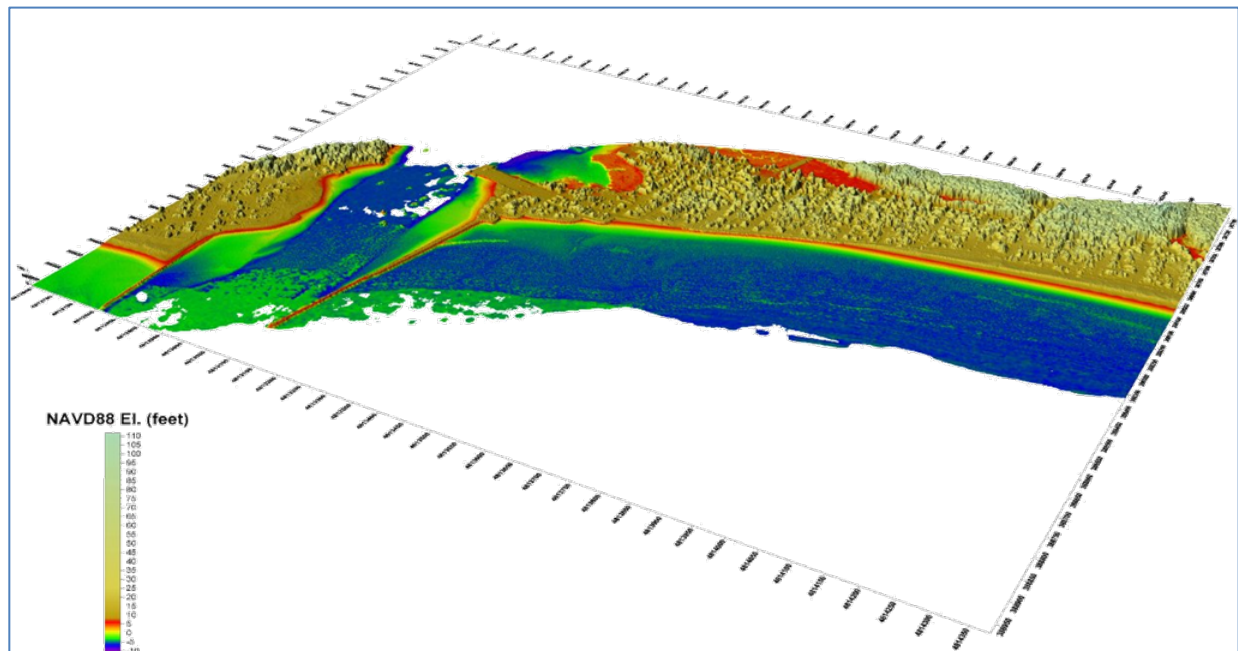


Appendix B

Saco River and Camp Ellis Beach - Saco, Maine

Coastal Engineering Appendix



U.S. Army Corps of Engineers – New England District
Water Management Section CE-NAE-EP-WM
696 Virginia Road
Concord, MA 01742

03-02-2012

Table of Contents

| | | |
|------------|---|-----------|
| 1.0 | INTRODUCTION AND REPORT DESCRIPTION | 8 |
| 1.1 | Report Description | 8 |
| 1.2 | Project Description | 8 |
| 1.2.1 | Coastal Environment..... | 14 |
| 1.2.2 | Coastal Geology | 14 |
| 1.2.3 | Geographic Setting..... | 15 |
| 1.3 | Federal Responsibility..... | 15 |
| 2.0 | PROJECT BACKGROUND/COASTAL ENGINEERING HISTORY | 17 |
| 2.1 | Coastal Engineering History | 17 |
| 2.2 | Post 2002 Coastal Engineering Study Analysis | 17 |
| 3.0 | DATA SOURCES AND DATA COLLECTION SUMMARY | 19 |
| 3.1 | Bathymetric Data | 19 |
| 3.1.1 | Bathymetric Data Sources – Outside of Study..... | 19 |
| 3.1.2 | Bathymetric Data Collected by USACE..... | 19 |
| 3.1.3 | Bathymetric Data Collected by Woods Hole Group | 23 |
| 3.2 | Tide Data Collection..... | 25 |
| 3.2.1 | Tidal Observations | 26 |
| 3.3 | ADCP Survey..... | 28 |
| 3.3.1 | Survey Region | 28 |
| 3.3.2 | Survey Results..... | 29 |
| 3.3.3 | ADCP Summary/Result Discussion..... | 31 |
| 3.4 | Wave Data Collection..... | 32 |
| 3.4.1 | Instrument Deployment | 32 |
| 3.4.2 | Wave Observations..... | 33 |
| 3.4.3 | Wave Data Summary | 42 |
| 4.0 | SHORELINE MAPPING AND SEDIMENT TRANSPORT | 43 |
| 4.1 | Historical Shoreline Change Analysis..... | 43 |
| 4.1.1 | Data Sources | 43 |
| 4.1.2 | Discussion of Shoreline Change | 43 |

| | | |
|------------|---|-----------|
| 4.1.3 | Shoreline Change Discussion – Post 1998 | 47 |
| 4.1.4 | Shoreline Movement Summary | 55 |
| 4.2 | Sediment Transport – Without project..... | 56 |
| 4.2.1 | Grain Size Analysis | 56 |
| 4.2.2 | Methodology..... | 57 |
| 4.2.3 | Regional Sediment Transport | 57 |
| 4.2.4 | Sediment Budget – Sand Source..... | 59 |
| 5.0 | WAVE MODELING..... | 61 |
| 5.1 | Generation-Scale Wave Modeling..... | 61 |
| 5.1.1 | Wave Model Used..... | 61 |
| 5.1.2 | Input Winds..... | 61 |
| 5.1.3 | Bathymetry and Grid Generation | 62 |
| 5.1.4 | Model Calibration and Results..... | 63 |
| 5.1.5 | Significant Wave Events..... | 65 |
| 5.1.6 | Summary | 66 |
| 5.2 | Regional Wave Modeling..... | 67 |
| 5.2.1 | Wave Model Description | 67 |
| 5.2.2 | Bathymetry and Grid Generation | 67 |
| 5.2.3 | Wave Characteristics and Input Spectra..... | 68 |
| 5.2.4 | Regional (Transformation Scale) Model Validation..... | 70 |
| 5.2.5 | Average Annual Directional Approaches..... | 72 |
| 5.2.6 | High Energy Events | 74 |
| 5.2.7 | Regional Modeling Summary..... | 75 |
| 5.3 | Local Wave Modeling..... | 75 |
| 5.3.1 | Analysis Approach..... | 75 |
| 5.3.2 | Grid Generation | 76 |
| 5.3.3 | Wave Input Spectra..... | 77 |
| 5.3.4 | Model Calibration and Verification..... | 77 |
| 5.3.5 | Existing Conditions Simulations..... | 78 |
| 5.3.5.1 | Average Annual Directional Approaches..... | 79 |
| 5.3.6 | Alternative Simulations..... | 82 |

| | | |
|------------|---|------------|
| 5.3.7 | Local Wave Modeling Summary | 82 |
| 6.0 | ALTERNATIVE DESIGN..... | 83 |
| 6.1 | Development of Alternatives..... | 83 |
| 6.2 | Alternatives Considered..... | 83 |
| 6.2.1 | Alternatives Screening Process..... | 84 |
| 6.3 | Alternative Descriptions | 86 |
| 6.3.1 | No Action Alternative..... | 86 |
| 6.3.2 | Base Alternative: Beach Nourishment Alone..... | 86 |
| 6.3.3 | Alternative 6: Inshore Location of a 750-foot Spur Jetty..... | 86 |
| 6.3.4 | Alternative 25a: Segmented Breakwater..... | 87 |
| 7.0 | ALTERNATIVE SELECTION ANALYSIS..... | 89 |
| 7.1 | Wave Height and Energy Evaluation..... | 89 |
| 7.1.1 | Visual Assessment..... | 89 |
| 7.1.2 | Wave Height Change Assessment | 90 |
| 7.1.3 | Wave Energy Change Assessment | 91 |
| 7.1.4 | Final Alternative Performance Discussion | 92 |
| 7.2 | Sediment Transport – Alternative Analysis | 96 |
| 7.2.1 | Methodology..... | 96 |
| 7.2.2 | Sediment Transport Reduction..... | 97 |
| 7.2.4 | Salient Formation..... | 98 |
| 7.2.5 | Beach Nourishment Performance | 100 |
| 7.2.6 | Sediment Deficiency/Sediment Blockage Caused by Federal Navigation Project | 103 |
| 8.0 | STRUCTURAL ALTERNATIVE DESIGN..... | 105 |
| 8.1 | Risk Reduction (Level of Protection of Structures)..... | 105 |
| 8.2 | Armor Unit Size | 106 |
| 8.2.1 | Slope and Crest Stone | 106 |
| 8.2.2 | Toe Stone | 106 |
| 8.3 | Design Wave Height..... | 107 |
| 8.4 | Structure Toe/Base Elevation | 109 |
| 8.5 | Structure Side Slopes | 112 |

| | | |
|-------------|---|------------|
| 8.6 | Crest Elevation | 112 |
| 8.6.1 | Crest Elevation – Tide Range and Wave Transmission Discussion | 112 |
| 8.6.2 | Crest Elevation – Near Shore Breakwater Design Consideration | 115 |
| 8.6.3 | Crest Elevation – Foundation Settlement Consideration | 116 |
| 8.6.4 | Crest Elevation – Recommendation | 117 |
| 8.7 | North Jetty Reinforcement and Toe Protection | 117 |
| 8.8 | Stone Size Determination | 119 |
| 8.8.1 | Armor Stone Size Determination | 119 |
| 8.8.2 | Toe Stone Size Determination | 120 |
| 9.0 | BEACH FILL CROSS SECTION DESIGN..... | 131 |
| 9.1 | Introduction and History..... | 131 |
| 9.2 | Beach Fill Design | 131 |
| 9.3 | Existing Beach Profile Data | 132 |
| 9.4 | Beach Fill Design Methodology..... | 133 |
| 9.4.1 | Beach Fill Only Profiles..... | 133 |
| 9.4.2 | Beach Fill Profiles with Structures | 135 |
| 9.5 | Beach Depth of Closure | 136 |
| 9.6 | Sand Deficit | 137 |
| 9.7 | Beach Fill Deficit/Nourishment/Re-nourishment Formulation | 137 |
| 9.8 | Design Storm Selection | 138 |
| 9.9 | SBEACH Modeling | 138 |
| 9.9.1 | Modeling Approach | 138 |
| 9.9.2 | Model Set Up | 140 |
| 9.9.3 | Model Validation..... | 141 |
| 9.10 | Alternative Modeling | 146 |
| 9.11 | SBEACH Modeling Results..... | 148 |
| 9.12 | Minimum Required Beach | 150 |
| 9.12.1 | Minimum Berm Widths..... | 150 |
| 9.12.2 | Minimum Berm Volumes | 151 |
| 10.0 | BEACH FILL RENOURISHMENT ANALYSIS..... | 155 |
| 10.1 | Beach Fill Longevity Calculation Methodology..... | 155 |

| | | |
|-------------|---|------------|
| 10.2 | Beach Fill Volume Calculation – 20, 30, and 40 FT Berm Widths | 157 |
| 10.3 | Beach Fill Calculation – Beach Fill Only Re-nourishment Interval | 157 |
| 10.4 | Beach Fill Volume Calculation – Uncertainty Consideration | 158 |
| 10.5 | Beach Fill Volume “Size” Recommendation | 159 |
| 11.0 | SEA LEVEL CHANGE IMPLICATIONS | 160 |
| 11.1 | Sea Level Change – Project History..... | 160 |
| 11.2 | Sea Level Change – Project Application..... | 161 |
| 11.3 | Sea Level Change Projection..... | 161 |
| 11.4 | Sea Level Change Impact Discussion | 163 |
| 11.4.1 | Sea Level Change Impact – Without Project..... | 163 |
| 11.4.2 | Sea Level Change Impact – With Project - Structure Crest Elevations ... | 164 |
| 11.4.3 | Sea Level Change Impact – Structure Maintenance..... | 165 |
| 11.4.4 | Sea Level Change Impact – With Project Stone Size..... | 165 |
| 11.4.5 | Sea Level Change Impact – Beach Fill (Bruun Rule)..... | 165 |
| 11.4.6 | Sea Level Change Impact – With Project Beach Fill Storm Protection ... | 170 |
| 11.4.7 | Sea Level Change – Should it Be Addressed? | 170 |
| 11.5 | SLC Discussion Conclusions..... | 170 |
| 12.0 | ALTERNATIVE RISK ANALYSIS..... | 172 |
| 12.1 | Beach Fill Risk Based Alternative Analysis | 173 |
| 12.2 | SBEACH Risk Discussion | 173 |
| 12.3 | Alternative Risk Discussion | 174 |
| 13.0 | CONCLUSIONS AND RECOMMENDATIONS..... | 176 |
| 13.1 | Historical shoreline change..... | 176 |
| 13.2 | Bathymetry | 176 |
| 13.3 | Tides and Currents..... | 177 |
| 13.4 | Wave Climate | 177 |
| 13.5 | Sediments | 177 |
| 13.6 | Generation Scale Wave Modeling | 177 |
| 13.7 | Transformation-Scale Wave Modeling | 178 |
| 13.8 | Local Scale Wave Modeling | 178 |
| 13.9 | Sediment Transport Assessment | 178 |

| | | |
|-------------|------------------------------------|------------|
| 13.10 | Initial Alternative Screening..... | 179 |
| 13.11 | Final Alternative Screening | 179 |
| 13.12 | Structure Design..... | 180 |
| 13.13 | Beach Fill Design | 180 |
| 13.14 | Sea Level Change Impacts..... | 181 |
| 13.5 | Risk Analysis | 181 |
| 14.0 | REFERENCES..... | 182 |

1.0 INTRODUCTION AND REPORT DESCRIPTION

1.1 Report Description

This report has been written to be included as the Coastal Engineering Appendix to the feasibility report. Contained within the Coastal Engineering Appendix is a combination of both the US Army Corps of Engineers (USACE) Coastal Engineering effort and the Wood Hole Group effort. To help provide a complete Coastal Appendix much of the information provided in the WHG report (Appendix C) has been summarized within this appendix in Sections 3 through 7. Throughout this appendix the applicable WHG report sections are referred to in order to assist the reader in acquiring more detailed information as needed. Generally, the Corps' detailed analysis and coastal engineering reporting have been provided in Sections 8 through 13. Through the 8+ years of this more "recent" effort, both USACE and the WHG worked closely on this study and it would be difficult to define much of the work done as USACE work or WHG work. As noted, if the reader is interested in greater detail regarding the work performed by WHG they are referred to Appendix C.

This appendix is the final iteration of significant work performed by USACE and WHG. Much of the earlier work completed by USACE, with regards to the coastal engineering analysis, was excluded from this appendix since it was superseded by later efforts of USACE and the WHG. Earlier sets of data, results, reports, etc. should be considered obsolete and should not be used. The information contained herein should be taken as the final version of the analysis.

1.2 Project Description

Camp Ellis Beach, Maine is located directly adjacent and north of the Saco River Jetties (Figures 1-1 and 1-2) within the city of Saco, ME. The first structures were constructed in 1869 by the US Army Corps of Engineers (USACE). Through several significant lengthening projects and repairs the structures have reached their present day lengths of 6,660 feet for the North Jetty and 4,800 feet for the South Jetty. Both are constructed of stone and vary considerably in structure elevation and cross section.

Due to the interaction between the structures and the local coastal processes, it has been concluded that the beach north of the structures is being adversely affected. Based upon analysis to be discussed, it was determined that the main cause of accelerated erosion is reflected wave energy from the North Jetty and that the jetty structures are causing erosion by cutting off the supply of sand to the north. Under Section 111 – Mitigation of Damages Caused by Federal Navigation Projects, a feasibility study was undertaken to determine a solution to the erosion problem caused, or amplified, by the Corps' jetties. The original project area was the shoreline within 2,500 feet of the North Jetty, but it has been extended to 3,250 feet to cover the more recent erosion in the latter part of the decade.

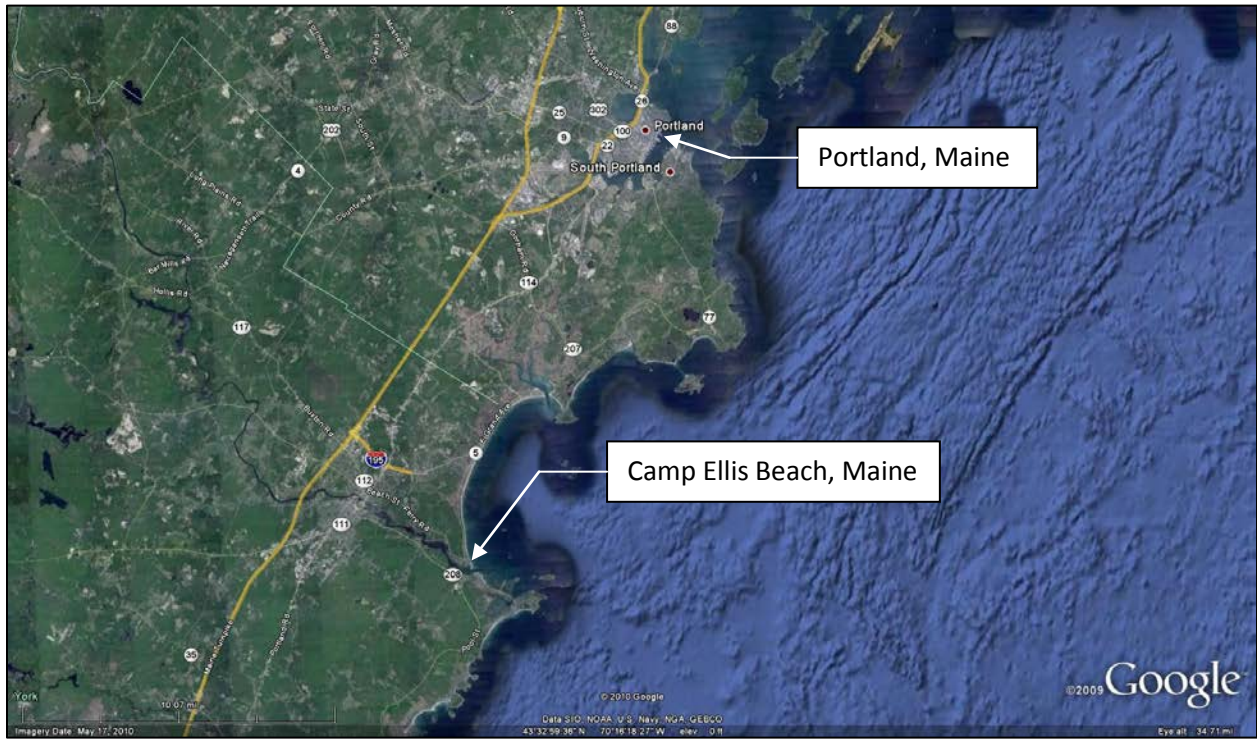


Figure 1-1 Camp Ellis Beach Location

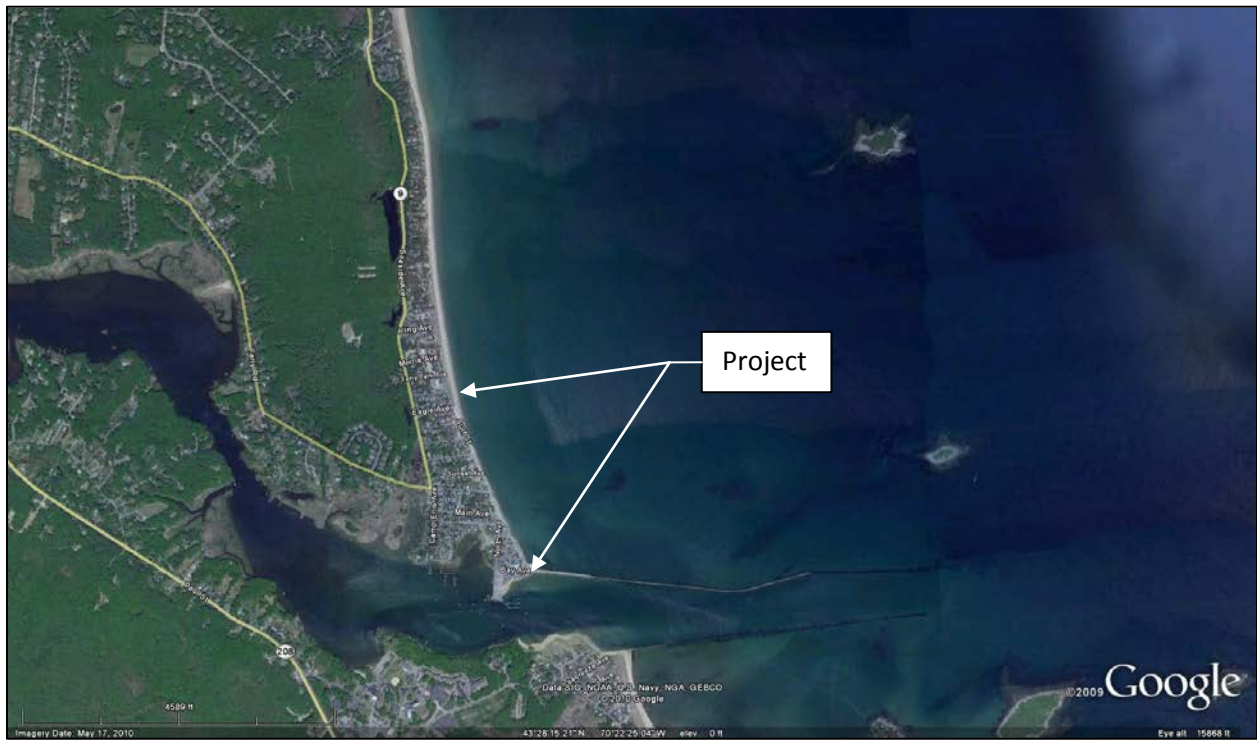


Figure 1-2. Camp Ellis Beach and Saco River Jetties

Over the last 100 years significant erosion has occurred with several rows of roads/houses being lost. Shown in Figure 1-3, is a map showing the lost property since 1908. To counter this erosion home owners and the City of Saco constructed several non permitted, non-engineered, temporary revetments to protect residences. These structures are shown in an aerial photo as Figure 1-4 and beach level photos in Figures 1-5 and 1-6. As will be discussed in Section 4.1.3, as the beach has continued to erode in front of the revetments all dry beach has been lost and the beach north of the revetments, which had be stabile has been severely eroding. This northern erosion has lead to the failure of a portion of the northern revetment (rebuilt), the abandonment of a road that was destroyed during a storm, and the addition of geotubes to extend the north revetment (Figures 1-7 to 1-9). It is theorized that since the sand source has been cut off by the jetties, the beach system at the south end of the project area became the default sediment source for the system. With that area depleted of sand the erosion problem has now moved north and will continue to move further north. As shown in Section 4, the Saco Bay system was stabile to accretionary in the past and with the information gathered and developed in this study it was concluded that the erosion problem occurring within the study area is caused by the federal navigation structures. The federal responsibility will be quantified within the report, and is summarized in Section 1.3.

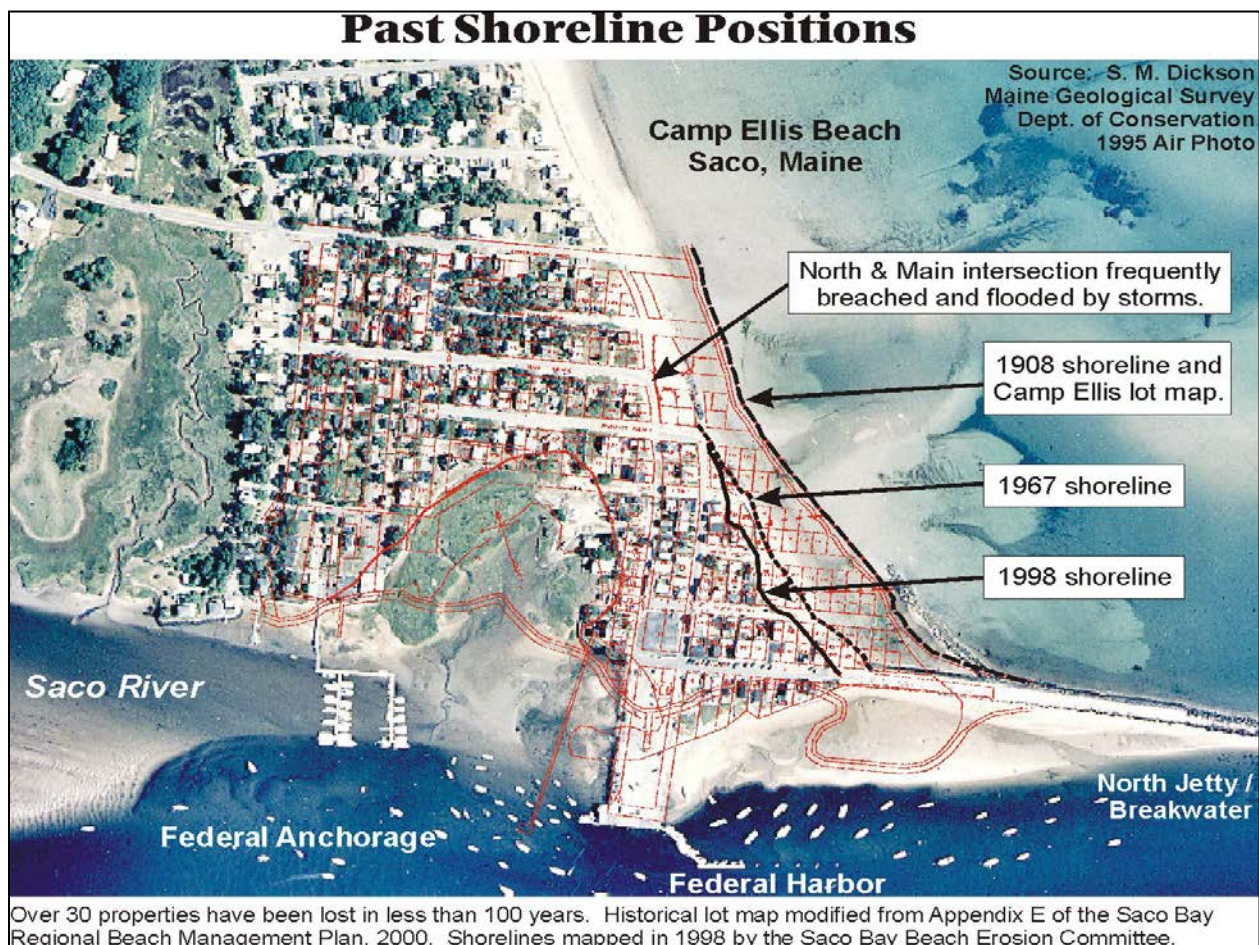


Figure 1-3. Historic shorelines and properties lost since 1908



Figure 1-4. Project area features.

While the revetments constructed do offer a level of protection against erosion, as shown in Figures 1-7 and 1-8 they are prone to failure, and will likely fail since they were not engineered or constructed as a revetment should be. The stone sizing, stone gradation, structure slope, crest elevation, under layer/filter layer, and toe elevation are all unknown/non-documented and/or not in accordance with applicable design criteria. For this reason, these structures cannot be relied upon for shoreline erosion protection, and will likely fail, as already shown, due to one or several failure mechanisms.



Figure 1-5. View of project area looking north from North Jetty – “temporary” revetments shown.



Figure 1-6. View of southern project area



Figure 1-7. Damage at northern end to road, houses, utilities, and revetment (looking north).



Figure 1-8. Damage at northern end to road, houses, utilities, and revetment (looking south).



Figure 1-9. North end of project with geotubes placed at the end of the “temporary” revetments.

1.2.1 Coastal Environment

The Saco Bay shoreline forms a littoral cell in the southeastern coastline of Maine. This embayment contains the largest sand beach and salt marsh system in Maine and has a mean tidal range of approximately 9 ft (2.7 m), with a spring range of 11.6 ft (3.5 m). The coast is characterized by three tidal inlets: the Saco River Inlet, Goose Fare Brook Inlet, and Scarborough Inlet, with sandy pocket beaches that provide economic, environmental, and recreational benefits (Slovinsky and Dickson, 2003).

1.2.2 Coastal Geology

The Saco embayment of Maine is considered a riverine-derived sandy barrier shoreline, which is defined as a barrier that has developed from the onshore and alongshore reworking of sediment deposited on the inner continental shelf by rivers (Fitzgerald, et al., 1994). Complete details on the local geology can be found in Chapter 2.0 of Appendix C.

Grain size data from samples taken at 140 stations throughout the estuary and nearshore region indicate that the dominant sediment type is medium-to-coarse grained sand with finer-grained sediment flanking the wider portions of the river (tidal flats and marshes, which are long-term sediment accumulation areas). South of the jetties the sediment is uniformly fine sand (Fitzgerald et al., 2002). Grain size within the Saco Bay system is finer in the northern reaches of the system than in the south. Fining grain sizes were found by Kelley et al. (1995), and more recently by the WHG sediment analysis (Section 4).

Riverine-associated inlets like that of Saco, contribute sand to the nearshore zone during periods of high riverine discharge. Sediment texture and bedforms show that larger spring freshet events overwhelm tidal flow in the estuaries and control net sediment transport directions. The magnitude, direction, and persistence of the current velocities indicate that freshets are important events in supplying coarse-grained sediment to the estuary mouth, filling harbor regions and/or shoaling jettied channels as in Saco (Fitzgerald et al., 2002).

1.2.3 Geographic Setting

Saco Bay, Maine (Figure 1-1) is an 8-mile long arcuate stretch of shoreline bound to the south by Fletcher's Neck and the Saco River, and to the north by Scarborough River and Prout's Neck (Figure 1-1). The majority of Saco Bay's coastline is densely developed consisting of small beachfront communities. As discussed in the previous chapter, the Bay represents the largest sand beach and salt marsh system in Maine and the Saco River has been considered one of the primary historical sources of sediment to the beaches within the Bay (Kelley et al., 1995; Slovinsky and Dickson, 2003). The Saco River estuary is located at the southern end of the sandy coastal system within the Saco embayment. Current day tidal influences extend 10 km upstream to the base of two dams at Factory Island. The Saco River mouth is shallow (< 3.0m or 10 ft) and is currently stabilized by two jetties. Recurring shoaling between the two jetties and in the harbor landward of the estuary mouth has necessitated a continual maintenance dredging program operated by the USACE.

1.3 Federal Responsibility

An important criteria for defining a Section 111 project for USACE is determining if the Federal Government's project is responsible for damages to the adjacent areas and if so how much damage is the Federal Government responsible for. For the Camp Ellis project this was done, and the supporting information has been provided in the following sections of this appendix. The analysis has shown that there are two major causes for the erosion. The first is a strong reflected wave coming off the North Jetty during storm conditions. This is detailed in Section 5.3.5 and in Appendix C Section 12.5.2, and it was found that the reflected wave increases wave energy on the beach by 40% directly north of the North Jetty and by 20% at the northern end of the project area. This energy, in conjunction with the very shallow approach angle to the beach causes a significant long shore current, which increases sediment transport potential. This basically causes increased erosion of the beach north of the jetty. The second impact of the jetties is the prevention of sand from the Saco River from reaching the beach system. Based on the length of the jetties, and the water depth at the end of the jetties, any sand that is transported through the jetties is essentially lost to the beach system. The volume of sand was not specifically quantified in this study effort, but this information was available in a report written by several well known professors and geologist from the state of Maine and regional universities and is discussed in Section 4.2.4. In this report a sediment budget for the Saco Bay system was developed and the conclusion of the study was that the federal navigation project was preventing 13,000 yds³ to 22,000 yds³ of sand from entering the beach system. USACE reviewed the report and other supporting information and concurs with that finding. Based on this information, the Federal Government is responsible for negating the reflected wave energy from the North Jetty, and supplying the Saco Bay

beach system with 13,000 yds³ to 22,000 yds³ of sand per year. The recommended alternative of a spur groin and beach fill addresses these two issues.

2.0 PROJECT BACKGROUND/COASTAL ENGINEERING HISTORY

2.1 Coastal Engineering History

The USACE involvement with the Saco River entrance dates back to the early 1800's with the initial project authorization in 1827. During the 1800's and 1900's the entrance channel jetties were lengthened, raised, tightened and reinforced. As discussed in Section 1.0 the structures are long for East Coast jetties, at 6,600 feet for the north jetty and 4,800 feet for the south jetty.

Through the lengthy history of the Saco River Jetties there have been several investigations or considerations by USACE regarding shoreline erosion adjacent to the structures. In each of those efforts, including the 1992 reconnaissance Section 111 investigation, it was concluded the jetties were not a significant contributor to shoreline erosion. This was due in large part to the USACE belief that sediment transport was from north to south in the southern half of Saco Bay. This was based upon geologic analysis, sediment source consideration, and the persistent historical shoaling problem within the Saco River federal navigation channel and the subsequent jetty lengthening already discussed.

The north to south transport direction was in direct disagreement with studies from the 1970s, 1980s and 1990s such as (Farrell, 1972), (Luepke and Grosz, 1986), (Kelley et al., 1989a), (Maine Geological Survey, 1995) and others, that made a very strong case for south to north transport.

Under Section 111 – Mitigation of Damages Caused by Federal Navigation Projects, a reconnaissance level study was undertaken to determine a solution to the erosion problem caused or amplified by the USACE jetties. Unfortunately there is little physical data. The Waterways Experiment Station (WES) performed a physical model study in 1995, which looked at a beach fill option, off shore sand berms, removal of the north jetty, and various shore parallel breakwaters that were attached to the north jetty. The two lengths looked at were 3,000 feet and 1,500 feet long (hence calling them breakwaters and not spur groins). It was determined that this was not feasible or desirable by the State of Maine. During the 1995 USACE study the physical model used coal dust to represent sediment transport. The study did not provide sand transport rates, but did provide some sand movement directions and helpful wave climate info. The study did show that the two breakwater options would reduce the long shore transport rate, but these two options are not feasible.

Since the USACE 1995 effort, several reports from the state of Maine researchers have been completed with the findings and conclusions being similar to earlier papers.

2.2 Post 2002 Coastal Engineering Study Analysis

Following the finding from the 1995 USACE physical modeling effort, the project basically reached a stalemate due to the lack of implementable alternatives in the 1995 effort and a large number of uncertainties in the coastal processes/alternative performance. In 2002, following a reanalysis of the existing work, it was concluded that the existing information was not adequate to move the study forward. This included existing condition erosion rates, sediment transport direction, necessary mitigation length, alternative design and performance, etc. Although significant money had been spent

in the past it was determined that a sizeable numerical modeling effort and shoreline change rate analysis was needed in order to answer the questions at hand and complete the Section 111 study.

In the spring of 2002 representatives from the New England District, North Atlantic Division, the New York District, City of Saco Personnel, the State of Maine, and congressional representative staffers met to discuss the status of the project. During that meeting a new study approach was presented and discussed amongst the group and it was decided at the end of the meeting that additional study was needed. The proposal included shoreline change mapping, wave numerical modeling, alternative design and evaluation, sediment transport analysis, and data collection. Following the meeting a detailed plan was developed for work to be performed both in house by USACE and by a contractor. The Woods Hole Group (WHG) from Falmouth, MA was selected as the contractor.

A 2-D hydraulic model was considered but was eliminated due to cost issues and the hypothesis that the most significant issues were related to wave energy. Additionally the conclusion by NAE and others from USACE (in 2003) was that 2-D modeling skill at the time would not be adequate since much of the area of interest is in the surf zone during storms, the complex bathymetry, and the strong wave current field that likely exists. With these considerations it was thought that 2-D models of the period would not likely provide reliable information. Today, if additional modeling was to be performed, a hydraulic model would be considered, and would we would look to CMS, Delft 3-D, or another system based model.

3.0 DATA SOURCES AND DATA COLLECTION SUMMARY

3.1 Bathymetric Data

With the WHG modeling effort extending from the coast into to outside of the Gulf of Maine, both detailed nearshore and broad scale, less detailed bathymetry was needed. A significant amount of bathymetric information was required to simulate the sea state from the large, coarser grids of the generation-scale model to the smaller, finer grids of the nearshore (local) wave models. Additionally, bathymetric data was needed for the design of the numerous structural alternatives investigated, and the design and evaluation of the various beach fill alternative components. To address this data need a combination of bathymetric data sources were used in this study. These sources included data collected by USACE through various technologies, data collected by other government agencies using various technologies, and by the WHG using various technologies. Provided in the sub sections of Section 3.1 is a general description of this data. Due to the length of this project study, the bathymetric data used was updated several times regarding the beach fill design. Using the most recent data was deemed necessary for having the most up to date design volumes and performance expectations. The data was not updated in the wave model due to the complexities of the wave modeling effort and the fact that the very near shore changes would have minimal impact on the wave modeling results.

3.1.1 Bathymetric Data Sources – Outside of Study

To achieve the coverage necessary for the wide area covered in the modeling effort multiple data sources were necessary. The data sources gathered from outside of USACE and WHG efforts are listed below:

- NOAA Hydrographic Survey Data and NGDC Marine Trackline Geophysics Data
- NOAA LIDAR Data (Coastal Service Center Topographic Data Interface)
- Naval Oceanographic Office Digital Bathymetric Data Base - Variable Resolution gridded bathymetry
- Supplemental Datasets from Bedford Institute of Oceanography and Brookhaven National Laboratory
- NOAA Medium resolution digital Shoreline and DMA World Vector Shoreline
- Defense Mapping Agency ETOPO5 Digital relief of the Surface of the Earth
- GEBCO General Bathymetric Chart of the Oceans
- USGS North American 30 arc-second Digital Elevation Model (DEM)

3.1.2 Bathymetric Data Collected by USACE

During the study numerous surveys were conducted by USACE survey crews using boat mounted acoustic survey gear (single beam and multi-beam) as well as beach transect surveys using both total station surveys and RTK GPS surveys. An additional significant survey conducted by the USACE Coastal Mapping Center was the 2007 bathymetric/topographic LIDAR survey. Shown in Figures 3-1 to 3-4 are the location maps of the several USACE beach surveys and the area of coverage used for the 2007 USACE LIDAR mapping effort.



Figure 3-1. Beach Profiles 2007



Figure 3-2. Beach Profiles 2009

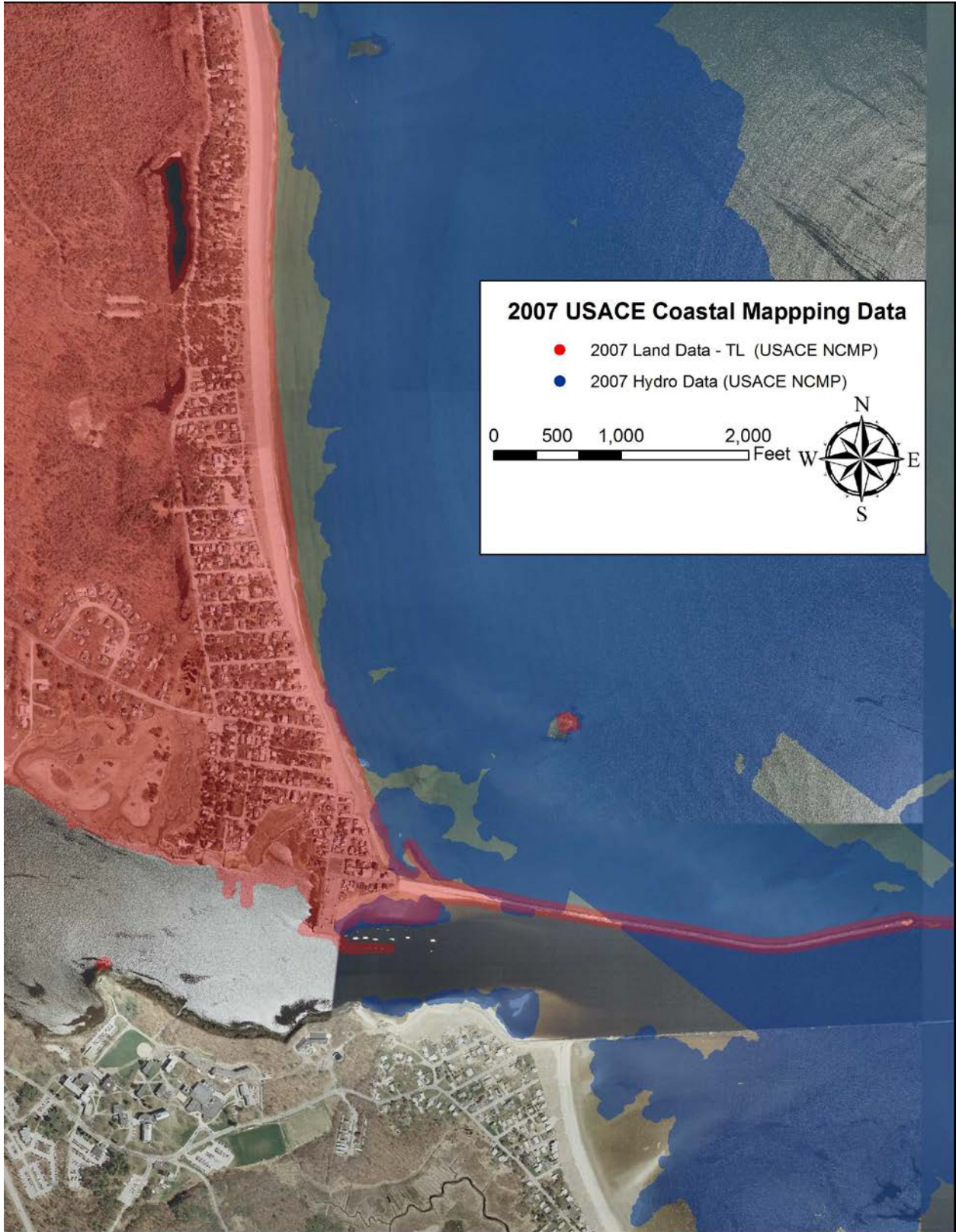


Figure 3-3. USACE Coastal Mapping Center LIDAR Data – 2007

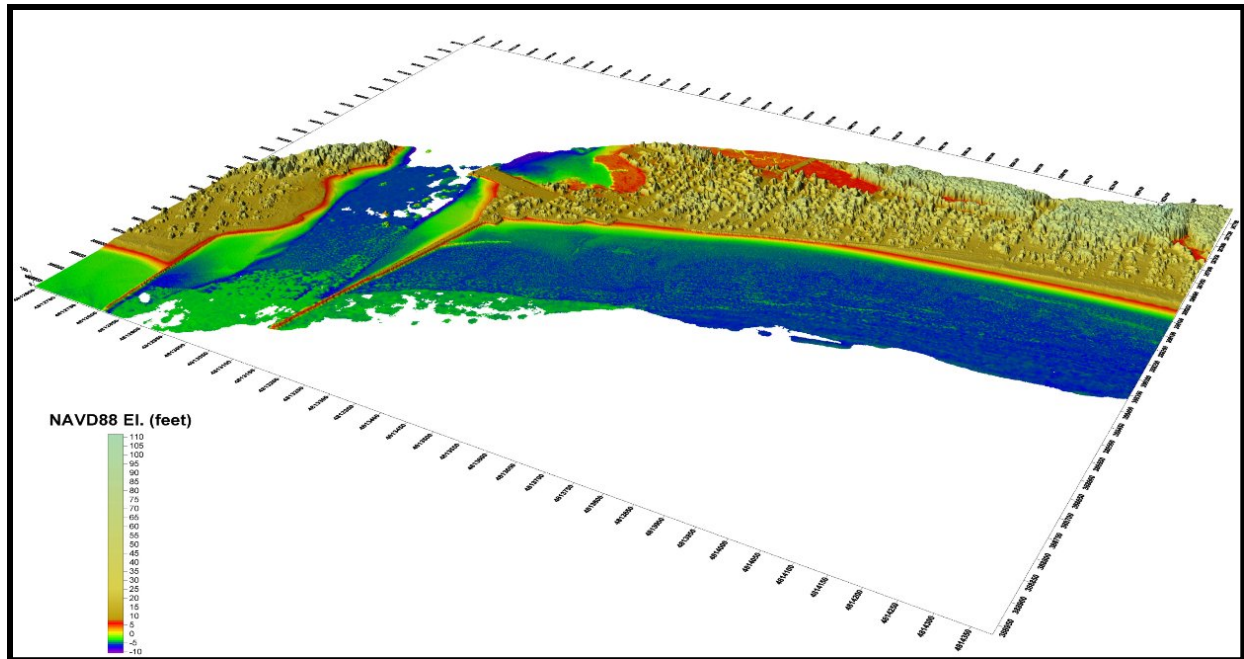


Figure 3-4. LIDAR data plot (combined USACE and NOAA LIDAR) – created by WHG.

3.1.3 Bathymetric Data Collected by Woods Hole Group

This section focuses on the bathymetric data set collected as part of this study by WHG, in the nearshore region of Camp Ellis Beach. For a more detailed discussion of the WHG survey effort refer to Section 4 of Appendix C.

3.1.3.1 Survey Methodology

CR Environmental, Inc. (CR) conducted a bathymetric survey of portions of Saco Bay and Wood Island Harbor near the mouth of the Saco River on May 13 and 15, 2003, for Aubrey Consulting, Inc. The survey was conducted from CR's 22-foot R/V C-Hawk. The boat was equipped with safety gear, multiple 12-volt power supplies, and a laptop computer equipped with navigation and data-logging software.

3.1.3.2 Bathymetric Observations and Summary

The survey extent was selected to focus on the region that was most critical for wave transformations in the nearshore vicinity of Camp Ellis Beach. In addition, this nearshore region had the most potential for recent physical changes. Therefore, in this region it was critical to get the most recent and accurate bathymetric information. Figure 3-5 and 3-6 presents the post-processed bathymetric data collected in May 2003. The contours indicate 2-foot elevation swaths relative to NGVD 1929. The color contours are presented as red (shallow) to dark purple (approximately -56 feet). A number of relevant features are shown in the bathymetric data that likely impact wave propagation and transformation in the vicinity of Camp Ellis Beach:

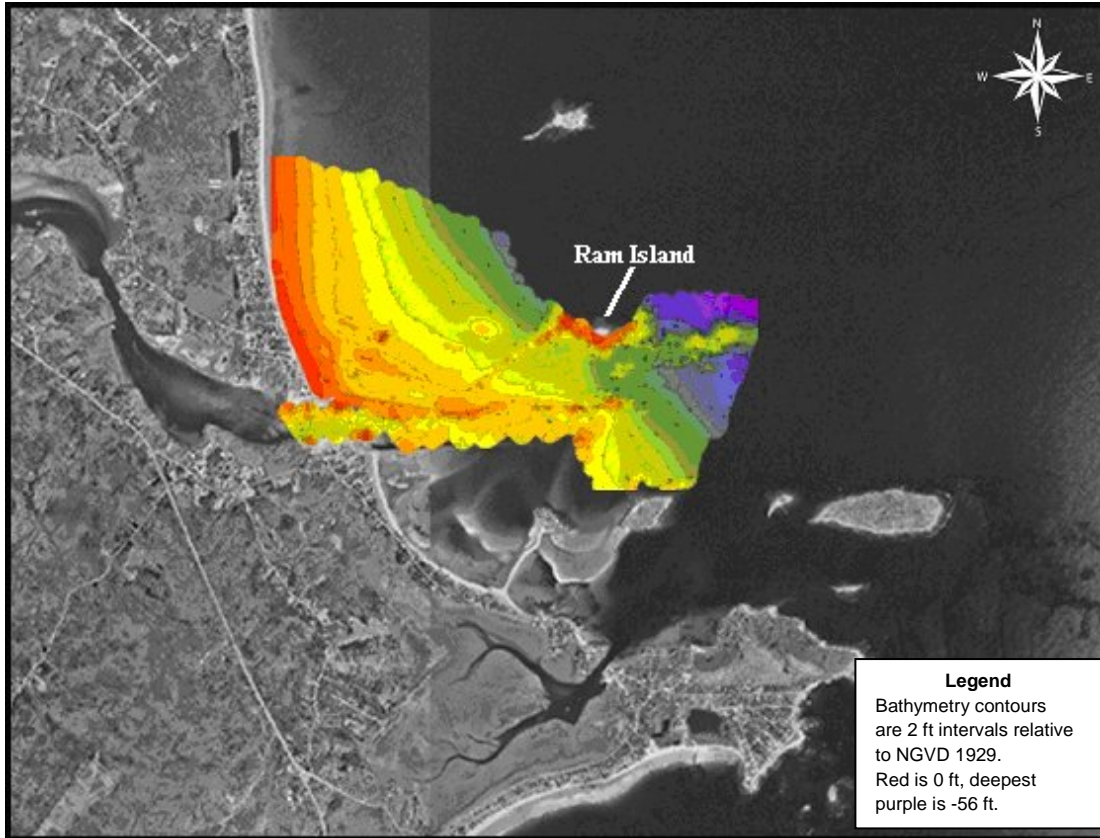


Figure 3-5. Nearshore bathymetric data collected in May, 2003.

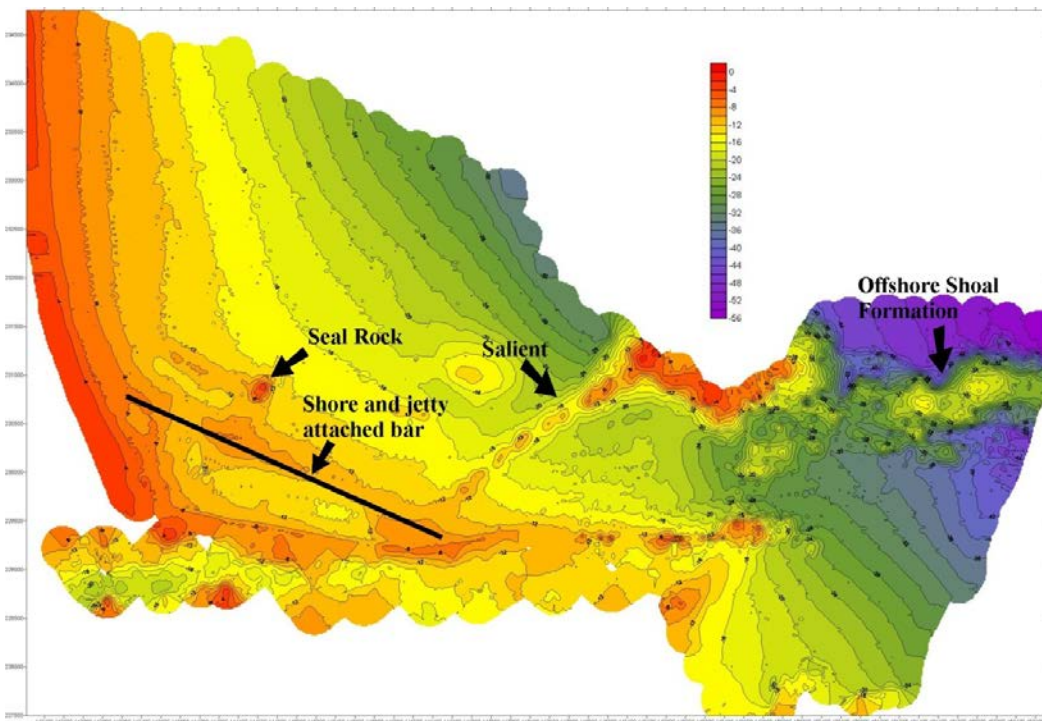


Figure 3-6. Bathymetric contour data and relevant features in the bathymetry offshore of Camp Ellis.

3.2 Tide Data Collection

Time-series of water surface elevation (calculated from pressure measurements) were obtained at four (4) locations (Figure 3-7) within the region during an interval of approximately two (2) months. The two (2) month deployment period allows for adequate characterization of all of the primary tidal constituents. Whereas short tidal records may be biased by climatic events, such as surges, wind, rainfall, atmospheric pressure, etc., this longer deployment samples a range of tidal events and is more representative of average tidal conditions within the estuary. This section briefly presents the data, including data collection procedures and instrumentation. For a more detailed discussion regarding tide data collection refer to Appendix C, Section 5.

These data were also collected within the Saco River estuary in case the USACE decided modeling of nearshore circulation, hydrodynamics, and tidal driven sediment transport modeling was eventually warranted. In the process of the study, it was determined that although the Saco river delivered sediment to the system the hydrodynamics of the Saco River had little influence on the sediment transport dynamics occurring at Camp Ellis Beach, where waves were the primary forcing mechanism for sand movement.

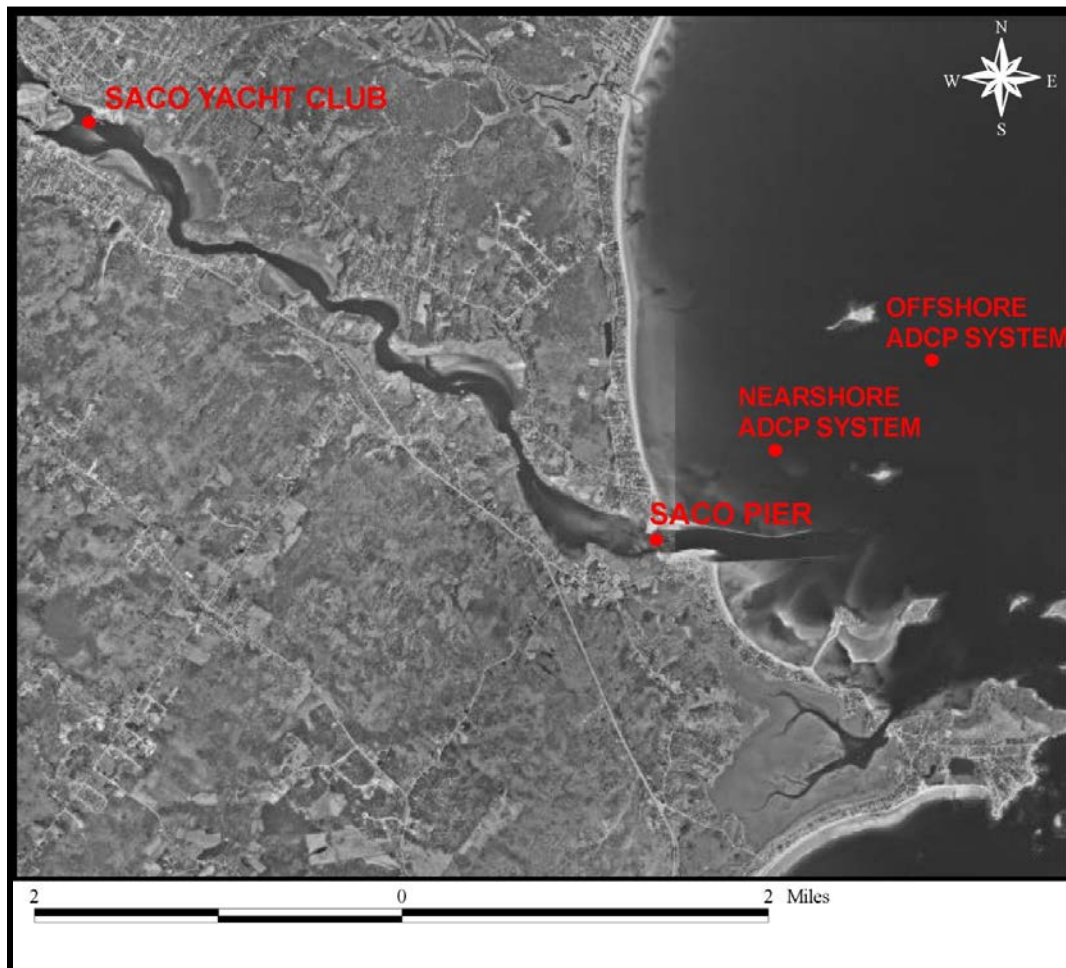


Figure 3-7. Tide gauge locations within the study area.

