoreline with rates ranging from a maximum recession of 6.5 feet per year between D-16 and D-17 to 3.5 feet per year of advance near D-10. In general, the shoreline between D-12 and D-21 is eroding while other parts of the Town's shoreline are either relatively stable or accreting. Average shoreline change rates used in this analysis are presented in Table 21.

Erosion of the shoreline was deemed to render structures imminently threatened once the berm encroached within 20 feet of the structure. As discussed in Section 11, the +6 feet NAVD contour represents the approximate elevation of the natural berm crest as it is representative of the average wave run-up elevation under normal conditions. The 20 foot criteria used to identify structures threatened by long-term erosion is generally the same definition used by DCM. Using LIDAR derived shoreline changes and the berm contour generated from the September 2013 profile survey, over the next 30 years approximately 54 structures and 20 swimming pools are at risk of damage due to the long-term erosion threat. This long-term erosion threat is limited to the area between D-14 and D-19 and no structures or pools are currently threatened given the existing berm crest location. The number of structures and pools at risk to long-term erosion are summarized in Table 30.

Table 30. Long-Term Erosion Threat – Existing Conditions

Profile		After 10 Years		After 30 Years	
From	To	Structure	Pool	Structure	Pool
D-01	D-14	0	0	0	0
D-14	D-15	0	1	13	6
D-15	D-16	1	3	13	5
D-16	D-17	1	1	13	3
D-17	D-18	0	3	9	4
D-18	D-19	0	0	6	2
D-19	D-34	0	0	0	0
D-01	**D-34**	**2**	**8**	**54**	**20**

15.2 Storm Damage Risk

The SBEACH model was used to identify parts of the Duck shoreline that require engineered features to protect upland structures from storm impacts. The vulnerability analysis detailed in the Erosion and Shoreline Management Feasibility Study (CPE, 2013) used profile survey data that preceded Hurricane Sandy and as a result did not capture the existing condition of the beach and dune system. Additionally, the vulnerability analysis was limited in scope as it used one typical profile for each of the ten shoreline segments analyzed. Although appropriate for a preliminary assessment, the formulation of a comprehensive shoreline protection and management plan requires a more detailed approach that includes multiple beach profiles to represent each shoreline segment. Therefore, as discussed in Section 6.3, a beach profile survey was conducted in September 2013 along the Town's shoreline to identify existing conditions which were then used to more thoroughly evaluate storm damage vulnerability using the SBEACH model.

The SBEACH model discussed in Section 14.1 was used to identify structures susceptible to storm damage. Initial conditions represent profile data collected September 2013, while model boundary conditions were defined using oceanographic and meteorological data collected at the FRF during Hurricane Isabel. The SBEACH modeled profile response was used to identify structures that could be impacted given the selected design storm. Using the 1-foot erosion criteria established in Section 14.1, the simulation identified 91 structures and 58 pools that are at risk of damage due to a storm similar to Hurricane Isabel. Of this total, 79 structures and 29 pools are located between D-10 and D-19. Maps that delineate the impact line and identify structures at risk to storm damage are provided in Figure 30. The number of structures and pools at risk to storm damage are summarized in Table 31. Other than identifying which buildings are vulnerable to storm damage, the analysis does not include other potential damages that are associated with storm surge (flooding), wave impacts, or wind. This storm damage risk assessment was conducted using the present position of the shoreline and profile condition. The number of structures at risk of storm damage would increase over time if the long-term erosion trends continue.

Notes:

1. 2012 Background imagery is from NC OneMap Imagery Service.

Legend:

Storm Impact Lines

- No Action
- Option 1
- Option 2
- Transects

Feet
0 50 100

1 inch = 100 feet

TITLE:

Storm Damage Risk Map
Duck, North Carolina
Sheet 1

Coastal Planning & Engineering
of North Carolina, Inc.
4038 Masonboro Loop Road
Wilmington, NC 28409
Ph. (910) 791-9494
Fax (910) 791-4129

Figure 30. Storm Damage Risk

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Notes:

1. 2012 Background imagery is from NC OneMap Imagery Service.

Legend:

Storm Impact Lines

- No Action
- Option 1
- Option 2

Transects

Feet
0 50 100
1 inch = 100 feet

TITLE:

Storm Damage Risk Map
Duck, North Carolina
Sheet 2

Coastal Planning & Engineering of North Carolina, Inc.
4038 Masonboro Loop Road
Wilmington, NC 28409
Ph. (910) 791-9494
Fax (910) 791-4129

Figure 30. Storm Damage Risk

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Notes:

1. 2012 Background imagery is from NC OneMap Imagery Service.

Legend:

Storm Impact Lines

- No Action
- Option 1
- Option 2

Transects

0 50 100 Feet

1 inch = 100 feet

TITLE:

Storm Damage Risk Map
Duck, North Carolina
Sheet 3

Coastal Planning & Engineering of North Carolina, Inc.
4038 Masonboro Loop Road
Wilmington, NC 28409
Ph. (910) 791-9494
Fax (910) 791-4129

Figure 30. Storm Damage Risk

G:\Enterprise\Dare\150440_DUCK Design and Permitting\DUCK\mxd\Duck_Storm_Damage_Risk_Map3.mxd

Table 31. Storm Damage Risk – Existing Conditions

Profile		Current Storm Damage Risk	
From	**To**	**Structure**	**Pool**
D-01	D-02	0	0
D-02	D-03	0	0
D-03	D-04	0	0
D-04	D-05	1	0
D-05	D-06	0	0
D-06	D-07	0	0
D-07	D-08	0	0
D-08	D-09	0	0
D-09	D-10	0	0
D-10	D-11	4	0
D-11	D-12	8	1
D-12	D-13	10	4
D-13	D-14	8	1
D-14	D-15	13	6
D-15	D-16	11	6
D-16	D-17	9	5
D-17	D-18	11	6
D-18	D-19	5	0
D-19	D-20	0	0
D-20	D-21	0	0
D-21	D-22	0	0
D-22	D-23	0	0
D-23	D-24	0	0
D-24	D-25	0	0
D-25	D-26	1	2
D-26	D-27	0	2
D-27	D-28	0	2
D-28	D-29	5	5
D-29	D-30	1	3
D-30	D-31	0	2
D-31	D-32	2	7
D-32	D-33	1	2
D-33	D-34	1	4
D-01	**D-34**	**91**	**58**

The risk of a storm comparable to the design storm (Hurricane Isabel) impacting the area over the next 30 years was evaluated to provide guidance for planning purposes. In this regard, assuming Hurricane Isabel has a 4% (25-year storm) to 5% (20-year storm) chance of occurring any given year, the risk of a similar storm impacting the Town of Duck within the next 5 years would be between 18% and 23%. Over the next 15 years, the risk would increase to be between 46% and 54%. The risk of several return period events (design storms) occurring within various time periods is provided in Table 32.

Table 32. Design Storm Risk

Time Period (years)	Return Period Event 1-Year	5-Year	10-Year	20-Year	25-Year	50-Year
1	100%	20%	10%	5%	4%	2%
2	100%	36%	19%	10%	8%	4%
3	100%	49%	27%	14%	12%	6%
4	100%	59%	34%	19%	15%	8%
5	100%	67%	41%	23%	18%	10%
10	100%	89%	65%	40%	34%	18%
15	100%	96%	79%	54%	46%	26%
20	100%	99%	88%	64%	56%	33%
25	100%	100%	93%	72%	64%	40%
30	100%	100%	96%	79%	71%	45%

15.3 Project Extents

Critical review of long-term erosion and storm damage threats discussed above suggests that the beach between D-10 and D-19 has the potential to realize the greatest benefit from a shoreline protection project. Though structures were characterized as impacted along other parts of the shoreline, the impacts were limited to either individual structures that extended further seaward relative to adjacent structures or the at risk structures were isolated from other at risk structures. In this latter case, the extension of the project to protect the isolated structures would not be economically justified.