3.2.1 Tidal Observations

Each tide gauge measured the water and atmospheric pressure above the instrument. In order to estimate the water level (gauge pressure), the atmospheric pressure was removed from the measured signal. The data were corrected using regional atmospheric pressure data from National Climatic Data Center (NCDC) Portland International Airport station. Subsequently, pressure data were converted to water surface elevation using the hydrostatic relationship based on the density of water. In order to reference the tide gauges to a common vertical datum, tide data from each gauge were referenced to the NAVD 1988 vertical datum. The tide gauges were surveyed to the instruments' pressure port via a local benchmark. Additionally, water surface elevation measurements were taken to provide a secondary means of referencing the water surface to NAVD 1988.

Figures 3-8 through 3-9 show examples of the tidal observations from one of the tide data stations and from one of the ADCP gages. Figure 3-10 shows a portion of the measured tidal observations during the deployment period for both locations within the Saco River. During a typical day, one of the high tides is higher than the other, and one of the low tides is lower. The spring and neap tides are also easily observed in the signal scale (Figures 3-8 and 3-9). During spring tide, also known as a moon tide, the tidal range is approximately 11.5 to 12.0 ft (3.5 to 3.7 m). However, during neap tide, the tidal range is reduced to approximately 7.0 ft (2.1 m). The observations at the upstream tide gauges (red line) indicated that only minor tidal attenuation compared to the downstream observations (blue line) as the tide propagates upstream. The tidal range at the upstream gauges is damped less than 0.6 ft (0.2 m), and the average daily reduction is even smaller.

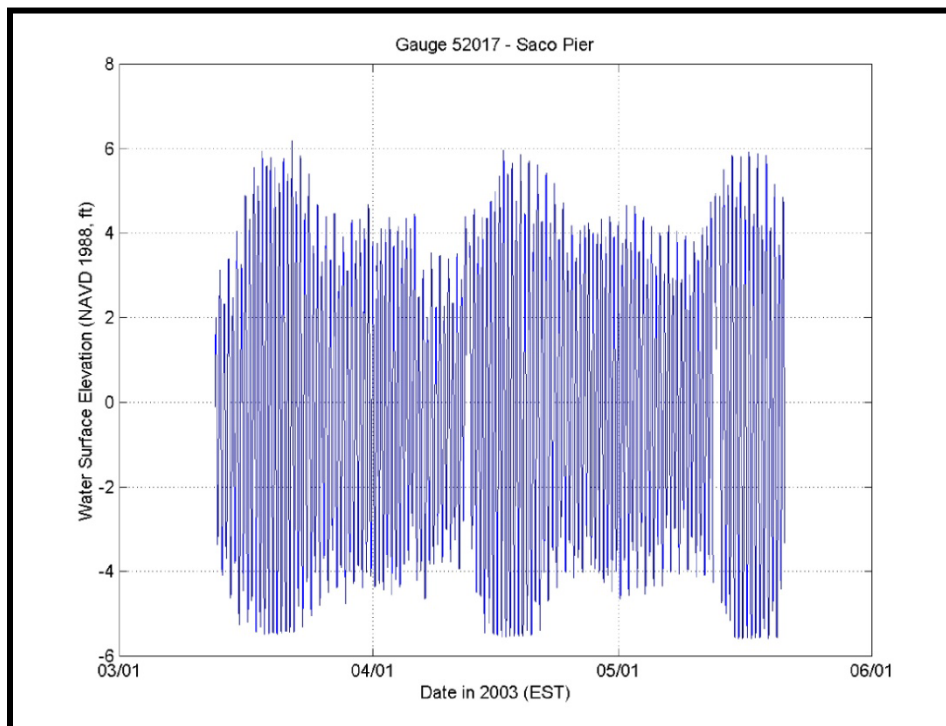


Figure 3-8. Measured water surface elevation at Saco Pier (Gauge 52017).

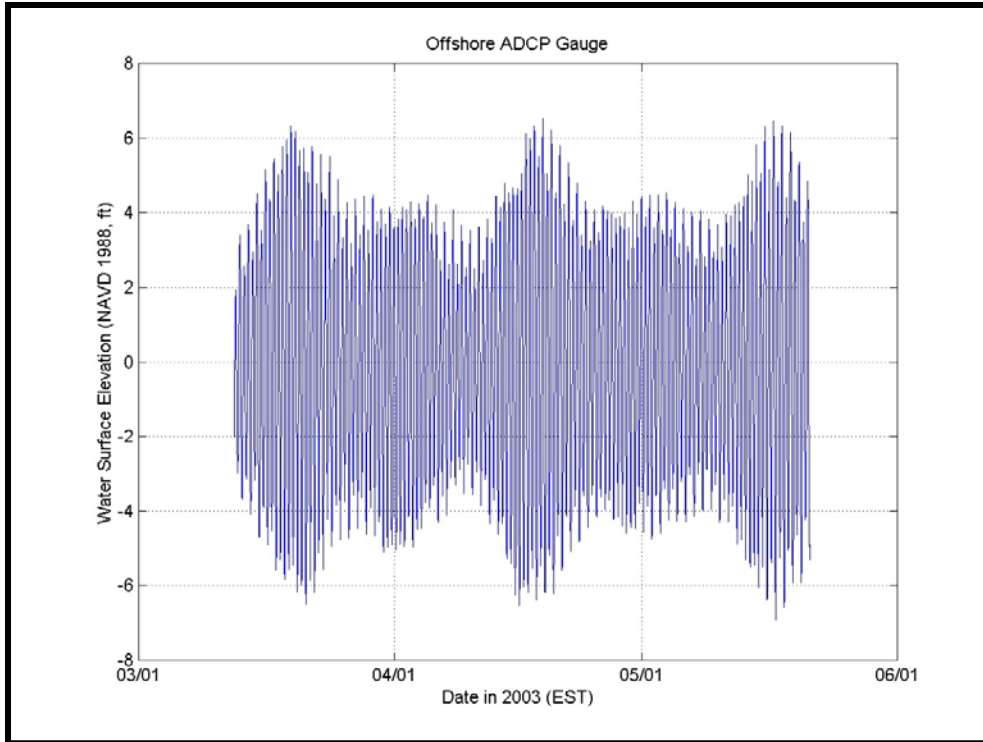


Figure 3-9. Measured water surface elevation at the offshore ADCP station.

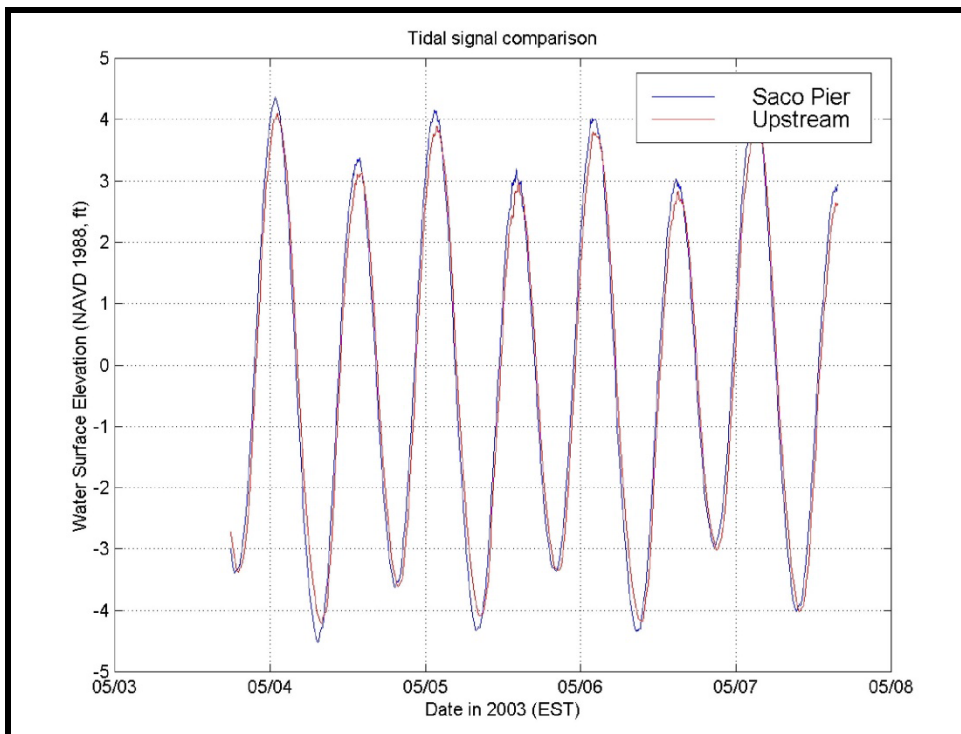


Figure 3-10. Measured water surface elevation of Saco River, inland of the jettied channel and upstream at the Yacht Club (time-zoom).

3.3 ADCP Survey

Tidal currents at selected locations in the river and offshore of Camp Ellis Beach were measured during approximately one complete lunar semi-diurnal tidal cycle (12.4 hours). The observations were obtained using an Acoustic Doppler Current Profiler (ADCP) mounted to a survey vessel. Six (6) transects were surveyed repeatedly, providing a resulting data set that provides a view of the temporal variation in spatial structure of tidal currents in the Saco River. The collection of these data served two purposes:

- Evaluation of the average tidal current magnitude in the vicinity of Camp Ellis Beach, and thereby the potential influence of tidal currents on the sediment transport rates and patterns in the nearshore zone. If the tidal currents are sufficiently small, then the wave forces might be considered the dominant forcing mechanism for sand transport at Camp Ellis Beach.
- Although not being directly evaluated in the present study, the tidal current measurements within the Saco River provide the ability to develop a hydrodynamic and tidal current sediment transport model of the Saco River estuary system. This would include an estimated quantity of the amount of sediment delivered to the coastal region via the river.

For a more detailed discussion of the ADCP survey effort refer to Appendix C, Section 6.

3.3.1 Survey Region

The survey was performed on May 14, 2003. Six (6) transects were surveyed in the jettied channel and surrounding areas: a river transect across the channel inland of the jetties, a transect parallel to the north jetty along its southern side, a transect parallel to the north jetty on its northern side, a shore-parallel transect north of the northern jetty, diagonally from the northern tip of the previous transect to the eastern tip of the northern jetty and a channel-perpendicular transect from the eastern tip of the north jetty to the eastern tip of the south jetty (see Figure 3-11). These six transects formed a contiguous loop to observe the spatial and time varying current regime in the area. Additionally, a seventh transect was surveyed to collect additional information during time periods when the tide level was too low to survey safely transects 3-5. This seventh transect progressed along the jettied channel from east to west.

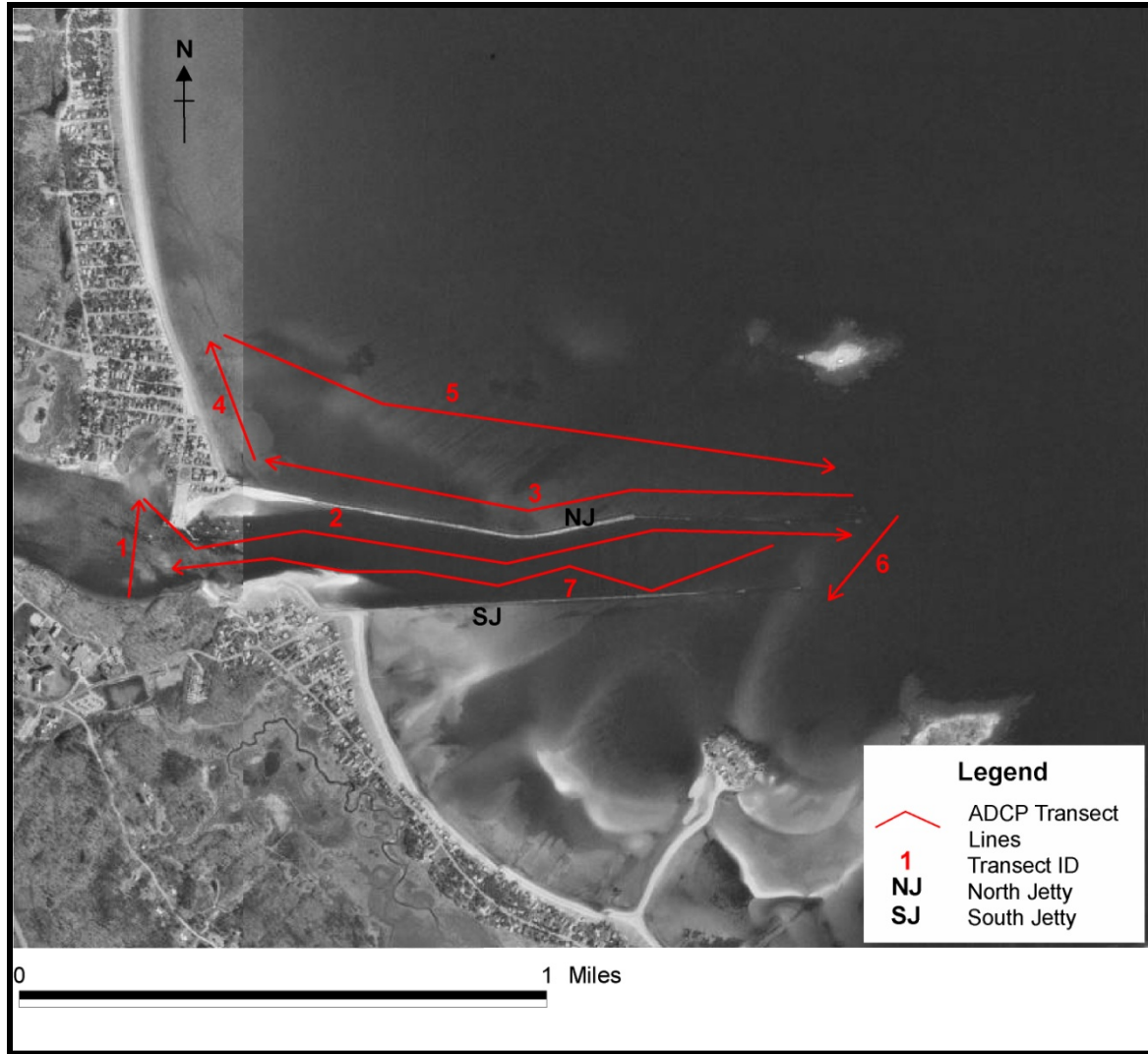


Figure 3-11. Location of ADCP transects.

3.3.2 Survey Results

3.3.2.1 Color Contour Plots

An example of the color contour plots have been provided as Figure 3-12 with the full set of color plots for each transect presented in Appendix 6-A of the WHG report (Appendix C). A detailed description of the ADCP data collection effort and results can be found in Appendix C, Section 6. The color contour plots represent measured conditions at the time of the survey. Each pair of plots present the spatial structure of flow through the transect at a discrete time period. Viewing a series of these plots for sequential intervals through a complete tidal cycle can offer a better understanding of how the spatial structure of flow varies with time.

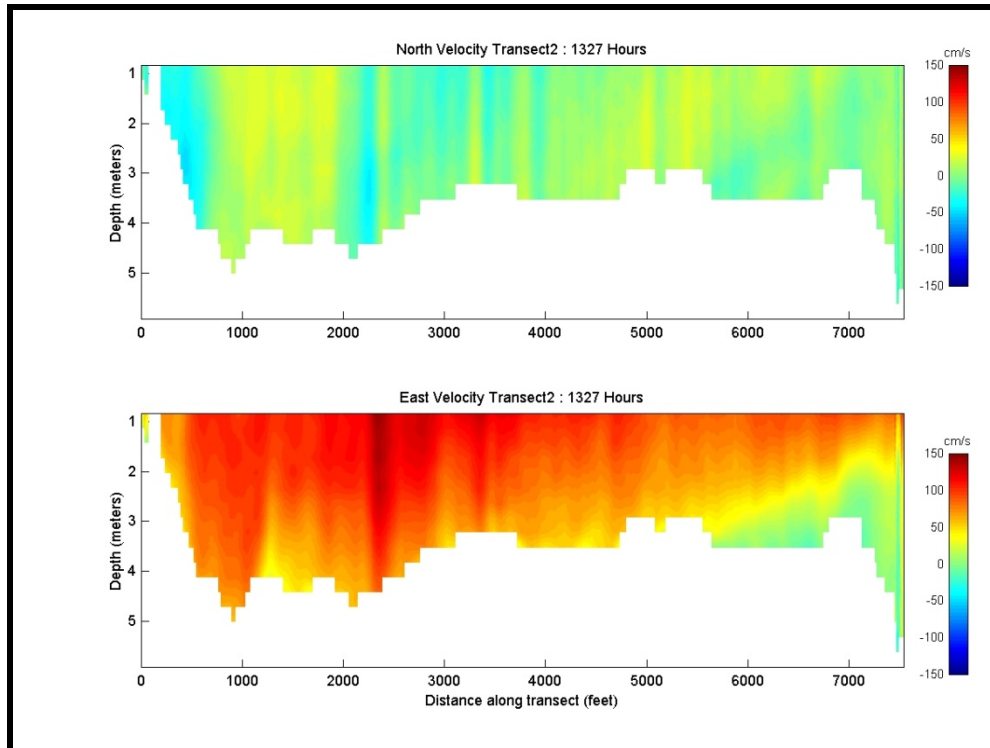


Figure 3-12. Color contour plots of north-south/east-west velocity (Transect 2 at 1:27 PM on 05/14/03).

3.3.2.1 Averaged Velocities

The velocities at selected nodes across each transect were calculated for each time step. Each transect was divided into eight (8) equal-length subsections; the center of each subsection was labeled individually as node 1 through node 8. For each node, vertically- and horizontally-averaged (east and north) velocity components were calculated for each time step. The vertical average of each ensemble consisted of the mean velocity for all valid bins. The result of this averaging procedure was a series of values showing the average velocity magnitude and direction for each loop of transects. In addition, the nodal averages included the average time of all ensembles in the subsection, average water depth of all ensembles in the subsection, and x-y position of each node. The values for each contiguous loop were plotted as arrows on separate geo-referenced maps to show the spatial current characteristics during each time step. An example plot has been shown as Figure 3-13 with a complete set provided in Appendix C, Sub-Appendix 6-B.

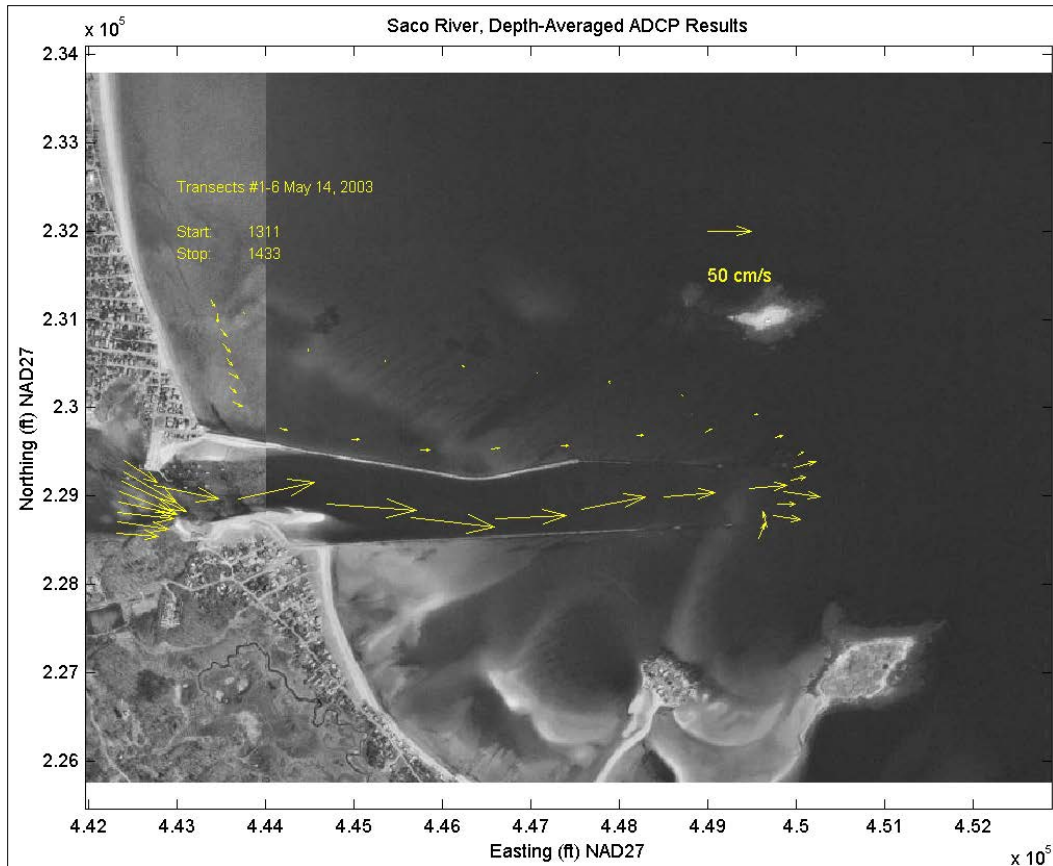


Figure 3-13. Plan view of depth-averaged currents observed at all six (6) transects during an ebb tide.

3.3.3 ADCP Summary/Result Discussion

- The relative strength of the currents within the jettied channel (≤ 150 cm/s or 4.9 ft/s) exceeded at the seaward end of the channel (≤ 30 cm/s or 1.0 ft/s). This feature is particularly obvious near the inland jetty site where channel-width reduction accelerates the flow (contraction flow) even further. This feature is important in considering sediment transport within the system and more importantly transport of sediment out of the river system. The flooding currents seem to favor a west to west northwest direction focused in the southern part, of the channel whereas the ebb currents favor a northeasterly direction in the northern part of the channel. Consequent dominant sediment transport out of the system likely removes sediment from the northern part of the channel and transport it to the northeast as it leaves the jettied channel.
- The magnitude of the tidal currents in the nearshore vicinity north of the jetties is minimal. This lack of significant tide-induced currents offshore of Camp Ellis Beach suggests that wave processes are the primary driver of sediment movement in the nearshore vicinity of Camp Ellis Beach.

3.4 Wave Data Collection

The wave data were a critical component of the overall project and were used to provide an understanding of wave propagation within the vicinity of Camp Ellis Beach, as well as to provide calibration and verification data for the numerical wave transformation models. Due to the complex bathymetry in the study area, accurate model calibration and verification required on-site information near these complex features (e.g., offshore islands, structures, rock outcrops, etc.). The collection program used two bottom-mounted Acoustic Doppler Current Profilers (ADCP). The ADCPs recorded directional wave information once every hour over a nine-week period. The ADCPs also collected directional current information and water level elevation every 10 minutes. The data collected by the two ADCPs were statistically evaluated and utilized to facilitate calibration and verification of the various wave transformation models (STWAVE, WAVAD, and CGWAVE) used in the numerical modeling portion of the project. For more detailed information regarding the wave data collection refer to Appendix C, Section 7. The results of the wave modeling that will utilize the wave data can be found in Section 5.

3.4.1 Instrument Deployment

The two bottom-mounted Acoustic Doppler Current Profilers (ADCPs) were deployed on March 12, 2003. Each ADCP instrument was secured to the ocean floor using a trawl-resistant mooring system (Figure 3-14). The ADCP trawl mount was anchored to the sea floor via four (approximately 90 cm long) screw anchors.



Figure 3-14. ADCP fitted in trawl-resistant bottom mount.

The Offshore ADCP was placed in the region seaward of Eagle and Ram Islands in approximately 56 ft (17 m) of water at Mean Tide Level (Figure 3-15). The ADCP operated at a frequency of 600 kHz and could resolve waves having a 2.9 second period and longer. The deployment diver reported that the sea floor is a flat, featureless gravel bottom. Observations made during retrieval of the ADCP did not indicate any measurable movement of the trawl mount during the recording period.

The second ADCP (Nearshore ADCP) was placed in the region landward of Eagle and Ram Islands in approximately 13 ft (4 m) water depth (Figure 3-15). The deployment diver reported that the sea floor was a flat, featureless sand bottom. Observations made during the retrieval of the ADCP showed that the platform was partially buried; however the ADCP head was not covered.

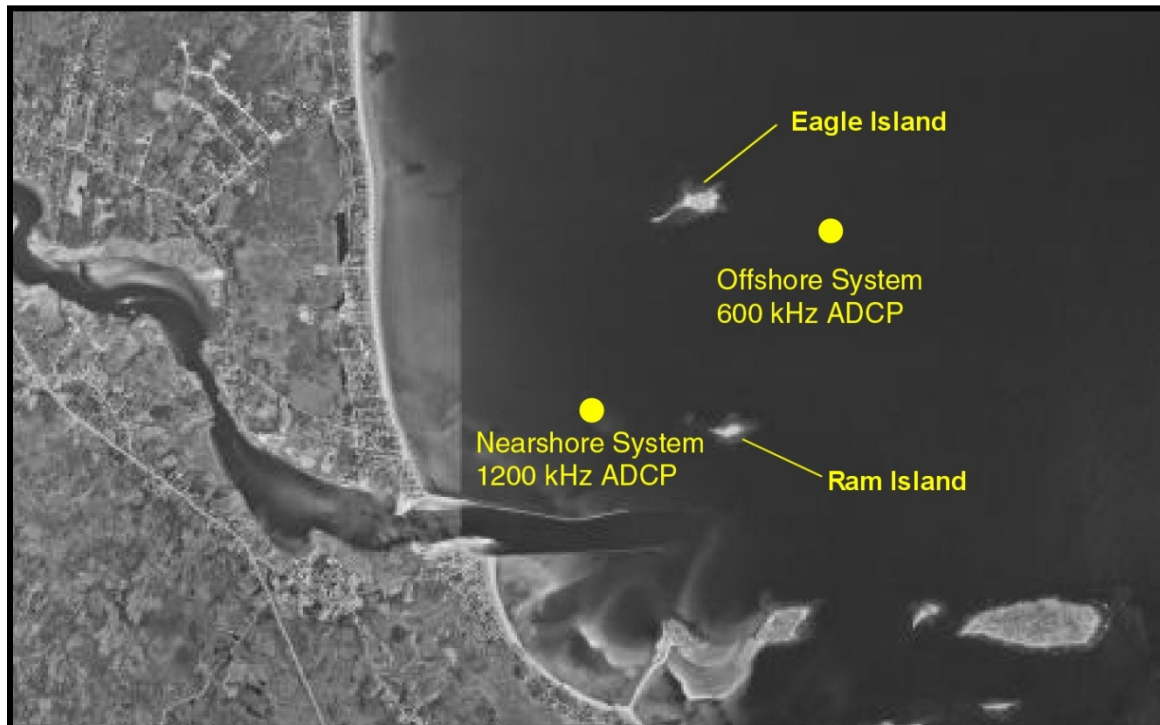


Figure 3-15. Approximate location of ADCP systems deployed offshore of Camp Ellis Beach.

3.4.2 Wave Observations

3.4.2.1 Offshore Wave Station

The Offshore ADCP made measurements from 1400 hrs March 12, 2003 until 1000 hrs May 21, 2003. There was no data loss during the reporting period (100% data return). About 97.85% of the data were usable.

The data indicate that significant wave height values (Figure 3-16) ranged from 0.6 to 7.4 ft (17 to 226 cm) during the reporting period. The most common wave heights were in the range between 1.2 and 1.6 ft (38 and 50 cm) (20.0%) and between 1.6 and 2.1 ft (50 and 63 cm) (19.0%). Significant wave heights exceeded 3.3 ft (1 m) 19.2% of the time. Peak wave periods (Figure 3-17) ranged from 3.0 to 14.2 seconds, with the majority of wave periods (21.1%) located in a band from 8 to 9 seconds. Of the remaining peak wave periods recorded, 48.6% of the time they were less than 8 seconds, and 30.3% of the time they were greater than 9 seconds.

Waves approached the offshore ADCP location primarily from the ESE. The largest percentage of waves came from the ESE (43.9%), with the second largest 22-degree band being from the SE (28.0%). These

directions are expected for the time of year of the deployment, and based on the geometry of the site location within the Gulf of Maine.

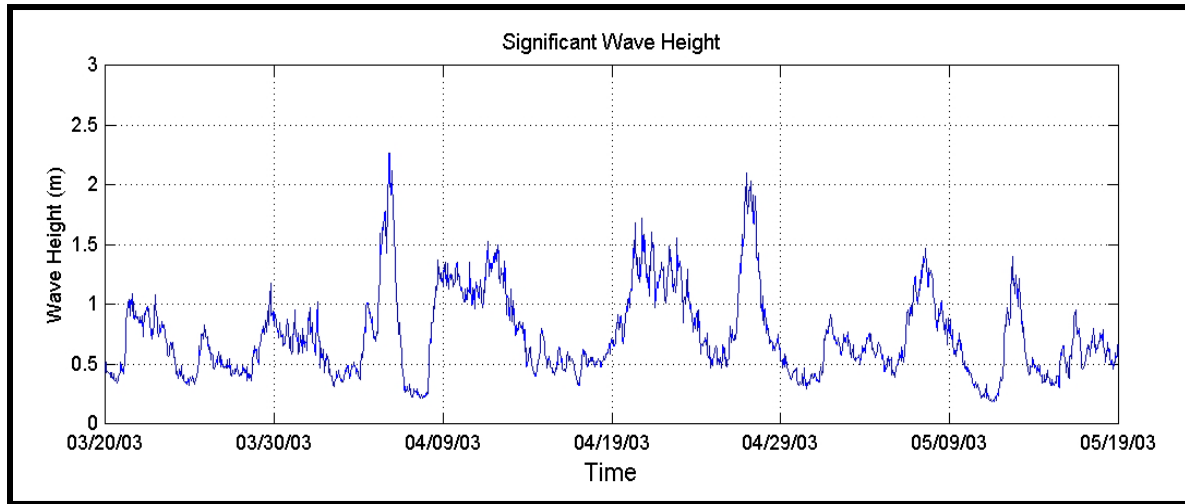


Figure 3-16. Offshore ADCP station significant wave height time series.

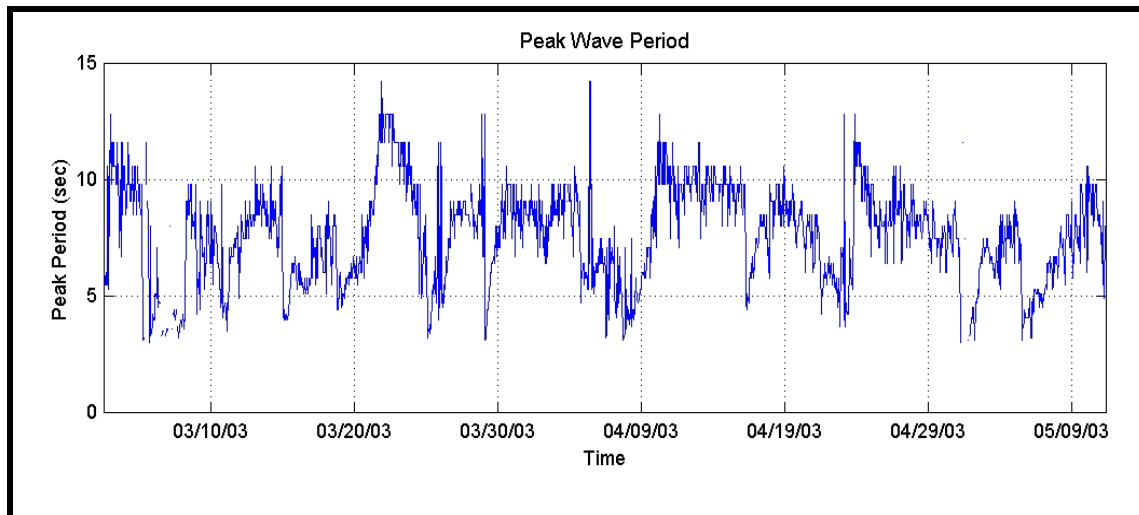


Figure 3-17. Offshore ADCP station peak wave period time series.

Figure 3-18 presents the directional distribution of wave height (cm) data (illustrated using a wave rose). The gray-scale sidebar indicates the magnitude of wave height, the circular axis represents the direction of wave approach (coming from) relative to North (0 degrees), and the extending radial lines indicate percent occurrence within each magnitude and directional band. The most common wave approach is from the ESE. Figure 3-19 presents a similar rose plot for the directional distribution of the peak wave period.

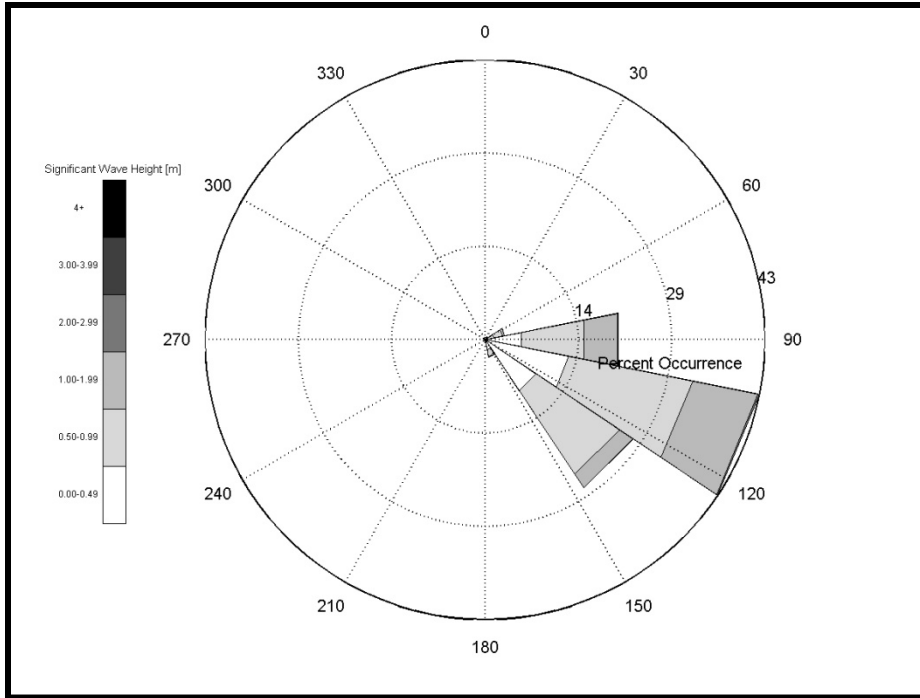


Figure 3-18. Wave rose of wave height data at Offshore ADCP station (March- May 2003).

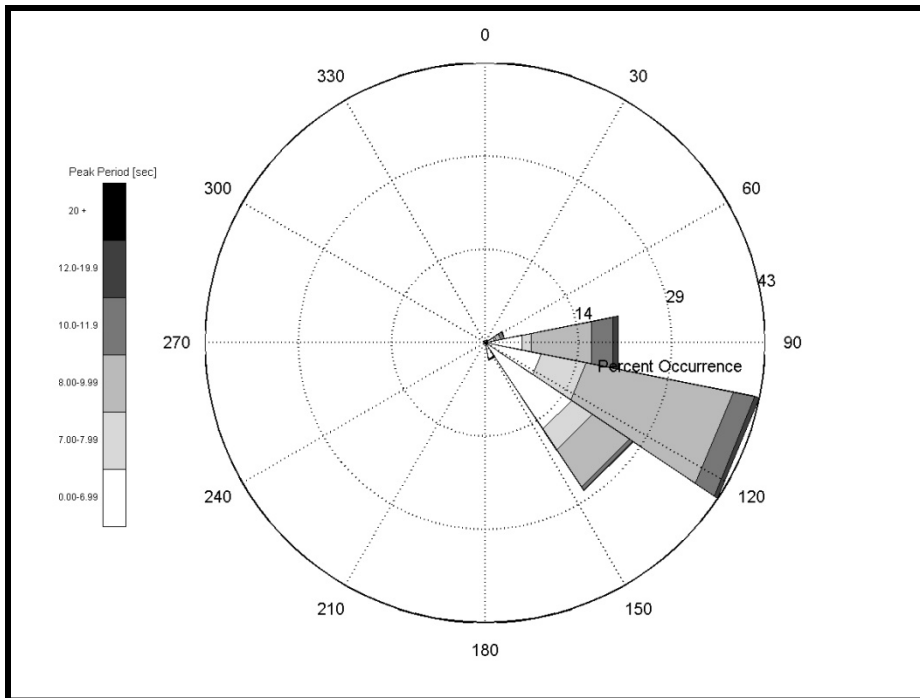


Figure 3-19. Wave rose of peak period data at Offshore ADCP station (March- May 2003).

In order to evaluate some of the higher energy time periods that occurred during the deployment, wave events were defined. For discussion purposes, higher energy wave events are herein defined as times when significant wave heights exceed 100 cm (3.3 ft) for more than 12 hours. Six higher energy wave

events were observed at the offshore ADCP site during the deployment time period, as evidenced in the time series of significant wave height shown in Figure 3-16. Table 3-1 details these events, including some of the pertinent event statistics.

Table 3-1. Wave events at offshore ADCP station (event threshold: $H_{sig} > 100$ cm for time >12 hours).

| Event ID Number | Start of Event | End of Event | Time event exceeded threshold (hours) | Mean H_{sig} during event (cm) | Max H_{sig} during event (cm) | Mean T_{peak} during event (sec) | Mean Peak Wave Direction during event (degrees) |
|-----------------|---------------------|---------------------|---------------------------------------|----------------------------------|---------------------------------|------------------------------------|---|
| 1 | 4/4/03 12:00:00 | 4/6/03 06:00:00 | 42 | 135 | 226 | 7.1 | 100 |
| 2 | 4/8/03 13:00:00 | 4/13/03 03:00:00 | 110 | 119 | 152 | 8.3 | 110 |
| 3 | 4/19/03 23:00:00 | 4/23/03 14:00:00 | 87 | 125 | 172 | 9.9 | 105 |
| 4 | 4/26/03 12:00:00 | 4/27/03 22:00:00 | 34 | 158 | 210 | 7.2 | 116 |
| 5 | 5/6/03 20:00:00 | 5/8/03 13:00:00 | 41 | 113 | 147 | 8.5 | 109 |
| 6 | 5/12/03 5:00:00 | 5/13/03 05:00:00 | 14 | 115 | 140 | 7.0 | 102 |

3.4.2.2 Nearshore Wave Station

The nearshore ADCP made measurements from 1500 hrs March 12, 2003 until 1200 hrs May 21, 2003. There was no data loss during the reporting period. However, only 97.2% of the data were usable, with the unusable data occurring when waves were quite small.

The data indicate that significant wave height values (Figure 3-20) ranged from 0.5 to 3.8 ft (15 to 117 cm) during the reporting period. The greatest percentages of wave heights were between 0.8 and 1.2 ft (25 and 38 cm) (35.1%) and between 1.2 ft and 1.6 ft (38 and 50 cm) (20.4%). Wave heights exceeded 2.5 ft (75 cm) only 6.8% of the time during the deployment time period.

Peak wave periods (Figure 3-21) ranged from 2.0 to 14.2 seconds, with the majority of wave periods (19.1%) located in a band from 8 to 9 seconds. Of the remaining peak wave periods recorded, 40.3% of the time they were less than 8 seconds, and 40.7% of the time they were greater than 9 seconds (but less than 15 seconds).

The nearshore ADCP depicted a wave environment that had little variation in approach direction over time (Figure 3-22). Waves approached the nearshore station between northeast and east 87.5% of the time. The greatest portion of those waves approached from the E (70.4%). Compared to the offshore ADCP station, which had a relatively consistent wave approach from the ESE, the nearshore ADCP station indicates a consistent E and slightly NE approach direction. Therefore, waves approaching the

Camp Ellis region appear to be transformed by the offshore island complex towards the Camp Ellis Beach area. Wave heights were reduced compared to the offshore ADCP station.

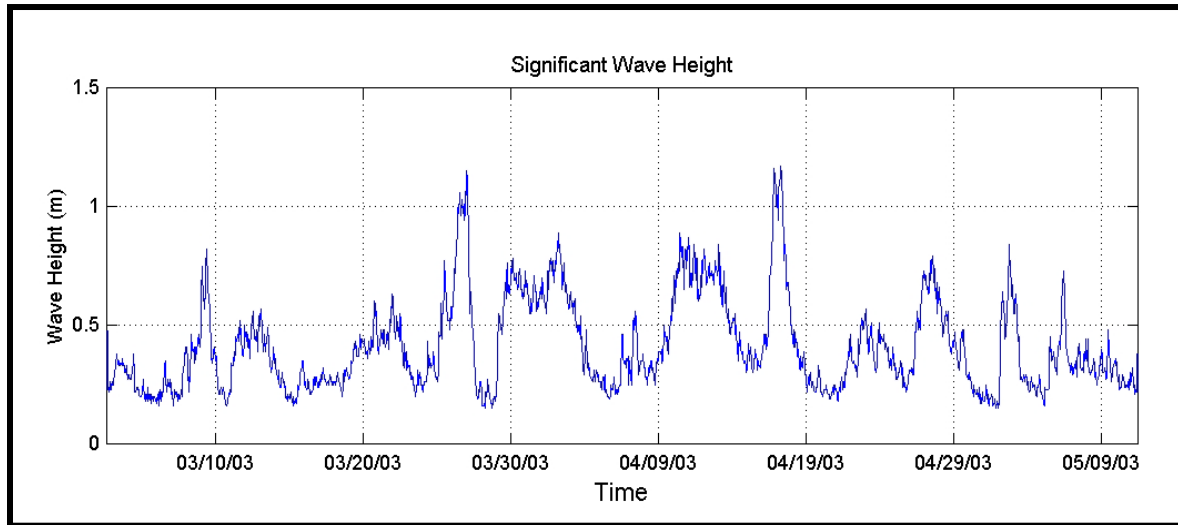


Figure 3-20. Nearshore ADCP station significant wave height time series.

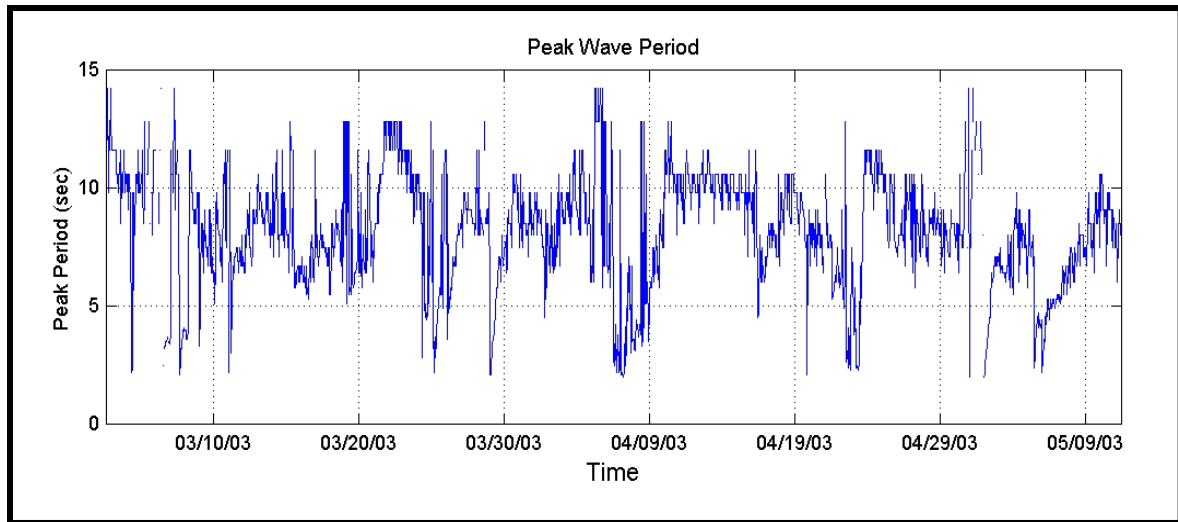


Figure 3-21. Nearshore ADCP station peak wave period time series.

Figure 3-22 presents the directional distribution of wave height (cm) data (illustrated using a wave rose). The gray-scale sidebar indicates the magnitude of wave height, the circular axis represents the direction of wave approach (coming from) relative to North (0 degrees), and the extending radial lines indicate percent occurrence within each magnitude and directional band. The most common wave approach is from the ESE. Figure 3-23 presents a similar rose plot for the directional distribution of the peak wave period.

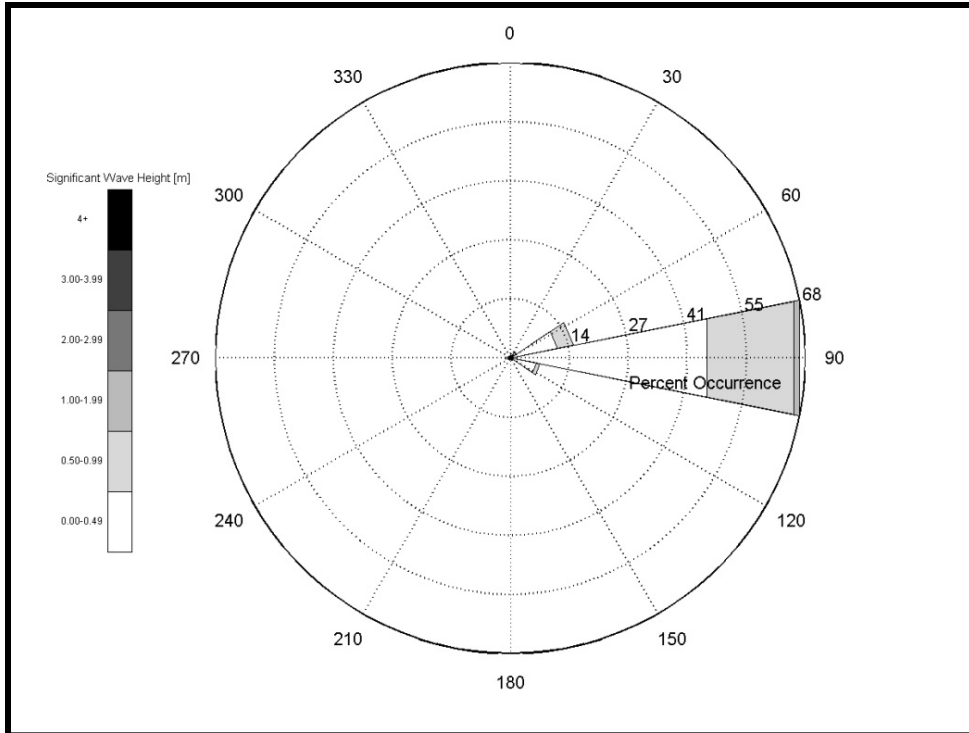


Figure 3-22. Wave rose of wave height data at Nearshore ADCP station (March- May 2003).

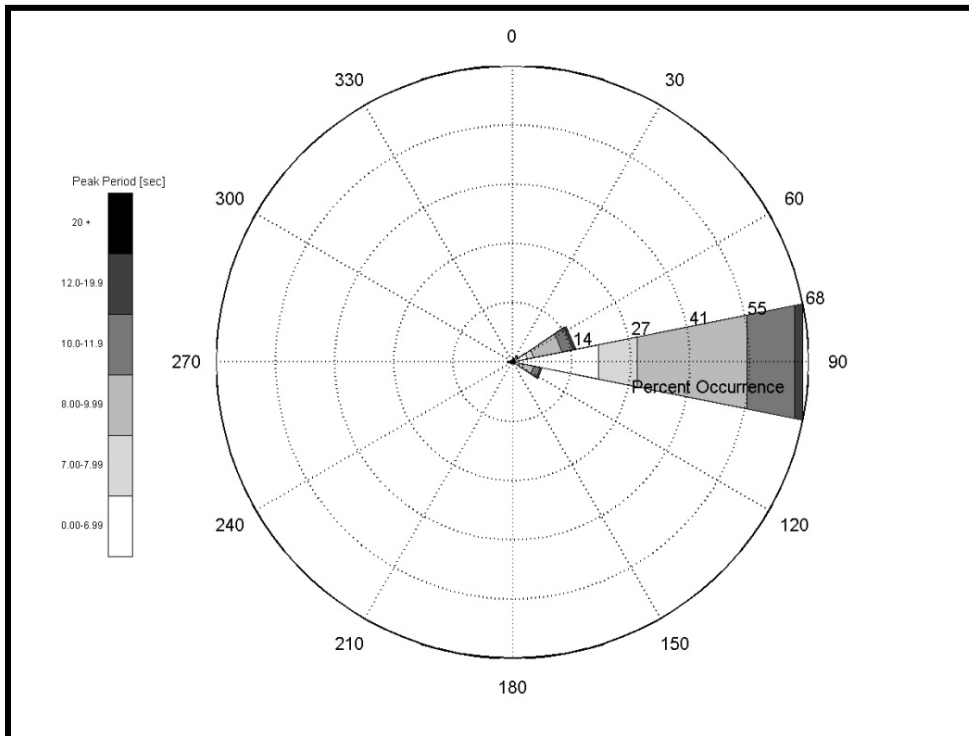


Figure 3-23. Wave rose of peak period data at Nearshore ADCP station (March- May 2003).

Again, in order to evaluate some of the higher energy time periods that occurred during the deployment, wave events were defined. For discussion purposes, wave events are herein defined as when significant wave heights exceed 2.5 ft (75 cm) for more than 12 hours. Four higher energy wave events occurred at the study site during the months of recording, as is evidenced in the time series of significant wave height shown in Figure 3-20. Table 3-2 details these events, including some of the pertinent event statistics. These four events are a subset of the six events that were measured at the Offshore ADCP. The offshore ADCP measured two events in May 2003 that did not exceed the required thresholds at the Nearshore ADCP station.

Table 3-2. Wave events at nearshore ADCP station (event threshold: $H_{sig} > 75$ cm for time >12 hours).

| Event ID Number | Start of Event | End of Event | Time event exceeded threshold (hours) | Mean H_{sig} during event (cm) | Max H_{sig} during event (cm) | Mean T_{peak} during event (sec) | Mean Peak Wave Direction during event (degrees) |
|-----------------|---------------------|---------------------|---------------------------------------|----------------------------------|---------------------------------|------------------------------------|---|
| 1 | 4/4/03 12:00:00 | 4/6/03 04:00:00 | 40 | 81 | 115 | 7.5 | 85 |
| 2 | 4/11/03 15:00:00 | 4/12/03 12:00:00 | 23 | 77 | 89 | 7.3 | 90 |
| 3 | 4/20/03 09:00:00 | 4/23/03 03:00:00 | 66 | 74 | 89 | 10.3 | 86 |
| 4 | 4/26/03 16:00:00 | 4/27/03 15:00:00 | 23 | 99 | 117 | 7.4 | 88 |

3.4.2.3 Additional Wave Data

The wave ADCP data collected during this project were compared to other available wave information in the area in order to ensure proper functionality of the observation systems. This section presents a brief comparison of the observed wave data to the NOAA buoy located offshore of Portland. Since the NOAA buoy does not record directional information, comparison is performed only between significant wave height and peak wave period.

The NOAA buoy recorded a mean significant wave height of 2.9 ft (0.91 m) and a maximum of 8.4 ft (2.56 m) during this time period. The time series of significant wave height as recorded by the NOAA buoy, as well as the Offshore ADCP and Nearshore ADCP, is presented in Figure 3-24. The significant wave heights recorded by the ADCPs indicate a reduced magnitude, as expected due to typical energy losses and dissipation effects as the waves approach the shoreline. Patterns in the significant wave heights are similar at all three locations, providing a good level of confidence in the observed data.

The mean peak wave period recorded by the NOAA buoy was 7.6 seconds with a maximum of 12.5 seconds. Figure 3-25 shows peak period recorded by the NOAA buoy and both the Offshore and Nearshore ADCP. The peak period also tracks well between the three locations. Differences between locations occur when the wave height is small or immeasurable. For example, during the time period

between approximately May 11 and May 13, the Nearshore ADCP observed minor wave heights, and subsequently non-representative wave periods.

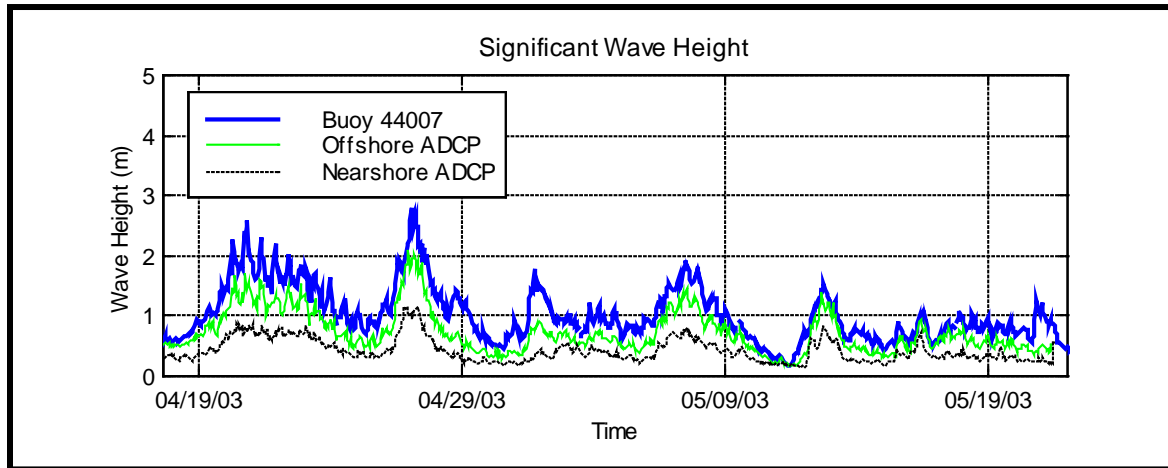


Figure 3-24. Comparison of significant wave height - NOAA buoy 44007 and the two ADCP stations.

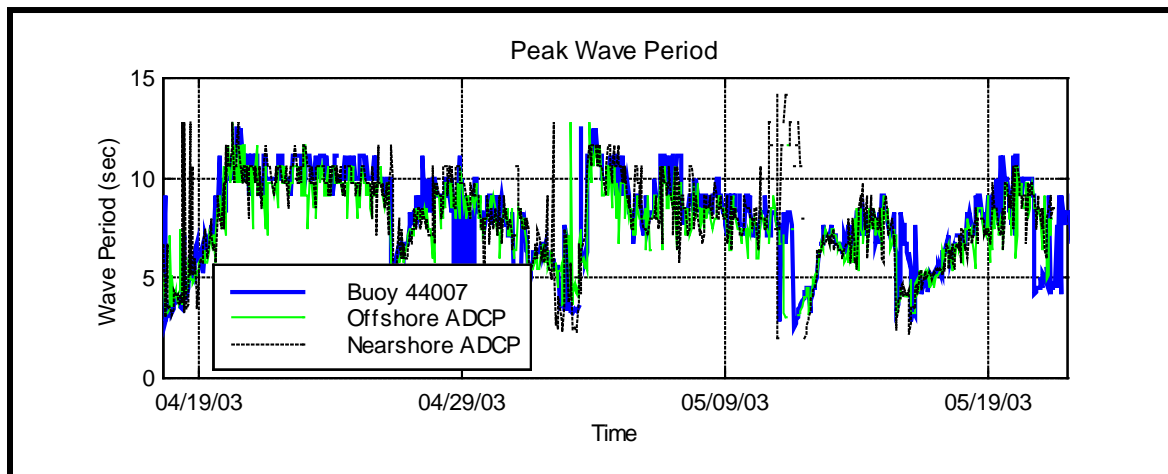


Figure 3-25. Comparison of peak wave period (sec) between NOAA buoy 44007 and the two ADCP stations during the deployment time period (March-May 2003).

3.4.2.4 Current Observations

Although wave observations were the focus of the field data collection program, the ADCPs were also programmed to resolve the water column into several elevation bins and determine the three components of the ambient current (u,v,w) within each bin. Only the horizontal components of the surface current are analyzed in this report (u,v).

For the Offshore and Nearshore ADCP gages, Figures 3-26 and 3-27, respectively, presents the directional distribution of current magnitude (cm/s) data (illustrated using a wave rose). The gray-scale sidebar indicates the magnitude of the current, the circular axis represents the direction of current approach (coming from) relative to North (0 degrees), and the extending radial lines indicate percent occurrence within each magnitude and directional band. For the Offshore gage, Figure 3-26 depicts a

general trend of current flow alternating between flows directed towards the Southwest and the Northeast due to the tidal fluctuations. For the Nearshore gage, Figure 3-27 depicts significantly smaller and less organized currents than at the Offshore ADCP station. The average current magnitude collected at the nearshore location was 0.2 ft/s (5.5 cm/sec). The maximum magnitude recorded at this location was 0.7 ft/s (20.3 cm/sec).

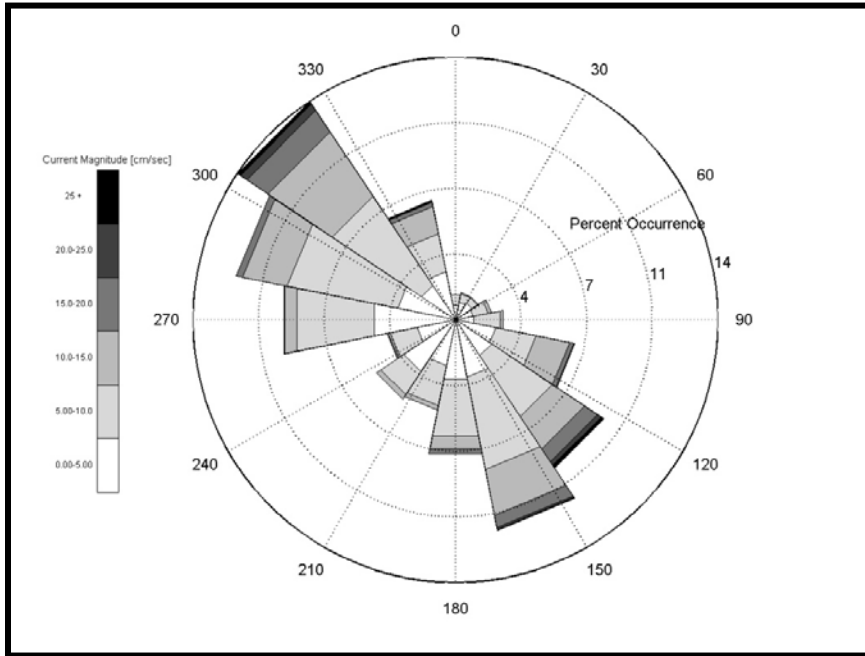


Figure 3-26. Rose plot of current observations at the Offshore ADCP station (March- May 2003).

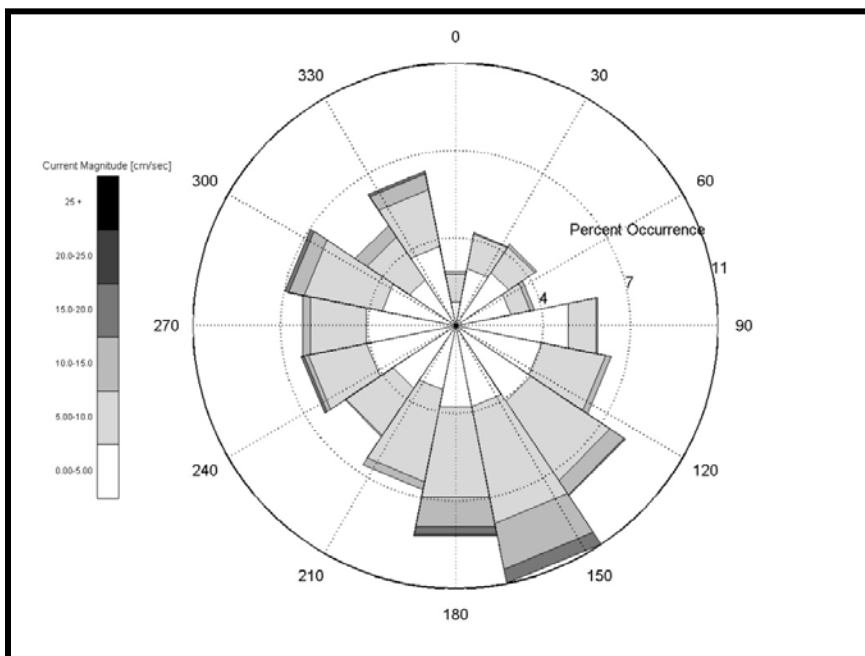


Figure 3-27. Rose plot of current observations at the Nearshore ADCP station (March- May 2003).

3.4.3 *Wave Data Summary*

Wave and current data were collected at target locations seaward and landward of the Eagle/Ram Island complex in order to assess accurately the wave transformations that occur due to the complicated physical nature of the islands and bathymetry in this region. Wave data during the relatively short deployment period, recorded waves approaching Camp Ellis Beach were primarily unidirectional, approaching from an ENE direction, even when waves approach from the ESE seaward of the islands. The wave ADCP data were compared to existing NOAA buoy data to provide a level of confidence in the observed data. The wave data were a necessity for calibrating and verifying the numerical wave transformation models. Data were used extensively to validate model performance, as presented in later sections.

4.0 SHORELINE MAPPING AND SEDIMENT TRANSPORT

4.1 Historical Shoreline Change Analysis

A computer-based mapping methodology, within a Geographic Information System (GIS) framework, was used to compile and analyze changes in the historical shoreline position between 1864 (T-Sheet compiled from 1849 to 1879) and 1998 for Saco Bay. The purpose of this task was to quantify changes in shoreline position using the most accurate data sources and compilation procedures available, and to characterize areas of erosion and accretion. For a more detailed discussion of historical shoreline change refer to Appendix C, Section 3. The shoreline change analysis does not include years beyond 1998 because at the time the analysis was done under the Woods Hole Group contract (in 2003) the 1998 ortho-rectified aerial photos were the latest available. Since 2003 there have been other photo sets collected as well as USACE LIDAR data sets. Unfortunately funds were not available to update the shoreline change mapping effort, and the lengthy delay beyond 2003 was obviously not envisioned by the PDT. To help address this, a narrative description with figures has been provided for the changes to the system post 1998, in Section 4.1.3.

4.1.1 Data Sources

For this project, four primary sources of data were used to evaluate changes in shoreline position during the period 1864 (1849-1879) to 1998. Shoreline data from 1864 (1849-1879) and 1944 were obtained from U.S. Coast and Geodetic Survey (USC&GS) historical T-sheets. Data for the 1864 T-sheet was collected over a period of time ranging from 1849-1879 and 1864 was chosen as a mid-point timeframe for this data. The shoreline data shown on these historical maps were surveyed using standard planetable surveying techniques. The 1965 and 1977 shoreline data were obtained from aerial photography flown by Col-East, Inc and James W. Sewall Company, respectively, and were obtained as overlapping 9x9 inch images covering the Saco Bay study area. The 1977, 1986, and 1995 shoreline data were obtained from aerial photography provided by the Maine Geological Survey and were obtained as overlapping 9x9 inch images covering Saco Bay. Before shoreline position data from various years can be compared for quantitative analysis of shoreline change, the photographs (or shoreline positions) must be geo-referenced and corrected for distortion. These corrections were accomplished utilizing computer aided cartographic mapping software.

4.1.2 Discussion of Shoreline Change

To evaluate trends in shoreline change at Saco Bay, various graphical representations have been developed. Shoreline positions for each of the available dates between the period of 1864 and 1998 were developed and an example figure is shown as Figure 4-1. For the full set of Figures the reader is referred to Appendix C Section 3.

Figure 4-1 shows the southernmost part of Saco Bay. Negative values correspond to shoreline erosion, whereas positive values correspond to shoreline accretion. These values represent the historic average shoreline change rate over the 134-year period. Figure 4-1 indicates that the area directly north of the Saco River (Camp Ellis Beach) has experienced significant erosion for a distance of approximately 2,500

to 3,000 feet north of the northern jetty. This erosion remained significant until approximately 1986. Subsequently, this region has remained relatively stable due to the placement of rock revetment and shoreline structures during the 1970s and 1980s. The shoreline north of this 2,500 to 3,000 foot stretch has been stable throughout the time period evaluated; however in more recent times (after 1998) this area has also shown erosion likely due to increased storm events and the lack of available sediments from beaches to the south (Camp Ellis). A sandbar emanating from the north jetty melds to the shoreline at the approximate location where the shoreline erosion begins to taper off. This bar, formed since the construction of the jetties, may be a potential sediment transport pathway for sand moving from south-to-north around the jetty.

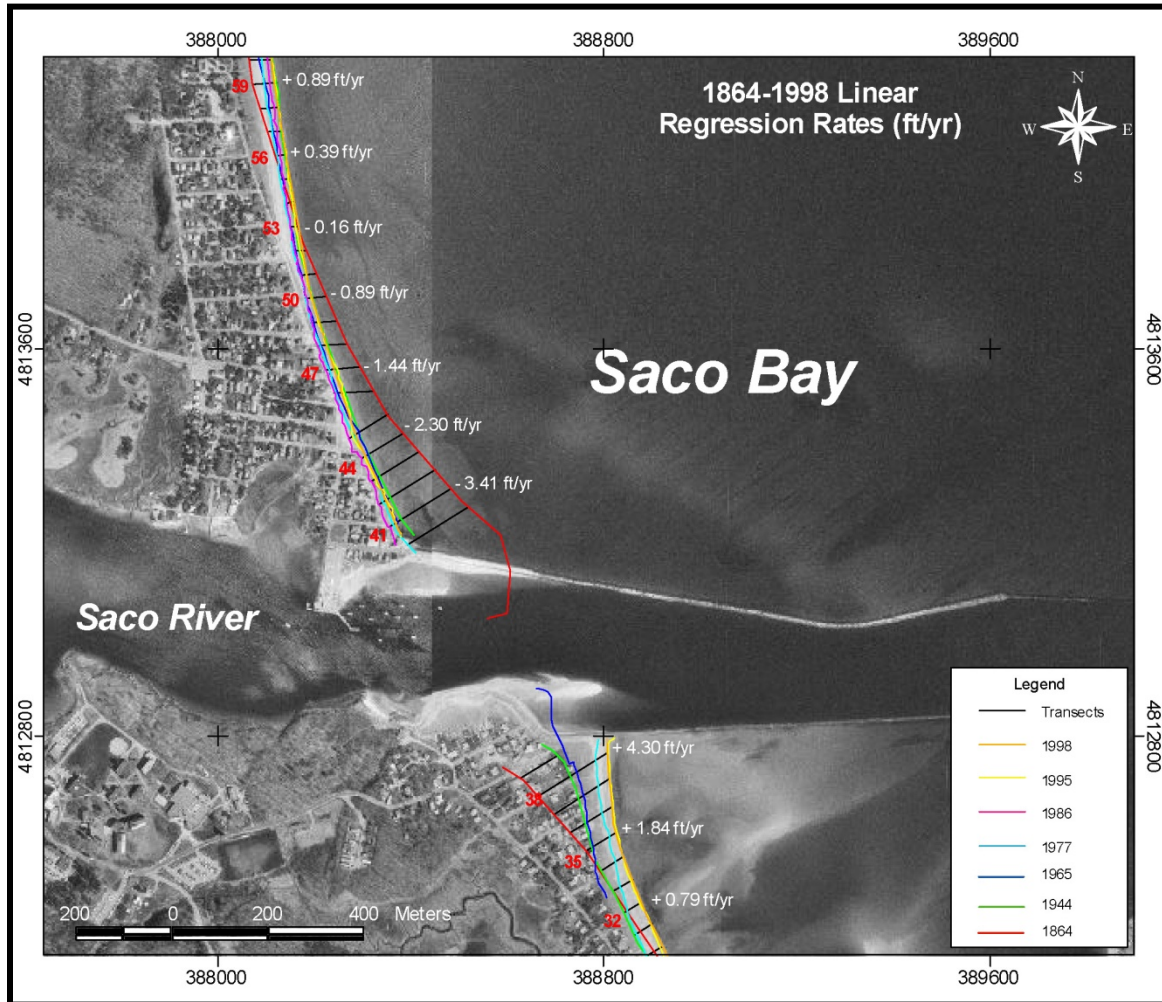


Figure 4-1. Historical shoreline positions and change rates (linear regression) from 1864-1998 for the region near the Saco River.

Figure 4-2 presents the shoreline positions and change rates for the area north of Camp Ellis Beach, containing Ferry Beach and Bay View. Although this stretch of shoreline has been relatively stable during the time period evaluated (up to 1998), erosion in this area has increased recently due to storm events and the lack of available sediment from the beaches directly to the south (Camp Ellis). Shoreline change rates indicate slight accretion, with a stable point at the salient landward of Eagle Island. This

salient feature may also reduce wave energy on both sides of the salient, producing areas more susceptible to accretion.

Figure 4-2 displays the rates of shoreline change computed using the end-point (dotted line) and linear regression (solid line) methods for the entire time period of 1864 to 1998. The x-axis indicates the rate of shoreline change, where positive values indicate accretion, and negative values indicate erosion. In general, the transect numbers increment from south to north and Figure 4-5 presents the shoreline change rates for the entire Saco Bay region. The figure shows that the end-point and linear regression methods yield similar rates of change. This similarity is an indication that the changes in shoreline position have been linear and steady through time and that there is little temporal variability in the data throughout the 134-year period at a given location. The only significant erosion within the entire Bay is at Camp Ellis Beach. Moderate accretion is evident at Goosefare Brook, the region directly south of the Saco River, and the area directly west of the Scarborough River.

To better define the shoreline morphology related to time period, the response of the shoreline was examined during different time intervals. The discussion of the historical shoreline change analysis was broken into both a historical (1864-1944) and contemporary (1944-1998) time period to compare the shoreline response to the overall shoreline change trends (1864-1998). Figure 4-3 presents the historic (red line), contemporary (green line), and overall (blue line) shoreline change rates for Saco Bay. In general, patterns and trends of erosion and accretion remain the same over all three time periods.

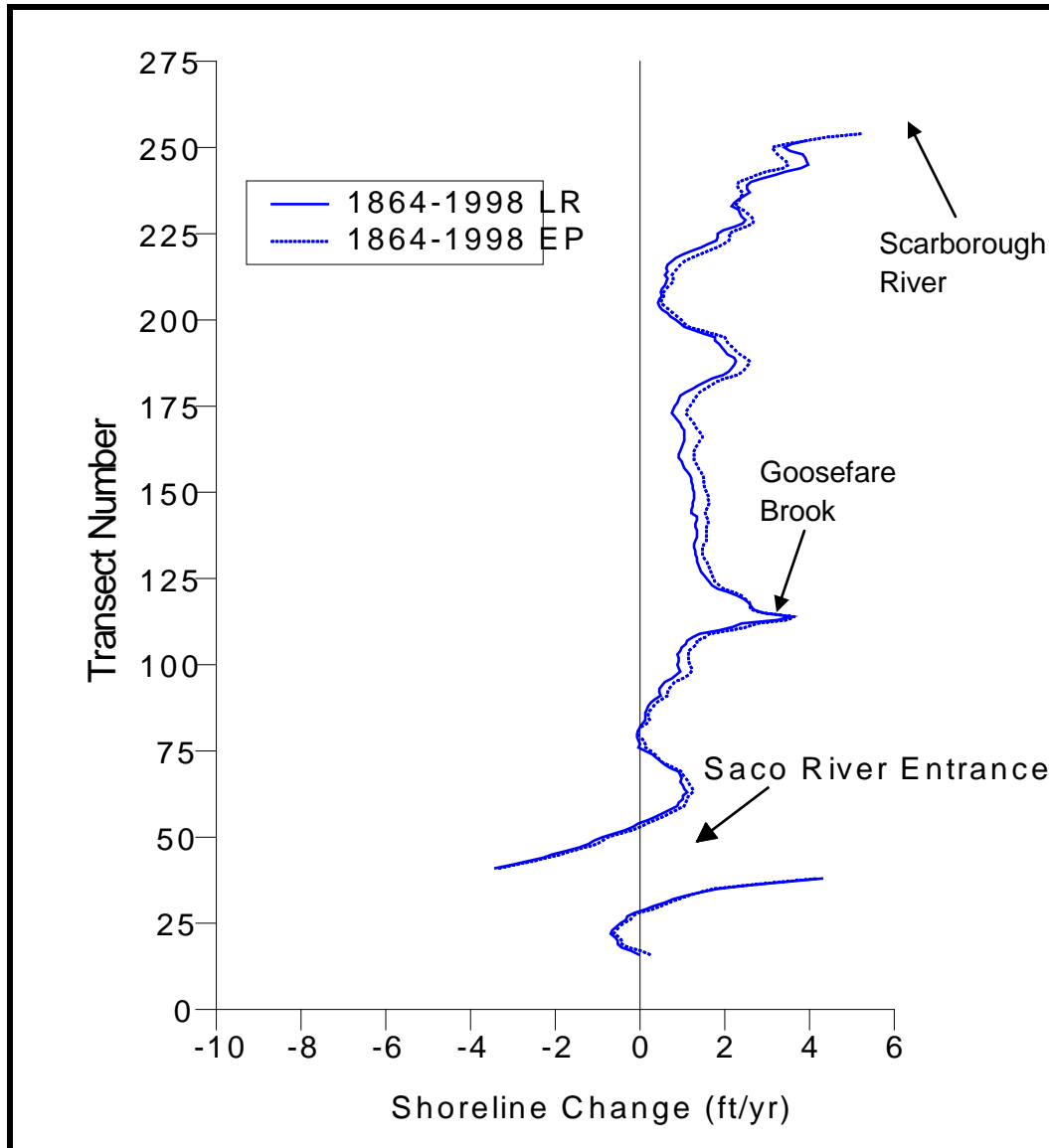


Figure 4-2. Shoreline change rates calculated using the linear regression (solid line) and end point methods (dotted line) for the period of 1864-1998.

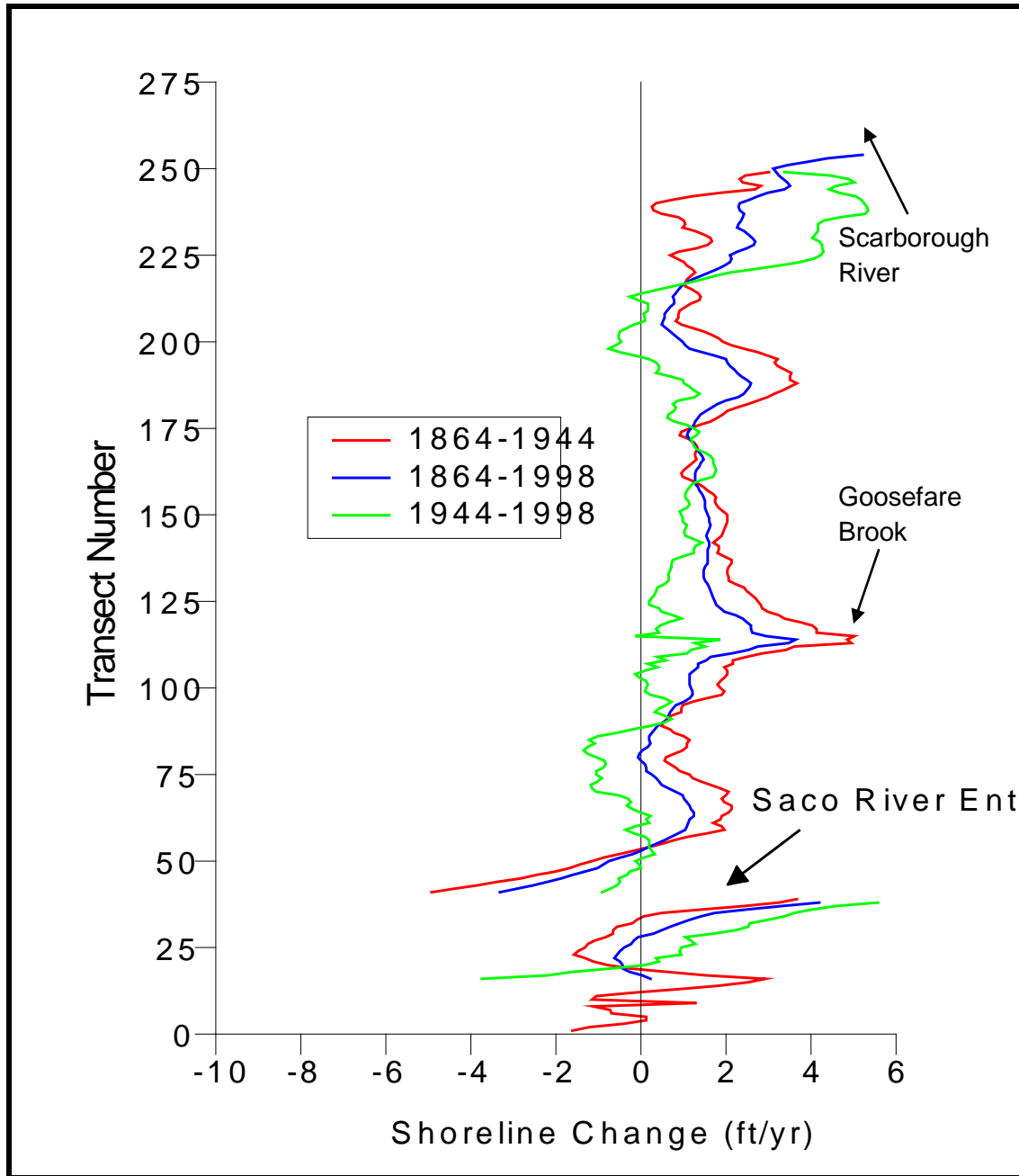


Figure 4-3. Shoreline change rates calculated using the end point method for the time periods of 1864-1944 (red), 1944-1998 (green), and 1864-1998 (blue).

4.1.3 Shoreline Change Discussion – Post 1998

As discussed in Section 4.1, the shoreline change/mapping did not extend past 1998 due to the aforementioned reasons. To help address the changes since 1998 the below narrative has been provided and as well as figures.

Since 1998, the study area has undergone significant changes that have had severe negative impacts to the shoreline and the properties along the coast. The loss of beach in front of the “temporary”

revetment structures discussed in Section 1 has been significant. As shown in Figure 4-4 and 4-5, which are 2003 and 2010 aerial photos, respectively, taken from Google Earth, the entire dry beach has been lost in the southern area. During the early 2000 time frame the southern part of the study area had a dry beach at mid tide and today there is no dry beach at any tide. This has been witnessed in the field by the PDT and it has steadily worsened through the lengthy study period. The limiting factor in comparing the photos is tide height but as highlighted there is evidence of beach loss.

Significant changes have also occurred to the northern area of the study. In the time frame of 2002 to 2005 (when most of the Wood Hole Group work was done), the acute erosion problem extended 2,500 feet to the north of the North Jetty. This was the original focus area of the project during the most recent study period (post 2002). Since that time, the erosion problem has migrated to the north, which necessitated lengthening the project study area to 3,250 feet north of the North Jetty. This 750 foot increase in project length is a 30% increase. It is theorized that the reason the problem has been migrating further to the north is that the available sand for transport to the north has largely been removed from in front of the temporary revetments to the south and therefore sand is now being removed from further north. Basically with the Saco River sand source being cut off, the area at the southern end of the project became the sand source for the system, and that has now been depleted. As discussed in Section 4.2.3, the sediment flux potential is very significant in this area. The changes to the northern over the last decade are very noticeable when looking at Figures 4-4 and 4-5. The beach width has been dramatically reduced, a road has been lost, trees that are evident in 2003 are not evident in 2010, the temporary revetment was severely damaged, repaired, and lengthened with geotubes, and the dune line has been eroded (shown in Figure 4-6 as well). To highlight the dune loss at the northern end of the project the dune/vegetation line was mapped in Google Earth from a 1998 photo and for the 2010 photo. The results are shown in Figure 4-6. As shown there has been an approximate 40 foot retreat of dune in the last 12 years. That is an erosion rate of 3.3 feet/year. This area had been stable to accretionary, as shown in Section 4.1.3, but now is experiencing the loss rate experienced in the past in the southern area of the project (as discussed in Section 4.1.3).



Figure 4-4. Google Earth Aerial Photo 2003 – Pre- recent changes highlighted.



Figure 4-5. Google Earth Aerial Photo 2010 – Post-changes highlighted.



Figure 4-6. Dune line erosion 1998 to 2010 and feature changes – Google Earth Photo 2010.

To further highlight the impacts to the north end of the project, where the most dramatic changes have occurred, Figures 4-7 to 4-11 have been provided. The figures show the north end before damage and post damage through several different cycles of repair to the road, houses, and revetment. The photos cover from 2005 up to 2010. The photos are all looking north from within the north end of the project area. It can be seen in the figures, through time there has been extensive damage, the road was abandoned, the “temporary” revetment failed, geotubes were added to the north end of the revetment, and dune line has retreated significantly. Figure 4-10 is particularly telling with the damage shown. It reiterates the point that the areas that are “protected” from the “temporary” revetments are still very vulnerable, and that the revetments are not designed or constructed for long term protection. The level of protection from these structures is uncertain and very unreliable.



Figure 4-7. North end of the project looking north following a storm – road is damaged.

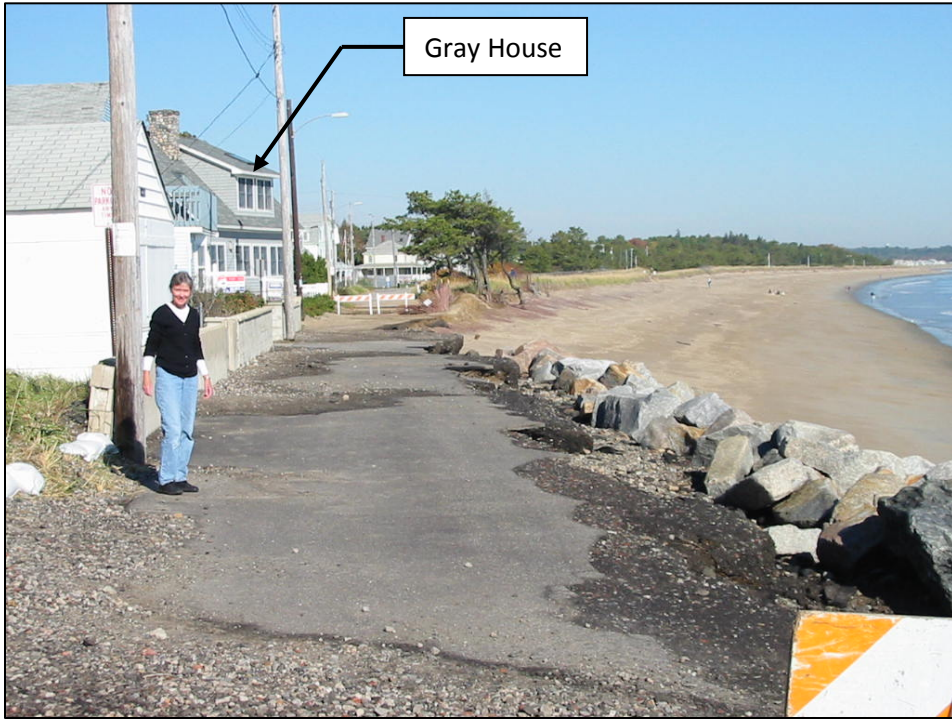


Figure 4-8. North end of project looking north (2006) – road is repaired but closed to traffic.



Figure 4-9. North end of the project – road lost (February 2007).



Figure 4-10. North end looking north - post storm April 2007. Destroyed road, houses and utilities and damaged revetment.



Figure 4-11. North end of project looking north (2010) - geotubes in place, the road is lost, trees are lost, dune is eroded.

4.1.4 Shoreline Movement Summary

An analysis of historical shoreline change was performed for an 8-mile shoreline segment along Saco Bay, Maine. The data used to compile the analyses were derived from aerial photography, historical maps, and digital ortho-photographic quads. Rates of historical shoreline change were calculated at 265 shore-normal transects from Biddeford Pool to Prouts Neck.

Between 1864 and 1998, the shoreline accreted adjacent to the southern jetty in Saco Bay. Accretion rates along this stretch ranged from +0.74 ft/yr at transect 32 to +4.3 ft/yr at transect 38, adjacent to the southern jetty. Between 1944 and 1998 the shoreline accreted more rapidly at rates exceeding 5 ft/yr. A small erosional area exists at Hills Beach (at approximately transect 25). Rates of erosion in this region are approximately -1 ft/yr.

The shoreline adjacent to the northern jetty (Camp Ellis Beach) experienced significant erosion. The shoreline between 1864 and 1998 eroded at rates between -3.4 ft/yr (at transect 41) and -0.2 ft/yr (at transect 53). The more contemporary time period (1944-1998) shows continued erosion, but at a reduced rate (approximately -1.0 ft/yr and less). This relative stabilization is primarily due to man-made intervention in the form of heavy structural stabilization (seawalls, revetments, etc.) and sand nourishment efforts in this area (placing sand dredged from the Saco River in the beach).

Between transects 53 and 101, the area between Ferry Beach and Bay View, there is a relatively stable section of shoreline from 1864 to 1998. Here shoreline change rates range from -0.2 ft/yr to +0.9 ft/yr. However, in the more contemporary time period, the shoreline becomes erosive, with rates averaging approximately -1 ft/yr. This may be due to the reduced sediment supply that is available for transport to this region from Camp Ellis Beach.

Between 1864 and 1998, a seaward growth of the shoreline occurred in the northern portion of the bay, and most notably closer to the area of Scarborough Inlet. This region has experienced accretion throughout the entire 134-year study period with accretion rates reaching +3.5 ft/yr. For recent history (1944-1998) a similar trend, with an increased magnitude, exists.

In general, in more recent history mapped (1977-1998) the entire Saco embayment (except for the study area) is experiencing shoreline accretion at reduced rates than during the historical time-period from 1864 to 1998. This may indicate a paucity of sediment supply compared to historic time periods, as less sand has been delivered from the Saco River to the beaches since the jetties (specifically the northern jetty) does not allow the sediment to be transported to the beach.

During the last decade (1998 to 2010), the shoreline mapping was not updated for the reasons provided, but based on the witnessed changes, the Google Earth photos, dune line mapping in Google Earth, and the changes/damages/destruction documented in the provided photos, the project area has continued to experience beach loss and the erosion problem has now migrated further north into areas that had

been stable or accretionary. It is theorized that the available sand for transport from the south end of the project has been depleted which has caused the more northerly erosion problem.

4.2 Sediment Transport – Without project

This section evaluates the regional sediment transport within Saco Bay, the local sediment transport in the vicinity of Camp Ellis Beach for without project conditions. For with project conditions, sediment transport, will be discussed in Section 7.2 of this appendix. The wave modeling effort used to support this analysis will be discussed in Sections 5.3 and 7.1 of this appendix. For greater detail refer to Appendix C, Section 12.

4.2.1 Grain Size Analysis

As part of the sediment transport analysis grain size was investigated by both WHG and by USACE. During the WHG effort, a total of 7 surface grab samples were collected during a site visit on October 7, 2003, Samples were collected along the coastline of Saco Bay and analyzed to determine the sediment distribution throughout the region. An example of the grain size data curves can be seen in Figure 4-12 with the grain size data provided as Table 4-1. For a full discussion of the WHG grain size analysis effort the reader is referred to Appendix C, Section 12.1.

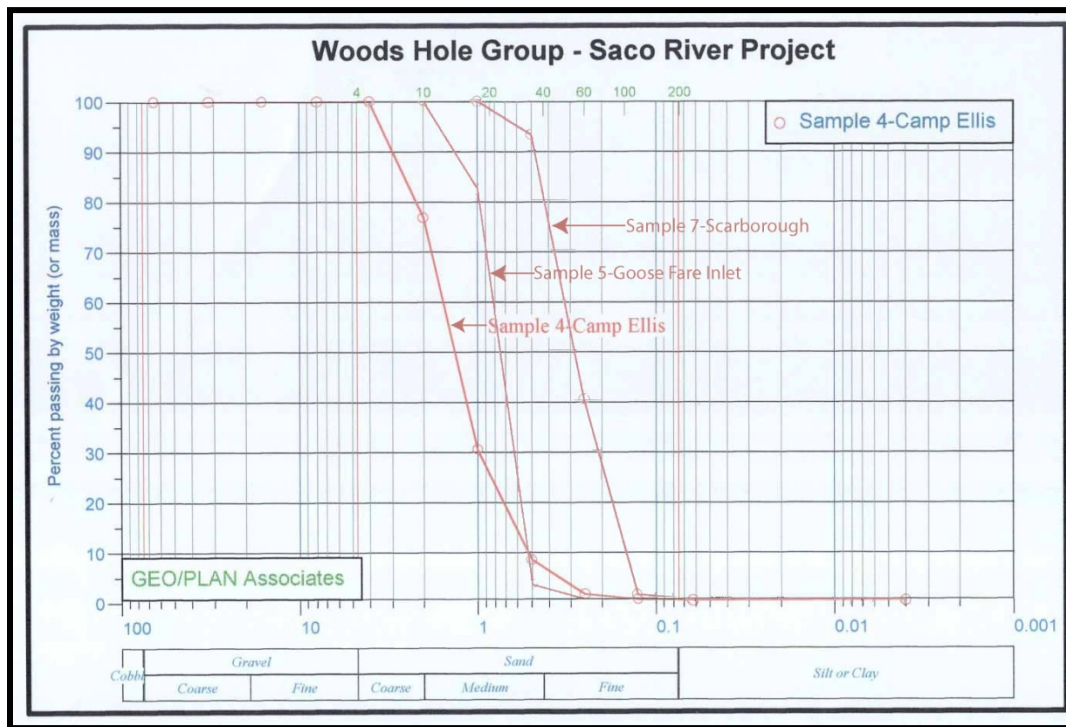


Figure 4-12. Comparison of sediment samples from locations along Saco Bay shoreline. The lower x-axis indicates the grain size in mm, while the upper x-axis shows the standard sieve sizes.

Table 4-1. Table of Sediment Properties for Saco Bay samples.

| Sample | Location | d ₅ [mm] | d ₁₆ [mm] | d ₅₀ [mm] | d ₈₄ [mm] | d ₉₅ [mm] | |
|--------|----------|------------------------|-------------------------|-------------------------|-------------------------|-------------------------|--|
| | | | | | | | |

| | | | | | | | |
|---|---------------------------------|------|------|------|------|------|-------|
| 1 | Adjacent to existing spur jetty | 0.58 | 0.63 | 1.08 | 1.73 | 1.91 | +0.65 |
| 2 | South of northern jetty | 0.15 | 0.28 | 0.63 | 0.97 | 1.54 | +1.01 |
| 3 | North of northern jetty | 0.19 | 0.30 | 0.60 | 0.90 | 1.34 | +0.87 |
| 4 | Camp Ellis Beach | 0.37 | 0.60 | 1.43 | 2.63 | 3.54 | +1.08 |
| 5 | South of Goosefare Brook | 0.51 | 0.56 | 0.74 | 1.15 | 1.71 | +0.55 |
| 6 | North of Goosefare Brook | 0.28 | 0.37 | 0.55 | 0.96 | 1.48 | +0.74 |
| 7 | Scarborough | 0.14 | 0.17 | 0.29 | 0.46 | 0.60 | +0.71 |

A general fining of sediment is observed from south to north. As such, the median grain size is finer at the northern end of littoral cell (Scarborough) than it is at the southern end (Camp Ellis). This will support the north to south sediment transport direction that was found and that will be discussed later in this section.

4.2.2 Methodology

Sediment movement in the coastal zone, as well as the effects of coastal structures on shoreline processes, can be estimated by using various types of sediment transport models. These models may differ in their detail, in their degree of representation of the physics, in their complexity, and in other manners. All models also have a certain level of uncertainty since predicting sediment transport in a dynamic coastal environment is inherently difficult. Although no single model of sediment transport may be fully representative of all conditions, these sediment transport models still provide a useful tool for analyzing the effects of structures on local coastal processes. The sediment transport model presented used by WHG is a process-based model of the regional sediment transport trends in the presence of time-variable (in direction and height) waves. The model was developed by WHG.

The sediment transport model itself consists of a hydrodynamic component (to determine the wave-induced currents) and a sediment transport component (to quantify the amount of sediment moved by the wave-induced currents). The hydrodynamic component is based on a standard set of equations that are widely accepted and generally used, more specifically known as the steady-state depth-averaged mass and momentum equations for a fluid of constant density. These equations are standard in many surf zone applications (*e.g.*, Mei, 1983) and provide a state-of-the-art representation of the alongshore current. The sediment transport component is based on a recent peer-reviewed and published formulation by Haas & Hanes (2004), which has been shown to be consistent with recent complex formulae for wave-driven sediment transport and with the Coastal Engineering Research Center (CERC) formula for the total (laterally-integrated) alongshore sediment flux in the limit of a long straight beach subject to waves that are uniform alongshore. For a much more detailed explanation of the model the reader is referred to Section 12.3 of Appendix C.

4.2.3 Regional Sediment Transport

The regional wave modeling results (Section 5.2) were used as input into the non-linear sediment transport model. Wave results from each of the average annual directional spectra bin simulations were used to develop the complete summary of sediment movement for various wave conditions. Sediment

transport results were also combined to define the average annual sediment transport regime throughout the Saco Bay region.

Sediment flux represents the potential rate of sediment moving along the coast, where negative values indicate movement towards the north (from bottom to top of the figure) and positive values indicate movement towards the south (from top to bottom of the figure). This rate is presented in units of m^3/yr and can be used to quantify the annual sediment transport in reaches within Saco Bay. Subsequently the flux divergence is calculated, and indicates areas of erosion and/or deposition. A flux divergence represents erosion, while a flux convergence represents accretion. These calculations all assume that sediment is available for transport on the beach. If the shoreline is armored, or doesn't have a sediment source readily available, then the sediment transport rates are meaningless. Therefore the rates are likely conservatively high as they assume an infinite supply of sediment, and do not account for morphologic changes to the shoreline.

Figure 4-13 presents the average yearly sediment flux and flux divergence. The sediment flux indicates an average annual longshore transport rate to the north. However, the magnitude of the transport varies throughout the domain. A region extending from just north of the navigational structures to approximately 3 km to the north averages approximately 32,700 to 65,000 yds^3/yr (25,000 to 50,000 m^3/yr) towards the north. Increases in flux can be seen directly adjacent to the northern jetty, and landward of Eagle Island at the 3 km alongshore location. The flux divergence in this area would indicate a general trend towards shoreline erosion over this reach. In the center of the bay, extending approximately 3 km, the average annual sediment flux rate is small. This is a region that gross transport direction shifts depending on the angle of the incoming wave field, and generally are equivalent. There is a small net northward transport rate of 13,000 to 27,000 yds^3/yr (10,000 to 20,000 m^3/yr) and the flux divergence indicates a stable stretch of coastline. This reach of shoreline is generally not prone to erosion or accretion, and remains relatively stable under average annual conditions. This was also indicated in the historical shoreline change analysis (Section 4.1). The northernmost region of the bay is strongly influenced by the Bluff and Stratton Island complex and there are major fluctuations in both the sediment flux and divergence. The average sediment transport rate in this region is 52,000 yds^3/yr (40,000 m^3/yr) towards the north, with peaks of 92,000 yds^3/yr (70,000 m^3/yr) towards the north. However, these regions of sediment flux (and subsequently, accretion and erosion) likely change daily based on the incoming wave climate, and some of this fluctuation may be minimized (i.e., these smaller fluctuations might not be real). Gross sediment transport rates vary significantly for the various average annual approach directions, and reach maximums of 392,000 to 450,000 yds^3/yr (300,000 to 350,000 m^3/yr).

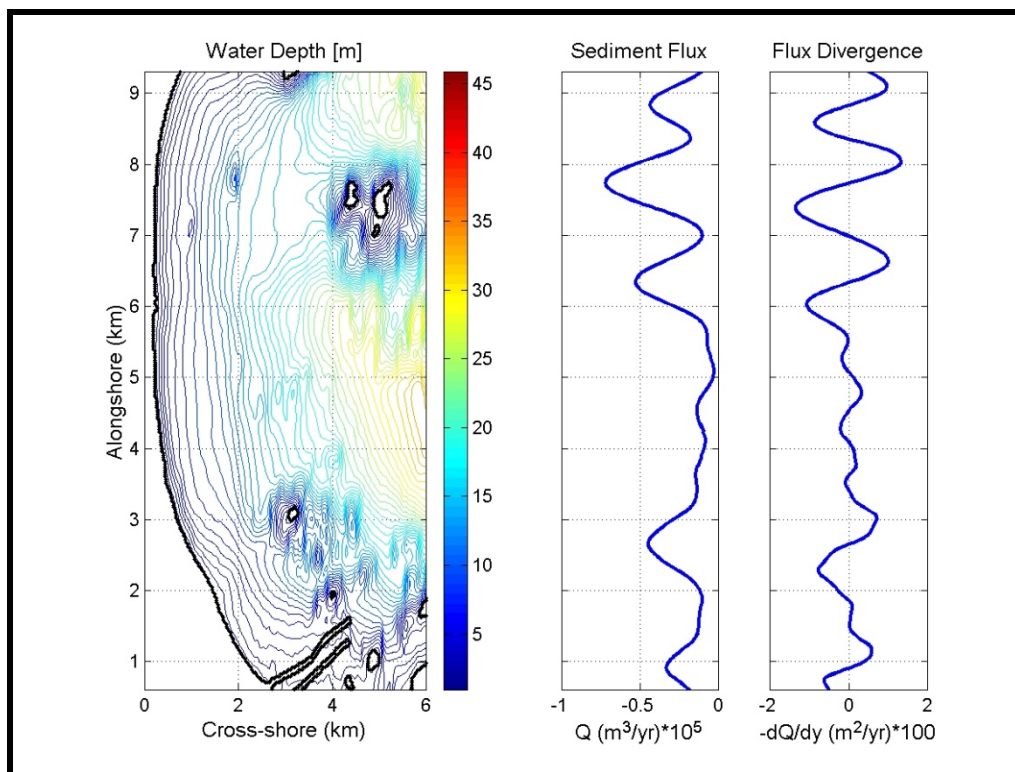


Figure 4-13. Annualized sediment flux and divergence for Saco Bay.

4.2.4 Sediment Budget – Sand Source

As mentioned in Section 2, the purpose of the Section 111 project is to determine if there are negative impacts from the Federal Navigation Project at Camp Ellis in Saco Maine, to determine the level of impacts from the project, and develop mitigation alternatives for negative impacts. As will later be discussed in Section 5.3, it was determined that the Jetties do cause significant impact to the adjacent shoreline through wave reflection off of the North Jetty and to a lesser degree Mach Stem effects along the North Jetty. In addition to impacts of the jetties on the wave climate, a concern of the stake holders, and to USACE, was the impact upon sediment supply to the Saco Bay littoral cell. Given the very long jetties, and the depth at which the jetties terminate, 20+ feet, it is unlikely any significant volume of sand is making it from the Saco River onto the southern Saco Bay beaches or from the beaches into the federal channel. Based on recent maintenance dredge records, 45,000 yds³ to 87,000 yds³ has been dredged every 9 to 14 years from the federal channel so it is apparent the project is capturing sand that would have made it to the beaches without a project. In all likelihood the only sand making it from the Saco River onto the Saco Bay beaches is dredged sand from federal navigation channel maintenance efforts. The volume that has been placed on the beach in the study area has varied since maintenance dredged sand has been split with approximately 50% going to Camp Ellis study area and 50% going to the Biddeford area (south of the jetties). Additionally, sand from project improvement efforts has also been placed on the beach as well. For planning consideration, given the maintenance records, it has been estimated that up to 80,000 yds³ from the navigation channel could potentially be placed in the project area every ten years.

4.2.4.1 Sand Budget Report

Knowing the volume of dredged material placed from maintenance activities was helpful to the project, but was not adequate for determining the volume of sand the federal navigation channel and jetties was preventing from reaching the beach. Basically the answer that was needed is how much sand was coming down the Saco River and being prevented from migrating onto the Saco Bay beaches. This was a complicated task when considering the amount of effort needed to determine this value. It is additionally complicated when trying to figure out how much sand is coming down the river now with the several dam projects installed along its length. Fortunately for the project, a sand budget for Saco Bay and the Saco River input was performed and published in Marine Geology International Journal of Marine Geology, Geochemistry and Geophysics in 2004 under the title "Sand Budgets at Geological, Historical and Contemporary Time Scales for a Developed Beach System, Saco Bay, Maine, USA. The authors of the paper and their affiliations have been provided below to demonstrate the knowledge source of the paper and the information:

- Joseph T. Kelley – Department of Earth Sciences, University of Maine
- Donald C. Barber - Bryan Mawr College, Department of Geology
- Daniel F. Belknap – Department of Earth Sciences, University of Maine
- Duncan M. FitzGerald – Boston University, Department of Geology
- Sytze van Heteren - Boston University, Department of Geology
- Stephan M. Dickson - Maine Geological Survey

In the paper, a sediment budget was worked up for Saco Bay and for the contributions from the Saco River. In the report, it was concluded that the Saco River contributed, or should contribute between 13,000 yds³ to 22,000 yds³ of sand per year to the Saco Bay beaches. Most of the sand would normally be transported during the spring snow melt/rain runoff events, or also known as the spring freshet. The paper also discusses the impacts of the jetties and the prevention of sand reaching the southern beaches in Saco Bay. Upon review of the paper, USACE agrees, in general with the papers findings regarding sand volumes from the Saco River and the conclusion that the jetties prevent sand from reaching the southern Saco Bay beaches. This conclusion was generally accepted by USACE before the paper was published.

4.2.4.2 Federal Sand Volume Responsibility

With the Saco River sand volume contribution determined in the previous section, the Corps was able to reach a conclusion on how much sand the federal government was responsible for to mitigate for the negative impacts from the federal navigation project. Basically, the federal government, through the USACE, is responsible to place between 13,000 yds³ to 22,000 yds³ of sand per year. If the dredging activities continue into the future and the 80,000 yds³ per ten years is placed on the beaches (8,000 yds³/yr) then the Corps needs to place an additional 5,000 yds³ to 14,000 yds³ per year to address the sand deficit caused by the federal navigation project.

5.0 WAVE MODELING

5.1 Generation-Scale Wave Modeling

5.1.1 Wave Model Used

The goal of the generation-scale modeling for Camp Ellis Beach was to simulate wave growth, dissipation and propagation in deep-water for use as input into the regional wave transformation modeling (Chapter 9.0). The results from this effort would be used to provide information at the higher resolution near shore transformation model offshore boundary condition during the calibration/validation process. A spectral wave model, WAVAD (Resio, 1990), was used for the generation-scale modeling. The model used input wind fields as the primary generating force for deep-water waves. The model output included wave spectra at equi-spaced points within the area of interest. All wave parameters, such as significant wave height, frequency of the spectral peak, and mean wave direction, were computed within these discrete elements.

5.1.2 Input Winds

The model used wind fields as the primary generating force for deep-water waves. The wind fields were created using the data from the National Aeronautics and Space Administration's QuikSCAT satellite (Figure 5-1).

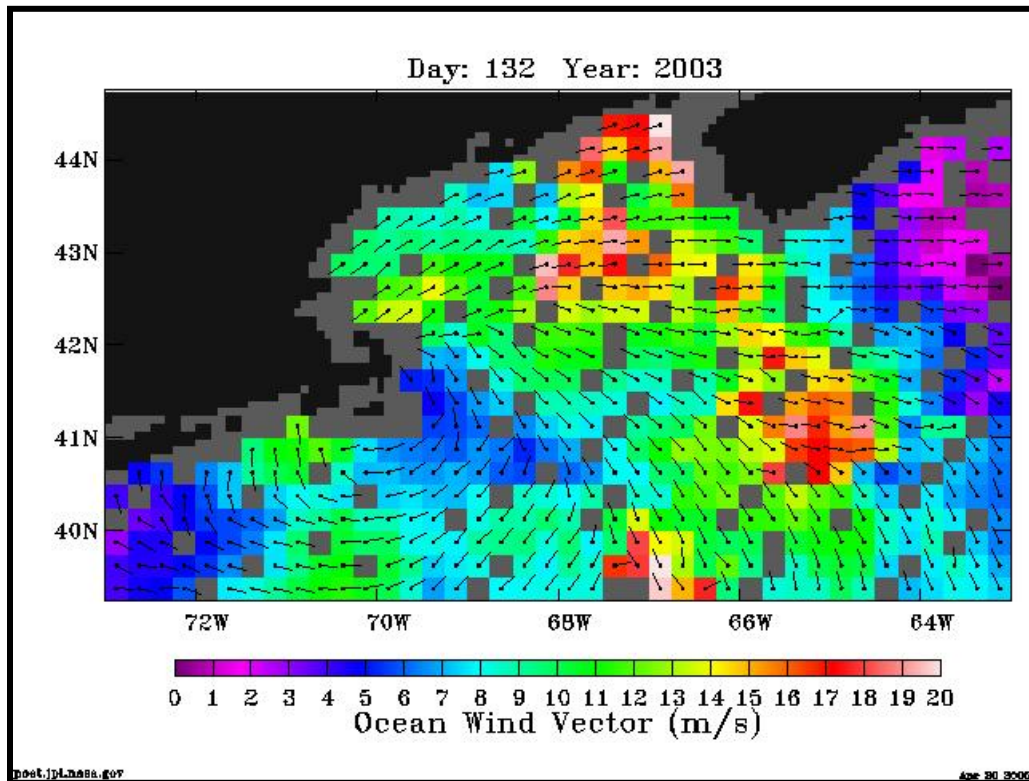


Figure 5-1. Example QuikSCAT wind field.

Wind speeds recorded at the nearshore buoy and the winds recorded at Portland International Jetport, ME were used to fill in the missing wind information in the shallow water regions. Ultimately, the nearshore region was more accurately simulated using the higher resolution nearshore transformation models such that the coarse resolution, wave generation model was not used in the nearshore region.

5.1.3 Bathymetry and Grid Generation

The WAVAD model required specification of bathymetry at each point in the computational grid. A series of two nested grids was applied to simulate the time period spanning the deployment of the two ADCPs. The larger grid has a resolution of 0.25 decimal degrees (17.3 miles), whereas the nested grid has a resolution of 0.05 decimal degrees (3.5 miles). Water depths within the grids were determined from the 30-arc second digital bathymetry constructed by the Coastal and Marine Geology Program of the United States Geological Survey (Figure 5-2).