To more accurately define the project extents, storm damage threats were further analyzed between D-10 and D-11 and between D-18 and D-19. Considering the inexact nature of the SBEACH analysis, Town engineers suggested including a 15 foot buffer when determining impacts at the project extents. This additional criterion identifies D-19 as the southern project limit and a location between D-10 and D-11 as the northern project limit. Applying this 15 foot buffer to the GIS footprints of the structures between D-10 and D-11 suggests that the project should extend north to the northern boundary of 140 Skimmer Way. Therefore, the project is designed to protect structures along the 7,970 feet of shoreline between 140 Skimmer Way (D-10.5) and D-19.

16 CROSS-SECTION DESIGN

The SBEACH model was used to evaluate various beach fill design cross-sections. Designs were evaluated on their ability to mitigate design storm impacts to structures fronting the beach. Designs tested include beach fill cross-sections with berms of varying width as well as beach fill cross-sections that include both a berm and dune, varying both the width and elevation of the dune and the width of the berm. The crest elevation of all berm designs was set at +6 feet NAVD, while dunes with crest elevations of +15 and +20 feet NAVD were tested. Berm widths were varied in 20 foot increments from no berm (a berm crest width of 0 feet) to a berm crest width of 100 feet, while dune widths were varied in 20 foot increments from no dune (a dune crest width of 0 feet) to a dune crest width of 100 feet. Therefore, a total of 65 design cross-sections were tested along each transect.

Design cross-sections were developed such that the landward toe of fill remains seaward of the structure line. The structure line inflection points were identified along profiles surveyed September 2013. While the line was delineated such that it remained seaward of structures located between profiles, some exceptions were made for individual structures that were clearly outliers to the general trend.

The slope of the dune on both the landward and seaward sides was set to 1V:5H, while the slope of the berm was set to 1V:15H. Both the dune and the berm were positioned to minimize volume while keeping the landward toe of fill seaward of the structure line. Any seaward extension of the shoreline was made by translating the profile seaward between the berm (+6 feet NAVD) and the depth of closure (-24 feet NAVD) as discussed in Section 14.1 and shown in Figure 29.

Each design cross-section was tested along all profiles surveyed in September 2013 using the same storm that was used to evaluate the existing conditions (see discussion in Section 14.1). *Impact points* along each profile were identified using the procedure detailed in Section 14.1. The distance between these *impact points* and the location where the structure line crossed the profile was calculated, with positive distances indicating that the *impact point* was seaward of the structure line and negative distances indicating that the *impact point* was landward of the structure line. A design matrix was then created to facilitate the identification of minimum beach fill cross-section designs that provide an adequate level of storm protection.

As one might infer, the design matrix showed that the placement of additional beach fill volume resulted in additional protection. However, the intent of the matrix was to help optimize the design cross-section to provide the greatest protection at the lowest cost (volume). Further review of the design matrix suggests that protection offered may be better related to the volume above a specific contour, such as the volume of sand above the berm. This became evident when comparing designs that resulted in similar protection. For example, the designs summarized in Table 33 offer similar protection but require different quantities of material to fill the design template.

Table 33. Cross-Section Comparison

Dune Design		Berm Design		Average
Elevation (ft,NAVD)	**Width (ft)**	**Elevation (ft,NAVD)**	**Width (ft)**	**Density (cy/ft)**
15	20	6	100	106.4
15	40	6	60	89.4
20	20	6	40	63.2

This design comparison suggests that the volume of fill required for the same protection decreases as the volume of fill placed above the berm increases. Considering that higher and wider dunes also increase volume, maximum dune dimensions were identified to drive the design. Review of the design matrix showed that designs containing dunes with a crest elevation of +20 feet NAVD and a crest width of 20 feet performed similar to volumetrically equivalent designs that contained dunes with a crest elevation of +15 feet NAVD and a crest width of 40 feet. Considering that both dune designs are able to meet project goals and are similar in volume, the surveys conducted September 2013 were inspected to identify a healthy dune crest elevation and profile shape. The September 2013 survey revealed that healthy profiles contained dunes with crest elevations around +20 feet NAVD. Therefore, all dune options evaluated in detail included a +20 foot NAVD dune with a crest width of 20 feet.

Based on this preliminary assessment of multiple beach fill design templates, two options were selected for detailed evaluation. These options, designated as Option 1 and Option 2, are described in detail below along with an assessment of their ability to meet the Town's stated goals and objectives. Both options include a dune with a crest elevation of +20 feet NAVD and a crest width of 20 feet. Option 1 includes a 60-foot wide berm at elevation +6 feet NAVD while Option 2 includes an 80-foot wide berm at elevation +6 feet NAVD.

Following the selection of the preferred design template, the plan layout of the selected option was modified based on the results of a one-year simulation using GENESIS. This modification is explained below.

17 BEACH FILL DESIGN OPTIONS

A standard beach nourishment design consists of two primary components:

1. Design Section. The design section is the cross-section required to meet project objectives.

2. Advanced Fill. Advanced fill is the sacrificial portion of the fill required to protect the design section from anticipated sediment losses.

This two section design is in accordance with the National Research Council (1995) recommendations. In this study, the various design sections consist of a dune with a 20 foot wide crest at an elevation of +20 feet NAVD fronted by a berm with a crest elevation of +6 feet NAVD; the berm crest width was varied to develop beach fill options that provide different levels of protection. Advanced fill requirements were defined using background erosion rates and modeled diffusion losses. Advanced fill calculations and optional design details are provided in the following sections.

17.1 Background Erosion Losses

A key component of a beach fill design is an assessment of periodic nourishment requirements needed to maintain the design profile during the interim period between nourishment events. The sediment budget developed in Section 13 details the various losses (longshore transport, relative sea level rise, etc.) associated with background erosion. Based on this analysis and the resultant transport curves (see Figure 27 and Figure 28), volumetric losses from a fill placed between D-10 and D-19 are estimated to be somewhere between 20,000 and 35,000 cubic yards of material each year. The difference between these estimates is the result of the increased erosion that occurred during Hurricane Sandy. Therefore, considering the 7,970 foot project length, somewhere between 2.5 and 4.4 cubic yards per foot per year of advanced fill is required to offset background erosion.

Shoreline change rates were compared to determine which background erosion rate should be used for project planning purposes, i.e. 2.5 or 4.4 cubic yards per foot per year. Annualized shoreline change rates were calculated along transects within the project area using the 1996 LIDAR, 2011 LIDAR, 2012 LIDAR, and 2013 profile survey data. All shoreline changes used the 1996 LIDAR shoreline as the baseline. Average shoreline change calculated within the project area was -1.8 feet per year using the 2011 LIDAR survey, -3.4 feet per year using the 2012 post Hurricane Sandy LIDAR survey, and -1.5 feet per year using the 2013 profile survey. A plot of the annualized shoreline change rates within the project area is provided in Figure 31.