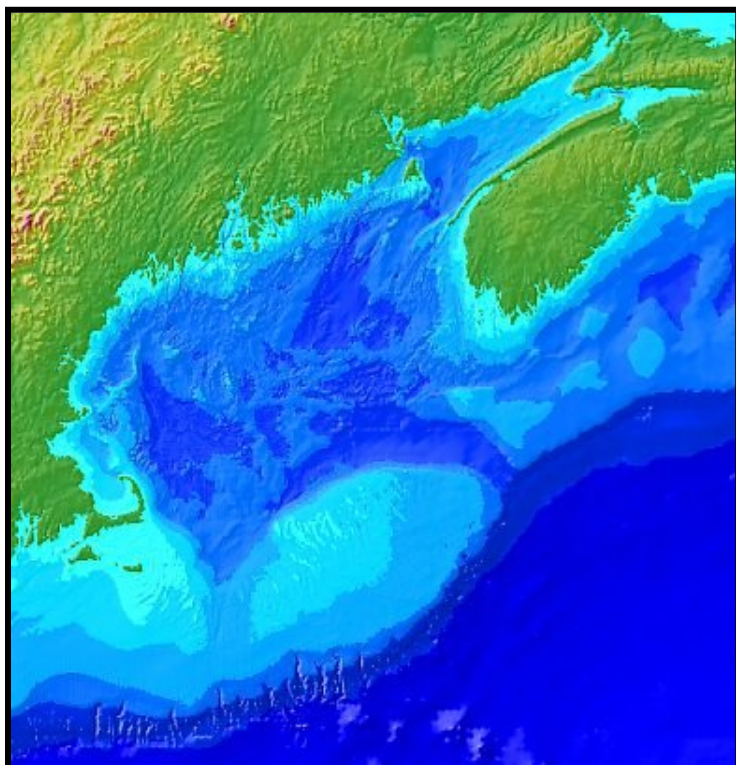


Figure 5-2. Digital bathymetry for the Gulf of Maine (<http://woodshole.er.usgs.gov/project-pages/oracle/gomaine/bathy/>)

The deep-water wave model used a rectangular grid. For simulations requiring finer resolution, the offshore wave model had a nesting capability. This nesting allowed the user to reduce the computational overhead of fine mesh calculations by utilizing a sequence of nested grids, each having a resolution finer than the preceding. An example of a grid has been provided as Figure 5-3.

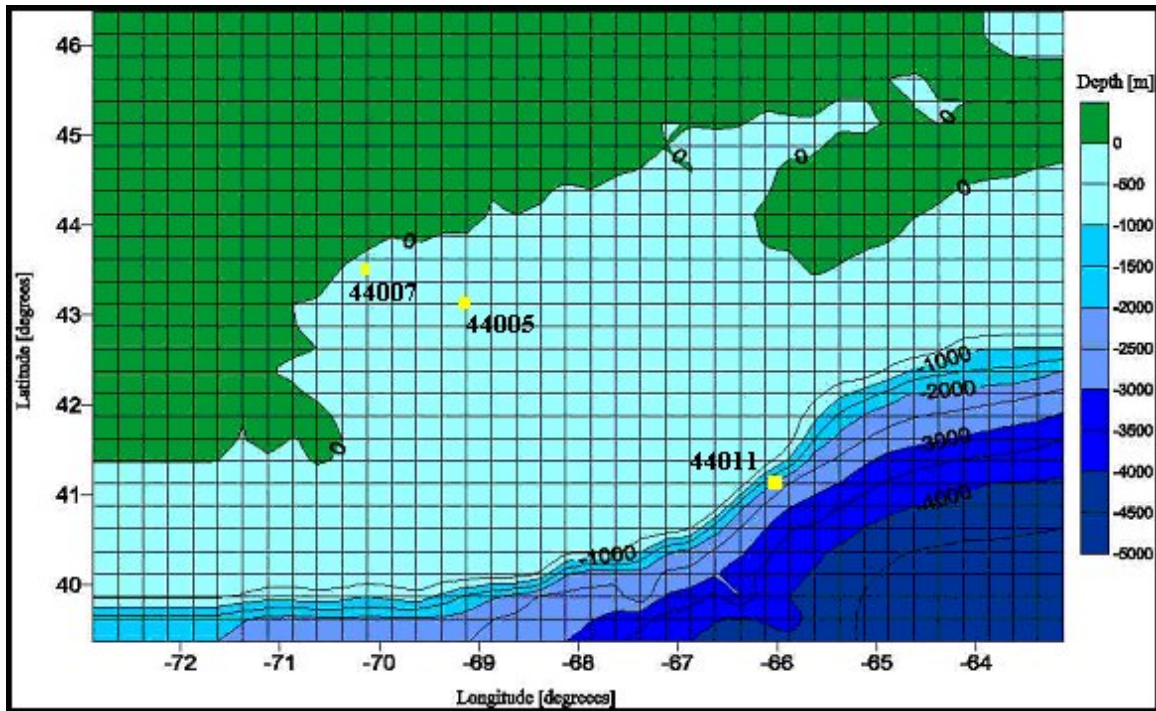


Figure 5-3. Nested WAVAD model grid.

5.1.4 Model Calibration and Results

Calibration of the wave model required an ability to compare observed wave heights with the modeled wave heights during the same time period. The generation scale model was calibrated through comparison to NOAA buoy data (locations shown in Figure 5-3) observed within the model region. The model was used to simulate the entire time period corresponding to the deployment measurements from the ADCPs (March 12, 2003 through May 21, 2003). Figure 5-4 shows a comparison between the modeled and the measured wave height and the modeled and measured wind speed during the deployment time period for the larger grid results and buoy 44011. Both the wave height and wind speed comparisons at Buoy 44011 show reasonable agreement during the entire time period. The overall trends in wave height are well identified, especially considering the input wind fields are only generated at 12-hour time steps. Figure 5-5 presents the calibration results compared to observations at NOAA buoy 44005. Again, the overall trends in wave height are well identified during the entire time period. The generation-scale model does have some difficulty resolving lower wave energies, corresponding to low wind time periods. Figure 5-6 presents the calibration results compared to observations at NOAA buoy 44007. Again, at this most shallow observation station, the overall trends in wave height are well identified during the entire time period. At this site, as at buoy 44005, the generation-scale model has difficulty resolving lower wave energies, corresponding to low wind time periods. Table 5-1 presents a summary of the error statistics (bias and RMS error) for the generation-scale wave modeling simulations based on the NOAA buoys. Although the errors appear larger than may be acceptable at the nearshore buoy locations, higher-resolution regional wave modeling (Section 5.2) was used for the near shore area to better resolve the wave conditions.

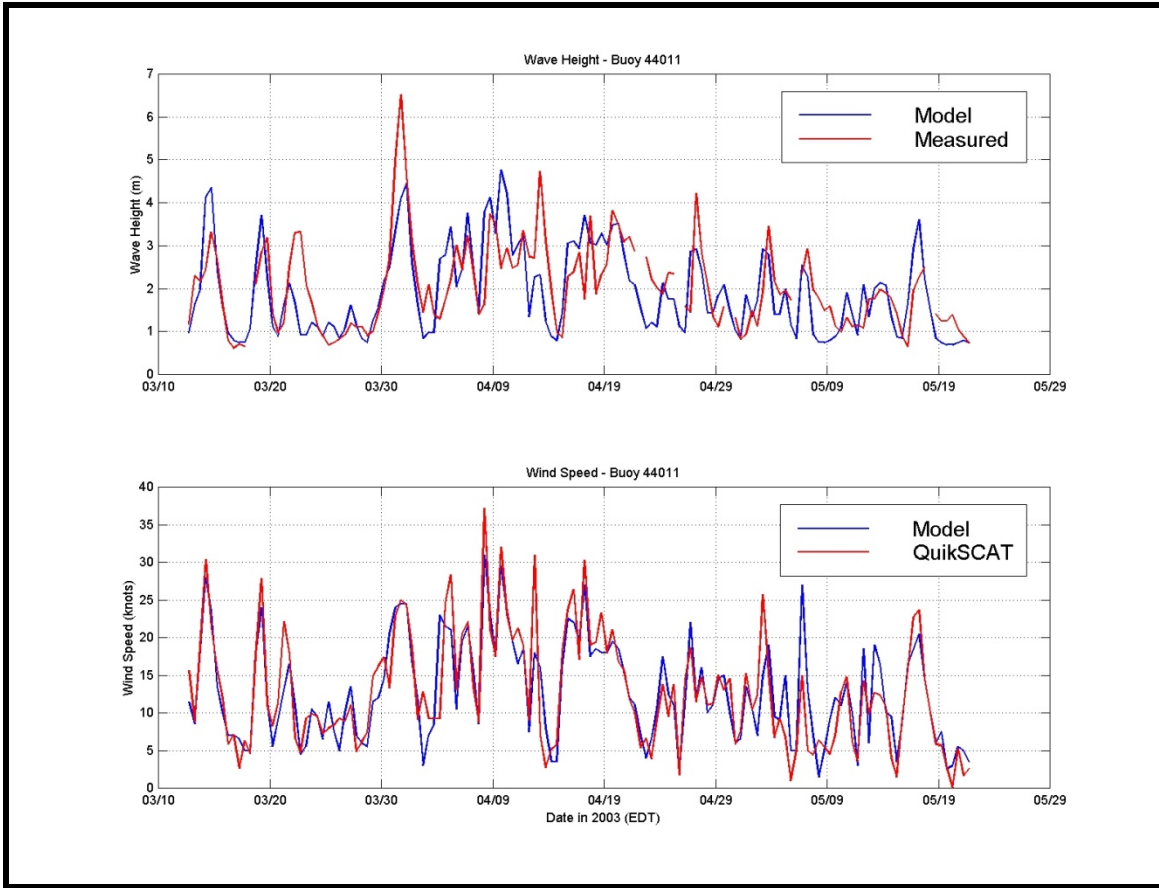


Figure 5-4. Comparison of modeled and measured wave height and wind speed at NOAA Buoy 44011.

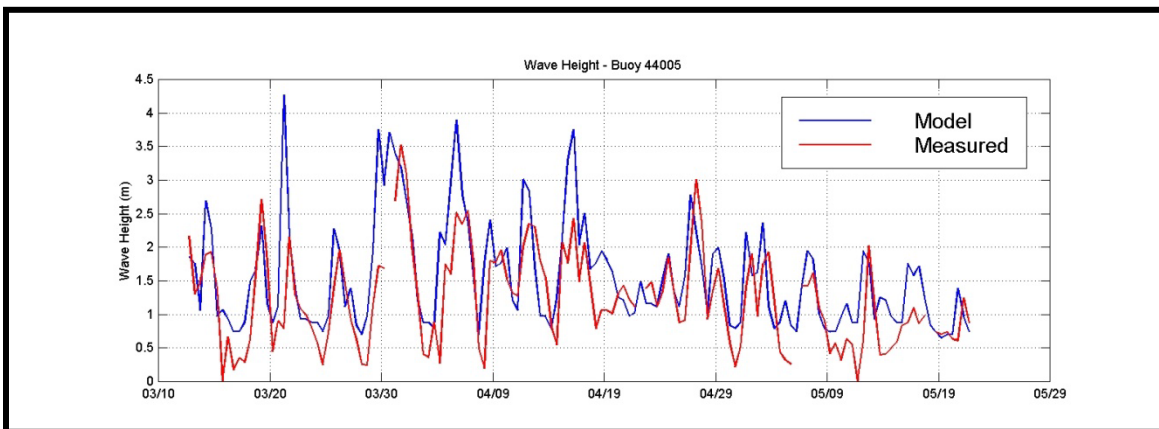


Figure 5-5. Comparison of modeled and measured wave height at NOAA Buoy 44005.

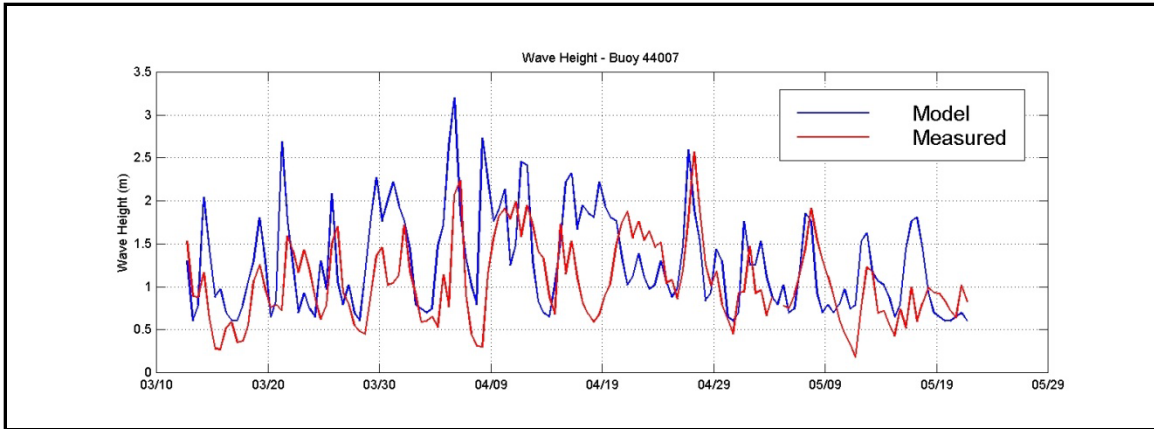


Figure 5-6. Comparison of modeled and measured wave height at NOAA Buoy 44007.

Table 5-1. WAVAD wave height model errors based on NOAA Buoys.

| NOAA Buoy | Bias (m) | RMS Error (m) |
|-----------|----------|---------------|
| 44011 | 0.09 | 0.84 |
| 44005 | -0.32 | 0.68 |
| 44007 | -0.24 | 0.62 |

5.1.5 Significant Wave Events

In addition to the overall comparison of waves during the entire deployment time period, two distinct higher energy wave events were evaluated to determine the efficacy of WAVAD in determining wave height and spectra evolution during potential storm events. Figure 5-7 presents the details on the passage of one event. The top two panels present the modeled and measured wave height record at NOAA Buoy 44005 and Buoy 44007 (the buoys located closer to shore) during the specific time period of the event passage. Both of the modeled buoy locations indicate reasonable and acceptable agreement with the maximum wave height as recorded at the respective NOAA buoys. However, there is a slight difference in the time at which the storm peak occurs. The difference in the peak of the wave event (or the time offset) that occurs between the modeled and measured wave data is likely due to the inability to resolve the input wind conditions at temporal resolution greater than 12 hours and the time offset associated with the satellite’s ascending and descending passes. However, based on the shape of the modeled and measured events, the model does represent wave growth and decay at the correct rates and correctly predicting the energy was important. The time series showing the passage of the event indicates that an energetic event is capable of pushing energy into lower frequencies as the event intensity increases. This movement of energy, from higher to lower frequencies, is a good indication that the model is representing spectral development and wave height increases (growth) correctly.

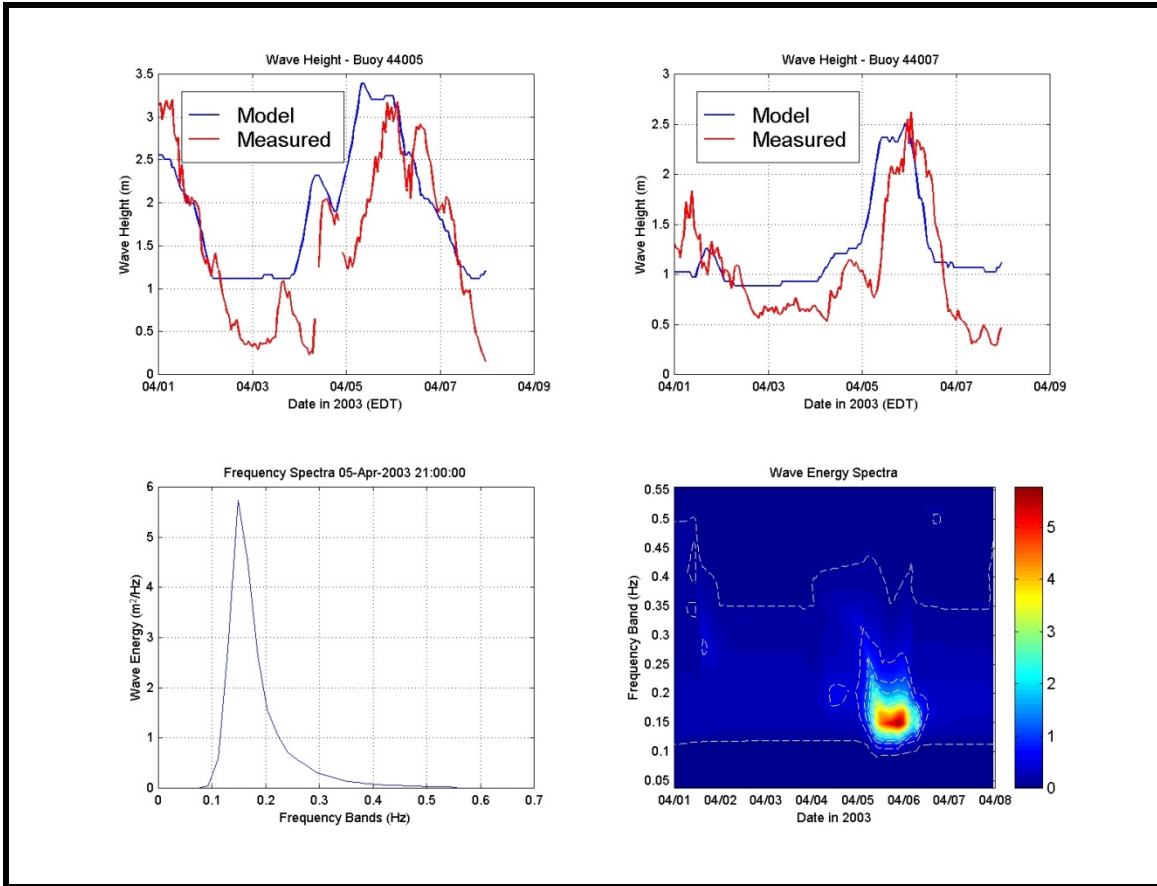


Figure 5-7. Evaluation of a higher energy event spanning April 01, 2003 – April 08, 2003, comparing modeled and measured wave heights and wave energy spectra. Upper left panel shows panel model versus buoy 44005; upper right panel shows model versus buoy 44007; lower left panel shows peak modeled wave spectrum; lower right panel show time series wave energy spectra.

5.1.6 Summary

Because of the lack of temporal and spatial similitude between locally observed wave information and available wave data sources, a generation-scale wave model was required to develop input into the detailed, shallow-water transformation-scale (regional) wave model. The generation-scale numerical model used satellite observed wind fields as input and was calibrated and verified using local point wave observations. The calibration between the measured and modeled wave heights was visually successful and quantifiable error statistics were within acceptable bounds for this scale of modeling. In addition, assessment of specific high-energy storms during the deployment time period indicated energy transfer across frequency bands during the passage of an event. This spectral information was only used as input conditions for validation of the regional-scale modeling effort. Additional wave data sources (e.g., WIS Data) were used to simulate long-term wave impacts on the Camp Ellis shoreline.

5.2 Regional Wave Modeling

The offshore wave climate during the time of the deployment was determined in Section 5.1. However, in order to evaluate local sediment transport pathways, as well as to assess impacts and identify potential alternatives to mitigate the erosion at Camp Ellis Beach, an understanding of the regional wave climate is required using a more refined wave model. This section evaluates the transformations waves experience as they propagate towards the coastline. A spectral wave model was used to propagate random waves from offshore to the nearshore region and investigate potential changes to the wave field caused by the bathymetry. So while the generation-scale wave model was used to develop the offshore wave climate for the time period when nearshore wave conditions were observed, the transformation-scale (regional) wave model was used to propagate and transform waves into the Saco Bay region for the wave observation time period and longer-term average conditions and storm events. Subsequently, results from the transformation-scale (regional) model are used to drive the nearshore models (Section 5.3), which is used to directly evaluate the nature of the waves specifically in the Camp Ellis region.

5.2.1 Wave Model Description

The spectral wave model STWAVE version 3.0 (Smith, Sherlock, and Resio, 2001), developed by the U.S. Army Corps of Engineers Waterways Experiment Station, was employed to evaluate changes in wave propagation across the nearshore region fronting Saco Bay beaches. STWAVE is a steady state, spectral wave transformation model, based on a form of the wave action balance equation of Jonsson (1990). STWAVE simulates the behavior of a random sea surface by describing wave energy density as a function of direction (directional spectrum) and frequency (frequency spectrum). The two-dimensional wave spectrum is discretized into separate wave components, which constitute an essential part of the input for STWAVE. Through a combination of the various wave directions and frequencies, STWAVE is able to simulate the behavior of a natural, random sea. Using the generation-scale wave modeling results for validation and existing wave data offshore of Saco Bay for developing appropriate offshore wave conditions, input data was generated to specify the wave boundary conditions. Then, using local bathymetry to create an accurate grid, the model was able to propagate waves within the Saco Bay region.

5.2.2 Bathymetry and Grid Generation

The transformation-scale (regional) modeling used digital bathymetry from the National Ocean Service (NOS), combined with 1-m LIDAR data, and a high-resolution nearshore bathymetric survey conducted near the mouth of the Saco River on May 13 and 15, 2003 (Section 3.1). Existing National Oceanographic and Atmospheric Administration (NOAA) data were obtained from the National Ocean Service (NOS) Office of Coast Survey Hydrographic Survey Geophysical Data System (GEODAS). These bathymetric surveys were combined to define the region offshore of Saco Bay. The compilation of these surveys was used to provide data for grid creation in the offshore regions (Figure 5-8).

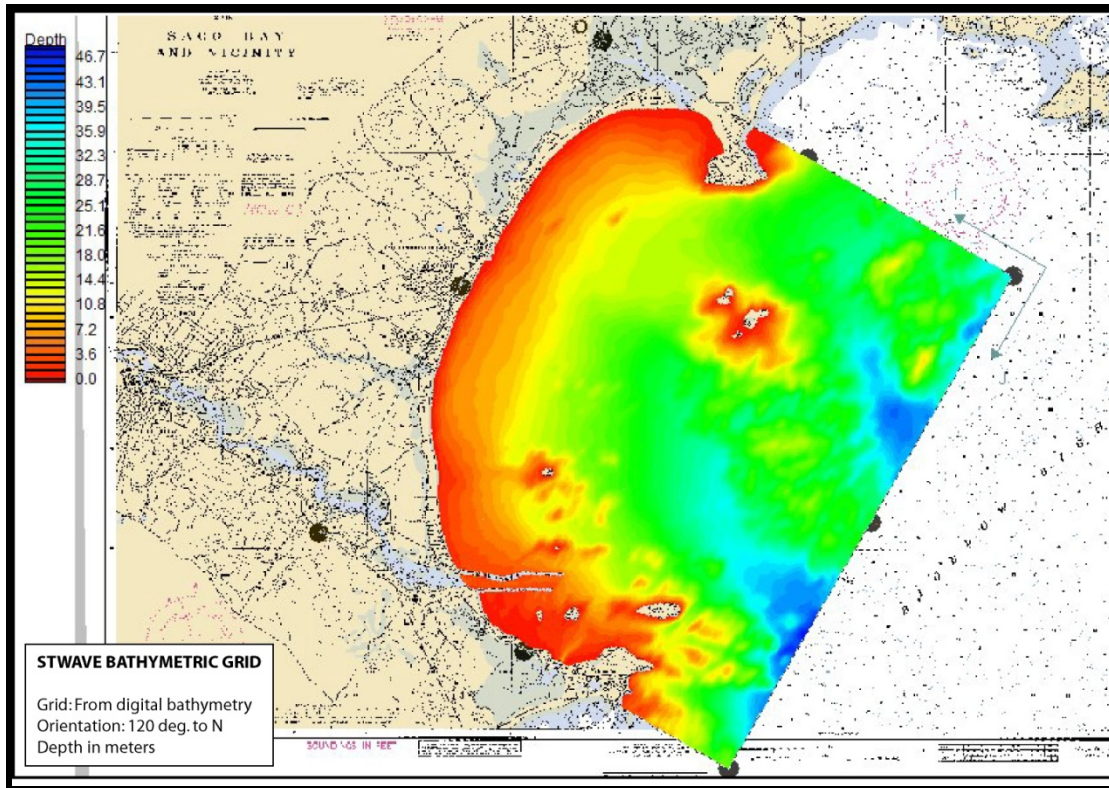


Figure 5-8. Bathymetric grid used for the STWAVE modeling (meters MTL).

5.2.3 Wave Characteristics and Input Spectra

Transformation wave modeling can only be as accurate as the input data; therefore, a key component of accurate wave modeling is the analysis and selection of input wave data. Figure 5-9 presents the location of the existing wave data sources in the vicinity of Saco Bay. These data sources include: U.S. Army Corps of Engineers (USACE) Wave Information Study (WIS) time series of wave data and wind data, National Oceanographic and Atmospheric Administration (NOAA) wave buoys and Gulf of Maine Ocean Observing System (GoMOOS). Both these data sources (NOAA and GoMOOS) consist of non-directional wave information. Due to the directional limitations of the existing buoy information, the Wave Information Study (WIS) time series of wave data and wind data was used to describe the wave climate offshore the Saco Bay region. WIS, performed by the U.S. Army Corps of Engineers (USACE), has met a critical need for wave information in coastal engineering studies since the 1980s and is widely accepted for design purposes for United States shorelines by many coastal engineers and scientists. Whereas the generation-scale wave modeling presented in Chapter Section 5.1 provides the wave climate during the time of the instrument deployment, the WIS data sets offers a long-term synopsis of the wave climate offshore of Saco Bay.

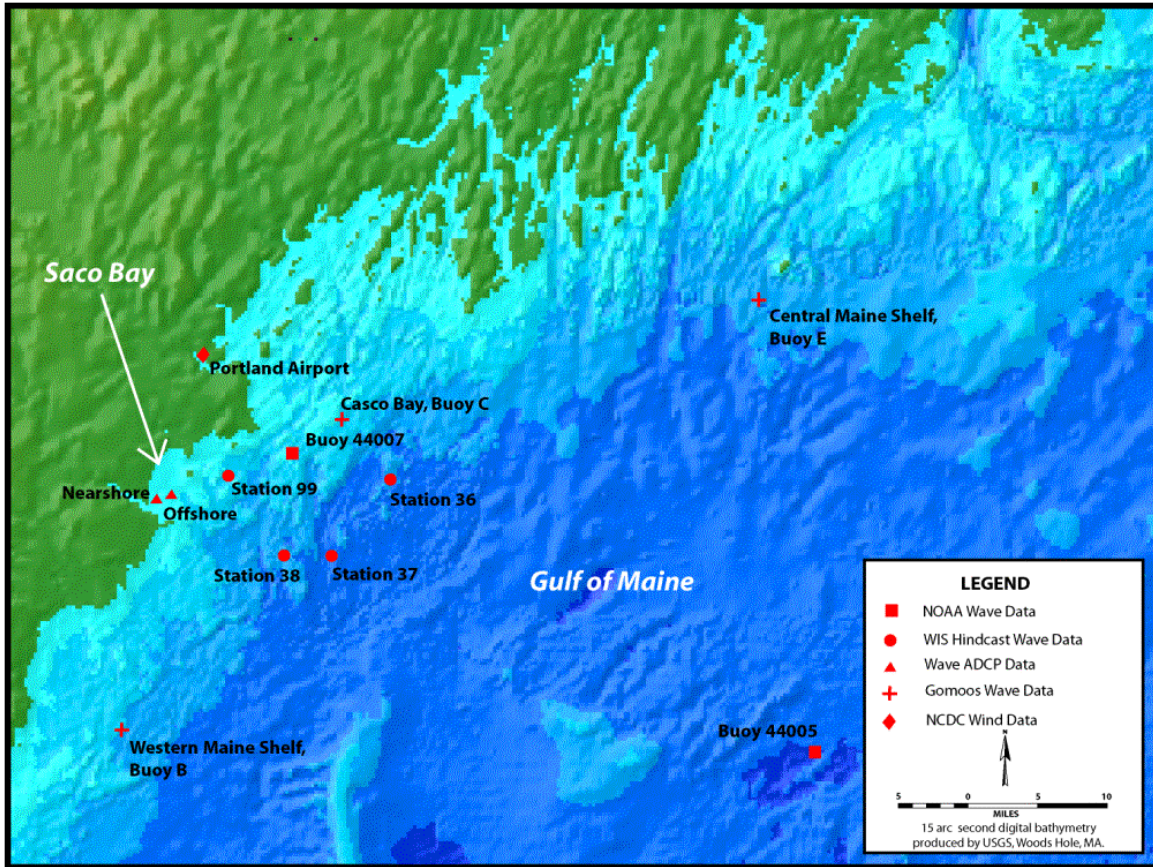


Figure 5-9. Location of existing wave observations and hindcast positions within the Gulf of Maine. The two triangle markers in the nearshore region represent the wave data collected for this study.

Figures 5-10 presents the distribution of significant wave height (illustrated using a wave rose plot) for WIS station Au2099 (99). The figure for station 99 is similar to the figure for station 36 so it was not shown. The primary clustering of wave directions tends to be propagating towards the shoreline from the east-southeast, with a less frequent, but larger energy component arriving from the northeast and south-southwest, likely due to northeasters and hurricanes, respectively. Therefore, the longshore sediment transport clearly cannot be defined by one single wave value, but rather is a compilation of a wide variety of waves that drive sediment movement both in the northward and southward directions along the shoreline in Saco Bay. WIS station 99 is a preferred choice for development of wave input conditions due to the proximity to the shoreline. Since the two WIS stations illustrated limited differences, Station 99, positioned near the offshore boundary of the model domain was used to develop annualized conditions.

As a reminder, the results of the wave transformation-scale (regional) modeling will explore the changes that occur to the wave distribution as they propagate towards the coast, and specifically in the vicinity of Camp Ellis Beach. The transformation-scale (regional) wave modeling evaluates the propagation of these wave fields into the Saco Bay region.

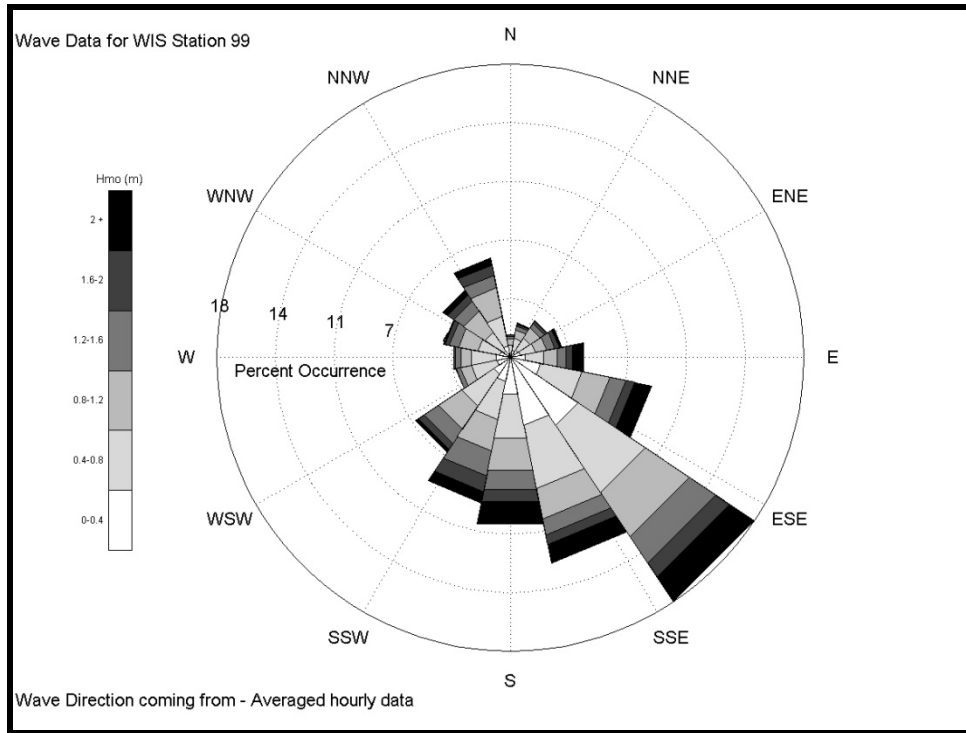


Figure 5-10. Twenty-year averaged wave rose for WIS Station 99.

5.2.4 Regional (Transformation Scale) Model Validation

Prior to using the model to transform long-term wave climate information into the Saco Bay region, the transformation-scale (regional) model was validated to ensure adequate performance of the transformation-scale (regional) model. Two-dimensional spectral output from WAVAD (generation-scale) (Section 5.1) was used directly as input into STWAVE for validation purposes. STWAVE was used to simulate the entire deployment (March-May 2003) time period (during which the waves were observed). Wave model results were compared to the wave measurements from the two nearshore ADCP systems to verify the performance of the model. Figures 5-11 shows a comparison of the modeled (green) and measured (black) wave heights at the nearshore ADCP site. The results for the offshore gage were not shown since they were similar. Figure 5-12 shows a comparison of the modeled (green) and measured (black) wave directions for the nearshore ADCP station. In general, the model predicts the correct directional approach; however, it is unable to rectify sharp changes in wave direction (events when the direction is directed offshore). However, periods when wave directions are heading onshore, the model does a reasonable job of predicting the wave direction. In addition, these differences (both in height and direction) may also develop since STWAVE cannot simulate some of the transformations (diffraction, reflection, etc.) that become important in the presence of coastal structures and nearshore islands. The time shifts and errors introduced in the generation-scale modeling may also transfer into differences in the transformation-scale (regional) wave model. Finally, differences during low-energy (small wave height) time periods add to the overall error, but are not important since when the wave energy is low, the model's performance is not as critical. These differences illustrate the need for the finer scale, nearshore, local wave modeling effort discussed in Section 5.3

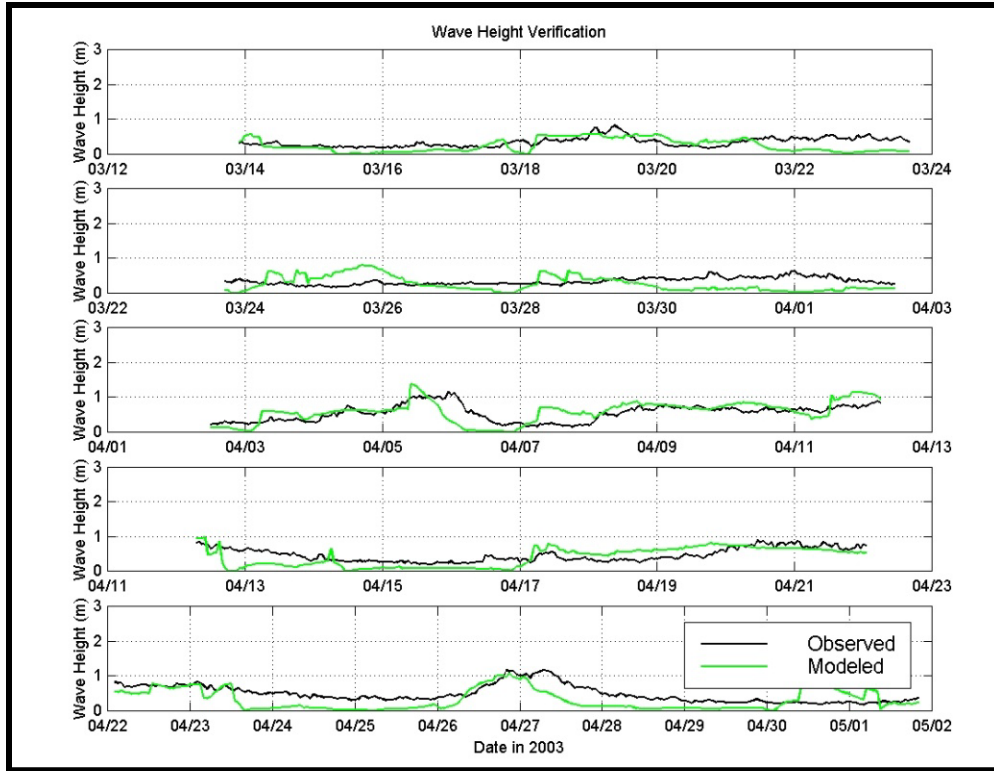


Figure 5-11. Comparison of observed and modeled wave height at the Nearshore ADCP station.

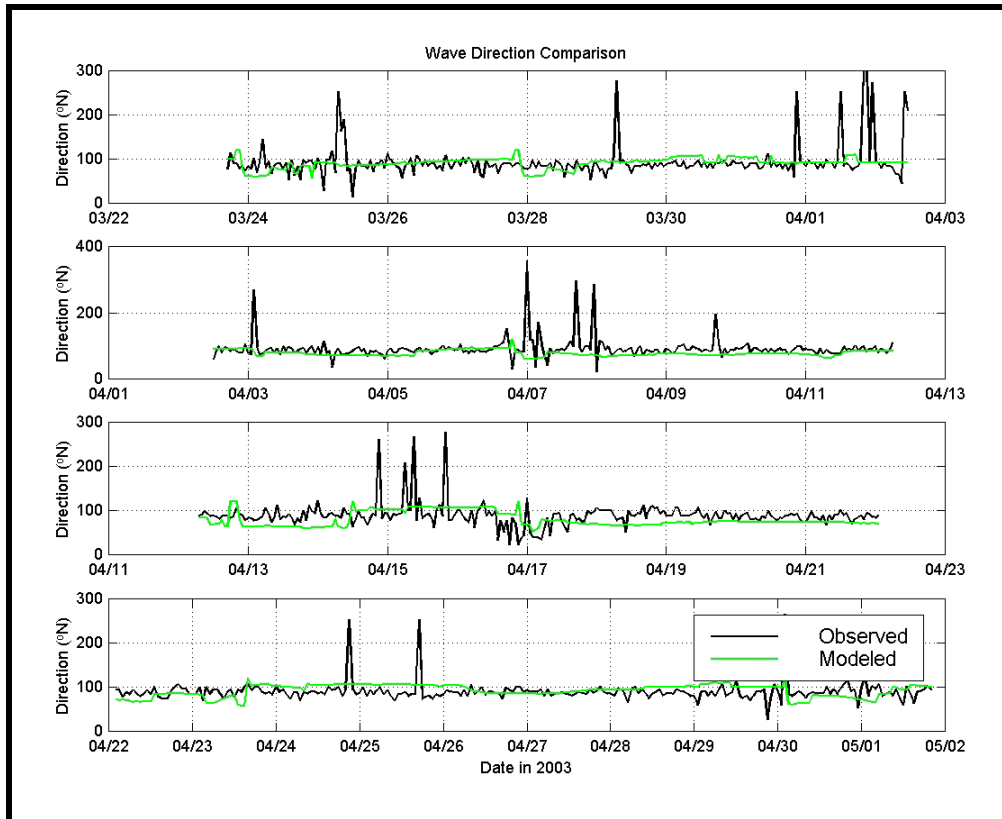


Figure 5-12. Comparison of observed and modeled wave direction at the Nearshore ADCP station.

Figure 5-13 shows the visual results of a spectral comparison for April 27, 2003 at 0400 at the offshore ADCP station. The observed data contains more energy at the peak frequency however the total energy matches well. Differences in energy between model and observed spectra were 5-15%.

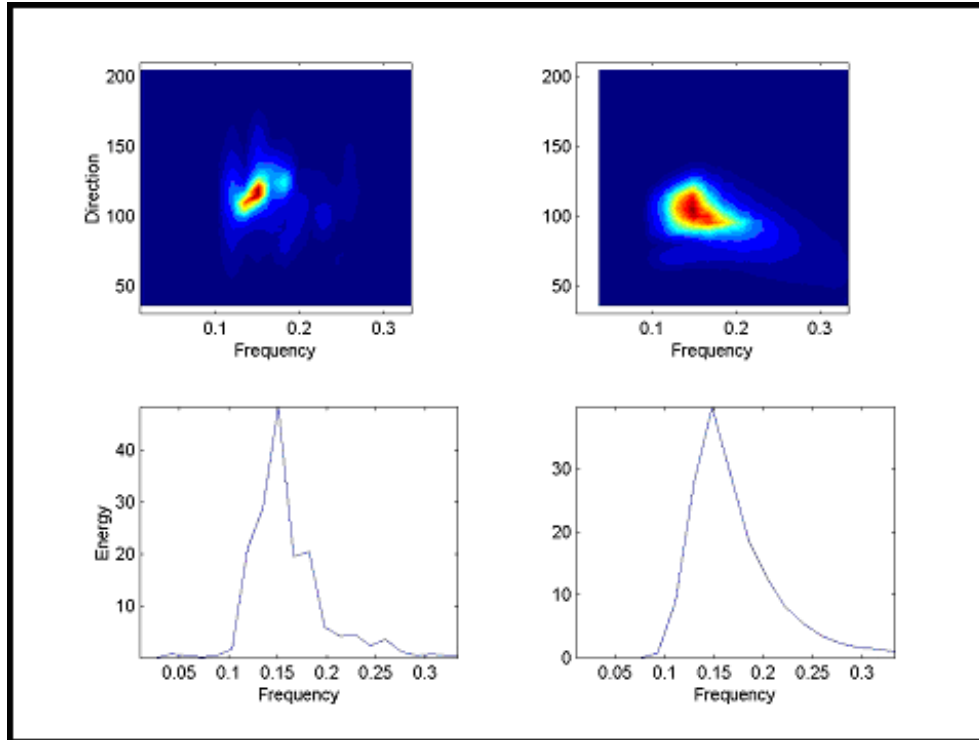


Figure 5-13. Comparison of observed and modeled two-dimensional spectra. Offshore ADCP station spectra are presented in the left panels, while STWAVE output of modeled spectra are presented in the right panels.

5.2.5 Average Annual Directional Approaches

In order to determine long-term wave conditions and for use in sediment transport calculations, spectral data from WIS station 99 were used to derive energy-conserving annual average directional spectrum. Data are segregated by direction of approach, and an energy distribution, as a function of frequency, is generated from all the waves in each directional bin. The energy associated with each frequency is then summed to create an energy distribution for each approach direction.

Figure 5-14 illustrates STWAVE results for waves approaching from the east-southeast (110 to 130 degree bin), one of the most commonly occurring of the typical condition cases, for the entire modeling domain. The model simulation was conducted at depths and shoreline positions corresponding to mean water level since the representative average annual cases represent static time periods. In general, the wave height decreases as waves move onshore due to the effect of bottom friction. The offshore islands also significantly influence the waves as they propagate towards the shoreline. Refraction and sheltering processes are clearly evident in the wave height results. There is also a fair amount of wave energy variation along the shoreline. The wave energy variation both result in alongshore variability in

the direction and magnitude of sediment transport along the shoreline. On a regional scale, the jetties at the Saco River tend to have a smaller influence than the offshore islands and it is difficult to determine the details of the wave transformations in the vicinity of Camp Ellis Beach. The structures become more important on the local scale (Section 5.3), as do the smaller nearshore islands (e.g., Eagle and Ram Islands). Although the offshore approach direction is east-southeast, the nearshore approach direction in the vicinity of Camp Ellis Beach is from the northeast, a similar trend as seen in the wave observations and for modeled wave directions even further from the south.

In order to arrive at an accurate estimation of the sediment transport in the region, results from the wave model can be used to generate the sediment transport flux. This would include waves coming from all directions and having various wave heights and periods. The combination of all the directional approach cases allows for an assessment of the average annual wave climate. As such, the STWAVE results were used to generate wave-induced currents (from radiation stresses) and regional sediment transport results for the entire Saco Bay Region (from headland to headland), as well as provide input into the nearshore (local) wave model. The results of all the approach directions are used, in concert with the percent occurrence, to compute the annual sediment transport in the region (Section 4.2).

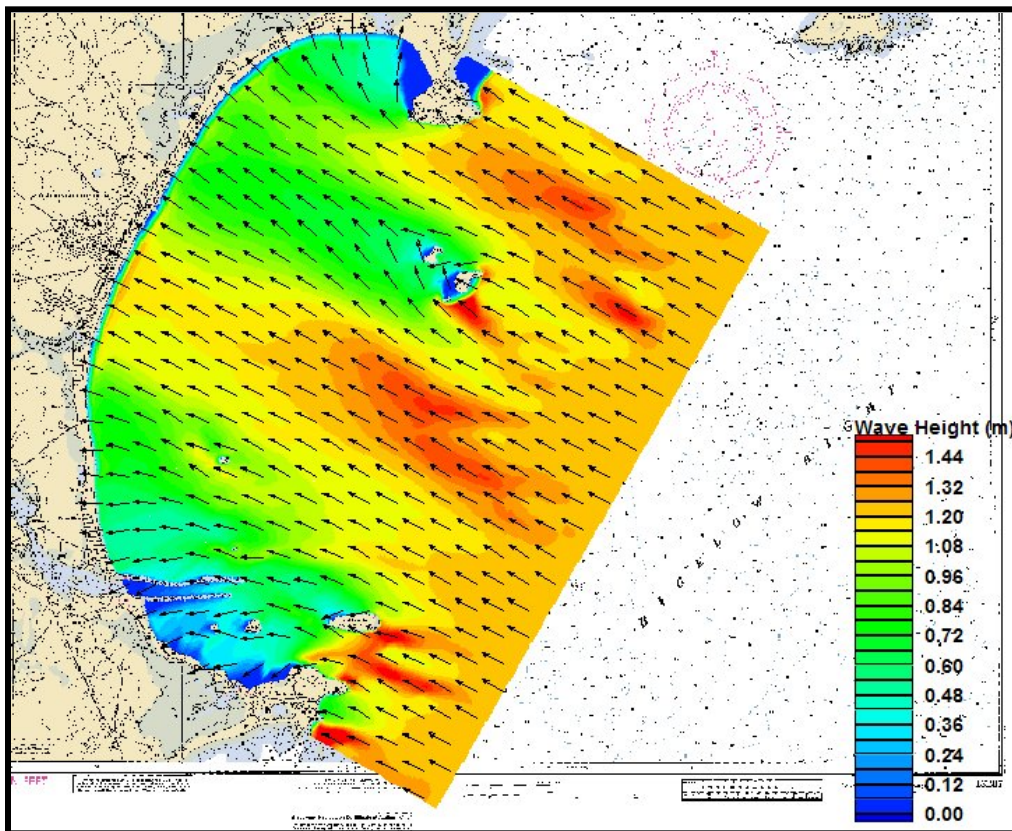


Figure 5-14. Example of STWAVE modeling results for existing conditions using an east-southeast (110 to 130 degree) approach directional spectra bin. The arrows indicate the wave direction.

5.2.6 High Energy Events

Since high-energy events have a significant impact on many physical processes (and in most cases, dominate sediment transport), it is crucial to include storm simulations in wave modeling to assess the potential impact of a storm on the shoreline and the potential sediment transport within Saco Bay. Two distinct types of storms, northeasters and hurricanes, affect the study area. Despite their infrequent occurrence, hurricanes have the potential to produce devastating impacts along the coastline. To represent historical storm conditions in Saco Bay, one historical hurricane event and two historical northeaster storm events were modeled. Figure 5-15 presents the result from the simulation of the Perfect Storm (10/31/1991). Wave heights are significantly higher than during the annual average directional cases, as the offshore heights are in excess of 7.5 m in locations.

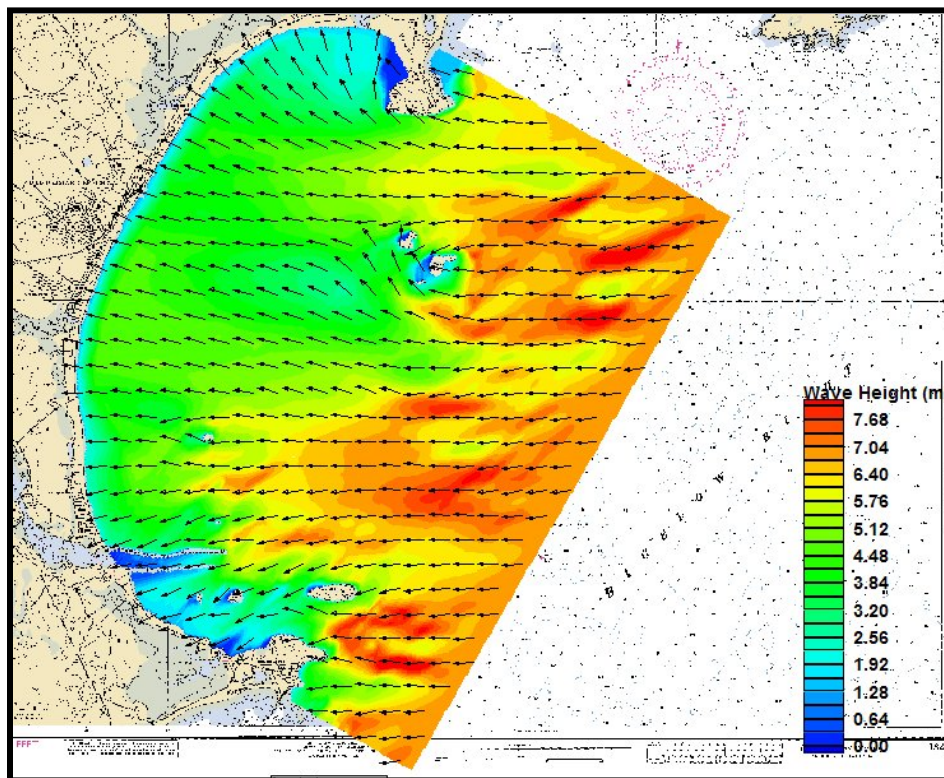


Figure 5-15. STWAVE modeling results for the Perfect Storm (10/31/1991).

In addition, return-period storm events (10-year, 50-year and 100-year) were developed and simulated in STWAVE to provide varying levels of storm events expected to occur at this location. Wave height was used as the metric to define return period storms for the wave modeling. The return-period storm wave height was developed using the Generalized Extreme Value (GEV) method. This method provides reliable estimates of extremes without assuming the distribution type is known (Resio, 1989). For more details refer to Section 9.4.2.3 of Appendix C. Table 5-2 provides the wave parameters for the return period storms defined. As discussed in respective later sections, where water level was critical to the analysis or design, the corresponding return period water level was used as well. It is understood the joint occurrence, or joint probability, of return period parameters such as waves and water levels results in an event that likely has a return period greater than the individual return periods. However, with

nor'easters in New England, typically, if a storm is strong enough to generate a return period water level, the storms intensity is such, that significant wave energy will surely occur as well. The offshore wave height may not be at the same return period level, but the near shore waves are likely to be near maximum due to shoaling and depth limited wave conditions.

Table 5-2. Return period storm wave parameters

| Storm Event | Hs (m) | Tp (sec) | Direction (degrees) | Storm Surge (m above MTL) |
|----------------------------|---------------|-----------------|----------------------------|----------------------------------|
| 10-year | 6.2 (20.3 ft) | 14.4 | 60 | 2.4 (7.9 ft) |
| 50-year | 7.1 (23.3 ft) | 15.4 | 60 | 2.6 (8.5 ft) |
| 100-year | 7.5 (24.6 ft) | 15.9 | 60 | 2.7 (8.9 ft) |
| Perfect Storm (10/31/1991) | 6.9 22.6 ft) | 14.3 | 37 | 2.4 (7.9 ft) |
| Hurricane Bob (08/20/1991) | 5.8 (19.0 ft) | 11.1 | -20 | 1.8 (5.9 ft) |
| Northeast March 6-7,2001 | 5.6 18.4 ft) | 11.1 | 50 | 2.4 2.9 ft) |

5.2.7 Regional Modeling Summary

A nearshore, transformation-scale (regional) wave model was used to propagate the offshore wave climate into the Saco Bay region. The model was verified using spectral output from the generation scale modeling results (Section 5.1). Once validated, the transformation-scale (regional) model was used to simulate average annual directional cases (developed from WIS data), specific historic storms events, and return-period storms. Results of the transformation-scale (regional) model are used to develop regional sediment transport fluxes and divergence, while providing spectral input for the local wave modeling effort.

5.3 Local Wave Modeling

As discussed, the regional, transformation-scale wave model (STWAVE – Section 5.2) is a half-plane model, and therefore, only represents waves propagating towards the coast. Waves that may be reflected from the coastline or structures are not included. In addition, STWAVE does not account for a number of wave transformation processes that are prevalent in the vicinity of Camp Ellis Beach (e.g., diffraction, reflection, etc.). Due to the limitations inherent in STWAVE, it was important to advance to a higher-resolution, phase-resolving model that embodied the reflection processes and could more accurately determine the nearshore structural interactions.

5.3.1 Analysis Approach

The goal of the local, nearshore wave modeling effort for Camp Ellis Beach was to simulate combined wave refraction/diffraction, wave reflection and wave dissipation by friction and breaking as well as including nonlinear amplitude dispersion. Therefore, two-dimensional spectral output from the transformation-scale (regional) model (which transformed the wave spectral energy from deep water to shallow water) was used as input into the nearshore (local) wave model. The nearshore (local) wave model, CGWAVE, is utilized to evaluate the local physical processes, (e.g., wave reflection, wave-induced

currents, wave dispersion, nearshore wave refraction and diffraction, etc.), and subsequently the engineering alternatives.

5.3.2 Grid Generation

Figure 5-16 presents the modeling grid used for the nearshore (local scale) wave modeling consisting of 262,940 nodes and 523,555 elements. The total model domain has dimensions of approximately 3 nautical miles by 3 nautical miles. The mesh is comprised of triangular elements. The nodal spacing within the mesh is dependent upon the wavelength. In the nearshore zone, resolution is approximately 10 m (32.8 ft) or about 8 nodes per wavelength. Figure 5-17 contains a zoomed in view of the mesh around the navigational structures and Ram Island, illustrating the density of the nodes and elements. The bathymetric data sources used in the generation of the model domain nodal depths came from the sources discussed in Section 3.1.

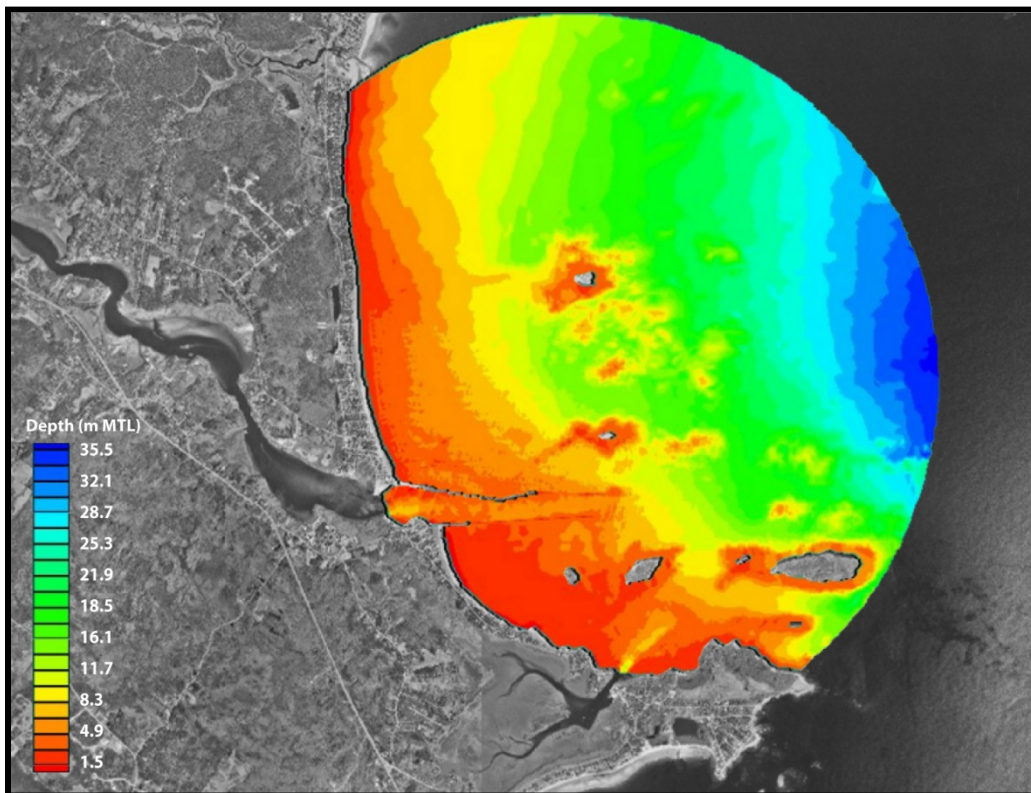


Figure 5-16. CGWAVE model domain for existing conditions (meters-MTL).

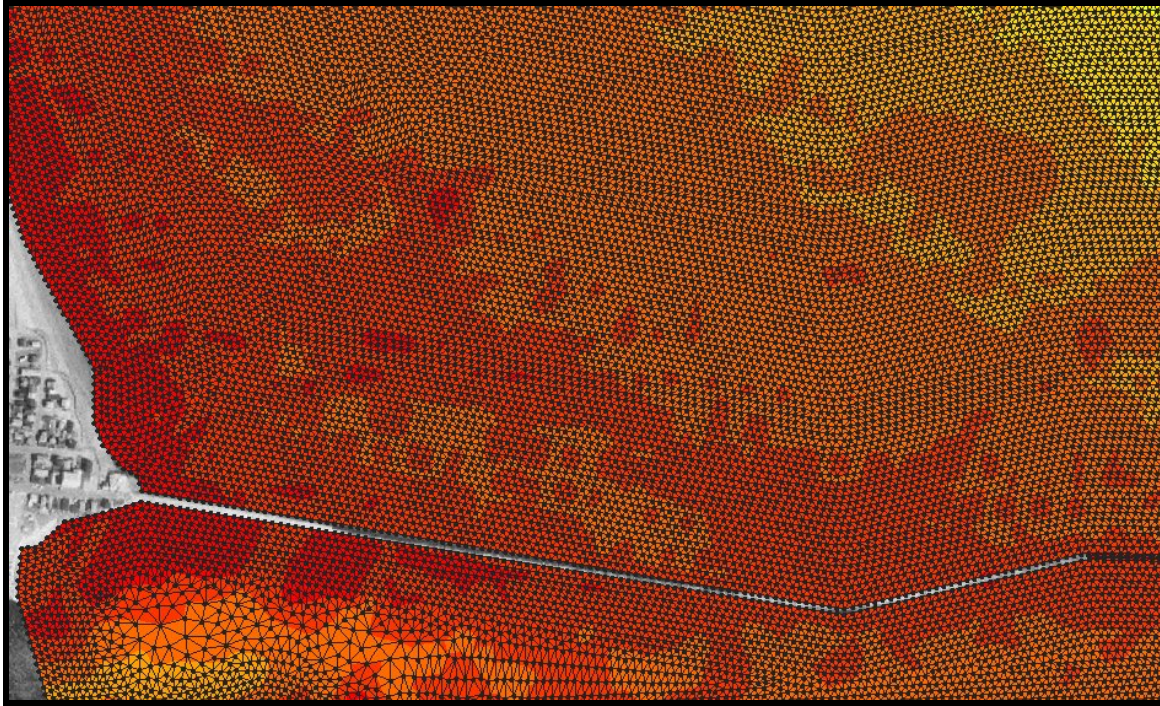


Figure 5-17. Detail of mesh density within the vicinity of the northern jetty.

5.3.3 Wave Input Spectra

CGWAVE does not allow for direct input of a complete wave spectrum; however, spectral input can be simulated through a combination of multiple directional/frequency paired components. Therefore, the two-dimensional wave spectra specified at the offshore boundary was assembled based on the output of the regional model extracted at that same location. As the number of spectral wave components increases, the more resolved the wave spectra, and potentially more accurate the wave results. However, a single direction/frequency pair required approximately 8-12 hours of simulation time, so each wave climate simulation required careful selection of the spectral components.

5.3.4 Model Calibration and Verification

Prior to simulation of the existing conditions and for the various alternatives, the nearshore model was calibrated and verified to the observed wave data collected during March-May 2003 (Section 3.4). Due to the significant computational time restrictions present in the nearshore (local) wave modeling, the entire deployment time period could not be simulated. The selected time periods represented the passage of higher energy wave periods (relative to the deployment period). These time periods included April 4, 2003 at 0700 hours and April 27, 2003 at 0400 hours.

In order to get reasonable agreement between the observed and modeled results, coefficients within the model were adjusted through an iterative process. This consisted of adjustment to the wave breaking coefficient, the frictional coefficients, and reflection coefficients within a reasonable range of bounds, until agreement was achieved between the observed and modeled data. The model was also applied using non-linear interactions, which was a critical component of the overall model calibration

process. Without the non-linear factors, the model was unable to resolve many of the complex wave interaction in the nearshore region of Camp Ellis Beach (e.g., intersection of incident and reflected wave trains).

The wave height and direction at each nodal location were compared to the recorded wave height and wave direction at the corresponding ADCP locations. Table 5-3 contains the comparison between measured and modeled wave height and direction based on the offshore and nearshore ADCP stations for April 4, 2003 at 0700 hours. The percentage error of the modeled values is also presented. The model slightly under predicted the wave heights (approximately 10% error), and performed reasonably well in relationship to the wave direction (between 5-8% error).

Table 5-3. Modeled and measured wave height and wave direction for April 4, 2003 at 0700 hours.

| | Offshore ADCP Location | | Nearshore ADCP Location | |
|----------|------------------------|----------------------------------|-------------------------|----------------------------------|
| | Wave Height [m] | Wave Direction (0° = True North) | Wave Height [m] | Wave Direction (0° = True North) |
| Measured | 1.59 (5.2 ft) | 96 | 0.86 (2.8 ft) | 88 |
| Modeled | 1.41 (4.6 ft) | 88 | 0.77 (2.5 ft) | 84 |
| % Error | -11.3% | 8.3% | -10.4% | 4.5% |

The April 27, 2003 at 0400 hours time period was subsequently used to verify the model's performance using the same coefficients determined during the calibration time period (April 4, 2003 at 0700 hours). Table 5-4 contains the comparison between measured and modeled wave height and direction based on the offshore and nearshore ADCP stations for April 27, 2003 at 0400 hours. The verification of the model indicated slight over prediction of the model wave height (approximately 1% error) at the nearshore location and small errors in wave direction (approximately 2% error).

Table 5-4. Modeled and measured wave height and wave direction for April 27, 2003 at 0400 hours.

| | Offshore ADCP Location | | Nearshore ADCP Location | |
|----------|------------------------|----------------------------------|-------------------------|----------------------------------|
| | Wave Height [m] | Wave Direction (0° = True North) | Wave Height [m] | Wave Direction (0° = True North) |
| Measured | 1.95 (6.4 ft) | 114 | 1.07 (3.5 ft) | 95 |
| Modeled | 1.92 (6.3 ft) | 112 | 1.08 (3.5 ft) | 97 |
| % Error | -1.5% | 1.8% | 1.0% | 2.1% |

5.3.5 Existing Conditions Simulations

The existing conditions simulations were performed on the same grid as that used in the calibration and verification procedures, using the same coefficients and parameters. Both average annual directional spectra and high energy events were simulated. The methodology established during the calibration and verification procedure for developing input spectra was followed for both simulation sets.

5.3.5.1 Average Annual Directional Approaches

The same average annual directional approaches that were simulated in the regional, transformation-scale wave model were also simulated in the nearshore (local) wave model. All of the directional approach simulations were performed on both a grid referenced to MHW and a grid referenced to MLW for sediment transport purposes. The following wave discussion focuses on the MHW simulations, which represent the more energetic cases.

Figure 5-18 shows example sea surface results from the nearshore (local) wave model for a southeastern approach spectrum. Dark blues represent the wave crests, while whites represent the wave troughs. The nearshore (local) wave model simulations can be used to evaluate the interaction of the waves with the complex nearshore bathymetry, the navigational structures, the islands, and the shoreline. The impact of the nearshore islands, shoals, and structures, as well as diffraction/refraction patterns and the crossing of various wave trains, is clearly evident in the sea surface results. Of particular interest are the wave patterns in the vicinity of Eagle and Ram Islands, which have considerable influence on the propagation of the wave field. The complex bathymetric area between the two islands also results in a significant modification to the offshore wave trains.

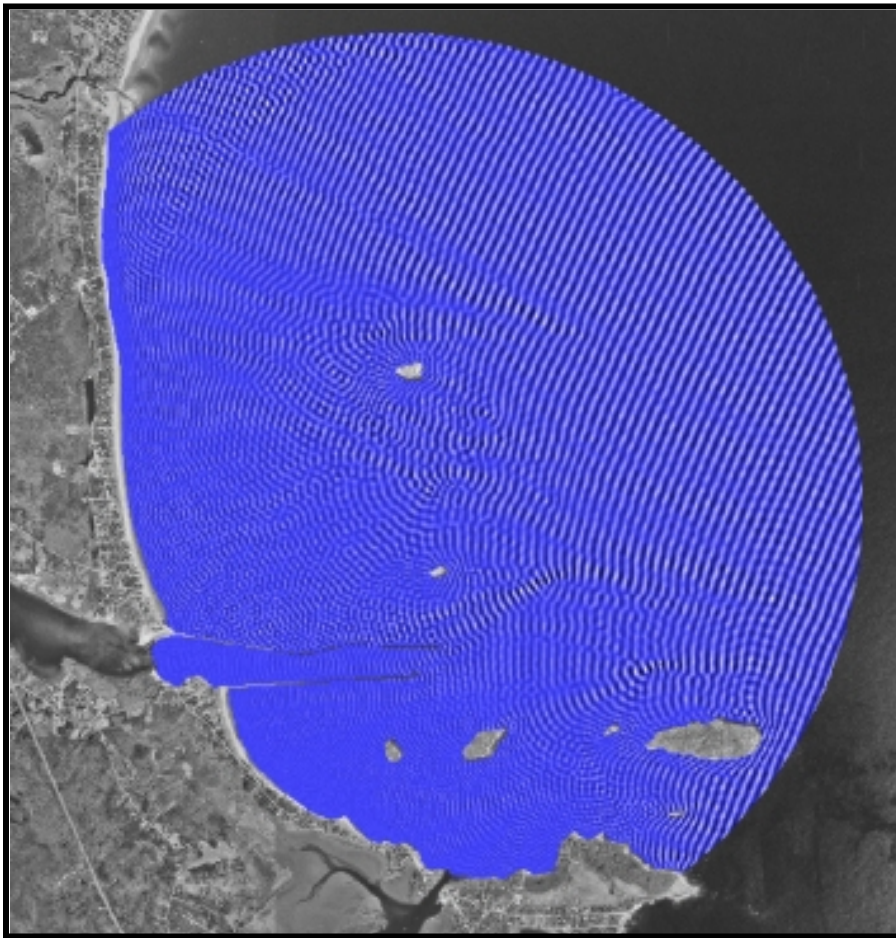


Figure 5-18. Example results of seas surface output from the nearshore (local) wave model (CGWAVE). The simulation is for a southeastern approach spectrum at Mean Tide Level.

Evaluation of the sea surface results for the existing conditions average annual approach directions reveal some important wave transformation aspects:

1. The significant wave reflection off of the northern jetty is identified as the waffle type sea surface north of the structure (Figure 5-19). These waffle-like patterns are formed due to the interaction of the incident and reflected wave trains. For a portion of the shoreline directly adjacent to the northern jetty, the beach is impacted not only by the incident wave energy, but also by the reflected wave energy. As such, approximately 610-914 m (2,000-3,000 ft) of shoreline experiences a significant increase in energy. As discussed in Section 12.5.2 of Appendix C, the reflected wave energy accounts for approximately 40% of the wave energy directly north of the North Jetty and it diminishes down to approximately 20% at the north end of the project.
2. In nearly all cases, independent of offshore direction of approach, the nearshore waves propagated directly towards the Camp Ellis Beach region and the northern jetty. The transformations due to the complex bathymetry between the islands, and the islands themselves, resulted in a nearly uniform approach towards the region of highest erosion and reflection. This was evident in both the observed data collected at the nearshore ADCP station and the nearshore (local) wave modeling results.
3. Mach-Stem waves (waves traveling along the structure) propagating along the northern jetty can be seen in most cases. Although this does not represent a large amount of energy, it does produce an additional wave process that impacts the coastline, and specifically the corner where the shoreline and northern jetty meet.

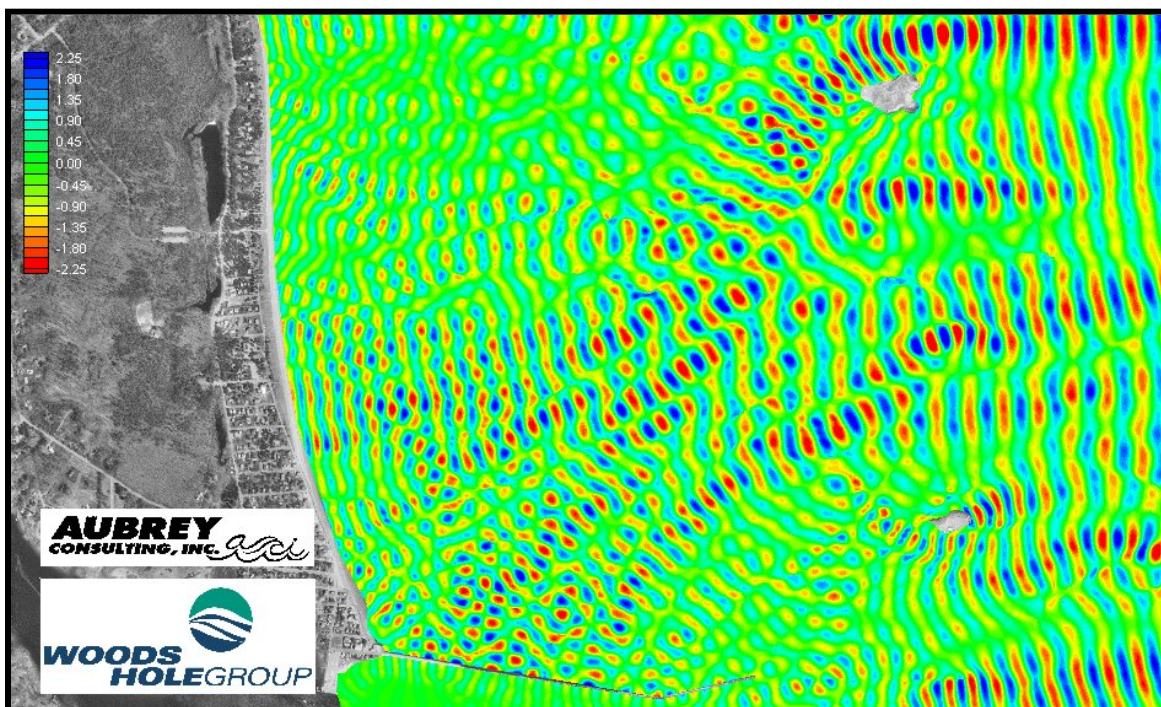


Figure 5-19. Sea surface results from the nearshore (local) wave model for the east-northeast (75 to 90 degree) approach bin. Blues indicate wave crests, while reds and yellows indicate wave troughs.

Results from the nearshore (local) wave model were utilized to produce local sediment transport estimates, generate energetics based performance evaluations, and provide an initial screening of the

proposed engineering alternatives (as presented in Chapter 7.1). Results from the directional approach simulations were used as part of the alternative analyses.

5.3.5.2 High Energy Event Simulations

In addition to the average annual approach directions, high-energy events were also simulated to provide a more complete picture of the existing conditions impacting Camp Ellis Beach. These simulations consisted of both return period design storms (10-, 50- and 100-year) and historical storm events (Hurricane Bob, Nor'easter of 2001 and "The Perfect Storm"). Each return period simulation was executed on a model grid that had increased water depth due to increased storm surge elevations.

Figure 5-20 presents the sea surface results for the 10-year return period storm in the direct vicinity of Camp Ellis Beach. The blues indicate wave crests, while the reds and yellows indicate wave troughs. The scale in the upper left corner of the figure presents the sea surface disturbance in meters. The storm case, consisting of increased wave heights and water depths in the vicinity of the shoreline, presents a more well-structured wave field when compared to the average annual approach directions. The interaction and shoreline impact of the incident and reflected wave trains is clearly evident. Many of the same wave transformation features discussed for the average annual approach cases are also evident in the storm scenarios. For the additional storm model run figures the reader is referred to Section 11.6.2 of Appendix C. The results from the nearshore storm simulations were used to quantify storm impacts on sediment transport and beach nourishment performance, as well as provide critical assessment of the alternatives (e.g., alternative performance during a significant storm event).

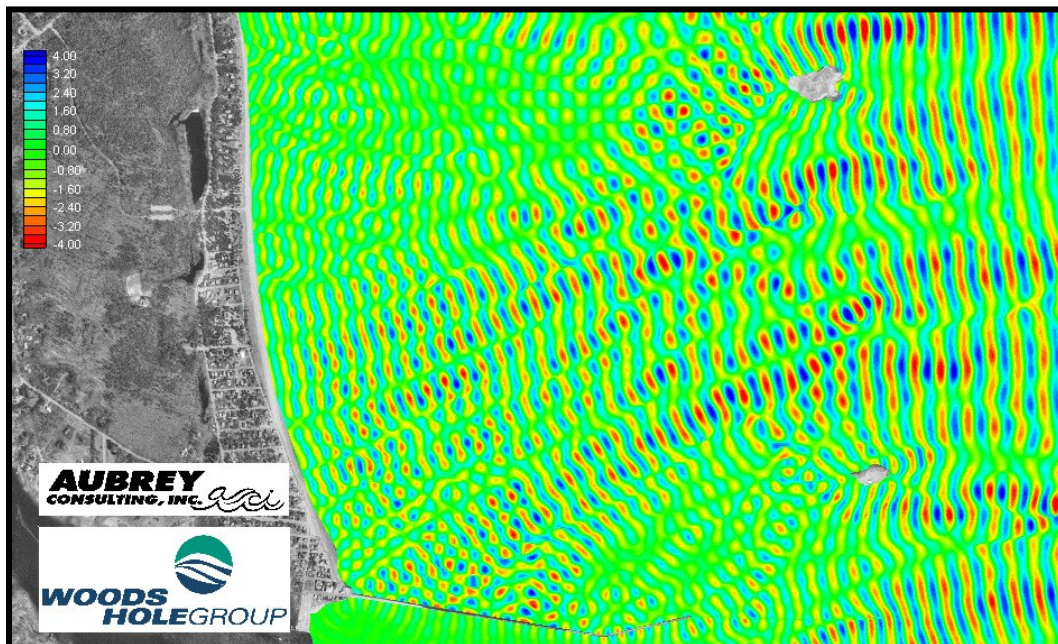


Figure 5-20. Sea surface results from the nearshore (local) wave model for the 10-year return period storm event. Blues indicate wave crests, while reds and yellows indicate wave troughs.

5.3.6 *Alternative Simulations*

The ultimate goal of the overall modeling system was application towards the evaluation of the wide range of alternatives presented in Section 6. The alternatives were geared towards mitigation of the ongoing erosion occurring at Camp Ellis Beach. The resolution of the local nearshore model allows for the simulation of these alternatives with accurate dimensions and layouts. In order to simulate the alternatives, the existing conditions model grid was numerically modified to include the proposed layouts.

The discussion regarding the design formulation and alternative development is provided in Section 6 and the evaluation of these alternatives, which includes the local wave modeling discussed in the previous sections of 5.3, is presented in Section 7.1.

5.3.7 *Local Wave Modeling Summary*

The regional model (STWAVE) presented in Section 5.2, only represents an intermediate step in the wave modeling system and although useful for identifying regional sediment transport trends, cannot be used for local sediment transport calculations for the Camp Ellis Beach region. Therefore, the numerical wave model CGWAVE was used to model the local, nearshore wave environment for Camp Ellis Beach. The nearshore (local) wave model was simulated using the same set of conditions developed for the transformation-scale (regional) modeling and captured the local physical processes, (e.g., wave reflection, wave-induced currents, wave dispersion, nearshore wave refraction and diffraction, etc.), and subsequently the engineering alternatives.

Evaluation of the sea surface results for the existing conditions revealed: (1) the significant wave reflection off of the northern jetty indicating the beach is impacted not only by the incident wave energy, but also by the reflected wave energy, (2) independent of offshore direction of approach, the nearshore waves propagated directly towards the Camp Ellis Beach region and the northern jetty, (3) Mach-Stem waves propagating along the northern jetty can be seen in most cases, (4) waves are refracted towards the northern jetty due to the jetty-parallel bottom contours, and (5) variations between annual average approach directions are important to understand the processes occurring at Camp Ellis Beach.

6.0 ALTERNATIVE DESIGN

The process of designing and evaluating the numerous alternatives investigated during the lengthy investigation evolved throughout the study. The work included in this appendix will cover the alternative development and analysis conducted post 2002 which utilized the WHG work. As with much of the earlier work, the early analysis was superseded and became outdated, and therefore it will not be discussed in this appendix.

Overall, a variety of factors were considered when designing and evaluating the various alternatives (e.g., cost, constructability, feasibility, performance, environmental impacts, etc.), with the overall objective focused on identifying the optimal solution. A significant and unexpected limiting factor in the placement and design of the various structural alternatives was the relatively poor sub-bottom geotechnical conditions. As discussed in Appendix D, there is a soft clay layer underneath much of the offshore project area that impacts the foundation stability of the structural alternatives. These weak foundation conditions resulted in limitations as to where structures could be placed and the size of the structures that could be placed. However, the focus of this section is primarily on the coastal engineering design layout of the alternatives related to providing protection and maintaining a beach at Camp Ellis. The alternatives were designed to mitigate the ongoing erosion occurring at Camp Ellis Beach and not for storm damage reduction. It is understood that there will be storm damage reduction benefits, but they were not considered in the analysis. For detailed structure design i.e. stone size, cross section, crest elevation, etc. refer to Section 8. For the detailed beach fill design and analysis refer to Section 9 and 10. The assessment and performance of the alternatives is discussed in Section 7 and Section 10.

6.1 Development of Alternatives

The Saco River and Camp Ellis Beach Section 111 Project alternatives were developed jointly between the USACE New England District, Woods Hole Group, Maine Geological Survey (MGS), and members of the Saco Bay Implementation Team (SBIT). During this iterative process, many viable solutions were discussed and considered, and an initial series of alternatives was selected for the analysis procedure. Careful consideration was given to all factors associated with each alternative. For example, potential impacts on the neighboring shoreline, engineering feasibility, likelihood of success, etc. were all considered in the final selection process. The alternatives that were viewed as the most highly effective were jointly selected for further analysis. Initially, a total of 11 alternatives were considered; however, this was expanded to 17 through the discussion and meeting process. Following some of the initial modeling results, the alternatives were expanded to a total of 23. Subsequent geotechnical evaluation resulted in the addition of 6 more alternatives. In the end twenty nine (29) potential solutions, including both structural and non-structural alternatives were developed.

6.2 Alternatives Considered

Table 6-1 presents a list of the alternatives considered, including the origin of each alternative. The base alternative is a beach nourishment project alone. However, since beach nourishment alone will not sustain an adequate level of protection against shoreline erosion (not storm damage protection), nor

does it directly address the impact caused by the northern jetty (increased energy due to wave reflection and a reduction in sediment supply through pushing sediment further offshore) additional project elements were considered in order to help create a more sustainable beach. Therefore, each alternative presented in Table 6-1 includes a beach nourishment component (to stabilize the shoreline and provide the lost sediment supply), constructed in concert with the alternative. In the table, reference to the northern jetty refers to the northern jetty of the Saco River, which is comprised of three distinct segments. Segment 1 is the shore-attached portion of the jetty that is exposed during all normal tide levels. Segment 1 is approximately 2,985 ft (910 m) in length. Segment 2 represents the northeast/southwest shift in jetty orientation and is approximately 1,050 ft (320 m) long. Segment 3 is comprised of the half-tide (i.e., exposed at low tide and submerged at high tide) portion of the northern jetty and is approximately 2,300 ft (700 m) in length. A spur jetty refers to a structure attached to the existing northern jetty, typically oriented perpendicular to the existing structure. A groin refers to a shore-attached structure that is built perpendicular to the shoreline and intended to trap sand flowing in the alongshore direction. In addition, references to an optimized location in Table 6-1 represent an iterative procedure performed during the modeling effort to identify the optimal performing location, if possible.

6.2.1 Alternatives Screening Process

As part of the alternatives analysis, a process was developed to perform an initial screening of all the alternatives presented above in order to streamline the modeling and analysis evaluation, focusing on only the alternatives that were reasonably meeting the performance goals. This initial screening process focused on wave height changes and energy reduction within the local region. Potential adverse impacts to neighboring beaches, navigation, and the Camp Ellis region were also evaluated. The alternatives that indicated the best potential for performance success were passed forward by the project team (WHG, USACE, SBIT, MGS) to a more detailed alternatives analysis and final assessment. The final screening and alternatives analysis consisted of a more detailed level of wave evaluation and assessment of the sediment transport. Due to the number of alternatives investigated only the final array of three (3) alternatives has been presented in the next sub section of this Appendix (Section 6.3). For a full review of the numerous alternatives refer to Section 10 of Appendix C.

Table 6-1. Alternatives considered in the alternative analysis procedure.

| Alt. ID | Description | Origin |
|----------------|--|---|
| Base | Beach nourishment alone | Base alternative |
| 0 | Northern jetty removal (segments 1, 2, and 3) | Initial alternative set |
| 1 | Northern jetty extension (segment 3) removal | Initial alternative set |
| 2 | Northern jetty extension (segment 3) removal and additional lowering of 600 m (1,970 ft) | Initial alternative set |
| 3 | Seaward location of a 230 m (750 ft) spur jetty | Initial alternative set |
| 4 | Optimized location of a 152 m (500 ft) spur jetty | Initial alternative set |
| 5 | Optimized location of dual 152 m (500 ft) spur jetties | Initial alternative set |
| 6 | Inshore location of a 230 m (750 ft) spur jetty | Initial alternative set |
| 7 | Inshore location of a 230 m (750 ft) spur jetty coupled with northern jetty extension (segment 3) removal | Initial alternative set |
| 8 | Inshore location of a 230 m (750 ft) spur jetty coupled with shore-based terminal groin | Initial alternative set |
| 9 | 1 st configuration of T-Head Groins | Initial alternative set |
| 10 | 2 nd configuration of T-Head Groins | Initial alternative set |
| 11 | Offshore Breakwater (seaward location) | Secondary alternative set |
| 11a | Offshore Breakwater (nearshore location) | Secondary alternative set |
| 11b | Offshore Breakwater (intermediate location) | Secondary alternative set |
| 12 | Offshore Breakwater (landward location) coupled with seaward location of a 152 m (500 ft) spur jetty | Secondary alternative set |
| 13 | Comb configuration of 15 m (50 ft) spur jetties | Secondary alternative set |
| 14 | Offshore borrow pit | Secondary alternative set |
| 15 | Seaward location of a 230 m (750 ft) spur jetty with an angled orientation | Secondary alternative set |
| 16 | Northern jetty roughening (segments 1, 2, and 3) | Secondary alternative set |
| 17 | Submerged shoal/rock outcrop | Secondary alternative set |
| 18 | Offshore Breakwater (landward location) coupled with landward location of a 152 m (500 ft) spur jetty | Developed based on highest performing previous alternatives |
| 19 | Seaward location of a 230 m (750 ft) spur jetty, northern jetty extension removal, and jetty roughening | Developed based on highest performing previous alternatives |
| 20 | Alt. 11a with estimated full salient formation | Estimated based on expected shoreline response |
| 21 | Alt. 11a with estimated partial salient formation | Estimated based on expected shoreline response |
| 22 | Combination of 230 m (750 ft) spur jetty with two nearshore 114 m (375 ft) segmented breakwater components | Developed based on results of initial geotechnical work |
| 23 | Combination of 152 m (500 ft) spur jetty with three 100 m (325 ft) segmented breakwater components | Developed based on results of initial geotechnical work |
| 24 | Alt. 23 with additional northern breakwater | Developed based on results |

| | | |
|------------|--|--|
| | segment of 100 m (325 ft) | of initial geotechnical work |
| 25 | Secondary configuration of 152 m (500 ft) spur jetty with three 100 m (325 ft) segmented breakwater components | Developed based on results of initial geotechnical work |
| 26 | Alt. 24 moving the northern most breakwater segment further north | Developed based on results of initial geotechnical work |
| 25A | Modification of Alt. 25, removing northernmost segmented breakwater | Developed based on concern of nearshore proximity of northernmost breakwater |

6.3 Alternative Descriptions

6.3.1 No Action Alternative

The no action alternative implies there would be no change to the present conditions at Camp Ellis Beach. This is an unacceptable alternative, as the existing shoreline would continue to be eroded, a sustainable beach and/or any protective action would not be undertaken, and the landward homes and structures would face potential damage/loss. This alternative does not address the required mitigation purview of the Section 111 Authority.

6.3.2 Base Alternative: Beach Nourishment Alone

The base alternative consists of placement of sediment on the beach area fronting Camp Ellis Beach. The current nourishment design consists of approximately 300,000 cubic yards of material extending approximately 3,000 ft (910 m) with the southern end of the project located at the northern jetty. However, as erosion continues, it is likely that the beach nourishment design volume will increase. As such, the final design and dimensions of the beach nourishment will be developed by the USACE, New England District. For the current study, this condition represents the base alternative; however, the beach nourishment is also considered to be a component of every alternative; that is, each alternative incorporates a beach nourishment project that will be constructed in concert with the other elements in that alternative. The base (beach nourishment alone) alternative is evaluated to determine if beach nourishment alone is an acceptable option for protecting the Camp Ellis Beach region.

6.3.3 Alternative 6: Inshore Location of a 750-foot Spur Jetty

This alternative (Figure 6-1) would consist of the construction of a 750 ft (230 m) spur jetty that would be attached to the existing northern jetty. The spur would be located approximately 1,475 ft (450 m) from the shoreline (approximately one-half the length of segment 1), determined by optimizing the location through multiple simulations. The spur would be oriented in a shore parallel (jetty perpendicular) orientation. This alternative would attempt to intercept the reflected wave energy, break a portion of the incident wave energy, and block Mach-Stem wave effects from transferring energy along the structure. Therefore, this alternative would potentially reduce the overall wave energy arriving at Camp Ellis Beach. In addition, the spur jetty should assist in reducing cross-shore sediment

transport from the beach seaward along the existing northern jetty. This alternative represents the optimal placement location of all spur alternatives.



Figure 6-1. Alternative 6: Inshore placement of 750-foot spur jetty.

6.3.4 Alternative 25a: Segmented Breakwater

This alternative (Figure 6-2) would consist of 2 detached breakwater segments and a spur jetty. The spur jetty is 500 ft (152 m) in length, 985 ft (300 m) from shore, and extends perpendicular to the existing northern jetty. Table 6-2 presents the dimensional details, including structure length, orientation (referenced from 0 degrees equivalent to north), distance from shore, gap distance from adjacent southern structure, and approximate depth, associated with each of the segmented breakwaters. This alternative would attempt to significantly reduce wave energy in the nearshore zone, to impede the reflected wave energy from the existing northern jetty, to extend beach nourishment life, and to produce salient formations that do not create a significant interruption in the littoral zone. This alternative was created to address the issue of the potential lack of suitable geologic conditions in the northern section of the proposed beach nourishment area for stable structural foundations and to alleviate potential concerns that the northernmost breakwater in Alternative 25 was too close to the shoreline.



Figure 6-2. Alternative 25A: Segmented breakwater and spur jetty configuration 4 (modified).

Table 6-2. Segmented breakwater dimensional parameters for Alternative 25A.

| Breakwater | Approx. Depth (MHW,m) | Length (m) | Distance from Shore (m) | Orientation (from 0° N) | Gap distance from adjacent south structure, end point to end point (m) |
|------------|-----------------------|------------|-------------------------|-------------------------|--|
| 1 | 4.75 | 125 | 265 | -25 degrees | 110 |
| 2 | 5 | 120 | 280 | -25 degrees | 105 |

7.0 ALTERNATIVE SELECTION ANALYSIS

The ultimate goal of the overall modeling system was the application towards the evaluation of the wide range of alternatives presented in Section 6. The alternatives were geared towards mitigation of the ongoing erosion occurring at Camp Ellis Beach. In order to simulate the alternatives, the existing conditions model grid was numerically modified to include the proposed layouts. The modified model grids (for each alternative) were simulated for the same set of wave conditions run on the existing conditions grid (i.e., average annual approach directional bins and high energy events). These simulation results were used in the initial screening analysis in order to evaluate the overall performance of each alternative and, in concert with the existing conditions simulations for each scenario, to generate differences in wave energy within the vicinity of Camp Ellis Beach.

Due to the number of simulations required to evaluate all potential solutions, the alternatives analysis consisted of an initial and final screening process. With twenty nine (29) alternatives, eight (8) average annual directional approach bins, and six (6) high-energy events, there is no reasonable way to simulate all scenarios for all alternatives. Therefore, the nearshore (local) wave model was used as the initial screening tool through evaluation of changes in wave energy, wave height, and wave direction prior to performing all simulations and associated sediment transport calculations.

7.1 Wave Height and Energy Evaluation

7.1.1 Visual Assessment

Visual assessment of the figures and model animations provided an overview of the modifications to the nearshore wave climate and wave propagation in the vicinity of Camp Ellis Beach. Figure 7-1 presents sea surface results for Alternative 6, consisting of a 750-foot spur jetty, for a 10-year return period storm scenario (storm defined in Section 5.2.6). The spur jetty intercepts a significant part of both the incident and reflected wave energy impacting Camp Ellis Beach. The area seaward of the spur experiences significant wave turbulence, due to reflected wave energy from both the spur and the existing northern jetty. Figure 7-2 presents sea surface results for Alternative 25, one of the segmented breakwater alternatives. This alternative indicates significant wave energy reduction along the shoreline, while intercepting a majority of the reflected wave energy off of the existing northern jetty.

Visual observations from these figures were one component of the initial screening analysis. Detailed discussions of the results for each alternative are presented in the following section as part of the initial screening analysis. The selected alternatives consist of those that indicated reasonable performance at reducing the wave energy with minimal negative impacts (as discussed in detail in the following section).

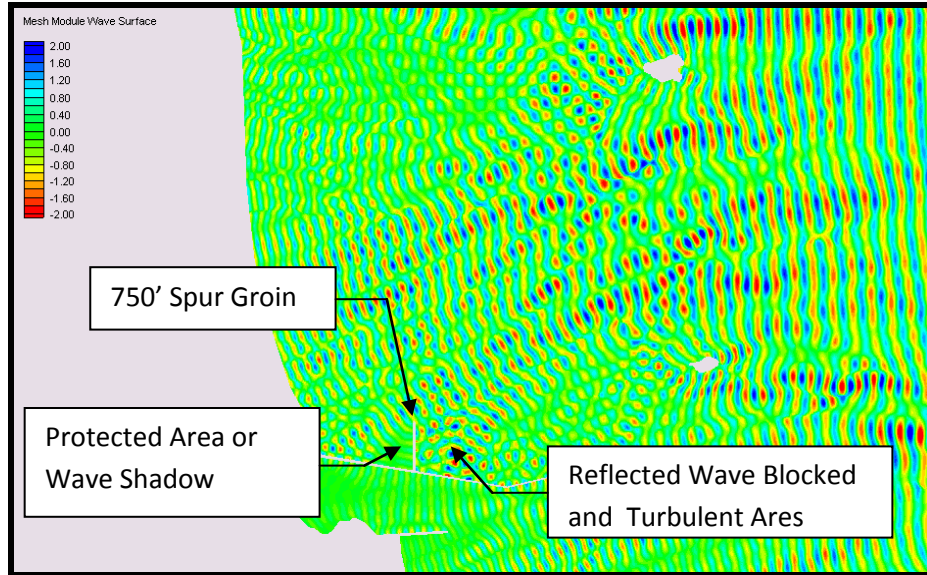


Figure 7-1. Sea surface results for Alternative 6 (750-foot spur jetty) for a 10-year return period storm. Blues indicate wave crests, while reds and yellows indicate wave troughs.

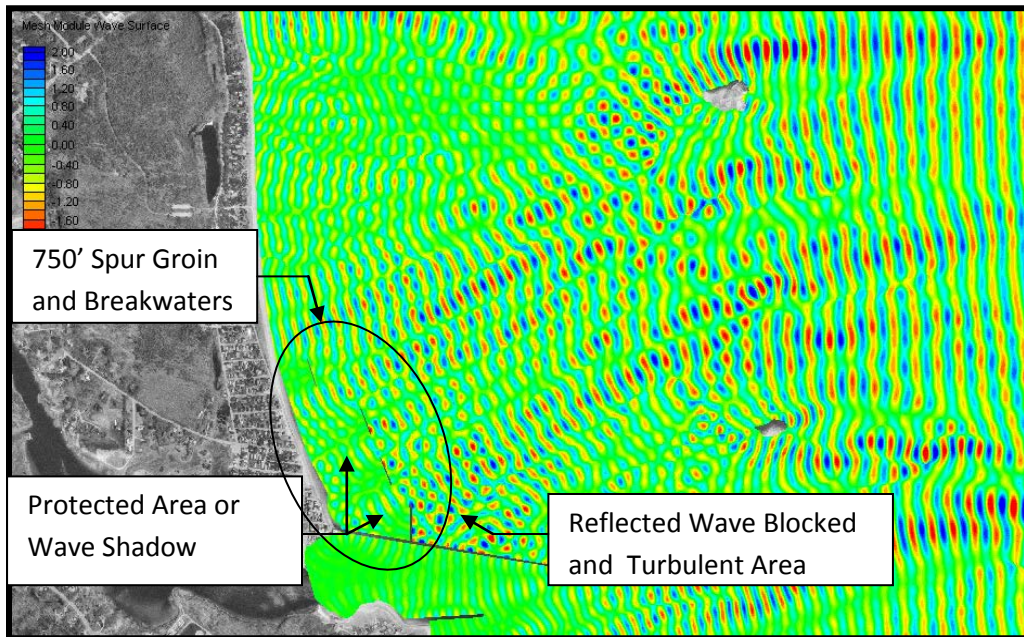


Figure 7-2. Sea surface results for Alternative 25 (one of the segmented breakwater alternatives) for a 10-year return period storm. Blues indicate wave crests, while reds and yellows indicate wave troughs.

7.1.2 Wave Height Change Assessment

Differences in wave heights (between existing conditions and alternative cases) were computed at each node within the model domain. Difference plots were then created (subtracting alternative wave heights from existing) that indicate regions of increased and/or decreased wave heights and assessed to determine the overall impact of the alternative on the wave height in the region. Figure 7-3 presents an example of the wave height change plots for Alternative 11a (an offshore breakwater) for a 10-year return period storm scenario (storm defined in Section 5.2.6). Positive values (yellows and reds) of wave

height change (m) indicate an increase in wave energy, while negative values (blues and purples) of wave height change (m) indicate a decrease in wave energy.

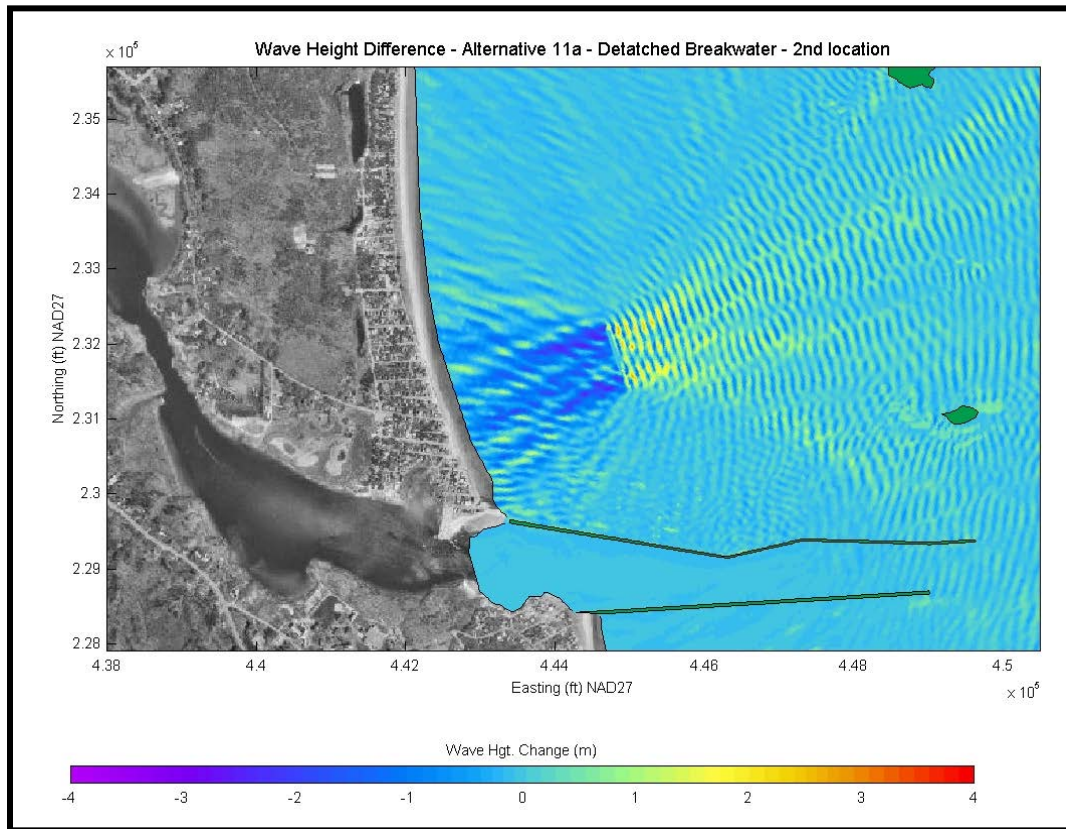


Figure 7-3. Wave height changes for Alternative 11a for a 10-year return period storm scenario.

7.1.3 Wave Energy Change Assessment

Wave energy change was evaluated in specific zones in the vicinity of Camp Ellis and the existing structures. Figure 7-4 shows the zones within which wave energy between existing conditions and alternative cases were compared. The results in Figure 7-4 show wave heights (m) for Alternative 11 (offshore breakwater) as a colormap behind the evaluation zones. The zones were selected to identify specific regions of potential concern. Assessment of impacts on adjacent shores, navigation, maintenance concerns, etc. An important aspect of any potential alternative and/or solution is the potential negative impacts that may be associated with the alternative. This may include, increased wave energy in other shoreline regions (e.g., Hills Beach or north of Camp Ellis), increased wave energy in the navigational channel, or alternatives that result in significant maintenance concerns.

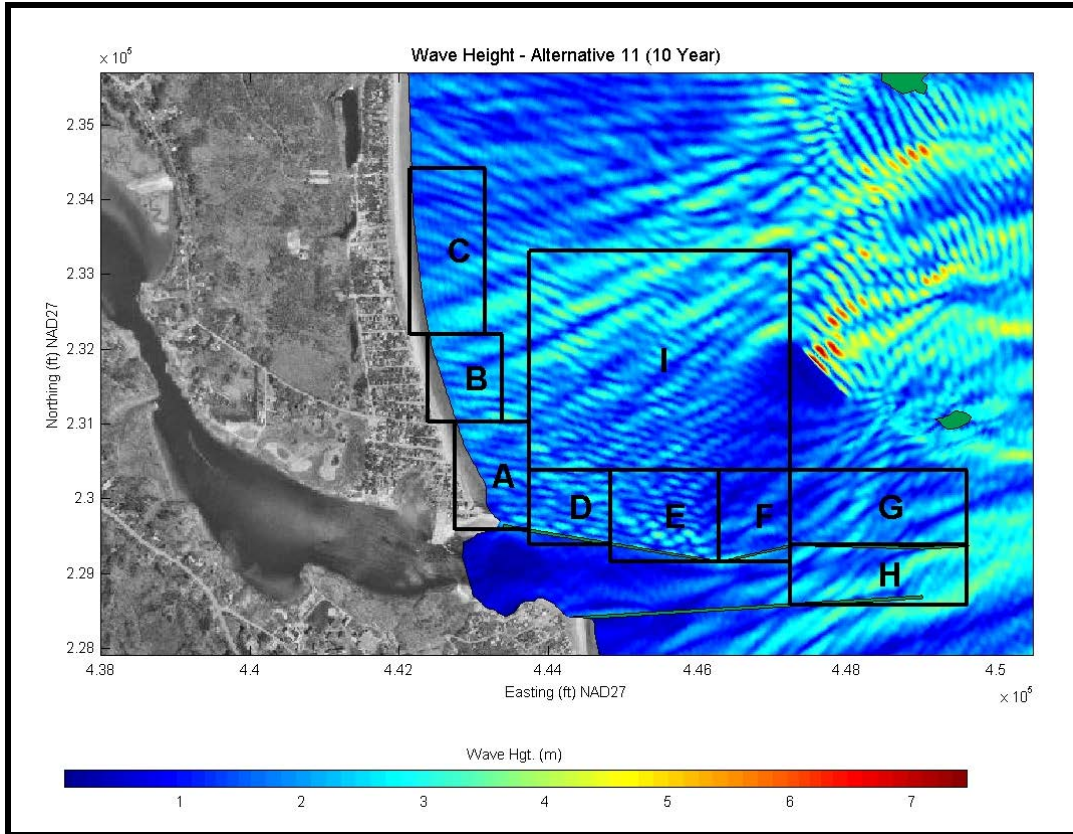


Figure 7-4. Zones used to evaluate changes in wave energy in the vicinity of Camp Ellis Beach and the Saco River Jetties.

7.1.4 Final Alternative Performance Discussion

Alternatives demonstrating the greatest potential for successfully reducing wave energy (and thus sediment transport), without resulting in negative impacts, were passed forward to the final screening analysis. The alternatives that were recommended for more in depth analysis were then simulated for all approach directions and storm cases to identify energy changes and to calculate sediment transport and beach nourishment performance. Additionally, criteria such as constructability, geotechnical foundation stability, cost, environmental permitting were also considered. Provided in the following sub-sections are the coastal engineering initial screening discussion for the final alternatives. The screening discussion for the full set of alternatives can be found in Section 11.8 of Appendix C.

7.1.4.1 Base Alternative: Beach Nourishment Alone

The base alternative plan was not simulated directly as an alternative. The plan does not contain any features that would reduce wave reflection or incident wave energy so there would be no changes to the wave climate along the project area beach. It was eventually concluded that this alternative was not complete or effective did not meet the criteria of a true alternative as it does not mitigate for the north jetty's effect on the local wave field and there was significant risk of failure during repeat storms. This was partially based on the historical performance of beach nourishment alone efforts, as material has quickly eroded due to the exacerbated wave energy on the Camp Ellis region. However, existing conditions simulations were used to assess the potential performance of this plan, and it was included in

further analyses from the standpoint of being a comparative measure for the other alternatives. The potential lifetime of beach nourishment alone can be used to assess the relative performance of other alternatives, as the same level of beach nourishment, is included in every alternative.

7.1.4.2 Alternative 6: 750-foot Spur Jetty

This alternative was designed through an iterative process of spur groin alternative layouts. Figure 7-5 presents the wave height difference plot for Alternative 6 under the average annual eastern (90-110 degree) approach bin. Wave height differences for the 10-year return period storm event are presented in Figure 7-6. For the average annual eastern approach scenario Alternative 6 showed significant wave energy reduction in zone A (17%), as well as wave energy reduction in both zones B and C (5%). The large reduction in wave energy in zone D (52%) also makes this alternative the best performing spur alternative that was simulated. Zones E and F show moderate energy increases (12-15%), as expected seaward of the spur. There also was no change in wave energy at the entrance to the navigational channel (zone H). Figure 7-5 clearly shows the interception of a good portion of the reflected wave energy throughout the directional array. Waves are intercepted that would propagate to a significant stretch of Camp Ellis Beach. For the 10-yr storm case (Figure 7-6), Alternative 6 also reduces wave energy within the same zones. The percentage of reduction is not as great, but the overall wave height reduction is still significant. Again, reduced energy is indicated for zones A-D. Overall, Alternative 6 was the best performing spur alternative. A significant amount of the reflected wave is intercepted; it reduces wave energy in critical zones A, B, and D, while it does not negatively impact zone C or the entrance to the navigational channel (zone H).

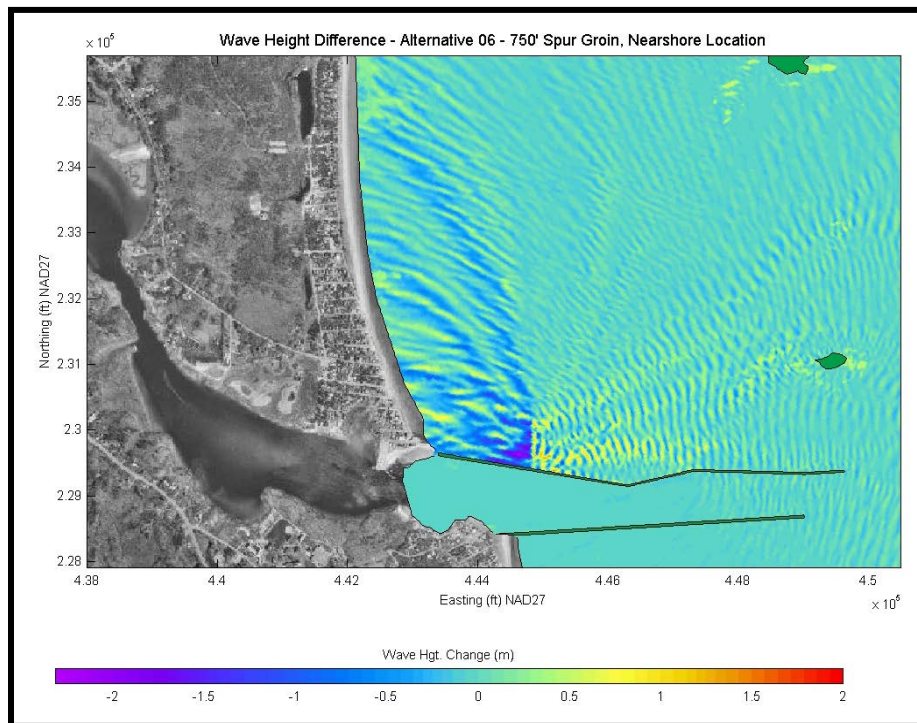


Figure 7-5. Wave height changes for Alternative 6 for an eastern (90-110 degree) wave approach bin.

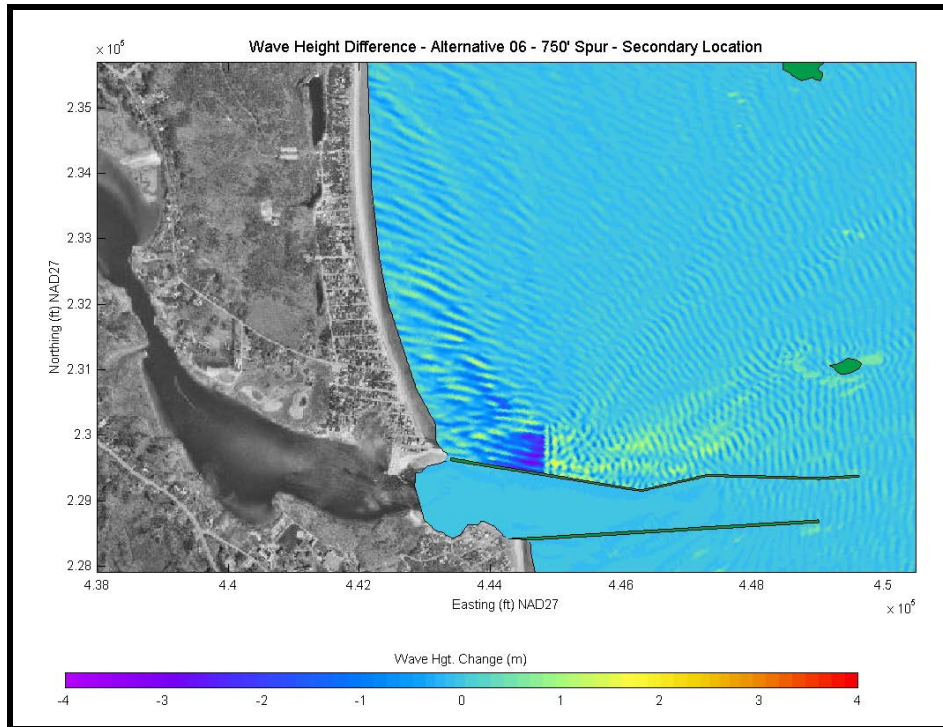


Figure 7-6. Wave height changes for Alternative 6 for a 10-year return period storm.

7.1.4.3 Alternatives 25A: Segmented Breakwater

Due to the proximity of the segmented breakwater alternatives to the shoreline, zone B was divided such that the presence of the segmented breakwaters did not divide the energy zone. This was required so that the energy zone wasn't identifying both the reduced energy landward of the structure, and the increased energy seaward of the structure. Ultimately, the concern is related to the wave energy at the shoreline. The smaller zone B is defined as B2. Figure 7-7 presents the adjusted energy zones used to assess the segmented breakwater alternatives and shows wave heights (m) for Alternative 25 as a color-map behind the evaluation zones.

Figure 7-8 presents the wave height difference plot for Alternative 25A for a 10-year return period storm scenario. The average annual eastern (90-110 degree) approach bin wave height differences are presented in Section 11.8 of Appendix C for all the alternatives.