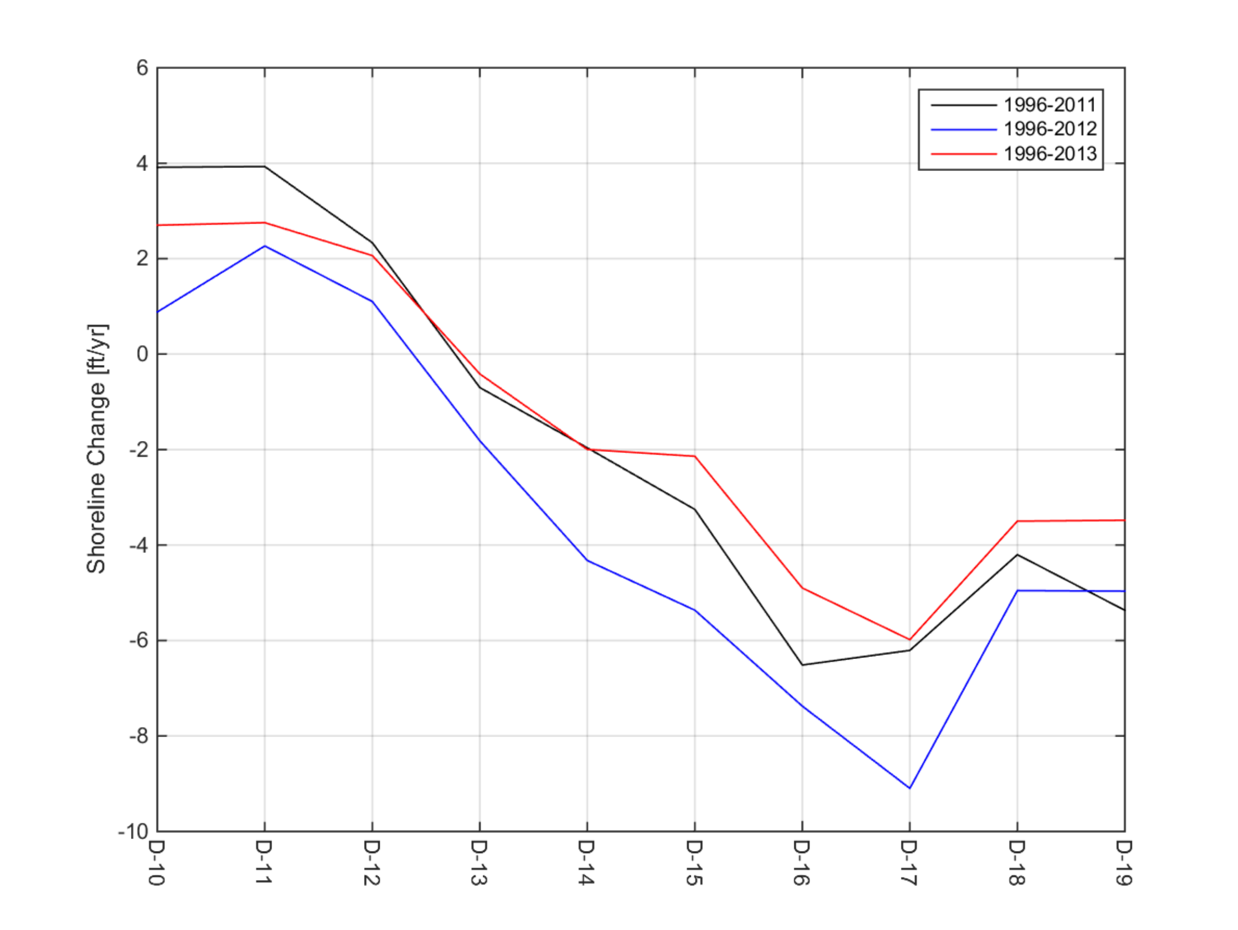

Figure 31. Project Area Shoreline Change Rate Comparison

As shown in Figure 31, the shoreline changes calculated for the 1996-2013 time period agree reasonably well with the 1996-2011 rates whereas the rate determined between 1996 and 2012, which included the immediate after effects of Hurricane Sandy, indicate higher recession rates along the entire project shoreline. This is indicative of the relative short-term impacts of a storm like Hurricane Sandy. As discussed in Section 12.2, the profile changes associated with Hurricane Sandy included the removal of material from the upper portion of the profile and deposition in a bar close to shore. The profile comparisons also showed erosion along the deeper portion of the profile seaward of the bar formed during the storm. With the passage of the storm and a return to normal wave conditions, the storm-created near shore bar migrated back on shore restoring some of the beach width lost to Sandy. Therefore, the background erosion losses calculated between 1996 and 2011 were deemed to be more appropriate to use for project planning purposes. Based on the estimated volume losses between 1996 and 2011, an additional 20,000 cubic yards, or 2.5 cubic yards per foot, of material should be placed in front of the design template to offset each year of background erosion.

17.2 Diffusion Losses

In addition to gradual loss of material due to background erosion, any beach fill placed along a shoreline is also subject to diffusion losses. Diffusion or spreading will occur with any sand placement activity as the nourished beach evolves into an equilibrium planform comparable to the adjacent shorelines (Dean, 2002). Diffusion losses are the result of the fill template spreading alongshore and occur when the fill material spreads outside the fill placement or project area. The feasibility study (CPE, 2013) included preliminary estimates of diffusion losses by employing a simple diffusion model. However, this design study employs the GENESIS model to obtain a better estimate of how the project will perform. As discussed in Section 14.2, GENESIS simulates the long-term planform evolution of the beach in response to wave conditions, coastal structures, and other engineering activity including beach nourishment.

Unlike the simple diffusion model used in the feasibility study (CPE, 2013), which was based on a single representative wave condition, diffusion in GENESIS is driven by actual wave data. To estimate diffusion losses, both the existing and design shoreline were modeled in GENESIS using the same boundary conditions, with the only difference being the initial position of the shoreline. Considering diffusion losses are greater for a larger shoreline perturbation, the design shoreline modeled was developed using the largest cross-section considered for development. Thus, the design shoreline represented the shoreline that would be in-place following the construction of a beach fill template between D-10 and D-19 that includes a 20 foot wide dune at +20 feet NAVD with an 80 foot wide berm at +6 feet NAVD. No taper was included so that diffusion loss estimates remain conservative and are viable for any taper design length.

The existing and design shorelines were modeled in GENESIS using 10 years of wave data that extends between September 2002 and September 2012. Volumetric differences were calculated at the end of each simulation year using modeled shoreline changes. These volumetric differences can be described as background erosion for the existing conditions, while the volumetric difference is a combination of background erosion and diffusion losses for the design simulation. Diffusion losses were estimated by subtracting the background erosion, or erosion modeled for the existing conditions, from the erosion modeled for the design conditions. Modeled erosion and diffusion loss estimates are provided in Table 34. Diffusion losses are expected to decrease with time as the planform perturbation becomes less pronounced, which is shown in Table 34 as the annualized diffusion loss decreases with time though the cumulative diffusion loss continues to increases. The GENESIS results suggest that roughly 45,000 cubic yards of material will be lost due to diffusion the first year, but this decreases to an annual diffusion loss of 30,300 cubic yards by year 4 and 21,400 cubic yards by year 10.