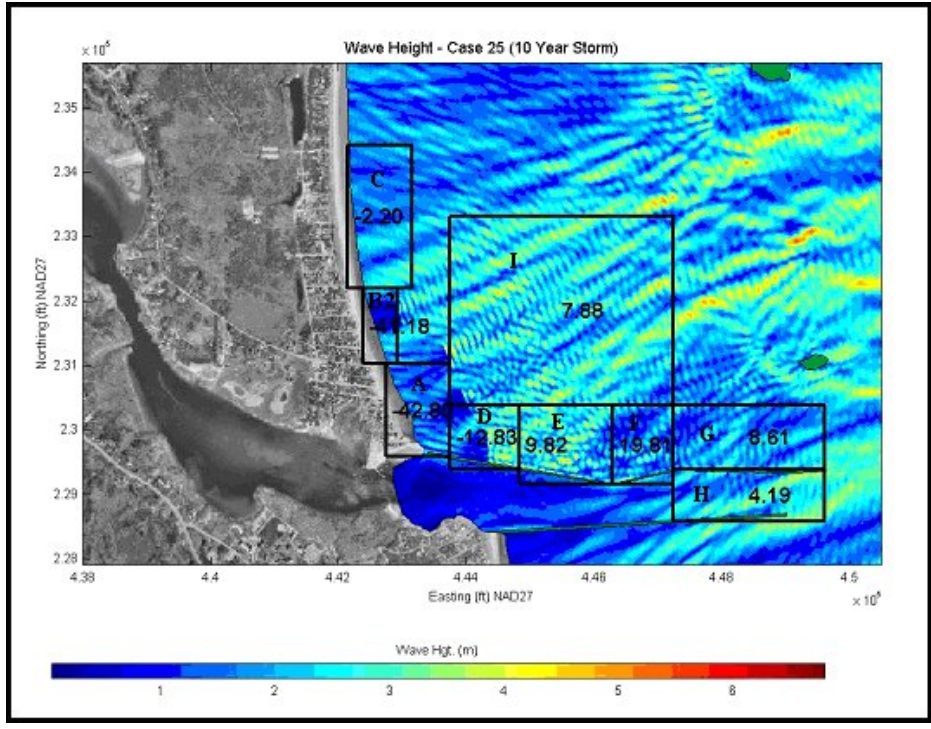


Figure 7-7. Redefined zones used to evaluate changes in wave energy in the vicinity of Camp Ellis Beach and the Saco River Jetties for the segmented breakwater alternatives.

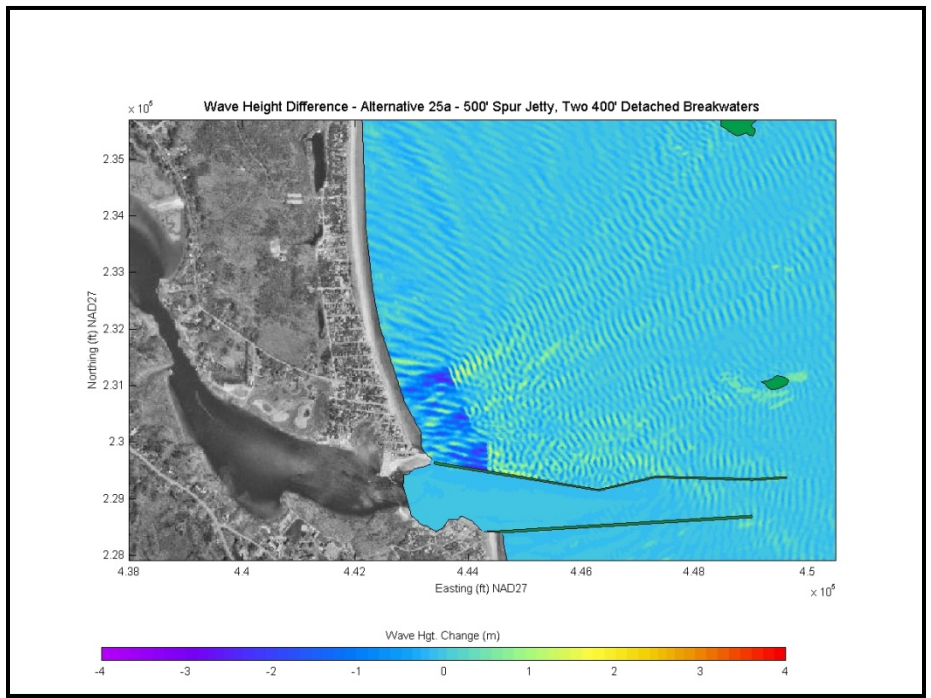


Figure 7-8. Wave height changes for Alternative 25A for a 10-year return period storm. A negative wave height change indicates a reduction in wave height, while a positive wave height change indicates an increase in wave height.

Table 7-1 presents the wave energy changes for Alternative 25A for both the average annual eastern approach direction and the 10-year return period storm. Zones seaward of the structure indicate wave energy increase. The energy reduction in Zone A for both the average annual and 10 year storm conditions is significant at 40%+. Nearshore breakwater alternatives that contained structures further to the north, that would have protected Zone B were initially investigated, and recommended, but due to geotechnical foundation issues of the sea bottom in this area, constructing breakwater structures was deemed to not be feasible. Therefore the scaled back near shore breakwater alternative of 25A was recommended as the near shore breakwater alternative. The difference in zone D for Alternative 25A is due to the more landward location of the spur jetty. As shown in Figure 7-7, the spur jetty for these alternatives bisects zone D, and therefore averages both the shadow zone and the reflected wave energy from the spur. When evaluating the shadow region only, a similar reduction (approximately 35%) is obtained.

Table 7-1. Comparison of energy changes for the segmented breakwater alternatives.

| Alternative | Avg. Annual Eastern Approach | | | | 10-yr return period storm | | | |
|-------------|------------------------------|------|-----|-------|---------------------------|------|------|-------|
| | % Energy Change in Zone | | | | % Energy Change in Zone | | | |
| | A | B2 | C | D | A | B2 | C | D |
| 25A | -47.3 | -4.7 | 1.3 | -14.6 | -42.0 | -0.9 | -2.7 | -13.0 |

7.2 Sediment Transport – Alternative Analysis

The methodologies to determine sediment transport were discussed in Section 4.2. In that section the existing conditions, and therefore without project conditions were discussed. In the following sub sections of Section 7.2, the alternatives passed forward from the initial screening analysis are evaluated herein to determine relative performance in terms of their ability to maintain a stable shoreline at Camp Ellis. This section compares the relative performance of the final alternatives and presents the information to allow the reader to determine, from a purely performance standpoint, the relative merit of each alternative.

7.2.1 Methodology

In order to determine the more complicated local sediment transport regime, including the potential ability of each alternative to maintain a healthy beach, the same zones as used in the wave energy reduction assessment were used to provide a local assessment of sediment transport. Specifically, the nearshore region zones of A, B, and C (Figure 7-7) were used. The values of sediment transport computed within these three boxes act as the basis for comparison to the alternative sediment transport results. It was the goal of all alternatives to reduce erosion (sediment transport) within zone A and B and to have limited impact upon the sediment transport within zone C. Areas represented by zones A and B, which are 1,000 ft (300 m) zones, have historically experienced significant erosion. Zone C, a 1970 ft (600 m) zone, is a region that exhibited more stable shoreline conditions in the past but more recently has experienced accelerated erosion rates.

Results from the nearshore (local) wave model were used to drive the local sediment transport analysis. These wave results capture the most physical processes in the vicinity of Camp Ellis each and provide high resolution (approximately 30 ft or 10 m) results. Therefore, the nearshore (local) wave model results provide the most accurate and detailed wave data to drive the sediment flux.

7.2.2 Sediment Transport Reduction

Changes in potential sediment transport flux were evaluated for each alternative, within each zone. Figures 7-9 and 7-10 present the change in potential sediment transport flux for several of the alternatives. It should be noted that in Figure 7-10 Alternative 23 can be used as a substitute for the performance of Alternative 25A. The numbers represent the percent change in potential sediment transport rate when compared to the existing potential sediment transport rate. A greater reduction in potential sediment transport means that a smaller amount of material would be leaving the region in the alongshore direction. As shown in Figures 7-9 and 7-10 there is a reduction in transport in zones A and B with a minimal amount of change in zone C. This means that there will be a potential decrease in sediment entering zone C from A and B; however, currently (and approximately over the last 40 years) there has been little to no sediment available for transport in these areas due to the severe historic erosion and revetment construction, as evidence, Zone C has begun to experience significant erosion under existing conditions for recent times (periods after 1998). Therefore, even with a reduced transport rate from zones A and B, zone C will experience a net increase in sediment with the influx from the beach nourishment program. This is important, since the overall project will have a net benefit on the shorelines to the north for the final alternatives considered as (1) currently there is a deficit of sand available for transport to the region such that erosion is expected to develop in this area with no action, and (2) the addition of sand to the system can only benefit the regions to the north, since even with a reduced transport rate, the net sand moving to the northern beaches would be greater than currently exists.

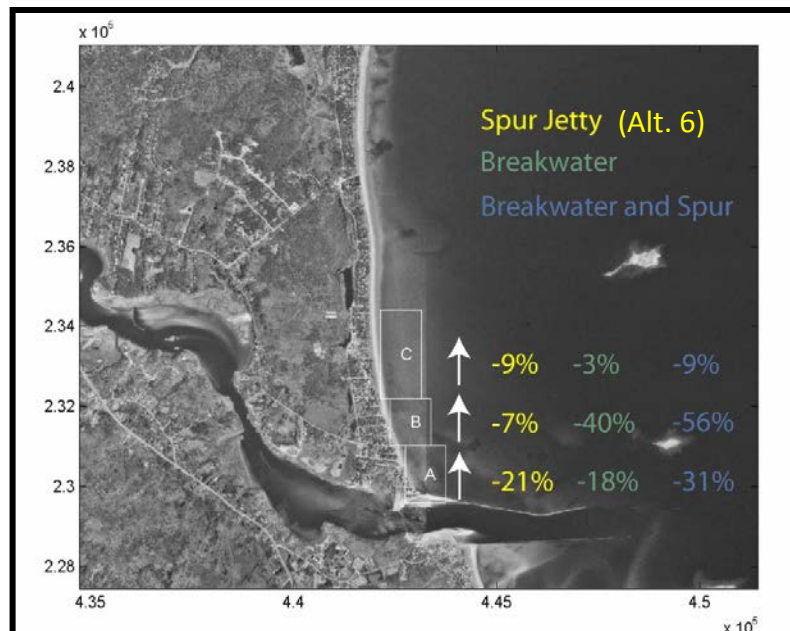


Figure 7-9. Changes in sediment transport flux for Alternatives 6, 11a, and 18.

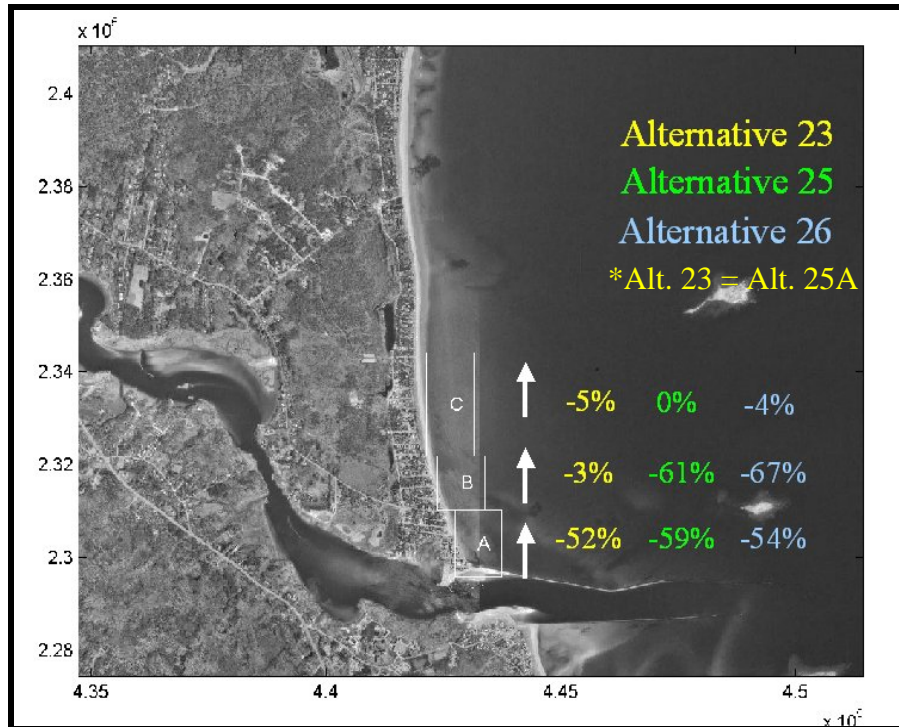


Figure 7-10. Changes in sediment transport flux for Alternatives 23 (Alt 25A), 25, and 26. The numbers indicate the percent change in sediment transport rate for each corresponding alternative.

The full explanation regarding the calculations of the sediment flux reduction can be found in Appendix C, Section 12.5.3.

7.2.4 Salient Formation

For alternative 25A, with the proposed installation of segmented breakwaters, estimates of shoreline response were developed to determine the impact of potential salient formation on wave energy and coastal processes. A salient is a coastal formation of beach material developed by wave refraction, wave diffraction, and longshore drift producing a bulge in the coastline behind an offshore island or breakwater. If the salient connects to the offshore feature/structure, it is termed a tombolo. Salient growth was determined to ensure that a tombolo would not form behind the proposed breakwater(s), and subsequently severely inhibit alongshore transport to the north. Finally, estimates of the volume of sediment that would comprise the salient are provided, and thus provide an estimate of how much material would be sacrificed from the beach nourishment to eventually form the equilibrium salient.

To determine the amplitude and shape of the salient formation, recent studies completed by Suh and Dalrymple (1987), Seiji, Uda, and Tanaka (1987), Aherns and Cox (1990), Noble (1978), and Hsu and Silvester (1990) that evaluated the formation of salients behind breakwaters were used. For the full explanation of the methodology used refer to Appendix C Section 12.5.4.

Figure 7-11 presents the salient formations for the segmented breakwater configurations of Alternative 25. Alternative 25A would not have the northernmost salient. The schematic shows the estimated

amplitudes and shapes of the salient formations behind the breakwater segments and provides a reasonable estimate of the potential shoreline modification. The total volumetric value (highlighted in the white box) is the summation of all the salients in these cases. Since the final beach nourishment template and width was unknown at the time of this report, this analysis was completed without including the shoreline advancement associated with the proposed beach nourishment. This would push the salient formations further seaward and would also result in slightly different amplitudes and volumes.

In all of the final breakwater alternatives, theory does not predict the formation of tombolos. Therefore, the effect on the natural longshore sediment transport will not be severe. In addition, the total volume of sediment estimated for full salient formation is less than 1/6th of the overall initial fill volume for Alternatives 11a and 18, and less than 1/5th of the initial fill volume for Alternatives 25A, 25, and 26.

The salients were numerically added to the model domain and were simulated for the full range of wave climate scenarios. These wave results were used in the development of the beach nourishment performance estimates presented in the section 7.2.5 for the breakwater alternatives.

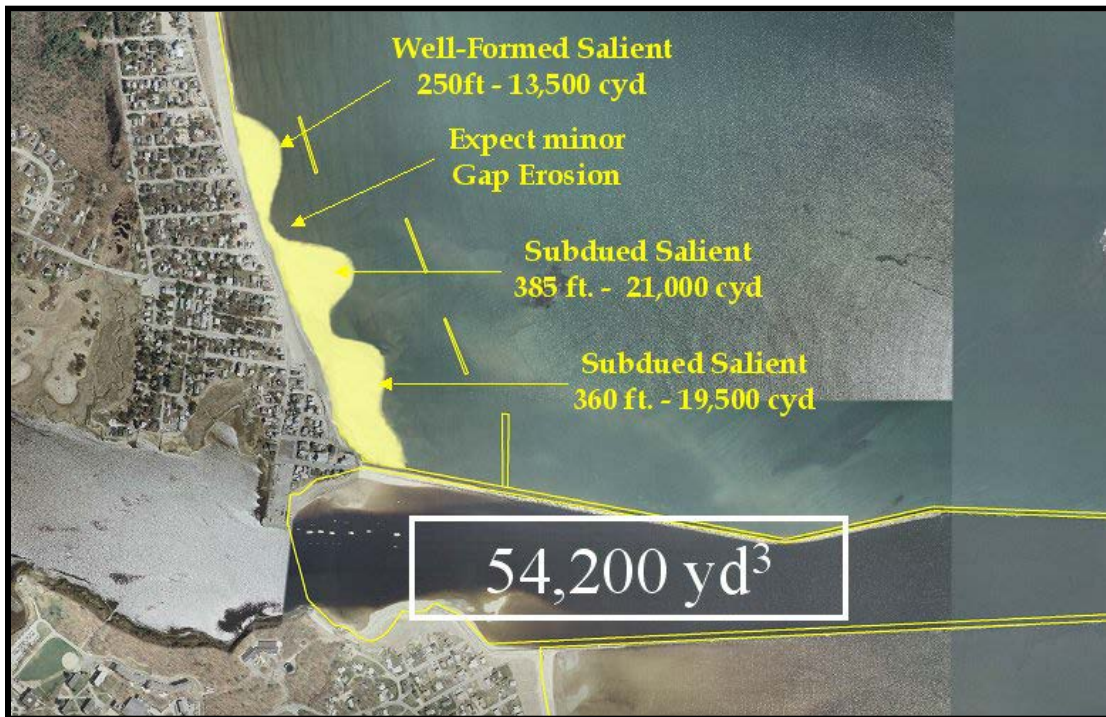


Figure 7-11. Estimated salient formation for the segmented breakwater configuration of Alternative 25.

7.2.5 Beach Nourishment Performance

7.2.5.1 Beach Nourishment Spreading

The basic fundamentals usually used to evaluate the performance of a beach nourishment do not necessarily apply to the conditions at Saco Harbor. Typically, nourishment performance is evaluated on an open coastline and does not include the effects of a neighboring inlet or coastal structures. The standard evaluation combines the conservation of sediment equation with the linearized transport equation. This formulation, called the Pelnard-Considére (1956) equation, is used in providing theoretical results to establish design and performance standards for nourishments (Equations 7-1 and 7-2). Due to the presence of the Saco River jetties, a modified version of the equation and analysis was necessary. Dean (2002) adapted the equations to evaluate sand movement in regions with inlets and/or structural influences. The detailed analysis can be found in Appendix C Section 12.5.5. This analysis used the methods and formulations provided for in the CEM Section V-4-1 g. page V-4-46.

$$M(t) = \frac{2\sqrt{Gt}}{l\sqrt{\pi}} \left(e - \left(\frac{1}{2\sqrt{Gt}} \right)^2 - 1 \right) + \operatorname{erf} \left(\frac{l}{2\sqrt{Gt}} \right) \quad \text{Equation 7-1}$$

where:

M(t) = proportion of sand remaining in the placed location
G = alongshore diffusivity parameter
t = time
l = project (nourishment) length

The alongshore diffusivity is presented by Pelnard-Considére (1956) and has been provided as Equation 7-2.

$$G = \frac{KH_b^{5/2} \sqrt{g/\kappa}}{8(s-1)(1-p)(h_* + B)} \quad \text{Equation 7-2}$$

where:

K = sediment transport coefficient (a function of sediment size)
B = berm elevation
H_b = breaking wave height
h* = depth of closure (in this case 8.7 m or 28.5 ft)
p = in-situ sediment porosity (approximately 0.35 to 0.40)
s = sediment specific gravity (approximately 2.65)
κ = ratio of wave height to water depth within the surf zone (approximately 0.78).

In an open coast situation, the nourishment will spread symmetrically about the centerline of the project as material is transported to both sides of the nourishment. However, placing the fill adjacent to a littoral barrier only allows the nourishment to move in one direction (to the north when considering placement of material next to the northern structure), compared to the initial plan form. Therefore, nourishment placed next to a littoral barrier can only spread in a single direction (Dean, 2002). In effect, the littoral barrier increases the time with which sediment must remain in the fill region. However, this does not necessarily mean that the lifetime of a nourishment placed next to a structure will be increased, since the down drift impact of the coastal structures must also be considered. Superimposed on top of the solution for the nourishment spreading is the solution for shoreline displacement at a littoral barrier (i.e., groin, jetty, etc.).

Table 7-2 presents the adjusted length of the nourishment for each alternative based on the influence of the structures (both existing and proposed). These adjusted lengths do not represent an actual physical extension of the nourishment; however, the adjustment is used to represent the influence of structures on the rate of dispersion in the sediment transport model. Typically, a structure such as the northern jetty at Saco River would limit the spreading to a single direction and the nourishment length would be doubled to account for this influence (Dean 2002). However, in the specific case of the Saco River northern jetty; the length of the jetty, especially on the northern side, has created nearshore contours along the onshore-offshore length of the structure. In some respects, this behaves like an alongshore stretch of coastline that contains a significant curvature, as evident by the contours and the way the waves refract into the structure. Waves approach a portion of the structure like a shoreline, increasing wave reflection and producing physical sediment transport that is more alongshore physics based in the cross-shore direction. Therefore, the existing northern jetty is considered to provide less of a boundary to the dispersion of sand, which will likely spread along the structure in the offshore direction. As such, doubling the nourishment length to account for the presence of the northern jetty (i.e., allowing no dispersion in the southern direction) does not seem realistic. In cases where a spur jetty is included, however, the combined ability of the northern jetty and spur to contain sand within the nourished zone is highly probable, and therefore, doubling of the nourishment length is advised. The selection of these lengths is based on engineering judgment and based on the results of the existing condition sediment transport along the northern structure.

Table 7-2. Adjusted nourishment lengths for assessment of beach nourishment performance

| Alternative | Adjusted Nourishment Length |
|----------------------------|-----------------------------|
| Beach Nourishment Alone | 990 m (3,250 ft) |
| 6 – Spur Jetty | 1,440 m (4,125 ft) |
| 25A - Segmented Breakwater | 1440 m (4,125 ft) |

7.2.5.2 Beach Nourishment Diffusivity Determination

Since the wave environment at Camp Ellis Beach is complex, calculation of the alongshore diffusivity (G value in Equation 7-1 which was calculated using Equation 7-2) was completed based on the wave

distribution (incident and reflected) for each average annual directional approach bin. The percent occurrence of each of these wave types (for each directional bin) was also computed. Values of alongshore diffusivity were then computed for each wave type, in each directional bin, and for both MHW and MLW cases. Finally, based on the overall percent occurrence for each wave type and each directional bin, a representative alongshore diffusivity was computed within each alongshore zone. For breakwater alternatives, this also included the calculation of alongshore diffusivity for both full and partial salient formations, and these formations were included in the nourishment performance. Salients were expected to form within 3 years following breakwater construction in all cases. This methodology produces a much more representative alongshore diffusivity value than the selection of the most commonly occurring wave height and direction, or averaged wave height and direction.

7.2.5.3 Background Erosion Rate Inclusion

As discussed in the CEM, the beach fill longevity formulation, discussed in the previous sections, only accounts for sediment being removed from the beach fill due to end losses, or material be transported from the edges of the beach fill. The loss rate does not account for sediment being removed due to the background erosion that previously existed before the fill was placed. The addition of background erosion is discussed in detail in the CEM in section V-4-1 g.3.c on page V-4-52. As highlighted in the following section a background erosion rate of 2 ft/yr was included in this analysis. The background erosion rate was determined in Section. When applying the background erosion rate it was realized that this rate would only apply to the Beach Fill Only alternative since alternatives 6 and 25A would reduce the wave climate, and wave direction, and therefore would impact the background erosion rate. Trying to determine the background erosion rate reduction for each alternative was difficult and the more conservative approach of using the same background erosion rate was used.

7.2.5.4 Beach Fill Longevity Formulation Results

In all cases a nourishment of 300,000 cubic yards and (762 m or 2,500 ft) in length was evaluated as the initial nourishment project. A berm elevation of (3.0 m or 10 ft-NAVD88) was used based on a preliminary engineering analysis performed by the USACE. The final beach fill alternatives used a beach fill length of 3,250 due to additional erosion that occurred during the study time period and a 12 ft-NAVD88 berm elevation due to modeling results discussed in Section 9.

Alternatives were compared to one another based on their ability to maintain a beach at Camp Ellis. Figure 7-12 presents the performance of a 300,000 cubic yard fill in terms of amount of material remaining, as a function of time, for the beach nourishment alone alternative (black line), Alternative 6 (blue line), and Alternative 25A (purple line). This includes background erosion corresponding to 2 ft/yr (0.6 m/yr). That is, in addition to the dispersion that is occurring, an additional 2 ft/yr (0.6 m/yr) is eroded due to the natural erosion of the beach (as indicated in the historical data analysis for the region directly north of the northern jetty – Section 4.1). The percent of initial material remaining is presented along the left hand axis, while the time (in years) is presented along the bottom axis. Although the performance curves are presented for an estimated 300,000 cubic yard nourishment, they can be scaled to represent any volume of proposed nourishment as the rate of dispersion is not a function of total volume, but is representative of the percent of the initial volume nourished.

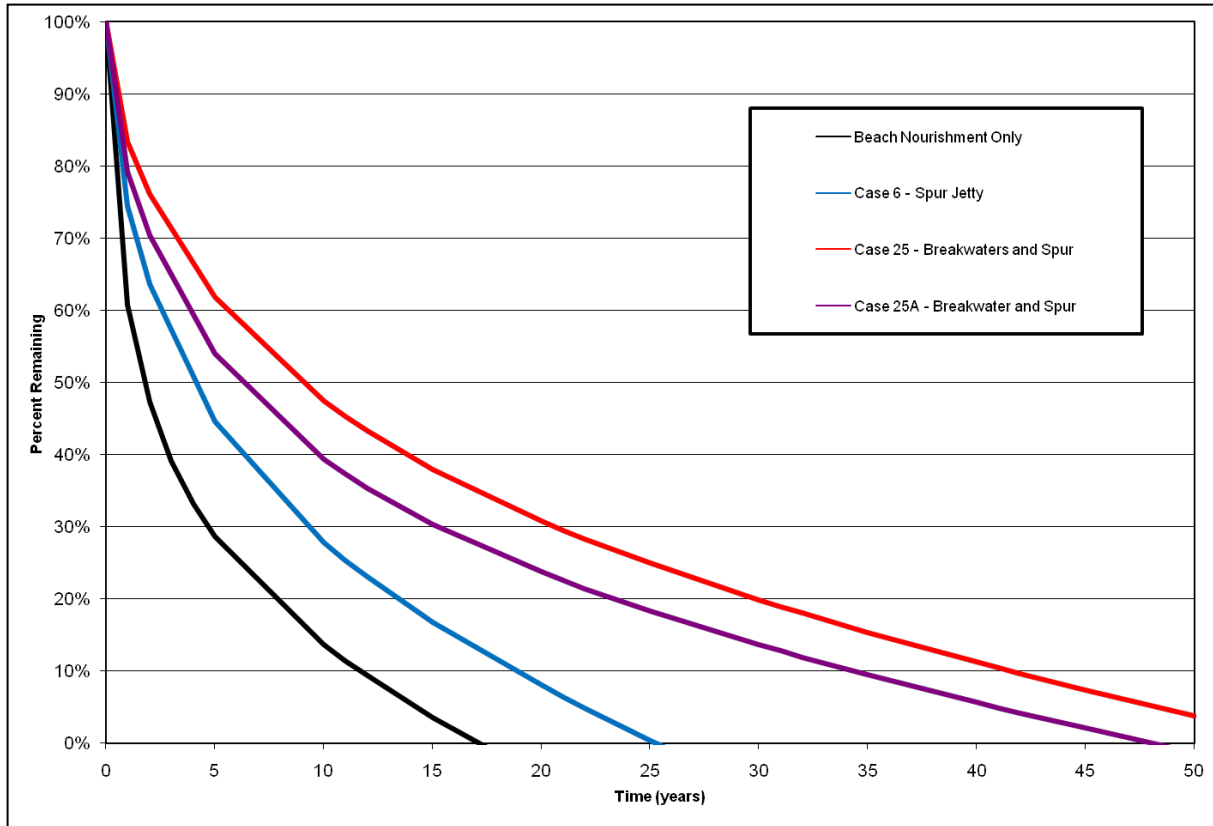


Figure 7-12. Beach fill longevity (graphical presentation)

7.2.6 Sediment Deficiency/Sediment Blockage Caused by Federal Navigation Project

As discussed in Section 4.2.4, a significant issue related to the federal navigation project is the prevention of sand movement from the Saco River to the southern Saco Bay beaches. In essence the federal navigation project is depriving the beach system of a critical sand source. In Section 4.2.4.2, it was estimated that the federal navigation project is preventing 13,000 to 22,000 yds³/yr from making it to the beach. In order to address this issue, the volume of sand used for the beach fill component of each of the final three alternatives was compared to the estimated sand deficit.

The beach fill volume (initial placement and renourishment volumes) for the 50 year study period for each alternative were determined later in this appendix in Section 10. Provided below in Table 7-3, is the calculated total volume for the historical sea level rise condition. As can be seen, the beach fill only alternative supplies significantly more sand volume than would need to be required to make up for the sand deficit caused by the federal navigation project. The high volume of sand for the beach fill only alternative is needed to address the high transports rates caused by the reflected waves coming off of the North Jetty. Alternative 6 requires less sand to maintain an adequate beach because the spur groin feature significantly diminishes the reflected waves off of the North Jetty. However, the volume required is still well within the estimated range of the sand deficit caused by the federal navigation project. Alternative 25 requires less sand than both the beach fill alternative and Alternative 6, but still

provides a volume of sand that meets the low end of the estimated deficit caused by the federal navigation project.

Table 7-3. Total fill volumes and annualized fill volumes.

| Alternative | Volume (yds ³) | |
|-------------|----------------------------|-------------------|
| | 50 Year Volume Total | Annualized Volume |
| Beach Fill | 2,027,170 | 40,543 |
| 6 | 865,098 | 17,302 |
| 25A | 652,219 | 13,044 |

8.0 STRUCTURAL ALTERNATIVE DESIGN

As discussed in Section 6, the geographic layout of the various rubble mound breakwater and spur groin structural alternatives was governed by several factors and was developed through several iterations of formulation and then model verification. Once the final set of structural alternatives was selected, the detailed design of the actual structures was conducted. For each structure the cross section and armor stone size was determined.

For the structural design return period storm levels were considered and both return period wave height and water level were used for the various structural components. As discussed in Sections 5.2.6, the corresponding return period wave height and water level was used. It is understood the joint occurrence, or joint probability, of return period parameters such as waves and water levels results in an event that likely has a return period greater than the individual return periods. However, with nor'easters in New England, typically, if a storm is strong enough to generate a return period water level, the storms intensity is such, that significant wave energy will surely occur as well. The offshore wave height may not be at the same return period level, but the near shore waves are likely to be near maximum due to shoaling and depth limited wave conditions. Therefore a joint occurrence was assumed to be reasonable.

8.1 Risk Reduction (Level of Protection of Structures)

The question or issue related to what level of flood protection/risk reduction the alternative structures has been designed for was discussed at length within the Project Development Team (PDT) and with several members of USACE HQ. Through those discussions it was concluded that the project intent was to address the erosion problem caused by the USACE jetties and was not a flood damage/risk reduction project. In basic terms, the jetties are increasing shoreline erosion, the various alternatives designed address the shoreline erosion issue, and that is the end of the project intent. Any flood prevention or risk reduction is a byproduct and was not determined by the PDT. Additionally, the project requirement of showing a plus one benefit cost ratio was waived through Congressional direction, so the PDT further concluded it would not be worthwhile to determine the reductions.

During the analyses presented in Sections 9 and 10, a 10-year storm was used to model the performance of the beach fill for the various alternatives (storm defined in Section 5.2.6). As explained in those sections, a 10-year storm was not selected for specific flood protection/risk reduction level, but instead to represent the beach fill performance under storm conditions. Basically, it is accepted amongst the coastal engineering community in New England that there really is not a major difference in the typical storm characterization metrics (water level, wave height, wave period) between a 1 year storm and a 10 year storm. The 10 year storm was chosen for conservatism. As shown in Table 8-2, and discussed in Section 8.6, the differences between a 1 year storm water level and a 10 year storm water level are minor, and based on the WHG information the waves are similar as well.

8.2 Armor Unit Size

8.2.1 *Slope and Crest Stone*

The first determination was that of armor unit stone size. The stable armor stone size of the rubble mound structures was calculated using Hudson's Equation (Equation 8-1). The key variables in the equation are wave height and the structure slope. As design wave height increases the required weight of the armor stone increases. As the slope of the structure decreases the required weight of the armor stone decreases.

$$W = \gamma_r H^3 / KD (\gamma_r/\gamma_w - 1)^3 \cot \theta \quad \text{Equation 8-1}$$

where:

W = armor stone weight (lb)

γ_r = unit weight of armor stone (pcf)

γ_w = unit weight of water at the site (salt or fresh)

KD = stability coefficient

H = design wave height for particular event being examined

θ = the structure slope from the horizontal

NS³ = minimum design stability number = (*KD cot θ*)

In the following sections the specific inputs for Hudson's Formula will be discussed and the formulations will be provided in Section 8.8.1.

8.2.2 *Toe Stone*

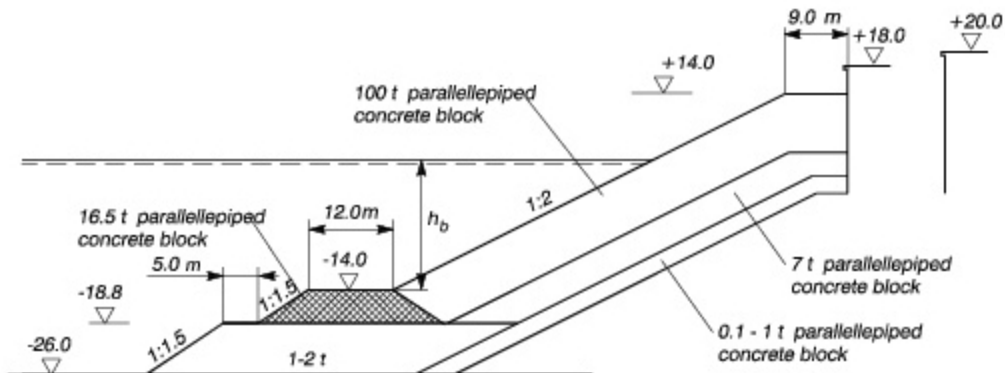
The toe stone was determined using a separate formulation than the Hudson formula discussed in the previous section. Toe stone, typically, experiences less wave breaking energy due to the fact it is most often submerged below the water surface. Basically the most intense forcing impacts the structure above the toe stone. However, in an area where there is a significant variation in tidal elevation, as the water surface lowers and approaches the toe stone, it is exposed to higher forcing levels. The counter to this process, is if the waves are depth limited as the water surface lowers, the impacting wave height is decreased and therefore the energy impacting the structure and toe is decreased. To account for these variables several toe stone sizing formulations have been developed that use water depth, wave height, and damage level. For this analysis the equation developed by Burcharth et al. 1995, as provided in the USACE Coastal Engineering Manual (CEM) as equation VI-5-108, has been used. The details regarding the equation have been taken from the CEM and provided below as Equation 8-2. To work through the actual calculations of the equation, the interactive, or "live", version of the CEM was used. This version of the CEM is in electronic format and allows users to enter values into the equation and get answers. This was a valuable tool since it allowed numerous water level and wave height combinations to be run, to try and determine the maximum needed toe stone size by trying to determine the maximum load scenario with water depth and wave height. In the following sections the specific inputs for toe stone formulation will be discussed and the formulations will be provided in Section 8.8.2.

$$N_s = \frac{H_s}{\Delta D_{n50}} = \left(0.4 \frac{h_b}{\Delta D_{n50}} + 1.6 \right) N_{od}^{0.15} \quad \text{or} \quad \frac{H_s}{\Delta D_{n50}} = \frac{1.6}{N_{od}^{-0.15} - 0.4 h_b/H_s} \quad \text{Equation 8-2}$$

where H_s Significant wave height in front of breakwater
 Δ $(\rho_s/\rho_w) - 1$
 ρ_s Mass density of stones
 ρ_w Mass density of water
 D_{n50} Equivalent cube length of median stone
 h_b Water depth at top of toe berm
 N_{od} Number of units displaced out of the armor layer within a strip width of D_{n50} . For a standard toe size of about 3-5 stones wide and 2-3 stones high:

$$N_{od} = \begin{cases} 0.5 & \text{no damage} \\ 2 & \text{acceptable damage} \\ 4 & \text{severe damage} \end{cases}$$

For a wider toe berm, higher N_{od} values can be applied.



8.3 Design Wave Height

In order to determine the design wave height for the structures, several factors were considered, and the wave modeling results from WHG were used. Provided in Table 8-1 are the wave heights taken from several storm events that were modeled. For the breakwater structures the incident wave heights were determined by finding the existing condition wave heights 30 to 40 feet in front of the proposed structure location. This distance was based on empirical calculations for breaker distance taken from the USACE Coastal Engineering Manual. Just as in physical modeling of structures, wave conditions for the model were determined without the proposed structures in place since the modeled wave heights would include the reflected wave height moving seaward from the structure and would result in an artificially high incident wave height. For the spur structures adjacent to the North Jetty, the wave height measurement came from the numerical model with the structure in place since the reflected wave heights coming off the north jetty into the spur groin would be additive to the waves directly

impacting the spur groin and vice versa. As shown in the wave modeling performed by WHG this jetty and spur junction experiences a multi directional wave pattern and is much more turbulent than the “regular” incident wave conditions. This area of “turbulence” has been shown as Figure 8-1. The model results for various storm wave conditions were taken from CGWAVE and were provided as significant wave height or H_s . The historical and return period storms are discussed in Section 5.2.6.

Table 8-1. Design Wave Heights
Design Wave Heights (H_s)

| Alternative | Structure | 10-Year | | 50-Year | | Hurricane Bob | | Perfect Storm | | Nor'easter of 2001 | | Average | | Maximum | | Design Wave |
|-------------|--------------|----------|-----------|----------|-----------|---------------|-----------|---------------|-----------|--------------------|-----------|----------|-----------|----------|-----------|-------------|
| | | existing | w/project | existing | w/project | existing | w/project | existing | w/project | existing | w/project | existing | w/project | existing | w/project | |
| CASE 6 | Spur Jetty | 7.5 | 12.1 | 8.9 | 12.1 | 7.5 | 11.2 | 11.8 | 13.8 | 10.8 | 13.5 | 9.3 | 12.5 | 11.8 | 13.8 | 12.5 |
| | | 8.9 | 12.1 | 9.8 | 12.5 | 7.5 | 10.5 | 9.5 | 12.1 | 10.5 | 12.8 | 9.3 | 12.0 | 10.5 | 12.8 | 12.5 |
| CASE 25A | Breakwater 1 | 7.5 | 7.2 | 8.5 | 9.5 | 6.2 | 8.9 | 11.2 | 14.1 | 8.5 | 12.1 | 8.4 | 10.4 | 11.2 | 14.1 | 10.5 |
| | | 8.2 | 12.1 | 9.2 | 11.2 | 7.2 | 10.8 | 10.2 | 13.8 | 9.2 | 13.5 | 8.8 | 12.3 | 10.2 | 13.8 | 10.5 |

Wave Heights in feet

The design wave parameter of H_s was used for several reasons with the depth limited wave height being used as a design check. When Hudson’s Formula was developed, only monochromatic waves were used in physical models so the correlation between H in Hudson to an irregular wave parameter needed to be determined. Based on a personal conversation with Dr. Jeff Melby of ERDC-CHL and work performed by Bob Carver of ERDC-CHL, it was concluded that H_s was a reasonable wave height parameter to use for the design wave H in the Hudson’s formula. Additionally, as shown in Figure 8-2, the bathymetry seaward of project structures is fairly flat with an approximated slope of 1V:280H. This has a tendency to “filter” the wave signal, with significant shoaling and breaking occurring seaward of the proposed structures. This is evident in the model as well as observed personally at the project site. It would be incorrect to multiply H_s by 1.27 to get $H_{1/10}$ or by 1.67 to get H_{100} for use in the design since the distribution is skewed do to near shore wave breaking. It was also reasoned that these structures do not provide protection for critical navigation or infrastructure directly behind them.

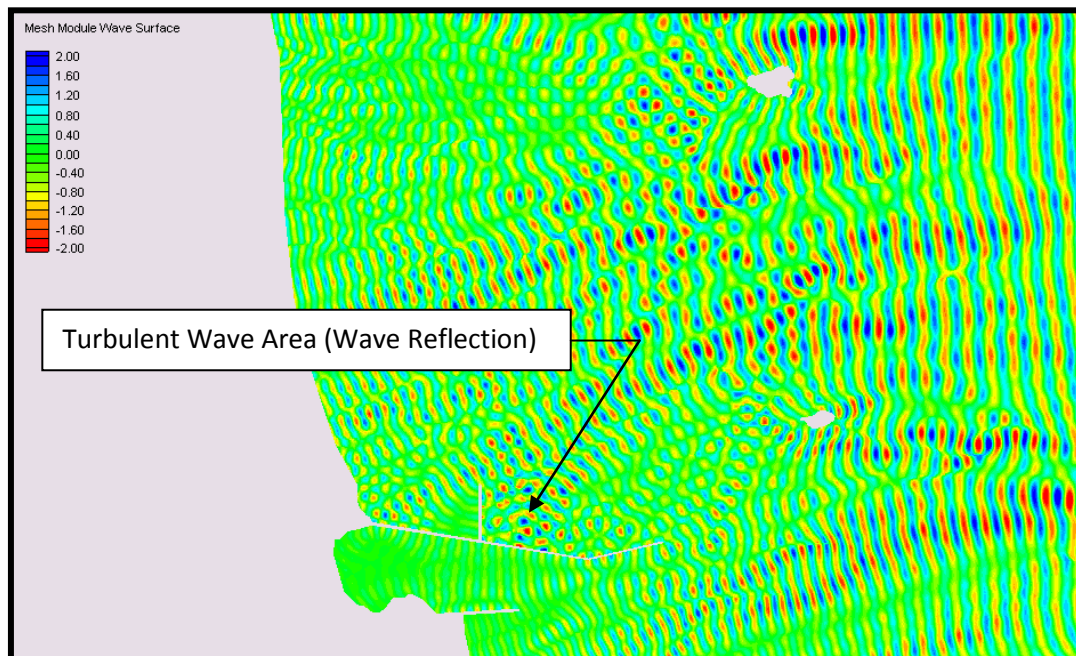


Figure 8-1. Numerical Wave Modeling Results for Alternative 6 – Spur Groin Wave “Turbulence”.

The purpose of the structures is to reduce the wave energy significantly, disrupt shore normal currents, and to reduce erosion. Even if the structures are damaged and are eventually reduced in crest elevation they will perform that function adequately. It also must be kept in mind that the local tide range of nearly 10 feet is a significant factor. With such a large tide range the duration of wave impact at the maximum level will be relatively short during storm events.

As a design wave height check, depth limited wave conditions were considered for each alternative. Water surface elevation was taken for a 10-year storm from Table 8-2, and water depth was determined using the bathymetry maps shown in Figures 8-2 to 8-5. As discussed in Section 5.2.6, where water level was critical to the analysis or design, the corresponding return period water level was used as well. It is understood the joint occurrence, or joint probability, of return period parameters such as waves and water levels results in an event that likely has a return period greater than the individual return periods. However, with nor'easters in New England, typically, if a storm is strong enough to generate a return period water level, the storms intensity is such, that significant wave energy will occur as well. The offshore wave height may not be at the same return period level, but the near shore waves are likely to be near maximum due to shoaling and depth limited wave conditions. The depth limited H_s wave height parameter was determined by multiplying 0.60 times the water depth. It was found, and as shown in Tables 8-11 to 8-20, that the depth limited waves are smaller than the design wave for the spur options, nearly the same for Breakwater 1 in Alternative 25A, and about 1.5 feet larger for Breakwater 2 in Alternative 25A. Consideration was given to using the depth limited wave condition for Breakwater 2 but it was decided that the wave model most likely gave the better answer. Additionally the multiplication factor of 0.60 for depth limited H_s is likely conservative since the range is typically considered 0.50 to 0.60. Given the relatively flat, wide bathymetry and the significant refraction occurring, the value is likely closer to the 0.50 multiplier. This would reduce the depth limited wave conditions to below the design wave conditions used in all cases.

8.4 Structure Toe/Base Elevation

The bottom elevations of the structures was taken from the latest bathymetry data available and included USACE LIDAR data and single beam/multi beam survey data collected by USACE and WHG. The bathymetry is shown in Figures 8-4 and 8-5.

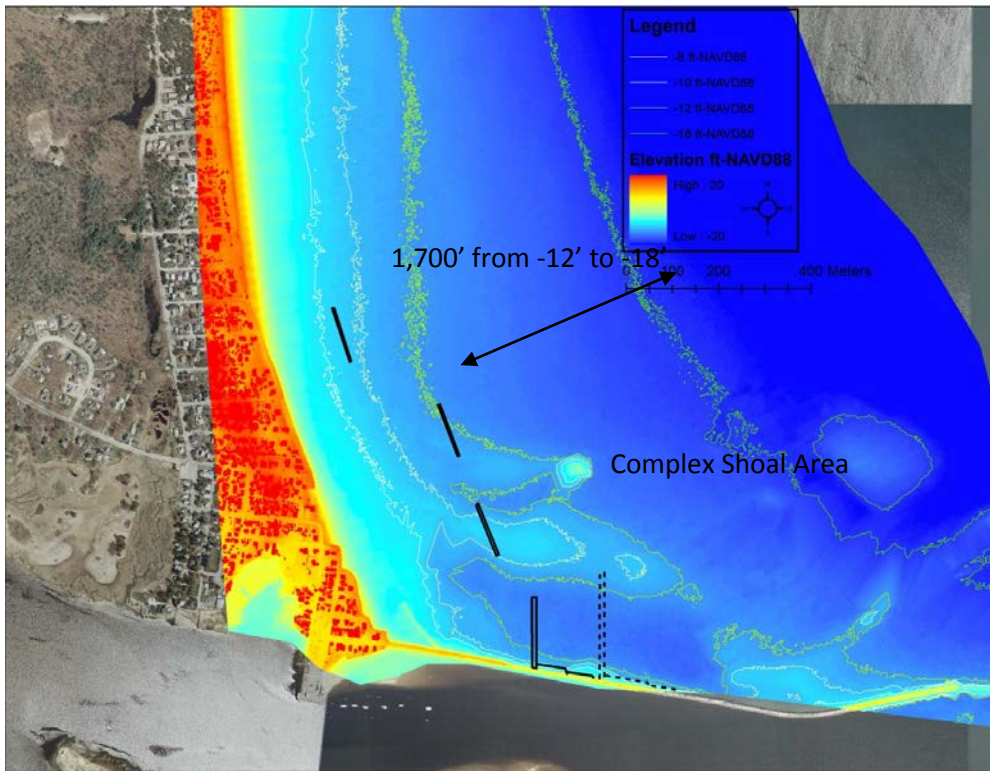


Figure 8-2. Bathymetry of study area.

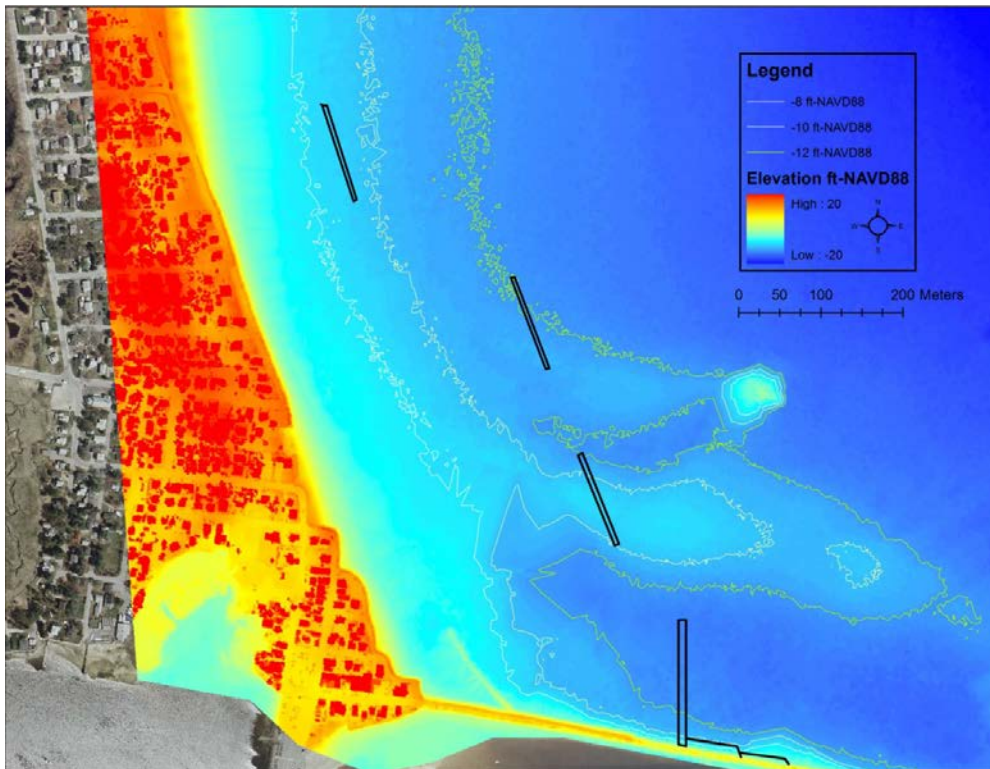


Figure 8-3. Alternative 25 with bathymetric map.

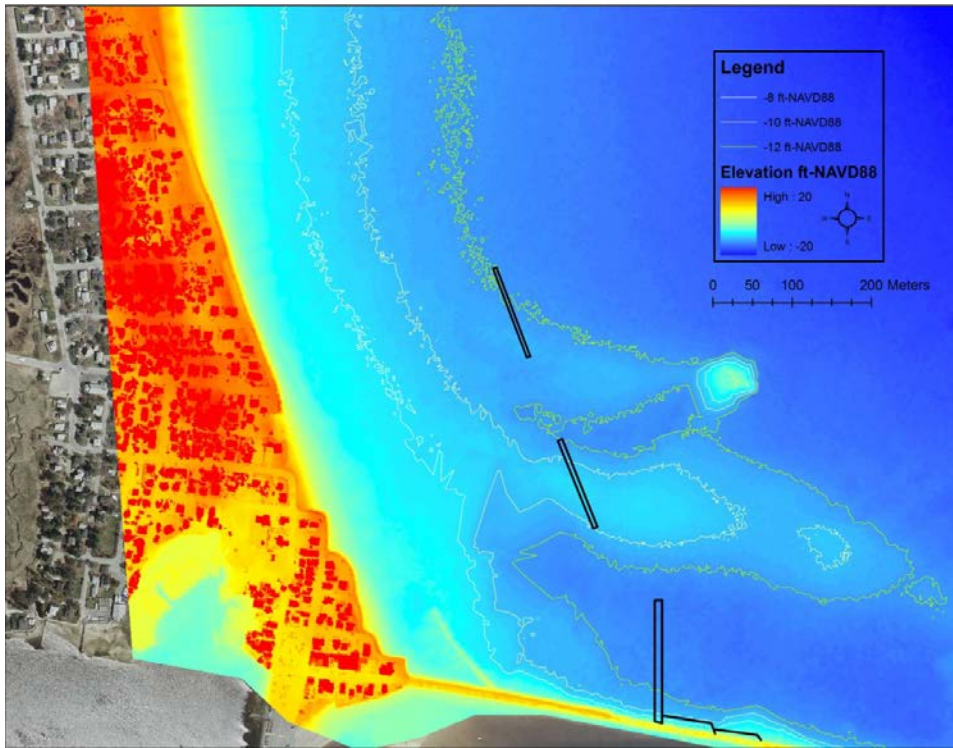


Figure 8-4. Alternative 25A with bathymetric map.

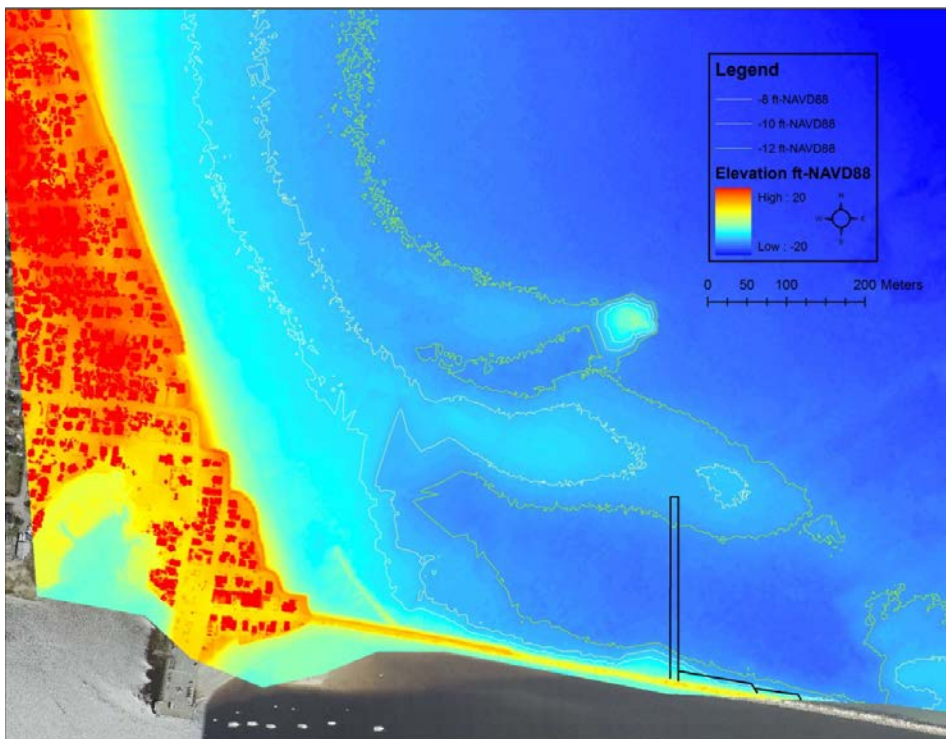


Figure 8-5. Alternative 6 with bathymetric map.

8.5 Structure Side Slopes

A structure slope of 1V:2H for both the front and back slope was ultimately selected after several iterations of structure slopes ranging from 1V:1.5H to 1V:3H for the spur groin structures in alternatives 6 and 25A. The first iteration of cross sectional design looked at 1V:1.5H but it was decided to go with a 1V:2H the shallower slope for the Spur Groin structures for the increased stone stability. This was reasoned to be especially important for the turbulent wave conditions and overtopping anticipated in the corner at the spur groin/North Jetty intersection. For the breakwater structures in Alternative 25A a 1V:2H slope was selected for the front slopes and a steeper slope of 1V:1.5H was chosen for the back side slope.

8.6 Crest Elevation

As part of the cross section design, it was necessary to select the crest elevation of the structures. The structure elevation was based upon return period and tidal water levels as well as shoreline erosion prevention. During the earlier design iterations t-head groins and the near shore breakwater alternatives were designed using the available literature from USACE and others, regarding structure length, gap distance, and distance from shore. Within that guidance was general recommendations for crest elevation but that was limited. From the literature, crest elevation appeared to be one of the lesser defined design criteria, and that the other criteria were of much more importance for beach behavior behind the structures. However, as discussed in Section 6, the final alternatives were designed outside of the better defined t-head groin/near shore breakwater design methods and instead used the CGWAVE numerical model, worked around the complicating subsurface geotechnical properties, and tried to work with the Maine coastal structure policy. As described in detail in Section 7.2.4 of this appendix and in greater detail in Appendix C Section 12.5.4, during the structural design effort and the beach impact effort, specifically salient formation, crest elevation was not considered. In researching USACE design guidance and other literature sources, a relevant USACE technical note was found - ERDC/CHL CHETN-II-45 March 2002 "Wave Transmission at Detached Breakwaters for Shoreline Response Modeling". The technical note describes a method for using wave transmission coefficients for shallow crested/submerged structures along with the GENESIS long shore transport model, and this approach was considered, but due to the relatively significant level of effort and need for additional modeling, this approach was ultimately not chosen. Time and funding played a large part in that decision, but also the consideration that the performance of the beach fill behind the structures was already estimated. Instead a more limited approach was used, as described in the following sections.

8.6.1 Crest Elevation – Tide Range and Wave Transmission Discussion

As previously discussed the tide range at the project is significant with a MLLW to MHHW range of 9.7 feet and an annual return period storm water elevation of 11.70 ft-MLLW or 6.56 ft-NAVD88. With the purpose of the structures to slow erosion of the beach fill component of the proposed alternatives, determining a design elevation was not straightforward. This meant that designing the crest was not as simple as calculating what crest elevation would allow a particular overtopping volume or allow a certain percentage of wave energy to pass. As a first cut, three crest elevations were investigated, 6 foot NAVD88, 9 foot NAVD88, and 12 ft-NAVD88, which were designated as the low, intermediate, and

high crest options, respectively. As can be seen in Table 8-2 the low crest would be above the MHHW line, but would be submerged during a 1 year storm, the intermediate would be approximately 1 foot above the 10 year storm, and the high option would be over 3 feet higher than the 100 year storm. In order to get some sense of storm performance, wave transmission was looked at by plotting the wave transmission coefficient (C_t) for a 7 foot and 10 foot wave height versus freeboard. Wave transmission formula by van der Meer and d'Angremond (1991) for rock armored low-crested, submerged, and reef breakwaters was used to determine wave transmission. These formulas were provided as Table VI-5-15 and Figure VI-5-54 in the USACE Coastal Engineering Manual (CEM) and for convenience the “live” equation version of the CEM was used to perform the calculations. The resultant calculations for the transmission coefficients have been provided as a plot in Figure 8-6. The 10 foot wave height is close to the design wave heights used for the structures as discussed in Section 8.3.

Table 8-2. Tide and Return Period Storm Water Elevations

| Tidal datums at CAMP ELLIS, SACO RIVER based on: | | | |
|--|-----------------------------|-----------|-------------|
| LENGTH OF SERIES: | 3 Months | | |
| TIME PERIOD: | July 1991 - September 1991 | | |
| TIDAL EPOCH: | 1983-2001 | | |
| CONTROL TIDE STATION: | 8418150 PORTLAND, CASCO BAY | | |
| Elevations of tidal datums referred to Mean Lower Low Water (MLLW) | | | |
| | MLLW (m) | MLLW (ft) | NAVD88 (ft) |
| 100-YEAR WATER LEVEL¹ = | 4.27 | 14.00 | 8.86 |
| 50-YEAR WATER LEVEL¹ = | 4.21 | 13.80 | 8.66 |
| 10-YEAR WATER LEVEL¹ = | 3.99 | 13.10 | 7.96 |
| 1-YEAR WATER LEVEL¹ = | 3.57 | 11.70 | 6.56 |
| MEAN HIGHER HIGH WATER (MHHW) = | 2.95 | 9.68 | 4.54 |
| MEAN HIGH WATER (MHW) = | 2.82 | 9.25 | 4.11 |
| North American Vertical Datum (NAVD88)⁵ = | | 5.14 | 0.00 |
| MEAN SEA LEVEL (MSL) = | 1.47 | 4.82 | -0.32 |
| MEAN TIDE LEVEL (MTL) = | 1.46 | 4.79 | -0.35 |
| MEAN LOW WATER (MLW) = | 0.10 | 0.33 | -4.81 |
| MEAN LOWER LOW WATER (MLLW) = | 0.00 | 0.00 | -5.14 |
| | Feet | + MHW | + MHHW |
| 2-year storm surge ² = | 2.04 | 11.29 | 11.72 |
| 10-year storm surge ² = | 3.46 | 12.71 | 13.14 |
| 50-year storm surge ² = | 4.16 | 13.41 | 13.84 |
| 100-year storm surge ² = | 4.56 | 13.81 | 14.24 |
| ¹ water level data taken from New England Tidal Survey | | | |
| ² data provided by WHG | | | |
| ³ water depth at proposed breakwater location is 9.90' MLLW | | | |
| ⁴ water depth at proposed spur groin locaiton is 5.35' MLLW | | | |
| ⁵ NAVD88 elevation determined from Portland Benchmark. Set relative to MTL. | | | |
| Sea Level Rise 0.63 feet/century (NOAA) | | | |

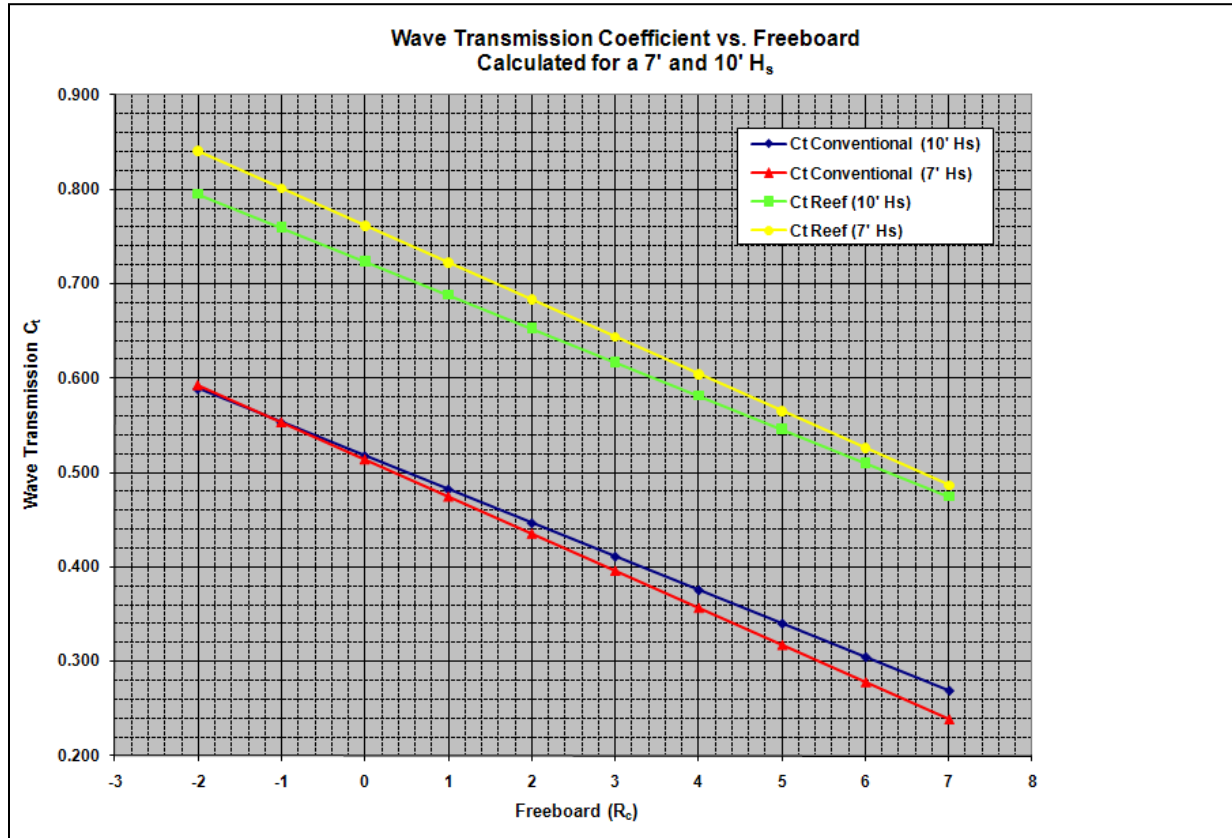


Figure 8-6. Wave transmission coefficient (C_t) vs. freeboard.

Looking at Figure 8-6 and Table 8-2, the low crested structure with an elevation of 6 feet NAVD88, and as a conventional breakwater, would allow 52% of the wave energy to pass on a 1 year return period storm i.e. 6.56 foot NAVD88 water elevation for a 1 year storm (0 ft freeboard). Significantly more energy passes with larger return period storms. For the intermediate crest elevation the wave transmission would be dropped down to approximately 42% for the 1 year return period storm (2.5 foot freeboard) and to 48% for a 10 year return period storm (1 ft freeboard). The high crest option at 12 ft-NAVD88 would allow 32% during the one year return period storm (5.5 ft freeboard) and 38% during a 10 year return period storm (4 ft freeboard).

To provide additional information to help select a crest elevation two more pieces of information were used related to water level. The first is the tide range that has been mentioned. The return period water elevations are for the maximum water elevations and therefore the duration of the storm at those elevations will be relatively short. Considering the 9 foot mean tide range, even with a storm surge from a strong nor'easter of 3 feet, the amount of time the water will be at the design elevation will be a matter of a few hours. If one considers a "typical" nor'easter with a storm duration of a few tide cycles, the number of hours at the higher elevations will be relatively short compared to the rest of the storm. The rest of the time the breakwaters will have a much greater freeboard and therefore a greater level of protection (Figure 8-7). The second consideration given to design water levels was the potential impact for wave and wind set up in the project area due to the corner created by the North Jetty and the shoreline. The water levels shown in Table 8-2 are from the Portland NOAA tide gage,

NOAA tide prediction transformations, and the New England Tidal Profile study from 1988. Within all of that information, there is no site specific information given for this project corner area. To achieve that level of detail a combination wave and hydrodynamic model would need to be run with a very fine spatial resolution which was not done for this project. Instead, it was decided to assume setup in the corner does occur based upon conventional coastal engineering science and wisdom.

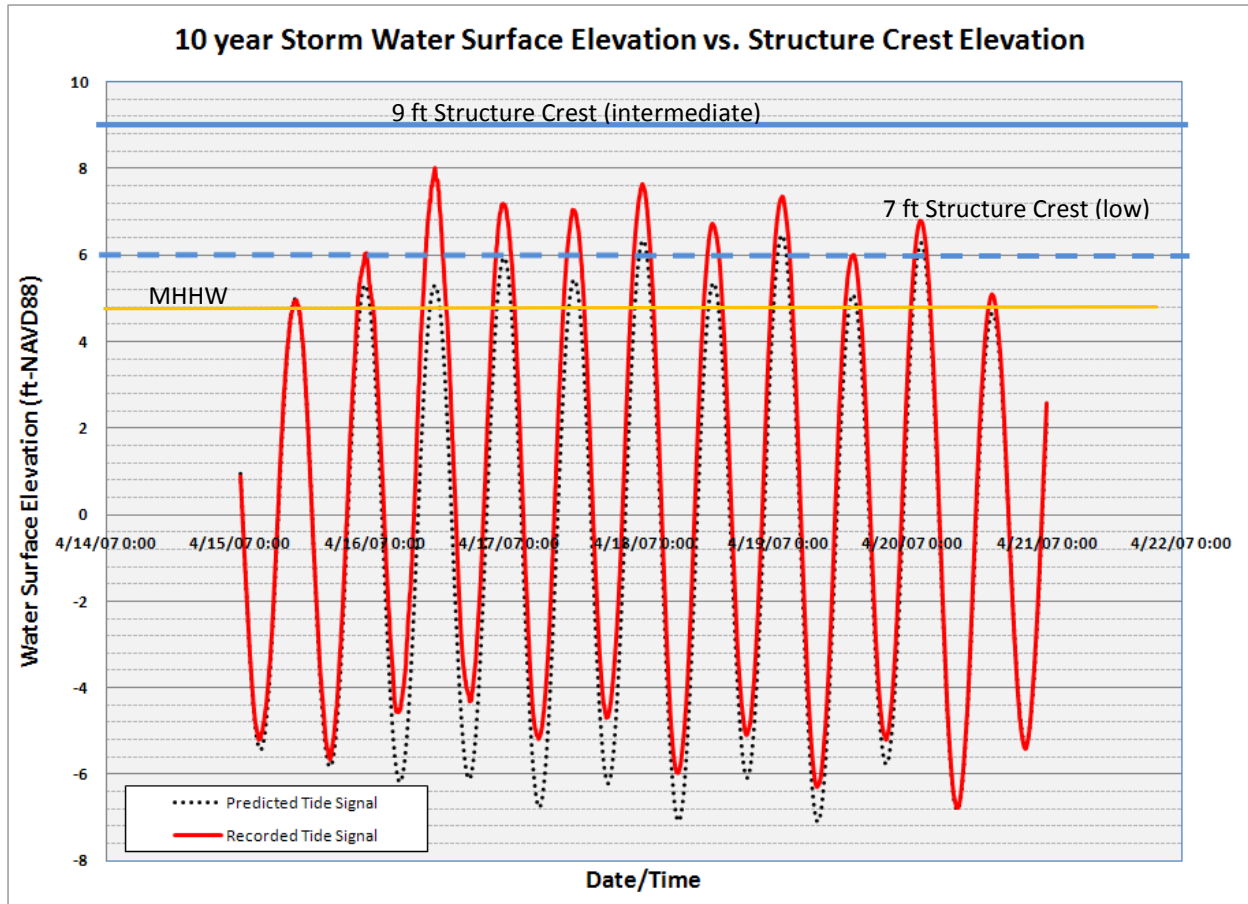


Figure 8-7. 10 year storm water elevation plot vs. crest elevation.

8.6.2 Crest Elevation – Near Shore Breakwater Design Consideration

As discussed, near shore breakwater design guidance focuses mostly upon structure length, spacing, and distance from shore. The discussion and research for optimal structure height versus shoreline protection appears to be limited, with perhaps the most applicable information found being the USACE technical note discussed earlier which required the use of the GENESIS numerical model. The available literature did state that lower crested structures would result in less protection, reduce the chance of tombolos, or reduce the size of salients. However, there was no information found saying what “low” crested was defined and similarly the term “emergent” breakwater is used without a definition provided. As discussed in Section 8.6, the Woods Hole Group did estimate salient growth behind the structures using the discussed formulations, but those calculations, due to the formulations used, did not address necessary crest elevation relative to that analysis. In the latest guidance found from the United Kingdom’s Department for Environment Food and Rural Affairs (DEFRA) Project SC060026/RI

2010 “Guidance for outline design of near shore detached breakwaters on sandy macro tidal coasts”, a figure relating crest elevation to salient formation was provided. The graph, which is provided below as Figure 8-8 is based on relative *submergence* of the crest. For the structures being considered the medium and high crest elevations were considered “emergent” above the MHHW and 10 year storm events which made the figure non applicable. The tests used for the results in Figure 8-8 were also done for a tide range to H_{m0} ratio of 2.5, which is not the case for this study. As shown in Section 8.6, the MHHW to MLLW tide range in the study area is 9.7 feet resulting in a tide range to H_{m0} ratio closer to 1. With this it was determined the DEFRA figure was not directly applicable to any of the alternative crest elevations being considered. However, the fact that the intermediate to high crest elevations were “emergent” suggests more than adequate structure elevation.

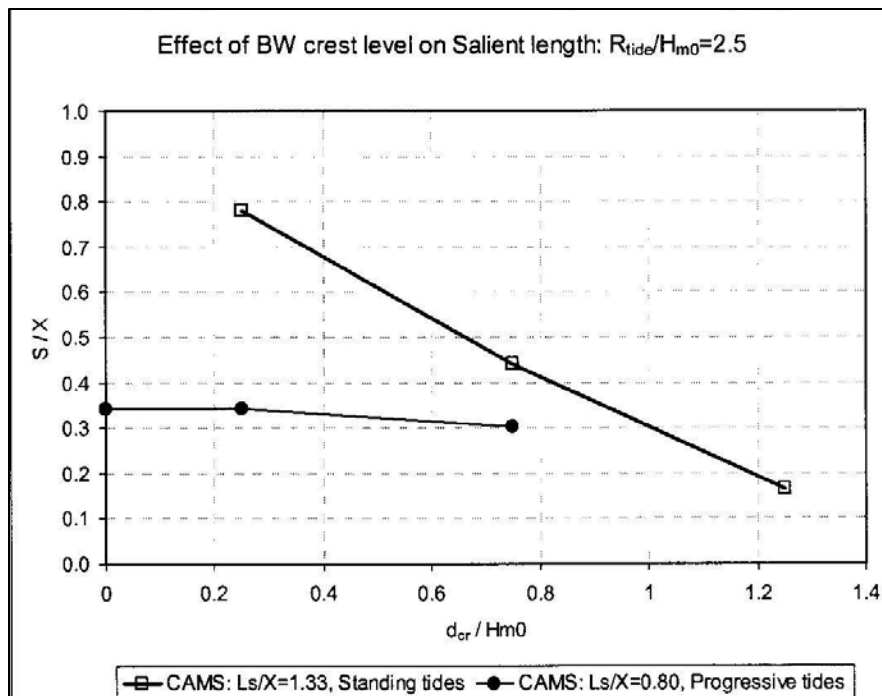


Figure 8-8. Effect of breakwater crest level (relative submerged depth at high water, d_{cr}/H_{m0}) for different breakwater length ($R_{tide}/H_{m0}=2.5$). Taken from DEFRA Project SC060026/RI 2010

Additional consideration was given to the distance that the near shore breakwaters were to shore. The available literature and design guidance contained within it would have the near shore breakwaters closer to shore than presently designed. For reasons already discussed, the structure layout was governed by geotechnical issues, wave modeling results, and permitting. With the structures located further offshore, the expected protection of the beach fill and therefore the performance of the beach fill was anticipated to be lower than breakwaters closer to shore. Based upon this, it was decided to keep the crest elevation higher. For details regarding beach performance with the various alternatives the reader is directed to Sections 9 and 10.

8.6.3 Crest Elevation – Foundation Settlement Consideration

As discussed in Section 7, foundation issues were a significant concern during the design of the structure layout, as well as the cross sectional design. Based on the work from the Geotechnical Section

(Appendix D) it was determined the proposed structures for Alternatives 6 and 25A would be stable and not experience foundation failure. It was also determined in the Geotechnical analysis that with proper foundation construction the breakwater structures and spur groins should not settle significantly. However, for conservatism, if two (2) feet of settlement does occur the intermediate crest elevation that was always above the 10 year storm water level becomes submerged part of the time. This can be seen in Figure 8-7.

8.6.4 Crest Elevation – Recommendation

Given all of the information provided and through consultation with the PDT, the intermediate crest elevation of 9 ft-NAVD88 was chosen and the reasons are summarized in list form below. All structures in each alternative were designed for the same elevation with the exception of reinforcement of the North Jetty. The details of this reinforcement have been included in Section 8.7.

1. Geotechnical loading conditions were a significant concern and keeping the crest elevation as low as possible to avoid overloading the foundation was a goal.
2. The fact that critical infrastructure or navigation was not being protected, wave transmission during the peak of a storm was considered acceptable.
3. Based on the available literature, especially the latest DEFRA document, it was concluded the high crest option was not needed.
4. The short duration of the elevated water surface during a storm, given the large tide range, was considered and provided further confidence the high crest elevation option was not necessary.
5. The low crested option was not selected due to the potential for frequent overtopping during storms, including annually occurring events. This concern was magnified by the likely occurrence of localized setup due to waves and wind current into the North Jetty/shoreline corner.
6. Information found during the literature search of detached breakwaters was interpreted as having structures that are submerged/or nearly submerged during “design” conditions for short periods is normal however higher crests will perform better at retaining sand. Based on the information available it was concluded the intermediate option satisfies those conditions.
7. The breakwater layout was not designed using typical near shore breakwater criteria, but instead based upon wave modeling results, geotechnical foundation issues, and the desire to keep the structures as far offshore as possible to address permitting agency concerns. Based upon these reasons, the breakwaters were evaluated having a crest elevation as high as possible without being too conservative, and that did not cause foundation loading problems.
8. The potential for foundation settlement lead to lowering the overall structure height but was balanced against maintaining adequate height for minimizing overtopping

8.7 North Jetty Reinforcement and Toe Protection

As mentioned in Section 8.3, the wave climate in the corner created by the spur groin structures in Alternatives 6 and 25A is expected to be elevated and extra turbulent compared to the more normal wave trains impacting the project area. As shown in Figure 8-1, this turbulence is denoted by the waffle pattern in this corner which is created by waves hitting the North Jetty, reflecting off it, and then

impacting the spur groin and also from waves impacting the Spur Groin, reflecting off of it, and then hitting the North Jetty. As discussed in Section 8.3, the design wave height for the spur groins in the final alternatives was increased versus the design wave heights used for the breakwater components in those alternatives. It was decided that the North Jetty required reinforcement due to this increase in wave climate. This reinforcement included using larger armor stones than already exist, shallowing the back side slope from a 1V:1.5H to a 1V:2:H, to increase armor stability, raising the crest elevation, and reinforcing the toe protection to prevent scour. The crest elevation was increased to prevent a large increase in overtopping.

The North Jetty will also be reinforced. The existing elevation of the North Jetty, where the spur groins would connect, is 7 ft-NAVD88. With the spur groin components being 9 ft-NAVD it was considered likely that more wave energy would be forced to overtop the North Jetty. Basically it would be the weak spot or relief valve for the increased wave energy. This would potentially cause damage to the North Jetty, and could impact navigation in the federal channel between the jetties. To address this issue the North Jetty slope will be reinforced with larger stone along with increasing the crest elevation to match the spur groin structure. In addition to the increased jetty slope protection, increased toe protection was provided. This was included in the area of extra slope protection and extended 200 feet beyond the slope protection. The length of North Jetty that was afforded this extra protection was determined from the CGWAVE numerical modeling effort with video images similar to Figure 8-1. Ideally, a physical model could have been used to confirm this design feature, and a physical model was recommended but the request was turned down due to cost and schedule implications. The plan form of the added protection can be seen in Figures 8-9 and 8-10 with the cross sectional views provided in Section 8.8. It should be noted the reinforcement is the same for each alternative and for each structure crest height option.



Figure 8-9. Alternative 6 plan form layout – with North Jetty reinforcement



Figure 8-10. Alternative 25A spur groin plan form layout – with North Jetty reinforcement.

8.8 Stone Size Determination

8.8.1 Armor Stone Size Determination

As discussed in Section 8.2, the Hudson formula was used to determine the design stone weight. In the previous sections, the design input values into the equation have been discussed and provided. The worksheets used for each structure, for each of the final alternatives have been provided in Figures 8-11 to 8-20. It worth noting that the Figures 8-11 to 8-20 are the worksheets used for stone size calculations and rough volume determinations and are not for construction purposes or for use in determining final stone/material quantities. Once again they are simply calculation worksheets. The stone size formulations have been provided for Alternatives 6 and 25A for the intermediate options. The high crest option would have the same stone sizes and layout, with the only exception being a higher crest. As seen in the worksheet figures, there are separate worksheets for the structure heads and for the reinforcement armor included along the north jetty seaward of the spur groin structures. Head sections of structures typically require larger/heavier stone than the trunk section of rubble mound structures due to more severe wave forcing. Waves have a tendency to be focused on corner and end features of structures as well as impart different forcing mechanisms to the structure. As a wave moves over the end slope, the orbital velocity of the wave in the water column creates an uplift force, which causes additional instability for the stone. Due to the focusing and additional forces experienced at the head of structures, the Hudson Formula uses a lower stability constant, K_d , and therefore the calculation increases the stone size at the head of the structure versus the trunk slope of the structure.

8.8.2 Toe Stone Size Determination

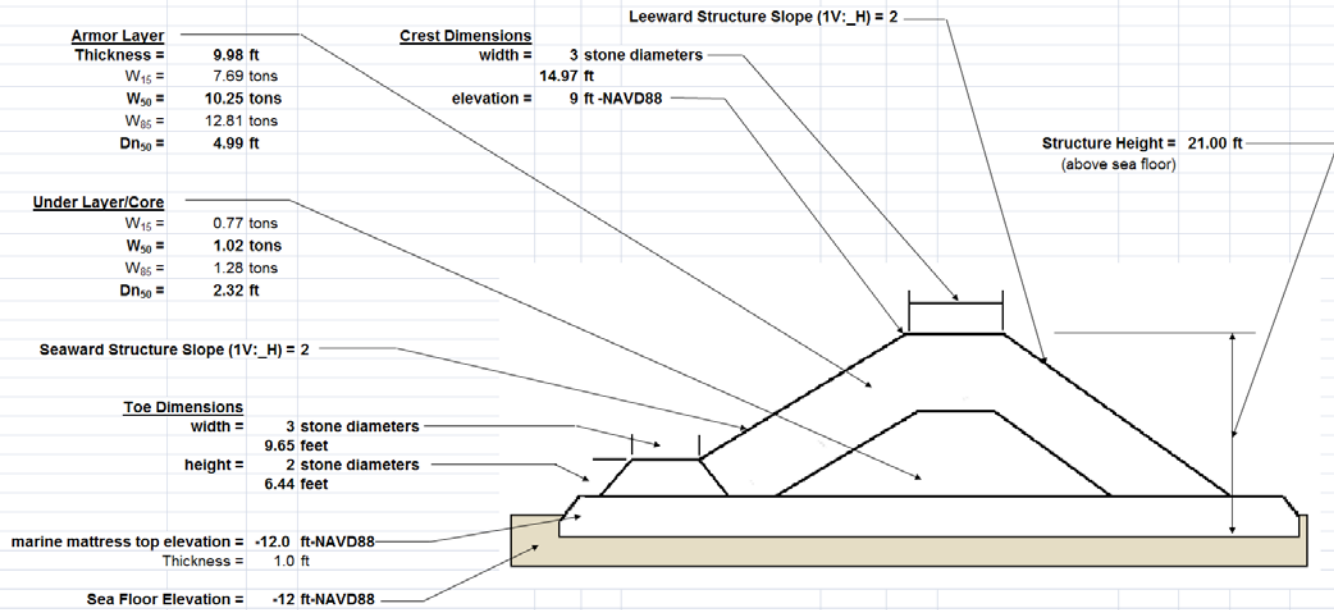
As discussed in Section 8.2.2 Equation VI-5-108 from the CEM was used to determine toe stone size. Due to the interaction of water depth, wave height, and toe stone elevation, an iterative process was undertaken to determine the appropriate toe stone size. As discussed in Section 8.2.2 the interactive version of the CEM was used to perform the calculations. The wave and water level information from the previous sections were used as boundary condition inputs with a range of intermediate inputs used to cover various forcing levels. As water level decreases, wave height decreases but the wave energy is closer to the toe, so to stone size increases. The maximum toe stone size found in this iterative process was selected for conservatism. Provided below in Table 8-3 is the worksheet output for the maximum stone size. The water depth above the toe (7.5 feet) and wave height (10 feet) was used (as highlighted in red) along with the no damage state. No damage was used for conservatism. The stone size was provided in D_{n50} , which was then converted to W_{50} , since weight is often the design specification used in armor stone design work. The calculated W_{50} was 5,407 pounds or 2.7 tons. The weight was rounded up to 5,500 pounds or 2.75 tons for convenience and the resulting diameter was then back calculated for structure design (3.22 feet). The final resulting stone size and diameter have been included in the structure worksheets provided in Figures 8-11 to 8-20.

Table 8-3. Toe stability/sizing results table

| Table VI-5-47/Equation VI-5-108 Stability of 2-layer toe berm (Burcharth) | | | | | | | |
|---|-------------------------------|-----------------------|---------------------------------------|---------|-----------------------------------|---------|--|
| Input Parameter | Value | Units | | | | | |
| Mean density of water (ρ_w) | 1.98902 | slugs/ft ³ | | | | | |
| Water depth at top of toe berm (Hb) | 7.5 | ft | | | | | |
| Rock density (ρ_s) | 5.14234 | slugs/ft ³ | | | | | |
| No damage | 0.5 | | | | | | |
| Acceptable damage | 2 | | | | | | |
| Severe damage | 4 | | | | | | |
| H_s (ft) | D_{n50} (ft) (no damage) | N_s | D_{n50} (ft) (acceptable damage) | N_s | D_{n50} (ft) (severe damage) | N_s | |
| 6 | 1.44187 | 2.6248 | 0.949113 | 3.98753 | 0.738598 | 5.12406 | |
| 7 | 1.8793 | 2.34949 | 1.30441 | 3.38496 | 1.05881 | 4.17013 | |
| 8 | 2.31672 | 2.17815 | 1.65972 | 3.04038 | 1.37903 | 3.65921 | |
| 9 | 2.75415 | 2.06123 | 2.01502 | 2.81731 | 1.69924 | 3.34086 | |
| 10 | 3.19158 | 1.97636 | 2.37032 | 2.66112 | 2.01946 | 3.12346 | |
| 11 | 3.629 | 1.91195 | 2.72562 | 2.54565 | 2.33967 | 2.96557 | |

Alternative 6 (intermediate crest elevation)

750 Foot Spur Groin (trunk section)



General Information

| | |
|--|-----------------|
| Bottom Elevation at Structure | -12.0 ft-NAVD88 |
| Design Tide Elevation | 8.0 ft-NAVD88 |
| Depth Limited Wave Height (0.60*water depth) | 12.0 ft |
| Crest Elevation | 9.0 ft-NAVD88 |
| Length of Structure | 700.0 ft |

Stone Weight Calculation

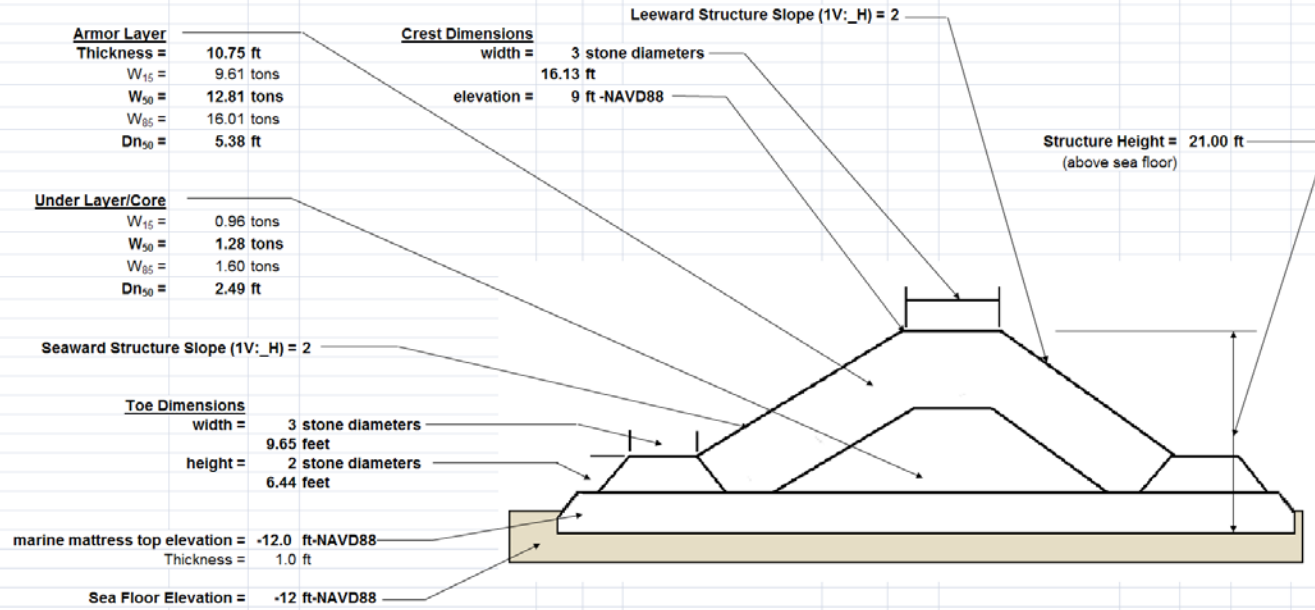
| | |
|------------------------------|---------------------------|
| Wave Height (H) | 12.5 ft |
| Specific Gravity (wr) | 165.0 lbs/ft ³ |
| Stability Coefficient (K) | 2.0 |
| Seaward Structure Slope (1V) | 2.0 |
| Leeward Structure Slope (1V) | 2.0 |
| Density of Water | 64.0 lbs/ft ³ |
| Bedding Layer Thickness | 0.0 ft |

| Armor Stone | Under Layer/Core Stone | Toe Stone |
|--|--|--|
| Armor Weight = 20,499 lbs = 10.25 tons | Underlayer Weight = 2,050 lbs 1.02 tons | Toe Stone Weight = 5,500 lbs = 2.75 tons |
| Cross Sectional Area of Armor Layer | Cross Sectional Area of Underlayer | Cross Sectional Area of Toe |
| Dn50 of Armor Stone 4.99 ft | Dn50 of Underlayer 2.32 ft | Dn50 of Toe 3.22 ft |
| Structure Height (above bedding) 21.00 ft | Height of Underlayer 11.02 ft | Toe Crest Width (# of units) 3 |
| Crest Width (# armor units) 3 | Underlayer Crest Width 9.98 ft | Toe Thickness (# of units) 2 |
| Armor Layer Thickness (# armor units) 2 | Underlayer Thickness (# of units) 2 | |
| Armor Layer Cross Sectional Area = 1,087 ft² | Cross Sectional Area = 353 ft² | Cross Sectional Area = 104 ft² |

Figure 8-11. Alternative 6 Spur Groin Trunk – intermediate crest elevation

Alternative 6 (intermediate crest elevation)

750 Foot Spur Groin (head section)



General Information

| | |
|--|-----------------|
| Bottom Elevation at Structure | -12.0 ft-NAVD88 |
| Design Tide Elevation | 8.0 ft-NAVD88 |
| Depth Limited Wave Height (0.60*water depth) | 12.0 ft |
| Crest Elevation | 9.0 ft-NAVD88 |
| Length of Structure | 50.0 ft |

Stone Weight Calculation

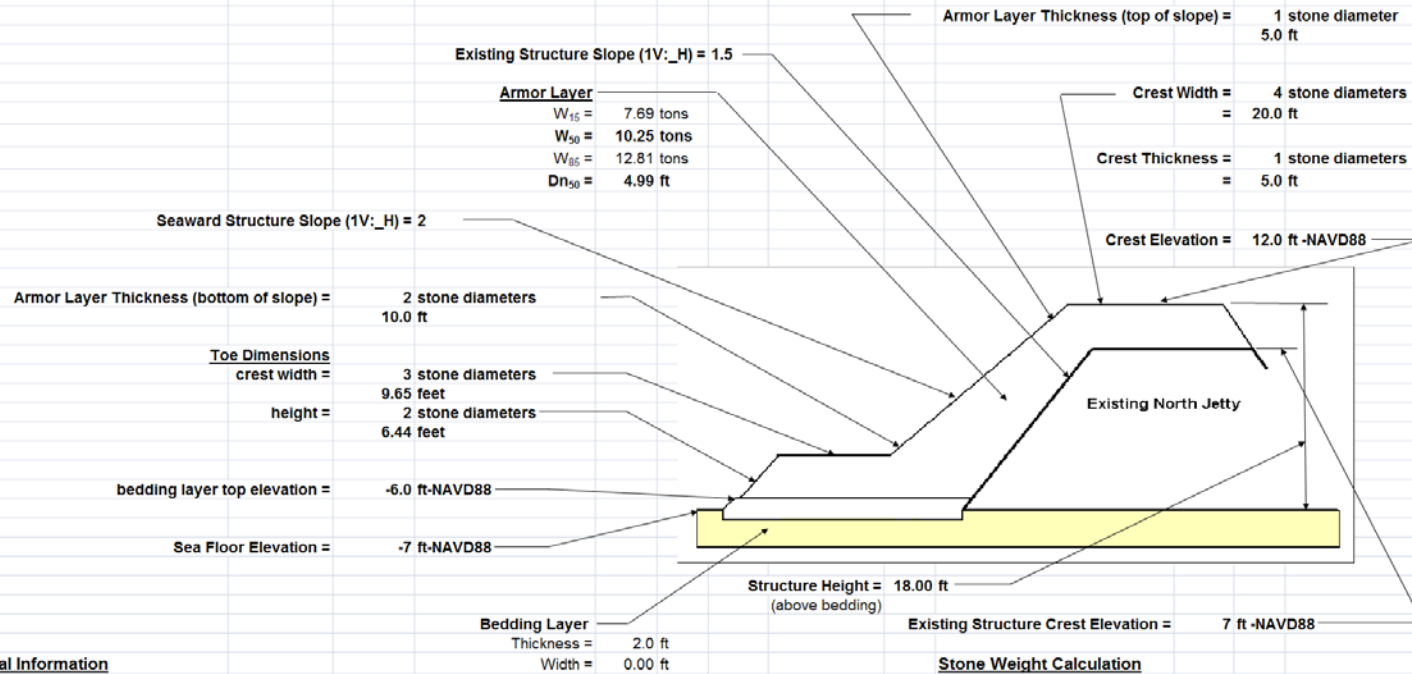
| | |
|------------------------------------|---------------------------|
| Wave Height (H) | 12.5 ft |
| Specific Gravity (w _r) | 165.0 lbs/ft ³ |
| Stability Coefficient (K) | 1.8 |
| Seaward Structure Slope (1V) | 2.0 |
| Leeward Structure Slope (1V) | 2.0 |
| Density of Water | 64.0 lbs/ft ³ |
| Bedding Layer Thickness | 0.0 ft |

| Armor Stone | Under Layer/Core Stone | Toe Stone |
|--|--|---|
| Armor Weight = 25,624 lbs = 12.81 tons | Underlayer Weight = 2,562 lbs 1.28 tons | Toe Stone Weight = 5,500 lbs = 2.75 tons |
| Cross Sectional Area of Armor Layer | Cross Sectional Area of Underlayer | Cross Sectional Area of Toe |
| Dn50 of Armor Stone 5.38 ft | Dn50 of Underlayer 2.49 ft | Dn50 of Toe 3.22 ft |
| Structure Height (above bedding) 21.00 ft | Height of Underlayer 10.25 ft | Toe Crest Width (# of units) 3 |
| Crest Width (# armor units) 3 | Underlayer Crest Width 10.75 ft | Toe Thickness (# of units) 2 |
| Armor Layer Thickness (# armor units) 2 | Underlayer Thickness (# of units) 2 | |
| Armor Layer Cross Sectional Area = 1,183 ft ² | Cross Sectional Area = 320 ft ² | Cross Sectional Area = 104 ft ² |

Figure 8-12. Alternative 6 Spur Groin Head – intermediate crest elevations

Alternatives 6 and 25A (intermediate and high crest elevation)

North Jetty Armor Reinforcement



General Information

| | |
|-------------------------------|----------------|
| Bottom Elevation at Structure | -7.0 ft-NAVD88 |
| Design Tide Elevation | 8.0 ft-NAVD88 |
| Depth Limited Wave Height | 9.0 ft |
| Crest Elevation | 12.0 ft-NAVD88 |
| Length of Structure | 200 ft |

Stone Weight Calculation

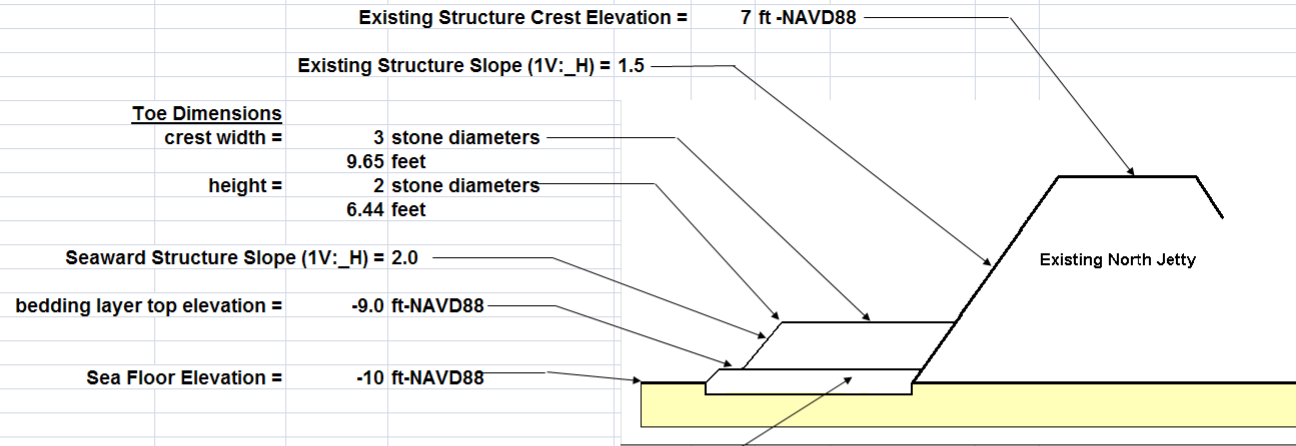
| | |
|--------------------------------|---------------------------|
| Wave Height (H) | 12.5 ft |
| Specific Gravity (wr) | 165.0 lbs/ft ³ |
| Stability Coefficient (K) | 2.0 |
| Seaward Structure Slope (1V:H) | 2.0 |
| Leeward Structure Slope (1V:H) | 1.5 |
| Density of Water | 64.0 lbs/ft ³ |
| Bedding Layer Thickness | 2.0 ft |

| Armor Stone | | Toe Stone | |
|--|-----------------------|------------------------------------|-----------------------|
| Armor Weight = | 20,499 lbs | Toe Stone Weight = | 5,500 lbs |
| = | 10.25 tons | = | 2.75 tons |
| Cross Sectional Area of Armor Layer | | Cross Sectional Area of Toe | |
| Dn50 of Armor Stone | 4.99 ft | Dn50 of Toe | 3.22 ft |
| Structure Height (above bedding) | 18.00 ft | Toe Crest Width (# of units) | 3 |
| Crest Width (# armor units) | 4 | Toe Thickness (# of units) | 2 |
| Armor Layer Thickness (# armor units) | 1 | | |
| Armor Layer Cross Sectional Area = | ft² | Cross Sectional Area = | ft² |

Figure 8-13. Alternatives 6 and 25A North Jetty Reinforcement –intermediate and high crest elevations

Alternatives 6 and 25A (intermediate and high crest elevation)

North Jetty Toe Reinforcement



General Information

| | |
|-------------------------------|-----------------|
| Bottom Elevation at Structure | -10.0 ft-NAVD88 |
| Design Tide Elevation | 8.0 ft-NAVD88 |
| Depth Limited Wave Height | 10.8 ft |
| Length of Structure | 200 ft |

Stone Weight Calculation

| | |
|---------------------------------|---------------------------|
| Wave Height (H) | 10.0 ft |
| Specific Gravity (wr) | 165.0 lbs/ft ³ |
| Stability Coefficient (K) | 2.0 |
| Seaward Structure Slope (1V:_H) | 2.0 |
| Leeward Structure Slope (1V:_H) | 1.5 |
| Density of Water | 64.0 lbs/ft ³ |
| Bedding Layer Thickness | 2.0 ft |

Toe Stone

| | |
|--------------------|-----------|
| Toe Stone Weight = | 5,500 lbs |
| = | 2.75 tons |

Cross Sectional Area of Toe

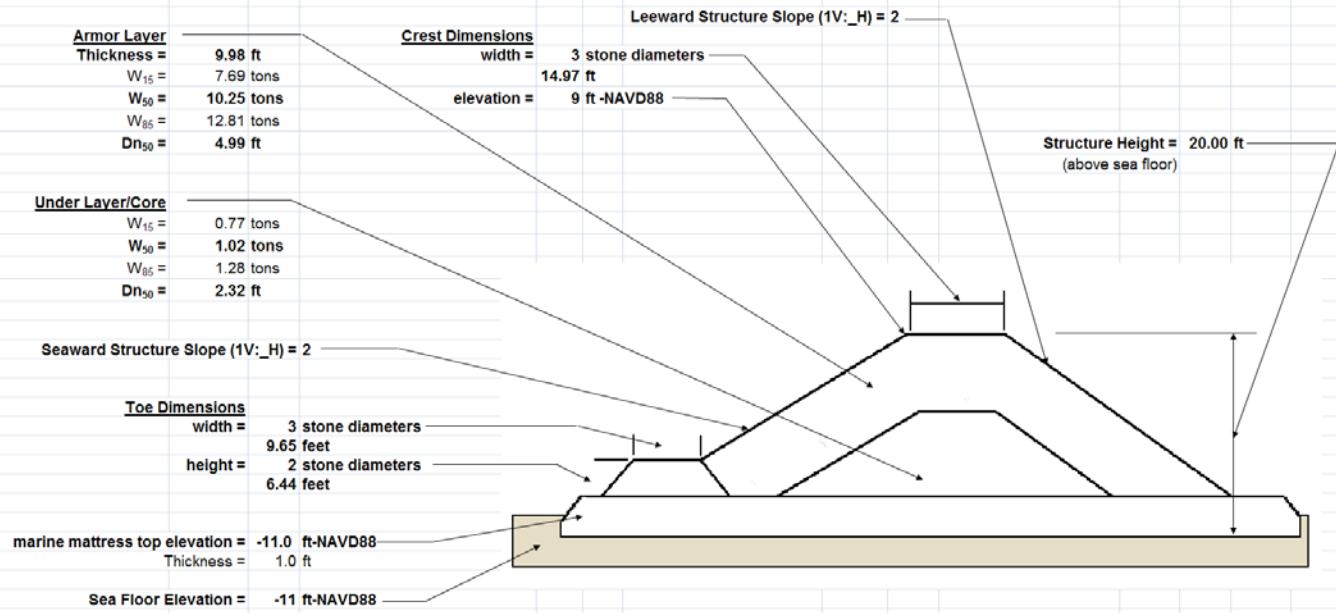
| | |
|---------------------------------------|---------|
| Dn50 of Toe | 3.22 ft |
| Toe Crest Width (# of units) | 3 |
| Toe Thickness (# of units) | 2 |
| Armor Layer Thickness (# armor units) | 2 |

| | |
|---|--------------------------|
| Armor Layer Cross Sectional Area = | 73 ft² |
|---|--------------------------|

Figure 8-14. Alternatives 6 and 25A North Jetty Toe Reinforcement – intermediate and high crest elevations

Alternative 25A (intermediate crest elevation)

500 Foot Spur Groin (trunk section)



General Information

| | |
|--|-----------------|
| Bottom Elevation at Structure | -11.0 ft-NAVD88 |
| Design Tide Elevation | 8.0 ft-NAVD88 |
| Depth Limited Wave Height (0.60*water depth) | 11.4 ft |
| Crest Elevation | 9.0 ft-NAVD88 |
| Length of Structure | 450.0 ft |

Stone Weight Calculation

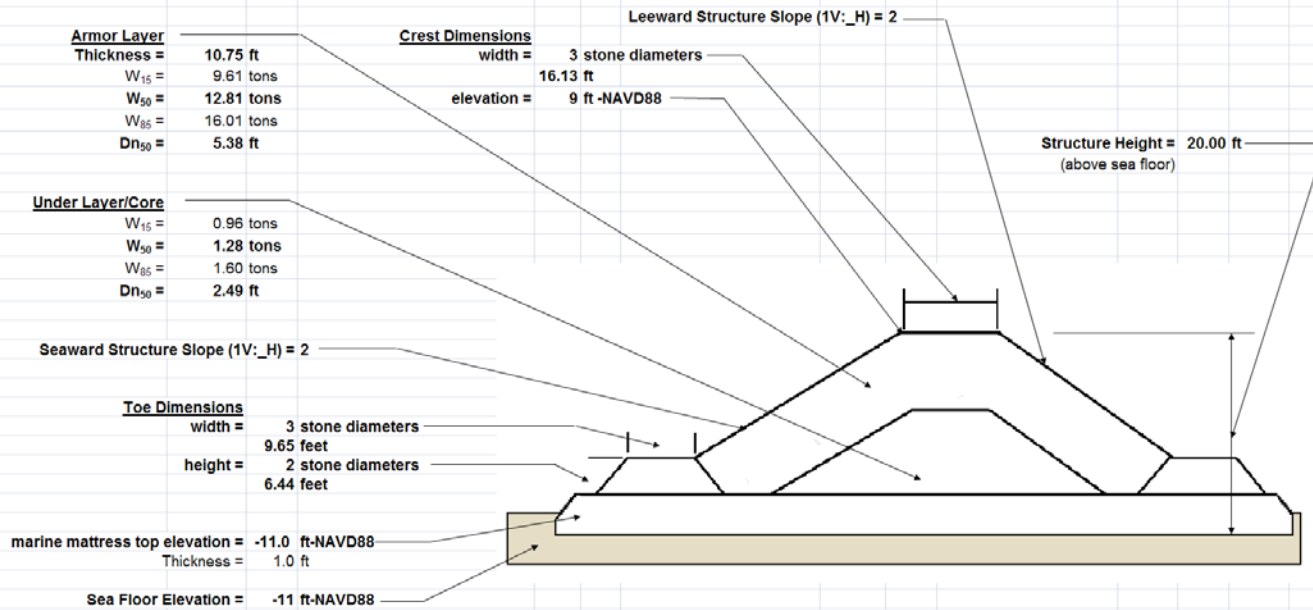
| | |
|------------------------------|---------------------------|
| Wave Height (H) | 12.5 ft |
| Specific Gravity (wr) | 165.0 lbs/ft ³ |
| Stability Coefficient (K) | 2.0 |
| Seaward Structure Slope (1V) | 2.0 |
| Leeward Structure Slope (1V) | 2.0 |
| Density of Water | 64.0 lbs/ft ³ |
| Bedding Layer Thickness | 0.0 ft |

| Armor Stone | Under Layer/Core Stone | Toe Stone |
|--|--|--|
| Armor Weight = 20,499 lbs = 10.25 tons | Underlayer Weight = 2,050 lbs 1.02 tons | Toe Stone Weight = 5,500 lbs = 2.75 tons |
| Cross Sectional Area of Armor Layer | Cross Sectional Area of Underlayer | Cross Sectional Area of Toe |
| Dn50 of Armor Stone 4.99 ft | Dn50 of Underlayer 2.32 ft | Dn50 of Toe 3.22 ft |
| Structure Height (above bedding) 20.00 ft | Height of Underlayer 10.02 ft | Toe Crest Width (# of units) 3 |
| Crest Width (# armor units) 3 | Underlayer Crest Width 9.98 ft | Toe Thickness (# of units) 2 |
| Armor Layer Thickness (# armor units) 2 | Underlayer Thickness (# of units) 2 | |
| Armor Layer Cross Sectional Area = 1,042 ft² | Cross Sectional Area = 301 ft² | Cross Sectional Area = 104 ft² |

Figure 8-15. Alternative 25A Spur Groin Trunk – intermediate crest elevation

Alternative 25A (intermediate crest elevation)

500 Foot Spur Groin (head section)



General Information

| | |
|--|-----------------|
| Bottom Elevation at Structure | -11.0 ft-NAVD88 |
| Design Tide Elevation | 8.0 ft-NAVD88 |
| Depth Limited Wave Height (0.60*water depth) | 11.4 ft |
| Crest Elevation | 9.0 ft-NAVD88 |
| Length of Structure | 50.0 ft |