Table 34. GENESIS Modeled Diffusion Losses

Year	Erosion (cy) Existing	Erosion (cy) Design	Diffusion Loss (cy)	Diffusion Loss (cy/yr)
1	25,000	70,000	45,000	45,000
2	45,000	117,000	72,000	36,000
3	63,000	163,000	100,000	33,300
4	82,000	203,000	121,000	30,300
5	99,000	238,000	139,000	27,800
6	116,000	273,000	157,000	26,200
7	132,000	305,000	173,000	24,700
8	146,000	335,000	189,000	23,600
9	161,000	364,000	203,000	22,600
10	183,000	397,000	214,000	21,400

Diffusion losses estimated as part of the Feasibility Study (CPE, 2013) were much higher than those calculated using GENESIS. This previous analysis estimated diffusion losses using Dean's simplified method (Houston, 1996). Dean's simplified method is a function of the fill segment length, berm width (or shoreline advance), active profile height, sediment grain size, and the breaking wave height. All of these input parameters, except the breaking wave height, can be defined using design information presented in this report. The diffusion analysis completed as part of the Feasibility Study (CPE, 2013) estimated a breaking wave height of 1.6 feet using SBEACH and offshore wave records. Though this is a reasonable estimate of the average breaking wave height, it results in a diffusion loss of approximately 210,000 cubic yards over the first year and roughly 780,000 cubic yards over 5 years. Considering that the breaking wave height employed in the analysis was calculated using linear wave theory and does not include energy dissipation due to friction and other processes, a sensitivity analysis of this variable was completed. Decreasing the breaking wave height to 1 foot decreases diffusion losses to 104,000 cubic yards during the first year, while decreasing the breaking wave height to 0.5 feet decreases diffusion losses to 40,000 cubic yards during the first year. This sensitivity analysis suggests that the model is highly sensitive to the breaking wave height. Therefore, in an effort to tune the analytic diffusion model, a breaking wave height of 0.55 feet was found to produce diffusion loss estimates similar to those calculated using GENESIS.

Table 35. Diffusion Model Comparison

	Cumulative Diffusion Loss (cy)						
Year	GENESIS	Hb = 2.0 ft	Hb = 1.5 ft	Hb = 1.0 ft	Hb = 0.5 ft	Hb = 0.6 ft	Hb = 0.55 ft
1	45,000	306,000	190,000	104,000	40,000	51,000	46,000
2	72,000	526,000	315,000	164,000	61,000	79,000	70,000
3	100,000	738,000	439,000	220,000	80,000	103,000	91,000
4	121,000	947,000	564,000	277,000	98,000	127,000	112,000
5	139,000	1,155,000	691,000	335,000	116,000	150,000	132,000
6	157,000	1,363,000	820,000	396,000	133,000	174,000	153,000
7	173,000	1,573,000	952,000	458,000	151,000	198,000	174,000
8	189,000	1,785,000	1,086,000	523,000	170,000	223,000	196,000
9	203,000	2,000,000	1,223,000	589,000	189,000	249,000	218,000
10	214,000	2,218,000	1,363,000	658,000	208,000	275,000	240,000

17.3 Taper Design

GENESIS was used to evaluate various taper or transition lengths at each end of the proposed beach fill project in an effort to minimize periodic nourishment requirements and thus periodic nourishment costs. This exercise considered if extending the taper would decrease periodic nourishment requirements sufficiently to offset the added placement cost associated with the initial construction of the taper. For the same design (as discussed in Section 17.2 above), four different taper lengths (no taper – abrupt stop, 500 feet, 1,000 feet, and 1,500 feet) were modeled using GENESIS. The performance of the various taper designs was simulated in GENESIS using 10 years of wave data that extends between September 2002 and September 2012. Diffusion losses for each taper length modeled in GENESIS were calculated using the methodology discussed in Section 17.2.

Diffusion losses modeled using GENESIS were compared with taper volumes to evaluate taper benefits. The combined northern and southern taper volumes for the 500 foot, 1,000 foot, and 1,500 foot taper designs are 27,000 cubic yards, 54,000 cubic yards, and 81,000 cubic yards, respectively. Taper benefits were calculated by subtracting the taper volume from the diffusion loss reduction volume; positive benefits indicate that the volume in the taper is less than the diffusion loss reduction volume while negative benefits indicate that the taper volume is greater than the diffusion loss reduction volume. Results of the taper benefit analysis are detailed in Table 36. Though diffusion losses are reduced with longer taper designs, this analysis suggests that the additional volume to construct the taper is greater than the diffusion loss volume reduction. Considering that there is no benefit to constructing a taper, the taper was designed for constructability purposes alone. Therefore, a 500 foot taper is recommended for constructability purposes.

Table 36. Taper Benefit Analysis

	Cumulative Diffusion Loss (cy)				Taper Benefit (cy)[1]		
Year	0 Feet	500 Feet	1,000 Feet	1,500 Feet	500 Feet	1,000 Feet	1,500 Feet
1	45,000	29,000	18,000	11,000	-11,000	-27,000	-47,000
2	72,000	54,000	39,000	28,000	-9,000	-21,000	-37,000
3	100,000	82,000	64,000	49,000	-9,000	-18,000	-30,000
4	121,000	102,000	83,000	66,000	-8,000	-16,000	-26,000
5	139,000	121,000	100,000	82,000	-9,000	-15,000	-24,000
6	157,000	138,000	116,000	97,000	-8,000	-13,000	-21,000
7	173,000	154,000	131,000	111,000	-8,000	-12,000	-19,000
8	189,000	170,000	147,000	125,000	-8,000	-12,000	-17,000
9	203,000	183,000	160,000	138,000	-7,000	-11,000	-16,000
10	214,000	195,000	171,000	149,000	-8,000	-11,000	-16,000

[1]Example: Taper Benefit for Year 1 and 500-ft taper = 45,000 cy loss for 0-ft taper – 29,000 cy loss with 500-ft taper – 27,000 cy to initially construct the taper = -11,000 cy.

Though the proposed beach fill project will include 500 foot taper sections at each end of the fill area, waves acting on the fill will generally transport material away from the immediate placement area and spread it along the adjacent shoreline. As spreading occurs along the constructed section of beach, adjacent shoreline segments benefit. The term feeder beach has been used to describe the process of overfilling a relatively short section of beach to provide nourishment to adjacent beaches as spreading occurs. The tuned analytic diffusion model presented in Section 17.2 suggests that the fill will spread both north and south out of the initial placement area and eventually spread over most of the Town's shoreline. Based on this simulation, the shoreline 2,000 feet beyond the fill placement limits could advance approximately 10% of the constructed width in the placement area. This example demonstrates the potential positive impact of fill spreading outside the initial placement area.

17.4 Development of Beach Fill Design Options

The evaluation of the two optional beach fill designs was based on the assumption that the full beach fill design profile will be in place when the design storm impacts the project area. This was accomplished by placing advanced fill in front of the design to compensate for background erosion and diffusion losses. Considering a five year nourishment interval, the volume of material that would be lost due to background erosion is expected to be somewhere around 100,000 cubic yards. Given a 500 foot taper, an additional 120,000 cubic yards should be added to the advanced fill volume to compensate for anticipated diffusion losses. Therefore, a total of 220,000 cubic yards (or 27.6 cubic yards per foot) of fill should be placed in front of the design so that at the end of the five year nourishment cycle only the advanced fill volume will have been lost leaving the project design template in place. Considering the 500-foot taper sections that are proposed for the northern and southern project extents, an additional 14,000 cubic yards of material will be needed to initially fill the transition section resulting in a periodic nourishment or advanced fill volume of 234,000 cubic yards.

Each option extends along 7,970 feet of shoreline between D-10 and D-19, which excludes the 500 foot tapers at both the northern and southern limits of the fill section. A typical cross-section plot is shown in Figure 32, while a plan view drawing of the fill placement area is presented in Figure 33. A summary of the fill volumes shoreline advance, and berm width is provided in Table 37.

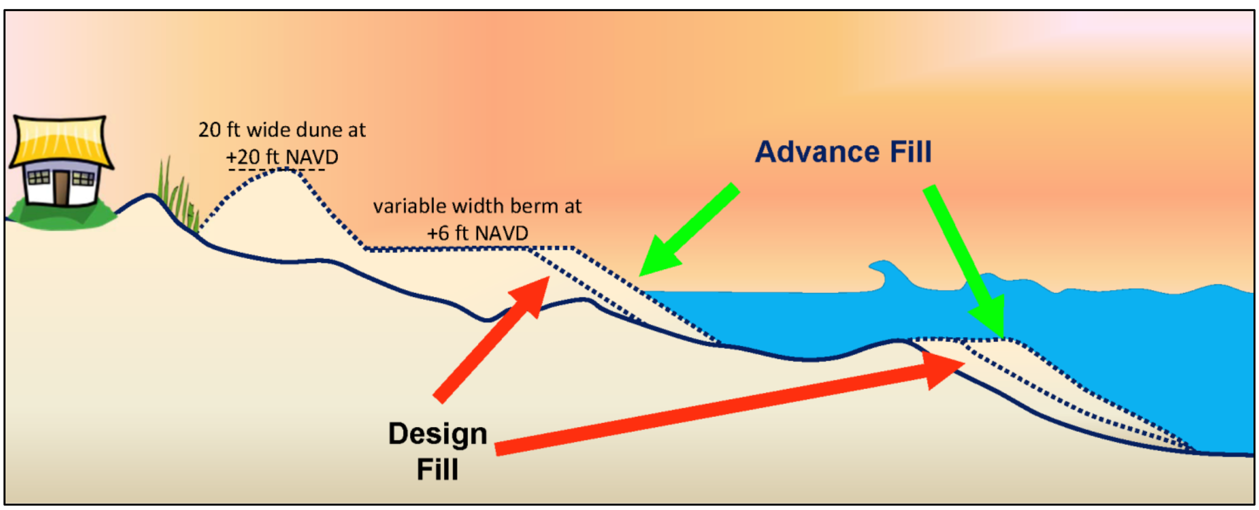

Figure 32. Optional Beach Fill Design Cross-Sections

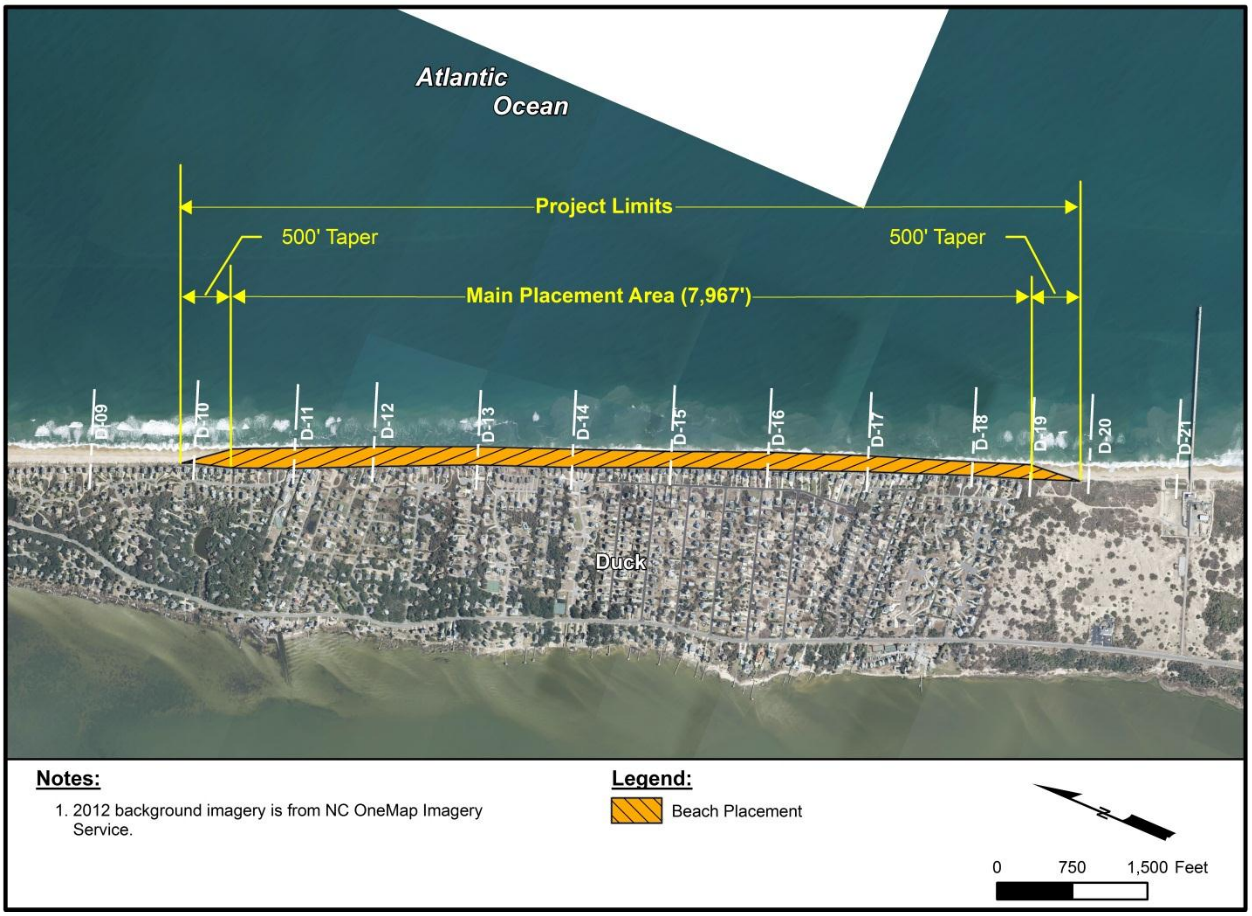

Figure 33. Beach Fill Plan View

Table 37. Optional Design Summary

	Volume (cy)		Shoreline Advance (ft)			Berm Width (ft)		
Design	**Design(1)**	**Fill(2)**	**Avg**	**Min**	**Max**	**Avg**	**Min**	**Max**
Option 1	806,000	1,040,000	65.0	28.8	92.0	53.6	28.7	60.0
Option 2	1,007,000	1,241,000	85.0	48.8	112.0	73.6	48.7	80.0

(1)Volume to construct the design template excluding tapers and advanced fill.
(2)Construction volume including tapers and advanced fill.

Both beach fill design options include a 20 foot wide dune with a crest elevation of +20 feet NAVD and a berm with a crest elevation of +6 feet NAVD. The nominal width of the berm for Option 1 and Option 2 are 60 feet and 80 feet, respectively. All landward dune crests were defined as either the +20 foot NAVD contour, if that was seaward of the structure line, or the location that minimizes volume where the landward toe of fill remains seaward of the structure line using a 1V:5H dune slope. The landward berm crest was identified by sloping down on a 1V:5H slope from the seaward dune crest; if this point was landward of the existing +6 feet NAVD contour, the landward berm crest was defined as the existing +6 feet NAVD contour. The seaward face of the design template extends from the seaward berm crest to the depth of closure and maintains the shape of the existing profile between these limits.

17.4.1 Option 1

Option 1 includes a 20 foot wide dune with a crest elevation of +20 feet NAVD and a 60 foot wide berm with a crest elevation of +6 feet NAVD. The average shoreline advance for Option 1 is 65 feet, while the maximum and minimum shoreline advance is 92 and 29 feet, respectively. Option 1 requires 806,000 cubic yards of design fill and 234,000 cubic yards of advanced fill for a total fill volume of 1,040,000 cubic yards. A summary of the Option 1 shoreline advance, design width, and fill density along each profile within the project area is provided in Table 38.

Table 38. Option 1 Design Characteristics

	Shoreline	Design Berm Width (ft)		Fill Density (cy/ft)		
Profile	Advance (ft)	Effective	Platform	Design	Advanced	Total
D-10	28.8	60.0	28.7	41.9	27.6	69.5
D-11	41.4	60.0	41.4	58.6	27.6	86.2
D-12	49.3	60.0	49.3	71.2	27.6	98.8
D-13	75.2	60.0	60.0	102.5	27.6	130.1
D-14	77.3	60.0	60.0	107.6	27.6	135.2
D-15	92.0	60.0	60.0	128.7	27.6	156.3
D-16	57.0	60.0	57.0	82.9	27.6	110.5
D-17	89.3	60.0	60.0	131.3	27.6	158.9
D-18	78.5	60.0	60.0	110.3	27.6	137.9
D-19	61.0	60.0	60.0	88.6	27.6	116.2
Avg	**65.0**	**60.0**	**53.6**	**92.4**	**27.6**	**120.0**
Min	**28.8**	**60.0**	**28.7**	**41.9**	**27.6**	**69.5**
Max	**92.0**	**60.0**	**60.0**	**131.3**	**27.6**	**158.9**

17.4.2 Option 2

Option 2 includes a 20 foot wide dune with a crest elevation of +20 feet NAVD and an 80 foot wide berm with a crest elevation of +6 feet NAVD. The average shoreline advance for Option 2 is 85 feet, while the maximum and minimum shoreline advance is 112 and 49 feet, respectively. Option 2 requires 1,007,000 cubic yards of design fill and 234,000 cubic yards of advanced fill for a total fill volume of 1,241,000 cubic yards. A summary of the Option 2 shoreline advance, design width, and fill density along each profile within the project area is provided in Table 39.

Table 39. Option 2 Design Characteristics

Profile	Shoreline Advance (ft)	Design Berm Width (ft) Effective	Platform	Fill Density (cy/ft) Design	Advanced	Total
D-10	48.8	80.0	48.7	65.4	27.6	93.0
D-11	61.4	80.0	61.4	81.7	27.6	109.3
D-12	69.3	80.0	69.3	94.3	27.6	121.9
D-13	95.2	80.0	80.0	126.5	27.6	154.1
D-14	97.3	80.0	80.0	131.3	27.6	158.9
D-15	112.0	80.0	80.0	153.2	27.6	180.8
D-16	77.0	80.0	77.0	105.9	27.6	133.5
D-17	109.3	80.0	80.0	156.8	27.6	184.4
D-18	98.5	80.0	80.0	134.0	27.6	161.6
D-19	81.0	80.0	80.0	111.8	27.6	139.4
Avg	**85.0**	**80.0**	**73.6**	**116.1**	**27.6**	**143.7**
Min	**48.8**	**80.0**	**48.7**	**65.4**	**27.6**	**93.0**
Max	**112.0**	**80.0**	**80.0**	**156.8**	**27.6**	**184.4**

18 PROJECT PERFORMANCE

The ability of Options 1 and 2 to reduce the threats of damage due to long-term erosion and storms were analyzed to identify parts of the shoreline where structures would remain vulnerable to the effects of chronic erosion and episodic storm events.

18.1 Long-Term Erosion Threat

Both design options include advanced fill to account for projected volume losses likely to occur over 5 years. At the end of 5 years or when the loss of some or all of the advanced fill poses a threat to the integrity of the design template (whichever occurs first), periodic nourishment would be accomplished to restore the advanced fill. Thus, in theory, the design template would be preserved in perpetuity thus eliminating potential damages to existing structures and infrastructure due to the effects of long-term erosion. Under existing or without project conditions, damages over the 30-year planning period were projected to include 54 structures and 20 pools. All of these potential long-term erosion impacts would occur within the proposed beach fill area. Implementation of either beach fill option would eliminate this potential loss.

18.2 Storm Damage Risk

Storm damage risks associated with the beach fill options were evaluated using a procedure similar to that presented in Section 15.2.

Initial conditions represent design cross-sections of the two beach fill options, while model boundary conditions were defined using oceanographic and meteorological data collected at the FRF during Hurricane Isabel. The SBEACH modeled profile response was used to identify structures that could be impacted given the selected design storm using the 1 foot erosion criteria established in Section 14.1. A map that delineates the impact line and identifies structures at risk to storm damage for the two design options is provided in Figure 30. The tax values and number of structures at risk to storm damage for the design options are summarized in Table 40 and

Table 41. Structure tax values were obtained from the Dare County website. Other than identifying which buildings are vulnerable to storm damage, the analysis does not include other potential damages that are associated with storm surge (flooding), wave impacts, or wind.

18.2.1 Option 1

Option 1 would result in the number of structures at risk to storm damage being reduced by 86% within the fill area, decreasing from 79 to 11. A summary of the Option 1 storm damage risk reduction is provided in Table 40.

Table 40. Option 1 - Storm Damage Risk Reduction

Profile			Structure	
From	To	Impacts	Benefits	Reduction
D-10	D-11	3	1	25%
D-11	D-12	5	3	38%
D-12	D-13	1	9	90%
D-13	D-14	0	8	100%
D-14	D-15	0	13	100%
D-15	D-16	0	11	100%
D-16	D-17	0	9	100%
D-17	D-18	2	9	82%
D-18	D-19	0	5	100%
Project Area		**11**	**68**	**86%**

18.2.2 Option 2

Option 2 would result in the number of structures at risk to storm damage being reduced by 94% within the fill area, decreasing from 79 to 5. A summary of the Option 2 storm damage risk reduction is provided in Table 41.

Table 41. Option 2 Storm - Damage Risk Reduction

Profile			Structure	
From	To	Impacts	Benefits	Reduction
D-10	D-11	1	3	75%
D-11	D-12	4	4	50%
D-12	D-13	0	10	100%
D-13	D-14	0	8	100%
D-14	D-15	0	13	100%
D-15	D-16	0	11	100%
D-16	D-17	0	9	100%
D-17	D-18	0	11	100%
D-18	D-19	0	5	100%
Project Area		**5**	**74**	**94%**

18.2.3 Comparison – Option 1 versus Option 2

Comparing the results of the assessment of Options 1 and 2, Option 2 would provide a higher degree of storm damage protection at a minimal increase in cost. In this regard, the additional volume of material needed to initially construct Option 2 is only 201,000 cubic yards greater than Option 1 yet Option 2 has the potential to provide storm protection to 6 more homes than Option 2. The difference in the amount of potential storm damage reduction over the 30-year evaluation period between Option 1 and Option 2 would be an increase in damage reduction of approximately 8%. While the cost to initially construct Option 2 would be more than Option 1, the cost of periodic nourishment of the two options would be the same. Given the inherent inaccuracies and uncertainties associated with the comparative analysis, the higher level of protection that would be provided by Option 2 should be viewed as simply adding a factor of safety to level of protection expected from the beach nourishment project. Based on this assessment, a third option, designated as Option 3, which is a variant of Option 2, was developed and selected as the preferred beach fill option. Option 3 is discussed below.

18.2.4 Option 3 – Recommended Option

Option 3 includes a 20 foot wide dune with a crest elevation of +20 feet NAVD and a variable width berm with a crest elevation of +6 feet NAVD. This design is essentially the same as Option 2 except GENESIS was used to align the shoreline and set the berm width. All options previously discussed were developed by placing the same cross-section template at different locations along the various profiles such that the volume was minimized and the landward toe of fill remained seaward of the structure line. This resulted in a design shoreline that did not have a shape similar to the existing shoreline but would likely equilibrate to a smooth shoreline essentially parallel to the existing shoreline within a year. In an effort to smooth (equilibrate) the design shoreline, GENESIS was used to estimate the alignment of the Option 2 shoreline following one year of adjustment. This adjusted shoreline was then used to set the seaward berm crest location for Option 3 and resulted in a berm varying in width from 44 to 87 feet. As with all other options, the seaward face of the design template extends from the seaward berm crest to the depth of closure and maintains the shape of the existing profile between these limits. The average shoreline advance for Option 3 is 67 feet. Alternative 3 requires 835,000 cubic yards of design fill and 234,000 cubic yards of advanced fill for a total fill volume of 1,069,000 cubic yards which is only 29,000 cubic yards more than Option 1 and 172,000 cubic yards less than Option 2. A summary of the Option 3 shoreline advance, design width, and fill density along each profile within the project area is provided in Table 42.

Table 42. Option 3 Design Characteristics

Profile	Shoreline Advance (ft)	Design Berm Width (ft) Effective	Platform	Fill Density (cy/ft) Design	Advanced	Total
D-10	13.2	44.4	13.1	23.6	27.6	51.2
D-11	46.4	65.0	46.4	64.1	27.6	91.7
D-12	66.5	77.2	66.5	91.0	27.6	118.6
D-13	83.8	68.6	68.6	112.4	27.6	140.0
D-14	85.3	68.0	68.0	116.4	27.6	144.0
D-15	84.6	52.5	52.5	119.6	27.6	147.2
D-16	83.9	86.8	83.9	114.8	27.6	142.4
D-17	86.4	57.1	57.1	127.8	27.6	155.4
D-18	63.8	45.3	45.3	92.6	27.6	120.2
D-19	51.4	50.3	50.3	76.7	27.6	104.3
Avg	**66.5**	**61.5**	**55.2**	**93.9**	**27.6**	**121.5**
Min	**13.2**	**44.4**	**13.1**	**23.6**	**27.6**	**51.2**
Max	**86.4**	**86.8**	**83.9**	**127.8**	**27.6**	**155.4**

Option 3 would result in the number of structures at risk to storm damage being reduced by 90% within the fill area, decreasing from 79 to 8. A summary of the Option 3 storm damage risk reduction is provided in Table 43.

Table 43. Option 3 - Storm Damage Risk Reduction

Profile From	To	Impacts	Structure Benefits	Reduction
D-10	D-11	1	3	75%
D-11	D-12	4	4	50%
D-12	D-13	2	8	80%
D-13	D-14	0	8	100%
D-14	D-15	0	13	100%
D-15	D-16	0	11	100%
D-16	D-17	0	9	100%
D-17	D-18	1	10	91%
D-18	D-19	0	5	100%
Project Area		**8**	**71**	**90%**

19 CONSTRUCTION

Either an ocean-certified self-contained hopper dredge with direct pump-out, a cutterhead suction dredge, or a combination of the two will likely be used to obtain material from the borrow areas to construct the beach fill project. The type of dredge utilized will depend on many factors, including competition in the bid process, pumping or haul distance, and depth and extent of dredging. The offshore borrow areas identified in Section 7 are subject to the most severe wave climate along the entire East Coast of the United States. Therefore, the potential for adverse sea conditions and adherence to construction schedule will be major factors to consider when selecting the dredging methods and equipment. A description of dredge types is provided below.

- **Hopper Dredges.** A hopper dredge is a self-propelled, maneuverable vessel that can independently load, transport, and unload dredged material. The hopper dredge has a trailer suction pipe with a draghead that strips layers of sediment and hydraulically suctions the material into the hopper. For the proposed project, material would be offloaded by direct pump-out through a submerged pipeline while the vessel is moored offshore.

- **Cutter Suction Dredge.** A cutter suction dredge can be self-propelled or require a barge for transport. During operation, the cutter suction dredge is anchored either by a spud at one corner or by wires held in place by anchors. In general, wires and anchors are used when operating in a wave environment. During dredging, material is hydraulically pumped up the suction pipe and discharged at a disposal site (may be upland or in-water) or to a barge for transport to the disposal site. Cutter suction dredges are limited by sea-state condition and do not perform well in areas of elevated sea states.

Once the material is discharged from the pipe onto the beach, onshore construction crews shape the material into the desired construction template. The material is typically managed in a way that reduces turbidity by constructing shore parallel dikes along which the water from the slurry flows, allowing additional time for material to settle out of suspension before the seawater returns to the ocean. Equipment such as bulldozers and front-end-loaders are used to shape sand on the beach and move pipes as necessary. At the location where the submerged pipeline comes ashore, the slurry flow is typically diverted with a 90-degree elbow to direct the flow towards the project area. As portions of the project are constructed, the pipeline is extended to allow for the next section of beach to be constructed.

The Town aims to complete the project in the shortest time practicable during a safe operating period and with the least environmental impact possible. Weather and sea-state conditions play a crucial role in the safety and efficiency of offshore dredging projects, particularly during the winter. The wave climate in the northern Outer Banks is reportedly among the most inclement on the United States Eastern Coast (Leffler et al., 1996). The Final Environmental Impact Statement (FEIS) written in association with the 2010 Nags Head Beach Nourishment project presents a detailed analysis of the local offshore wave climate. Data were obtained from the USACE FRF and are considered representative of conditions offshore. The USACE (2010) analyzed a three year record of wave heights between January 2003 and December 2005 collected at FRF630. The USACE reported that during the three year period analyzed, there was an annual average of 59 weather events producing wave heights in excess of 1.6 meters, and an average of 5.3 storm events producing wave heights greater than 3.4 meters. Two storm events, one of which was Hurricane Isabel, produced wave heights in excess of 7.0 meters. Historical data, as presented in Section 6.1.1, also shows that the wave climate in the northern Outer Banks varies seasonally.

The Nags Head EIS and feasibility study developed for the 2010-2011 Nags Head project suggest that, based on conditions encountered during two previous projects constructed in North Carolina, there is an inverse relationship between wave height and dredging efficiency (USACE, 2000; USACE 2010). Larger, steeper waves are frequently generated by winter storms and adversely impact dredging operations by decreasing safety, increasing downtime, and thus increasing total project cost. In the Nags Head FEIS, dredging efficiency for Dare County was calculated based on two other dredging projects completed in North Carolina and was estimated to range from 81% in July to only 46% in February (USACE, 2000). A complete, detailed analysis is included in the Biological Assessment developed for the 2010 Nags Head Beach Nourishment project (USACE, 2010, Appendix H – Attachment 8) and is incorporated here by reference.

Due to the aforementioned sea state conditions, dredging during the winter months (October to March) increases the risk to crews and equipment and reduces dredging efficiency. This can result in a longer construction period, which increases construction cost and potentially prolongs environmental impacts. Risks translate directly into costs whether the risks are related to safety, weather, financial, environmental, or other factors. The downtime associated with shutdown and redeployment represents the main factor contributing to inefficiency and the overall economics of the project. In a letter addressed to the Town of Nags Head, the Technical Director from the Dredging Contractors of America (DCA) stated "…it would be extremely dangerous and expensive" to conduct dredging operations during the winter months north of Oregon Inlet due to the high risk of dangerous wave and storm events and the associated potential for frequent shut-downs of dredging operations (CSE, 2005 – Attachment 6). The warmer months between April and September are relatively calm compared to the fall and winter months. This period also corresponds with recommended "environmental windows" during which time sand placement and hopper dredging is typically discouraged to avoid construction during periods of high biological activity within coastal waters and beaches along the United States Atlantic coast. In North Carolina, sand placement and dredging projects generally occur from November 16 through April 30 to avoid peak sea turtle and shorebird nesting seasons.

Year-round construction would provide the contractor the most flexibility and provide a safer and more economical work environment for offshore dredging activities in the Northern Outer Banks. The cost to initially construct the recommended beach fill project is estimated to be $15,408,000. This cost includes actual construction of the project and soft costs such as permitting, preparation of plans and specifications, and construction observations (Table 44). Construction of the project was assumed to coincide with the construction of similar beach fill projects along the Town of Kill Devil Hills and Kitty Hawk. Based on estimated production rates, the construction of the project will likely require approximately 3 months. This estimate is based on the production rates for hopper dredges achieved during the 2010-2011 Nags Head project. The production rate was adjusted to account for distances from the project areas to the identified borrow areas. The estimated timeframe assumes that material will be obtained from Borrow Area A; however, if Borrow Area C is used, the construction time may decrease.

The Duck project includes planned periodic nourishment to maintain the design beach fill template. The material for periodic nourishment will also be obtained from one of the two offshore borrow areas discussed in Section 7. The fill volume provided in Table 44 includes 5 years of advanced fill. As discussed in Section 17, this initial estimate of the 5-year nourishment requirement was based on the shoreline changes determined from LIDAR data and diffusion losses estimated using GENESIS. The actual performance of the restored beach and the periodic nourishment needed to maintain the design template should be determined from beach profile monitoring surveys taken at designated transects at least once a year.

The volume of material needed to maintain the Duck beach nourishment project every 5 years, which is currently estimated to be 234,000 cubic yards, is relatively small given the high cost for mobilization and demobilization of a dredge and ancillary equipment needed to perform the operation. With the Towns of Kill Devil Hills and Kitty Hawk planning similar beach nourishment projects which will also require periodic nourishment, periodic nourishment of the Duck project would be performed in conjunction with periodic nourishment of the other two towns. Based on preliminary estimates, the total volume of material needed to maintain all three projects would be close to 700,000 cubic yards every 5 years.

The estimated cost for providing periodic nourishment along the Duck shoreline every 5 years, assuming the operation would be combined with periodic nourishment of Kill Devil Hills and Kitty Hawk, is estimated to be $4,544,000 in today's dollars (2015). Excluding possible inflation of dredging costs, the total cost for periodic nourishment over the 30 year planning period would be approximately $22.7 million. The total project costs over the 30-year planning period, including initial construction and periodic nourishment would be approximately $38.1 million.

Table 44. Project Cost Estimate – Option

Volume (cy)	Permitting/soft cost	Construction Cost	Total Project Cost
1,069,000	$750,000	$14,658,000	$15,408,000

20 SUMMARY & RECOMMENDATIONS

Numerous beach fill design options were evaluated for their ability to reduce damages due to a continuation of long-term erosion and storms. The main fill for the project extends along 7,970 feet between profile station D-10 on the north and station D-19 on the south. Profile station D-10 is located near 140 Skimmer Way while station D-19 is located at the south property line of 137 Spindrift Lane. A 500-foot taper section would be provided on each end of the main fill to allow for a gradual merger of the fill shoreline with the existing shoreline. Note the south taper extends into the FRF property.

The number of beach fill options initially evaluated was 65, however, the number of viable options was reduced to 2 for detailed analysis. Based on this detailed analysis, the beach fill option designated as Option 2 was identified as providing the level of protection that would meet the stated goals and objectives of the Town of Duck. Option 2 was further refined with regard to the shoreline alignment and position that would ultimately be provided following initial construction of the project by simulating a one-year post-fill adjustment using the computer model GENESIS. The simulated shoreline alignment was then used to adjust the berm widths for the final design template which is designated as Option 3 in this report. The recommended beach fill configuration judged to provide the level of protection being sought by the Town of Duck would have a 20-foot wide dune at elevation +20.0 feet NAVD fronted by a variable width berm at elevation +6.0 feet NAVD.

The cost to initially construct the recommended beach fill project is estimated to be $15,408,000. This cost includes actual construction of the project and soft costs such as permitting, preparation of plans and specifications, and construction observations. Construction of the project was assumed to coincide with the construction of similar beach fill projects along the Town of Kill Devil Hills and Kitty Hawk. Periodic nourishment of the project, which would be needed about every five (5) years, would cost $4,544,000 per operation (current dollars) and would be performed in conjunction with the periodic nourishment of the Kill Devil Hills and Kitty Hawk projects. The storm impact line determined from the SBEACH analysis for Option 3 is provided in Figure 34. The estimated cost of the project over the 30-year period (in current dollars) would be $38.1 million.

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Notes:

1. 2012 Background imagery is from NC OneMap Imagery Service.

Legend:

Storm Impact Lines

No Action

Option 3

Transcets

Feet
0 50 100
1 inch = 100 feet

TITLE:

Storm Damage Risk Map
Duck, North Carolina
Sheet 1

Coastal Planning & Engineering
of North Carolina, Inc.
4038 Masonboro Loop Road
Wilmington, NC 28409
Ph. (910) 791-9494
Fax (910) 791-4129

Figure 34. Selected Option Storm Impacts

G:\Enterprise\Dare\150440_DUCK Design and Permitting\DUCK\mxd\Duck_Storm_Damage_Risk_Map2.mxd

Notes:

1. 2012 Background imagery is from NC OneMap Imagery Service.

Legend:

Storm Impact Lines

- No Action
- Option 3
- Transects

Feet
0 50 100
1 inch = 100 feet

TITLE:

Storm Damage Risk Map
Duck, North Carolina
Sheet 2

Coastal Planning & Engineering of North Carolina, Inc.
4038 Masonboro Loop Road
Wilmington, NC 28409
Ph. (910) 791-9494
Fax (910) 791-4129

Figure 34. Selected Option Storm Impacts

Notes:

1. 2012 Background imagery is from NC OneMap Imagery Service.

Legend:

Storm Impact Lines

No Action

Option 3

Transects

Feet
0 50 100
1 inch = 100 feet

TITLE:

Storm Damage Risk Map
Duck, North Carolina
Sheet 3

Coastal Planning & Engineering of North Carolina, Inc.
4038 Masonboro Loop Road
Wilmington, NC 28409
Ph. (910) 791-9494
Fax (910) 791-4129

Figure 34. Selected Option Storm Impacts

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