Stone Weight Calculation

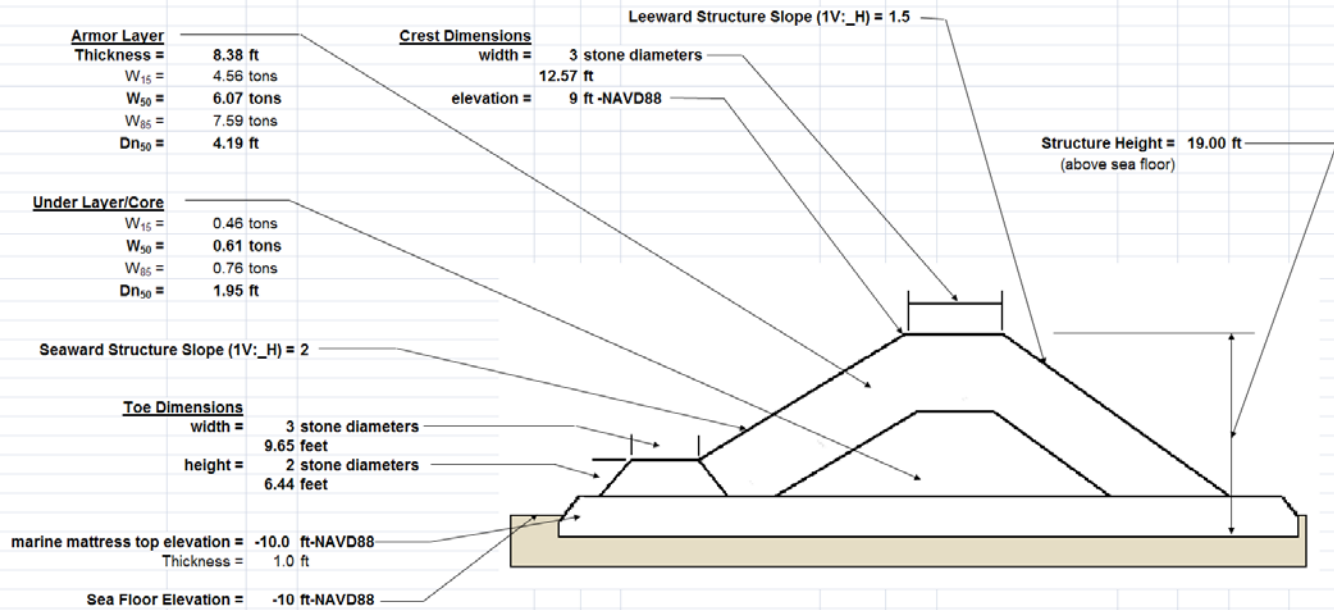
| | |
|------------------------------|---------------------------|
| Wave Height (H) | 12.5 ft |
| Specific Gravity (wr) | 165.0 lbs/ft ³ |
| Stability Coefficient (K) | 1.6 |
| Seaward Structure Slope (1V) | 2.0 |
| Leeward Structure Slope (1V) | 2.0 |
| Density of Water | 64.0 lbs/ft ³ |
| Bedding Layer Thickness | 0.0 ft |

| Armor Stone | Under Layer/Core Stone | Toe Stone |
|--|--|--|
| Armor Weight = 25,624 lbs = 12.81 tons | Underlayer Weight = 2,562 lbs 1.28 tons | Toe Stone Weight = 5,500 lbs = 2.75 tons |
| Cross Sectional Area of Armor Layer | Cross Sectional Area of Underlayer | Cross Sectional Area of Toe |
| Dn50 of Armor Stone 5.38 ft | Dn50 of Underlayer 2.49 ft | Dn50 of Toe 3.22 ft |
| Structure Height (above bedding) 20.00 ft | Height of Underlayer 9.25 ft | Toe Crest Width (# of units) 3 |
| Crest Width (# armor units) 3 | Underlayer Crest Width 10.75 ft | Toe Thickness (# of units) 2 |
| Armor Layer Thickness (# armor units) 2 | Underlayer Thickness (# of units) 2 | |
| Armor Layer Cross Sectional Area = 1,135 ft² | Cross Sectional Area = 271 ft² | Cross Sectional Area = 104 ft² |

Figure 8-16. Alternative 25A Spur Groin Head – intermediate crest elevation

Alternative 25A (intermediate crest elevation)

Breakwater 1 (trunk section)



General Information

| | |
|--|-----------------|
| Bottom Elevation at Structure | -10.0 ft-NAVD88 |
| Design Tide Elevation | 8.0 ft-NAVD88 |
| Depth Limited Wave Height (0.60*water depth) | 10.8 ft |
| Crest Elevation | 9.0 ft-NAVD88 |
| Length of Structure | 310.0 ft |

Stone Weight Calculation

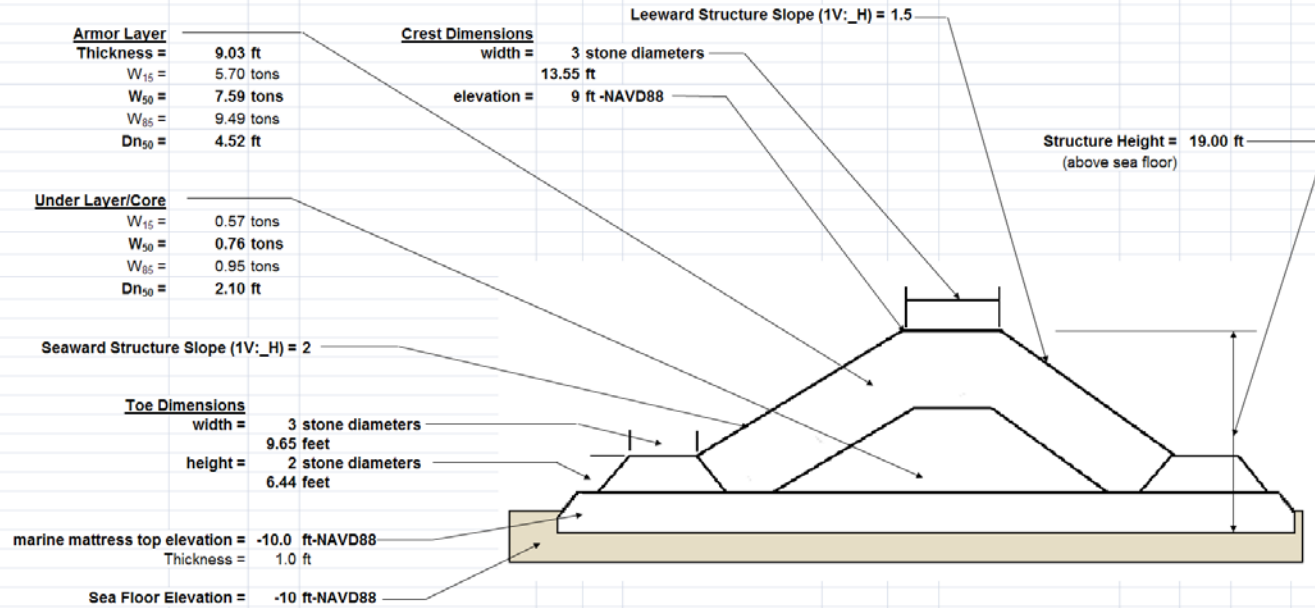
| | |
|------------------------------|---------------------------|
| Wave Height (H) | 10.5 ft |
| Specific Gravity (wr) | 165.0 lbs/ft ³ |
| Stability Coefficient (K) | 2.0 |
| Seaward Structure Slope (1V) | 2.0 |
| Leeward Structure Slope (1V) | 1.5 |
| Density of Water | 64.0 lbs/ft ³ |
| Bedding Layer Thickness | 0.0 ft |

| Armor Stone | Under Layer/Core Stone | Toe Stone |
|--|--|--|
| Armor Weight = 12,150 lbs = 6.07 tons | Underlayer Weight = 1,215 lbs 0.61 tons | Toe Stone Weight = 5,500 lbs = 2.75 tons |
| Cross Sectional Area of Armor Layer | Cross Sectional Area of Underlayer | Cross Sectional Area of Toe |
| Dn50 of Armor Stone: 4.19 ft | Dn50 of Underlayer: 1.95 ft | Dn50 of Toe: 3.22 ft |
| Structure Height (above bedding): 19.00 ft | Height of Underlayer: 10.62 ft | Toe Crest Width (# of units): 3 |
| Crest Width (# armor units): 3 | Underlayer Crest Width: 8.38 ft | Toe Thickness (# of units): 2 |
| Armor Layer Thickness (# armor units): 2 | Underlayer Thickness (# of units): 2 | |
| Armor Layer Cross Sectional Area = 749 ft² | Cross Sectional Area = 286 ft² | Cross Sectional Area = 104 ft² |

Figure 8-17. Alternatives 25A Breakwater 1 Trunk – intermediate crest elevation

Alternative 25A (intermediate crest elevation)

Breakwater 1 (head section)



General Information

| | |
|--|-----------------|
| Bottom Elevation at Structure | -10.0 ft-NAVD88 |
| Design Tide Elevation | 8.0 ft-NAVD88 |
| Depth Limited Wave Height (0.60*water depth) | 10.8 ft |
| Crest Elevation | 9.0 ft-NAVD88 |
| Length of Structure | 50.0 ft |

Stone Weight Calculation

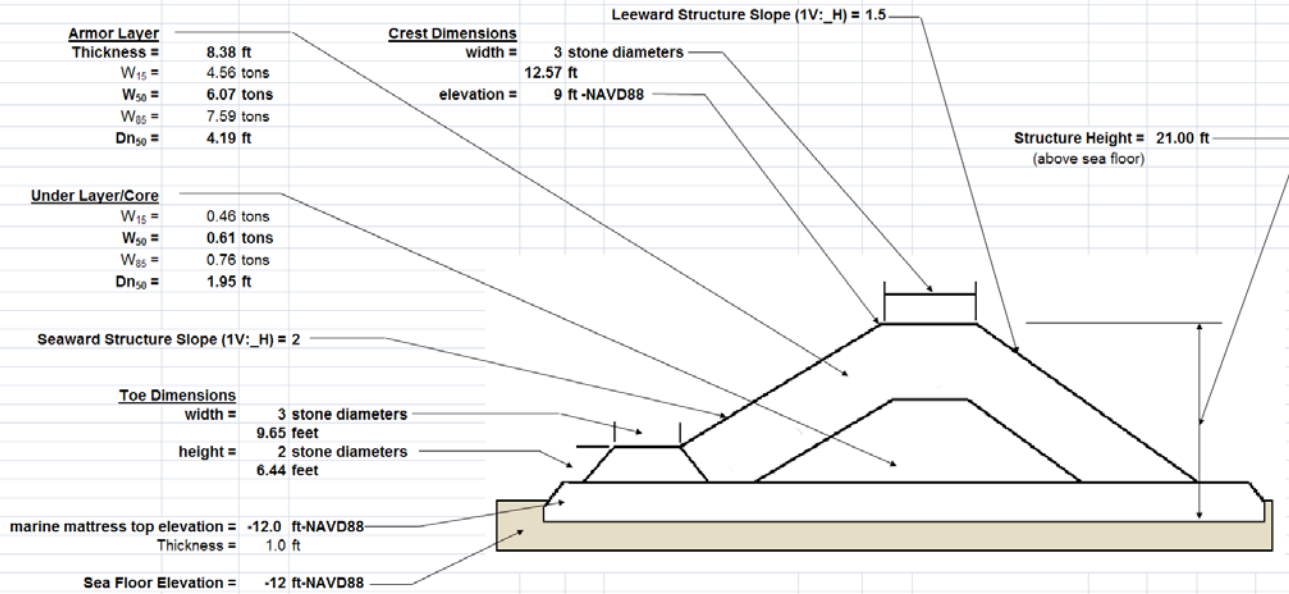
| | |
|------------------------------|---------------------------|
| Wave Height (H) | 10.5 ft |
| Specific Gravity (wr) | 165.0 lbs/ft ³ |
| Stability Coefficient (K) | 1.6 |
| Seaward Structure Slope (1V) | 2.0 |
| Leeward Structure Slope (1V) | 1.5 |
| Density of Water | 64.0 lbs/ft ³ |
| Bedding Layer Thickness | 0.0 ft |

| Armor Stone | Under Layer/Core Stone | Toe Stone |
|--|--|--|
| Armor Weight = 15,187 lbs = 7.59 tons | Underlayer Weight = 1,519 lbs 0.76 tons | Toe Stone Weight = 5,500 lbs = 2.75 tons |
| Cross Sectional Area of Armor Layer | Cross Sectional Area of Underlayer | Cross Sectional Area of Toe |
| Dn50 of Armor Stone: 4.52 ft | Dn50 of Underlayer: 2.10 ft | Dn50 of Toe: 3.22 ft |
| Structure Height (above bedding): 19.00 ft | Height of Underlayer: 9.97 ft | Toe Crest Width (# of units): 3 |
| Crest Width (# armor units): 3 | Underlayer Crest Width: 9.03 ft | Toe Thickness (# of units): 2 |
| Armor Layer Thickness (# armor units): 2 | Underlayer Thickness (# of units): 2 | |
| Armor Layer Cross Sectional Area = 815 ft² | Cross Sectional Area = 264 ft² | Cross Sectional Area = 104 ft² |

Figure 8-18. Alternatives 25A Breakwater 1 Head – intermediate crest elevation

Alternative 25A (intermediate crest elevation)

Breakwater 2 (trunk section)



General Information

| | |
|--|-----------------|
| Bottom Elevation at Structure | -12.0 ft-NAVD88 |
| Design Tide Elevation | 8.0 ft-NAVD88 |
| Depth Limited Wave Height (0.60*water depth) | 12.0 ft |
| Crest Elevation | 9.0 ft-NAVD88 |
| Length of Structure | 310.0 ft |

Stone Weight Calculation

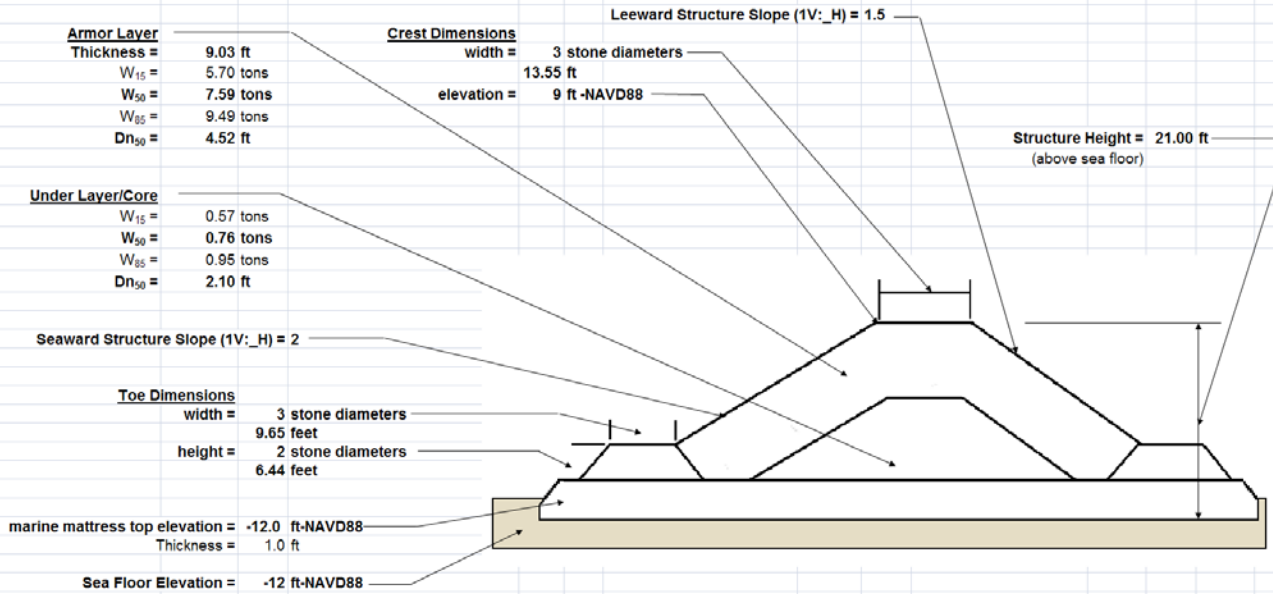
| | |
|------------------------------------|---------------------------|
| Wave Height (H) | 10.5 ft |
| Specific Gravity (w _r) | 165.0 lbs/ft ³ |
| Stability Coefficient (K) | 2.0 |
| Seaward Structure Slope (1V) | 2.0 |
| Leeward Structure Slope (1V) | 1.5 |
| Density of Water | 64.0 lbs/ft ³ |
| Bedding Layer Thickness | 0.0 ft |

| Armor Stone | Under Layer/Core Stone | Toe Stone |
|--|--|--|
| Armor Weight = 12,150 lbs = 6.07 tons | Underlayer Weight = 1,215 lbs 0.61 tons | Toe Stone Weight = 5,500 lbs = 2.75 tons |
| Cross Sectional Area of Armor Layer | Cross Sectional Area of Underlayer | Cross Sectional Area of Toe |
| Dn50 of Armor Stone 4.19 ft | Dn50 of Underlayer 1.95 ft | Dn50 of Toe 3.22 ft |
| Structure Height (above bedding) 21.00 ft | Height of Underlayer 12.62 ft | Toe Crest Width (# of units) 3 |
| Crest Width (# armor units) 3 | Underlayer Crest Width 8.38 ft | Toe Thickness (# of units) 2 |
| Armor Layer Thickness (# armor units) 2 | Underlayer Thickness (# of units) 2 | |
| Armor Layer Cross Sectional Area = 816 ft² | Cross Sectional Area = 384 ft² | Cross Sectional Area = 104 ft² |

Figure 8-19. Alternatives 25A Breakwater 2 Trunk – intermediate crest elevation

Alternative 25A (intermediate crest elevation)

Breakwater 2 (head section)



General Information

| | |
|--|-----------------|
| Bottom Elevation at Structure | -12.0 ft-NAVD88 |
| Design Tide Elevation | 8.0 ft-NAVD88 |
| Depth Limited Wave Height (0.60*water depth) | 12.0 ft |
| Crest Elevation | 9.0 ft-NAVD88 |
| Length of Structure | 50.0 ft |

Stone Weight Calculation

| | |
|------------------------------|---------------------------|
| Wave Height (H) | 10.5 ft |
| Specific Gravity (wr) | 165.0 lbs/ft ³ |
| Stability Coefficient (K) | 1.6 |
| Seaward Structure Slope (1V) | 2.0 |
| Leeward Structure Slope (1V) | 1.5 |
| Density of Water | 64.0 lbs/ft ³ |
| Bedding Layer Thickness | 0.0 ft |

| Armor Stone | Under Layer/Core Stone | Toe Stone |
|--|--|--|
| Armor Weight = 15,187 lbs = 7.59 tons | Underlayer Weight = 1,519 lbs 0.76 tons | Toe Stone Weight = 5,600 lbs = 2.75 tons |
| Cross Sectional Area of Armor Layer | Cross Sectional Area of Underlayer | Cross Sectional Area of Toe |
| Dn50 of Armor Stone 4.52 ft | Dn50 of Underlayer 2.10 ft | Dn50 of Toe 3.22 ft |
| Structure Height (above bedding) 21.00 ft | Height of Underlayer 11.97 ft | Toe Crest Width (# of units) 3 |
| Crest Width (# armor units) 3 | Underlayer Crest Width 9.03 ft | Toe Thickness (# of units) 2 |
| Armor Layer Thickness (# armor units) 2 | Underlayer Thickness (# of units) 2 | |
| Armor Layer Cross Sectional Area = 898 ft² | Cross Sectional Area = 359 ft² | Cross Sectional Area = 104 ft² |

Figure 8-20. Alternative 25A Breakwater 2 Head – intermediate crest elevation.

9.0 BEACH FILL CROSS SECTION DESIGN

9.1 Introduction and History

During the design of the numerous alternatives discussed in Sections 6 and 7, beach fill was considered as a component of many of the plans but it was not considered a viable stand alone alternative. This was the case due to the very short life span of previous sand placement in the project area, the complex wave environment and reflected waves from the North Jetty, experiences at other acute erosion, down drift inlet projects within and outside of USACE, and from discussions with subject matter experts within the Corps. The first approach to designing the beach fill was to use the same fill volume for each alternative, with limited analysis being performed on the beach fill since the major protection was expected to come from the various structural components. However, it became evident when evaluating and selecting the alternatives that the beach performance within the alternatives could play a more significant role than anticipated.

Part of the reason a more detailed beach fill design/performance analysis was not originally conducted was the complex wave environment and the complications introduced by the various structural components. After consultation with the USACE Coastal Working Group of the Hydraulics, Hydrology, and Coastal Community of Practice, also known as the Coastal Working Group (CWG), and several subject matter experts it was concluded the analysis tools available were not developed to handle situations as complicated as the ones in the project alternatives. However, simplification methods to help account for some of these complications were suggested to allow an analysis to be done for the beach fill performance in the various alternatives. The answers produced from this analysis are not exact and may not reflect the actual performance of the beach fill within the alternatives, however the information obtained from the analysis is likely correct in a relative sense between the alternatives.

The SBEACH model literature, which was used to model cross shore storm performance, specifically excludes the use of the model in areas such as the project area. However, through consultation with ERDC and the WHG, the model setup was adjusted to allow it to be used as an approximate representation of the project beach. The SBEACH modeling effort will be discussed in detail in Section 9.9.

9.2 Beach Fill Design

The beach fill designs went through several iterations throughout the multiple year investigation. The original intent of the beach fill component was to restore a beach within the project area that would provide the extra level of protection against erosion behind a structural alternative and restore a sand source to the system. The beach fill design was originally based on beach profile data from the late 1990s and early 2000s time frame. The original volume was estimated at 300,000 cubic yards along 3,000 feet of beach. This would provide a beach fill density of 100 cubic yds³/ft of beach. As more attention was brought upon the beach fill only alternative, and subsequently the beach fill components of the structural alternatives, greater detail was given to the design of the beach fill.

Ultimately the beach fill analysis sought to determine what was the minimum beach volume/berm width needed to protect the shoreline from further erosion for each of the alternatives. It was hypothesized that a smaller beach would be needed to be in place as part of the structural alternatives due to the level of protection from the spur and/or breakwaters than for a beach fill only alternative where there was no reduction in wave and current forcing. Additionally, the longevity of the beach fill and the number of renourishments over the 50 year project life was sought as well.

9.3 Exiting Beach Profile Data

The beach fill alternatives were designed in the USACE RMAP software package utilizing the most recent fall 2009 profiles. The first profiles that were evaluated were located in the project area and were classified as an eroded beach condition. Due to the relatively short project length of 3,250 feet, one profile was chosen to represent the eroded beach condition. It is worth noting that the beach fill length had to be extended an additional 250 feet from the original 3,000 feet due to additional shoreline erosion during the eight year study period. As part of the design process, it was desirable to have a healthy, or non eroded beach profile as a template. With such a profile the proper beach berm elevation and foreshore slope could be determined. Profile data was collected just north of the project, where a “healthier” beach profile exists (dune, berm, foreshore slope, and sub aerial slope). Shown in Figure 9-1 are the eroded and healthier northern profiles. It can be seen that the “healthy” northern profile does not extend offshore (was not collected in the field). In order to have a more complete, healthy profile, the northern profile was merged with the eroded profile by joining them at the matching offshore elevation. This was done based on equilibrium beach theory and the strong likelihood that the sub aerial profiles were similar. The extended healthier northern profile compared to the eroded profile can be seen in Figure 9-2.

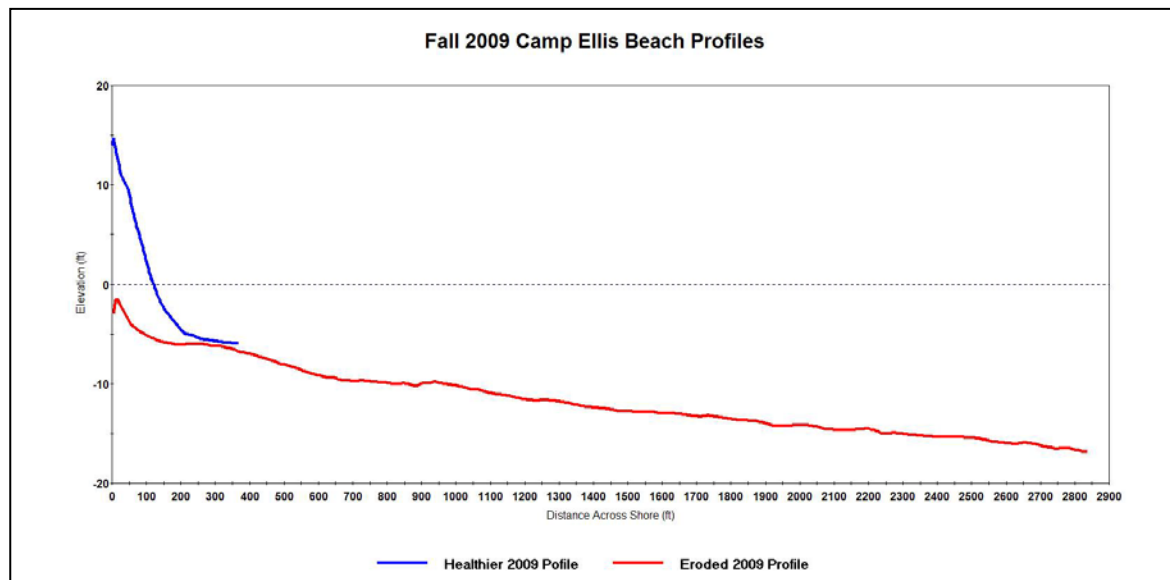


Figure 9-1. Fall 2009 Camp Ellis Beach profiles (elevation ft-NAVD88)

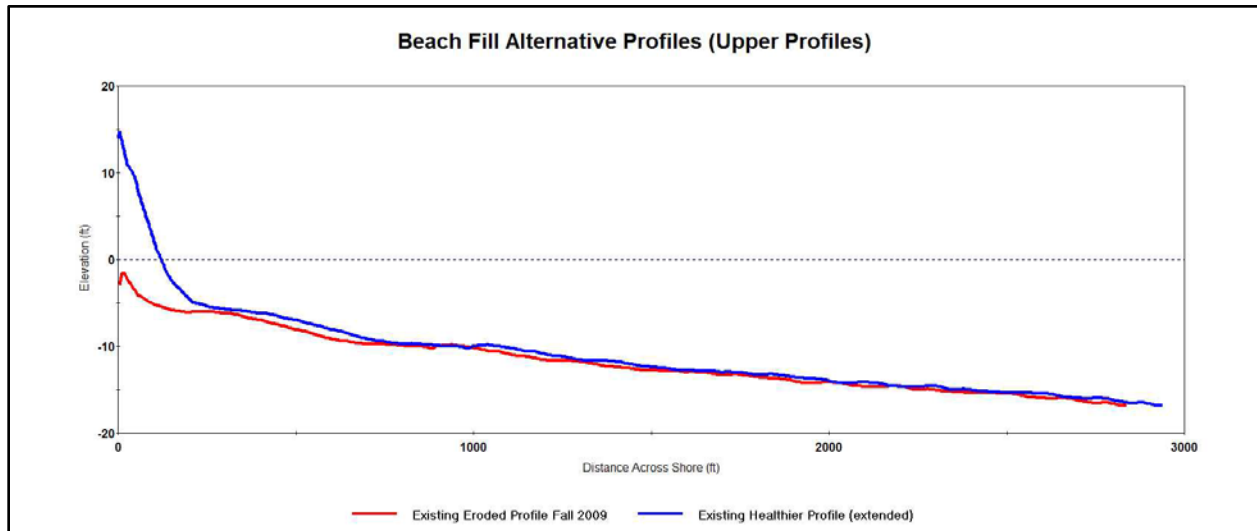


Figure 9-2. Extended healthy profile (elevation ft-NAVD88)

9.4 Beach Fill Design Methodology

9.4.1 Beach Fill Only Profiles

Beach fill alternatives with berm widths ranging from 10 feet to 70 feet, in ten foot increments were designed in RMAP. To design the profiles it was assumed that the sand being placed matched the existing sand grain size distribution. This decision was made because a specific borrow source of material for the beach fill had not been identified at the time of the beach fill design. The sand that is chosen could ultimately be a finer grain sand, which would result in a shallower sloped beach, or coarser which would result in a steeper sloped beach. The use of equilibrium theory, and assuming a matching grain size distribution, means that as sand is placed on the beach the beach profile will grow seaward, and will match the existing shape of the beach. This simply translates the existing beach profile seaward. The healthy profile from the north was used as a template for the beach fill profile. From this profile the beach berm elevation for beach fills was set at 12 ft-NAVD88 and the same foreshore and sub aerial beach profiles were used. It can be seen in Figure 9-3 that the 30 foot beach berm fill alternative is nearly identical to the healthier beach profile from the north. The 12 foot berm elevation was chosen due to the natural berm elevation found on the healthy profile and also as a result of the SBEACH modeling which will be discussed in Section 9.9. The 12 ft-NAVD88 berm elevation is approximately one foot higher than the natural berm elevation but the slight increase in elevation showed performance benefits during the storm modeling effort, specifically more beach material remained near shore and at higher elevations.

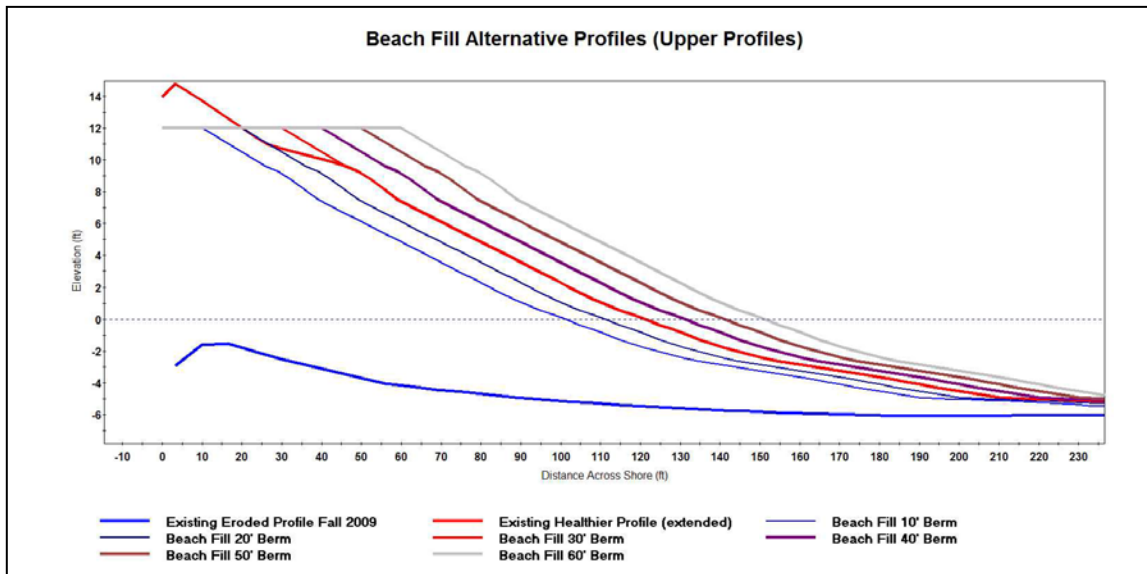


Figure 9-3. Beach fill alternative berm widths - upper profiles (elevations in ft-NAVD88)

Beach fill density and volumes per project length were determined in RMAP for each alternative by subtracting the difference between the alternative profiles and the existing eroded profile. The volumes have been provided in Table 9-1. For comparison purposes the volume difference between the healthier beach and the eroded profile has been included as well. Once again, the 30 ft berm width alternative is very comparable to the existing healthier profile to the north. The volumes for 1,625 feet were provided, which is one half of the beach fill length, and as will be seen in the beach fill modeling/performance analysis this number will be useful. It can also be seen in Table 9-1 that for each 10 ft of berm width increase of an equilibrated profile, the additional fill volume is approximately 10 yds³ (+/- 0.5 yds³)

Table 9-1. Beach Fill Only Volumes

| Profile/Beach Fill Alternative | Beach Fill Density | Beach Fill | |
|---------------------------------|-------------------------------|-------------------|-------------------|
| | yds ³ /ft of beach | 1,625 ft of beach | 3,250 ft of beach |
| 10' Berm | 79.81 | 129,686 | 259,373 |
| 20' Berm | 90.30 | 146,733 | 293,465 |
| 30' berm | 100.75 | 163,724 | 327,447 |
| North Profile (Healthy Profile) | 101.25 | 164,535 | 329,069 |
| 40' Berm | 111.16 | 180,642 | 361,283 |
| 50' Berm | 121.55 | 197,519 | 395,038 |
| 60' Berm | 131.92 | 214,373 | 428,747 |
| 70' Berm | 142.28 | 231,203 | 462,407 |

9.4.2 Beach Fill Profiles with Structures

For the structural alternatives there is cross shore beach equilibration benefits that would result from the placement of structures. Breakwaters typically limit cross shore movement of sand, creating a perched beach, which reduces the volume of sand necessary to fill out an equilibrated beach profile. An example of this has been provided as Figure 9-4.

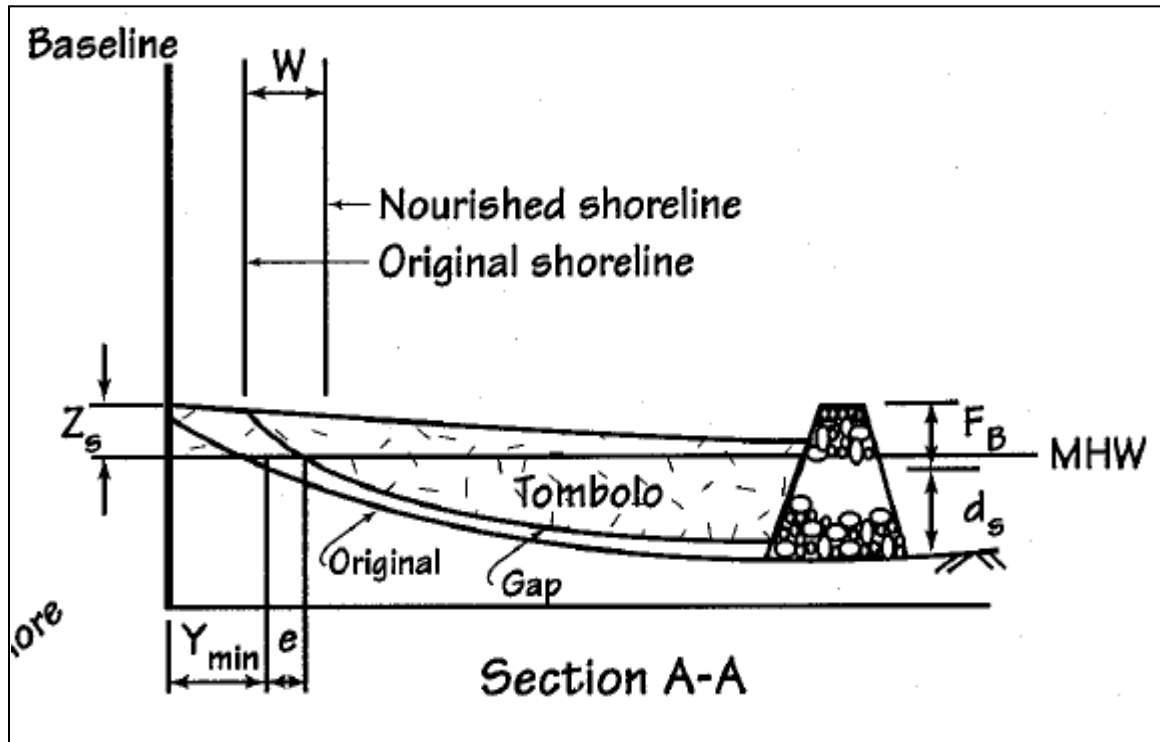


Figure 9-4. Beach profile cutoff/perched beach from near shore breakwater (Figure V-3-14 CEM).

The cross shore profile benefits were determined by simply cutting off the profile volume calculation at the distance offshore for the various spur and breakwater alternatives. For instance the spur and breakwaters for Alternative 25A are designed to be offshore 1,000 feet so the profile in the RMAP calculation was cut off at 1,000 feet. For the beach fill only profiles discussed in Section 9.4.1, the profile volume calculations were extended to a distance of 2,800 feet offshore. The volumes for the various beach berm widths for the various cutoffs have been provided in Table 9-2. Table 9-2 can be compared to Table 9-1 to determine the volume reduction for the structural alternatives versus the full beach fill profile. However, as discussed in Section 9.12.2, the profiles being cut off by the structures did not really impact the beach fill volumes significantly. This is due to the relatively thin layer of sand covering the beach beyond 1,000 and 1,500 feet and the fact that structural alternatives cover a maximum frontage of 1,400 feet of beach out of the 3,250 of project beach. Once again this will be discussed further in Section 9.12.2.

It should be noted however that the beach fill differences with structural components become more significant with rising sea level. Additional details are provided in the Sea Level Rise discussion in Section 12.

Table 9-2. Beach fill volumes with structural components.

| Beach Fill Volumes - 1,500 ft Profile Cut Off (Spur groin for Alt 6) | | | |
|---|-------------------------------|-------------------|-------------------|
| Profile/Beach Fill Alternative | Beach Fill Density | Beach Fill | |
| | yds ³ /ft of beach | 1,625 ft of beach | 3,250 ft of beach |
| 10' Berm | 68.25 | 110,898 | 221,796 |
| 20' Berm | 77.29 | 125,601 | 251,202 |
| 30' berm | 86.82 | 141,089 | 282,178 |
| North Profile (Healthy Profile) | 86.33 | 140,280 | 280,560 |
| 40' Berm | 95.35 | 154,947 | 309,894 |
| 50' Berm | 104.37 | 169,600 | 339,199 |
| 60' Berm | 113.37 | 184,221 | 368,443 |
| 70' Berm | 122.34 | 198,798 | 397,595 |
| Contour -14 ft | | | |
| Profile Length 1,500 ft | | | |
| | | | |
| | | | |
| Beach Fill Volumes - 1,000 ft Profile Cut Off (Alternative 25A) | | | |
| Profile/Beach Fill Alternative | Beach Fill Density | Beach Fill | |
| | yds ³ /ft of beach | 1,625 ft of beach | 3,250 ft of beach |
| 10' Berm | 60.25 | 97,905 | 195,809 |
| 20' Berm | 68.38 | 111,109 | 222,219 |
| 30' berm | 76.51 | 124,327 | 248,654 |
| North Profile (Healthy Profile) | 77.01 | 125,138 | 250,276 |
| 40' Berm | 84.68 | 137,608 | 275,217 |
| 50' Berm | 92.90 | 150,964 | 301,928 |
| 60' Berm | 101.11 | 164,302 | 328,604 |
| 70' Berm | 109.28 | 177,580 | 355,160 |
| Contour -14 ft | | | |
| Profile Length 1,000 ft | | | |
| | | | |

9.5 Beach Depth of Closure

An issue worth noting is that the depth of closure was likely not reached in the beach fill profile designs. As shown in Figure 9-2, the available survey data only extended to the elevation of -17 ft-NAVD88. During the SBEACH modeling effort discussed later in Section 9.9, the profiles showed movement and erosion at the furthest seaward extent of the profiles. This is not surprising given the beach profile elevation of -17 ft-NAVD88. This elevation is very likely shallower than the depth of closure given the wave climate determined in the wave modeling effort. This could mean that the volumes associated with the various beach fill alternatives may be under estimated since it may take more sand than calculated to fill the full beach profile. However, it is difficult to know if sand will be pulled out across the full profile since it is so wide and there are other influences likely involved in the development of

this wide and shallow sloped beach plan form. This uncertainty would be more significant with the beach fill only alternative since the structural based alternatives will help prevent cross shore movement of the beach, with the breakwater alternatives providing the maximum limitation of cross shore movement. Alternative 25A, with the near shore breakwater components will develop a scalloped shaped beach behind the structures and will limit the cross shore movement of the sand.

9.6 Sand Deficit

As shown in Figure 9-2 and Table 9-1, a comparison between the healthier profile to the north and the eroded profile in the project area shows that a significant volume of sand will be needed to restore the beach to a healthier beach condition. The volume comparison provided in Table 9-1 shows that the eroded profile had a sand deficit of 101.25 yds³/ft of beach as compared to the healthy profile just outside the project area. This deficit has likely changed since the profiles were collected in 2009. As will be discussed in upcoming Section 9.12, the modeling confirmed that a beach fill similar to the healthier beach profile is needed to survive a significant storm without further shoreline damage. This means that 101.25 yds³/ft or 329,000 yds³ for the 3,250 project length will be needed just to restore the beach to this healthier profile. This number is not exact, and is likely overstated since the beach transitions from the eroded profile to the healthier profile while moving north through the project. However, it does demonstrate that there was a significant sand deficit for the beach in the project area at the time of the 2009 surveys.

9.7 Beach Fill Deficit/Nourishment/Re-nourishment Formulation

As will be discussed in Section 9.12, one of the important pieces of information sought for plan formulation was the volume of sand needed to be in place, or the beach berm width needed to be in place, to survive a significant storm, and the different volumes needed between alternatives. This necessary volume or berm width can be considered a trigger point, or a minimum beach width/volume that the beach should not be allowed to erode below. Below this volume, for the particular alternative being considered, the shoreline will be impacted by a significant storm. These trigger point volumes/berm widths were determined in Section 9.12. If shoreline erosion is allowed to occur beyond the location that exists at the start of construction, the project intent of the Section 111 will not be met. Remember the intent of the Section 111 is to prevent the erosion that is being caused by the jetties. As also shown in Section 4.1, the beach system within Saco Bay is a stable system, beyond the erosion area adjacent to the federal navigation project.

These minimum beach widths/volumes should be maintained at all times and re-nourishment should be conducted before these minimum volumes are reached. Therefore, it was decided that for the purposes of this analysis the first beach fill should be broken up into two separate fills. Obviously the fills would be constructed at the same time, but to keep the fill volume discussion as clear as possible two fill names were used. The first, as designated in Section 9.12, is the "Non Sacrificial Fill". This will essentially restore the beach back from the severely eroded condition that existed in 2009. The second fill will be the "Nourishment Fill" which is the sacrificial fill that will be allowed to erode overtime to a point where the trigger is reached and a re-nourishment project is conducted. This second fill has been called "advanced nourishment" by other USACE districts. This Nourishment Fill and the maintenance of

this Nourishment Fill (re-nourishment fills) will be analyzed separately during the longevity performance analysis in Section 10. The first fill or Non Sacrificial fill can be considered a one-time construction effort which must be provided for in all alternatives.

9.8 Design Storm Selection

It was decided to select a 10 year return period storm for the beach fill analysis. The 10 year storm characterization was based on water level and wave height. It is understood the joint occurrence, or joint probability, of return period parameters such as waves and water levels results in an event that likely has a return period greater than the individual return periods. However, with nor'easters in New England, typically, if a storm is strong enough to generate a return period water level, the storms intensity is such, that significant wave energy will surely occur as well. The offshore wave height may not be at the same return period level, but the near shore waves are likely to be near maximum due to shoaling and depth limited wave conditions. Therefore a joint occurrence was assumed to be reasonable. The wave height hydrograph was provided by the Woods Hole Group at the thirty (30) foot contour offshore of the project site.

Only one storm hydrograph (wave height, wave period, water level) was used in this analysis effort and based on engineering judgment, the storm hydrograph used was representative of a significant New England coastal storm. It should also be noted that there is not a large difference in parameters between a New England coastal 2 year event and a 10 year event, or even a 20 year event. This is because strong nor'easters are a fairly frequent event in New England. The details regarding the specific 10 year event and storm hydrograph are provided in Section 9.9.2 in Figure 9-6.

The use of a 10 year storm should not be interpreted as this project providing protection for a 10 year event. A full risk based analysis was not completed for this project since the intent of the project is not flood risk reduction or storm damage reduction, but instead shoreline erosion prevention. As discussed, a more robust storm was used to determine the general performance of the beach fill during a storm, and the potential for erosion, not for reducing storm damage or flood risk. Once again, the intent of this Section 111 project is to mitigate for shoreline erosion problems caused by the USACE jetties.

9.9 SBEACH Modeling

In order to provide differentiation in performance between the various beach fill alternatives, an attempt was made to model the beach storm performance. Despite the known limitations of beach models, after lengthy discussions and consultations with ERDC and WHG, an approach that utilized SBEACH was chosen and used for analysis. However, due to limitations of the analysis, the results should be used more for relative comparison and less for actual values.

9.9.1 Modeling Approach

As discussed throughout the report, there are some intense coastal processes that occur within the project area that have been documented and demonstrated in the wave modeling effort. These processes include, Mach stem waves riding along the structure, wave focusing between the offshore islands, a significant reflected wave off of the north jetty that sends waves towards the beach at a very

shallow angle (strong sediment transport potential), and a complex hydraulic current field is suspected during storms. A hydraulic numerical model was not completed but looking at the bathymetry it is likely that a complicated hydraulic regime exists in the project area. The SBEACH model was not designed to handle the complex wave and hydraulic conditions in the project area nor beach performance near coastal structures. To address the complicated coastal forcing and the limited capabilities of SBEACH in these areas, a simplified approach was chosen with an understanding that several factors may need to be estimated. Therefore, results should be used more for relative comparison and less for exact values.

The basic approach to address some of the limitations was to run the SBEACH model with the wave climate behind the near shore breakwaters and spur groin options. Typically, SBEACH is run for the full beach profile and storm time series, however, for this case, the near shore breakwaters and spurs significantly reduces the wave energy behind them necessitating a cutoff. During the SBEACH model parameter setup the profile lengths were set to either 1,000 ft or 1,500 ft vs. the full 2,800 ft, depending on alternative scenario the beach was being modeled for. Also, the reduced wave heights resulting behind the breakwater and spur structures were used as the wave input. These adjustments were not needed for the beach fill only option since there were no interruptions to the wave energy impacting the beach. The wave reductions will be discussed in Section 9.9.3.3.

Since SBEACH cannot handle mach stem currents, reflected waves, or the complicated current regime of the existing beach condition, the modeling will likely over predict the performance or resiliency of the beach fill only alternative. Due to under predictions of the modeling process for the beach only alternative, engineering judgment was used to develop predicted beach replenishment volumes later on in the report (Section 10). This engineering judgment amounted to doubling the renourishment volumes determined in the modeling effort for the beach fill only alternative. Although this number is arbitrary the results are more likely to be correct than not applying engineering judgment. Once again this is further discussed in Section 10.0 where the modeling/beach fill longevity results are presented and in Section 12 during the Risk discussion. As mentioned previously, the Beach Fill Only Alternative was determined not to be a viable alternative once after all of the analysis was concluded. For Alternative 6, the spur and beach fill option, the modeled performance in SBEACH is more likely representative of the actual performance of the beach fill. The spur groin was shown in the WHG wave modeling to significantly block the reflected waves from the north jetty and the Mach Stem effects. This results in a situation that better suits the SBEACH model, and comes closer to meeting the conditions it was designed for. For Alternative 25A, the spur and breakwater alternative, the SBEACH modeling likely under predicts the performance of a beach fill. In addition to the benefits mentioned for the spur, the breakwaters will also cause a more stable beach form to develop behind them, as shown in Figure 9-5. To be conservative, no adjustments were made to decrease the beach re-nourishment frequencies of the spur/breakwater alternatives. This formation will perform better during storms than a more typical sloped beach without protection. This effect was not accounted for in SBEACH, which means it is likely that the beach will perform better than predicted.

To help provide a better understanding of the modeling limitations, a table was created for each alternative and related processes in the project area and can be found, along with a more detailed discussion, in Section 11.

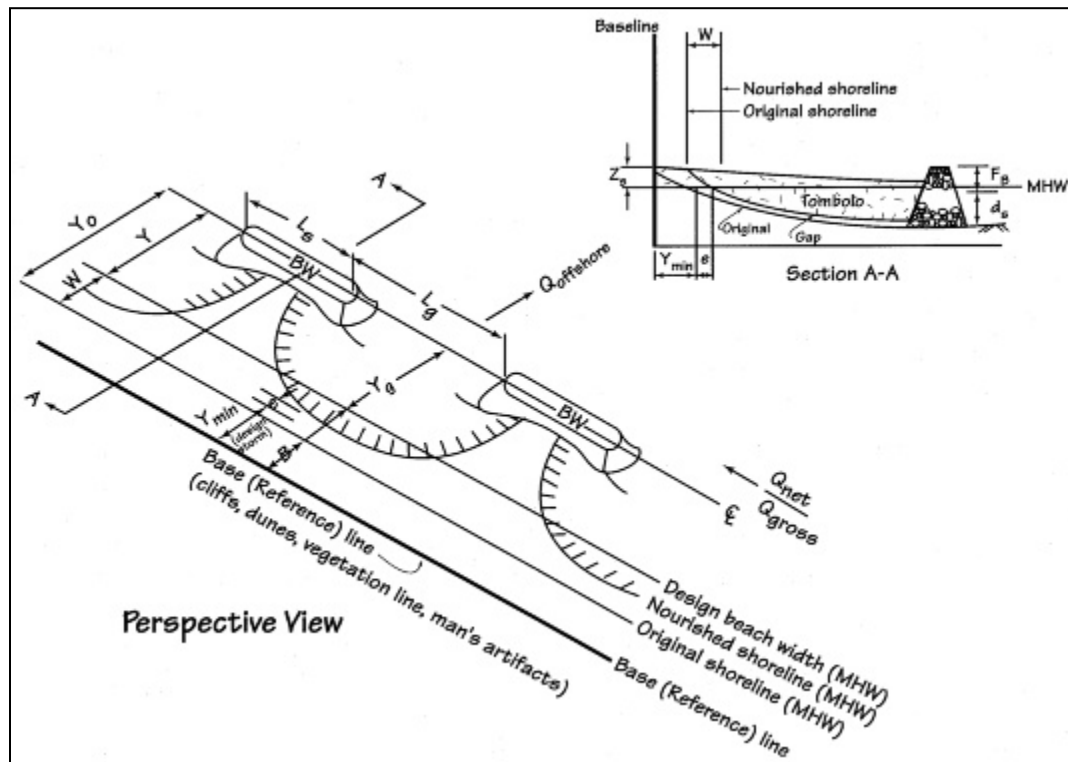


Figure 9-5. Near shore breakwater beach formation (Figure V-3-14 of USACE CEM)

9.9.2 Model Set Up

The SBEACH model setup consisted of importing the profiles to be modeled and developing the storm time series. The profiles were directly imported from the RMAP software and comprised of both the existing profiles (eroded and healthy) and the beach fill profiles. The 10 year storm that was used was actually taken from the Nantasket Section 103 study that was conducted for a Nantasket Beach in MA. There were numerous water level and wave condition time series developed for this area as part of the study. Given the similar storm climate, tide range, and relatively close geographic proximity it was decided to use modified storm time series from Nantasket instead of regenerating the time series for Camp Ellis which would have been difficult since there is no long term recorded tide data for Camp Ellis. During the WHG effort full time series were not developed, and only the peak storm conditions were determined and modeled. Based on the WHG work, the parameters that defined a 10 year storm event were characterized. This information was used to adjust the 10 year storm time series from Nantasket. Basically the time series were adjusted up or down to match the 10 year parameters at Saco. The 10 year storm time series used offshore of Camp Ellis can be seen in Figure 9-6.

As discussed the beach profiles lengths were adjusted to match the beach profiles landward of the alternative structures. The grain size was set based on the collected grain size data from USACE and

WHG. The rest of the parameters were left at a default setting with the exception of the avalanche slope angle. The default was allowing for very steep beach profiles that appeared unrealistic and that also caused the model to perform poorly. While there was no recorded storm beach profile data to compare to, during the model runs with the default cave parameter, smaller beaches often performed as well or better than larger beach fills. Better performance meant, a wider berm was maintained during the modeled storm runs for the smaller beach fill than for a larger beach fill. The model was run with a 5 ft grid cell width.

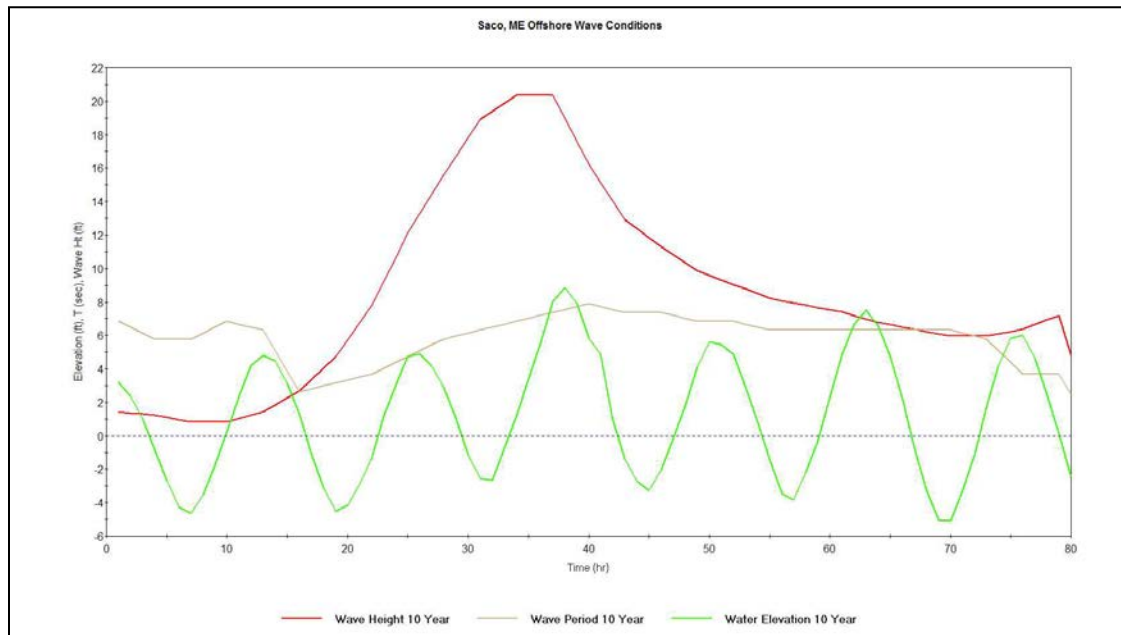


Figure 9-6. Camp Ellis generated 10 year storm time series (elevations in ft-NAVD88)

9.9.3 Model Validation

9.9.3.1 Healthier Beach Profile Model Performance

With the 10 year storm chosen and the profiles entered into SBEACH, the healthier existing beach profile from just north of the project was run. The starting and minimum profile during the storm can be seen in Figure 9-7.

The performance of this profile was of particular interest since it is just outside of the project area and just north of the area that is experiencing storm damage during storms. Before running the model it was hypothesized that this profile should perform fairly well in the model and not show excessive near shore erosion during the storm modeling. If it did then it could be concluded the SBEACH modeling was causing too much erosion. However, it was also hypothesized that there should be sufficient erosion that would bring the profile near to the point of allowing storm damage since just to the south in the project area, with a less healthy profile, there is documented storm damage. As can be seen in Figure 9-7, the healthier profile does experience noticeable erosion and the minimum profile, based on elevation and field observations would be on the verge of storm damage. The figure shows the starting profile and the minimum profile experience during the storm. Although this was not an ideal model validation,

it does lend support that the SBEACH model is performing somewhat adequately in this area. It should be noted, that at the 3,500 foot distance north of the jetties, this area is outside of the direct impacts of the reflected waves coming from the north jetty, with the exception of the stronger along-shore current being driven to the north. It should be noted that the storm profile shown in Figure 9-7 does not look like a “typical” storm profile since no sand bars are present and the tailing off of the offshore end of the profile. This “non-typical” model performance is believed to be the result of the wide, shallow, and shallow sloped bathymetry and because the profile does not reach the depth of closure. It is understood that this raises questions regarding model validity in the project area, and those concerns were also expressed by the PDT. However, this was the only tool available and calibration/validation data was not available so the results were used with an understanding of the limitations.

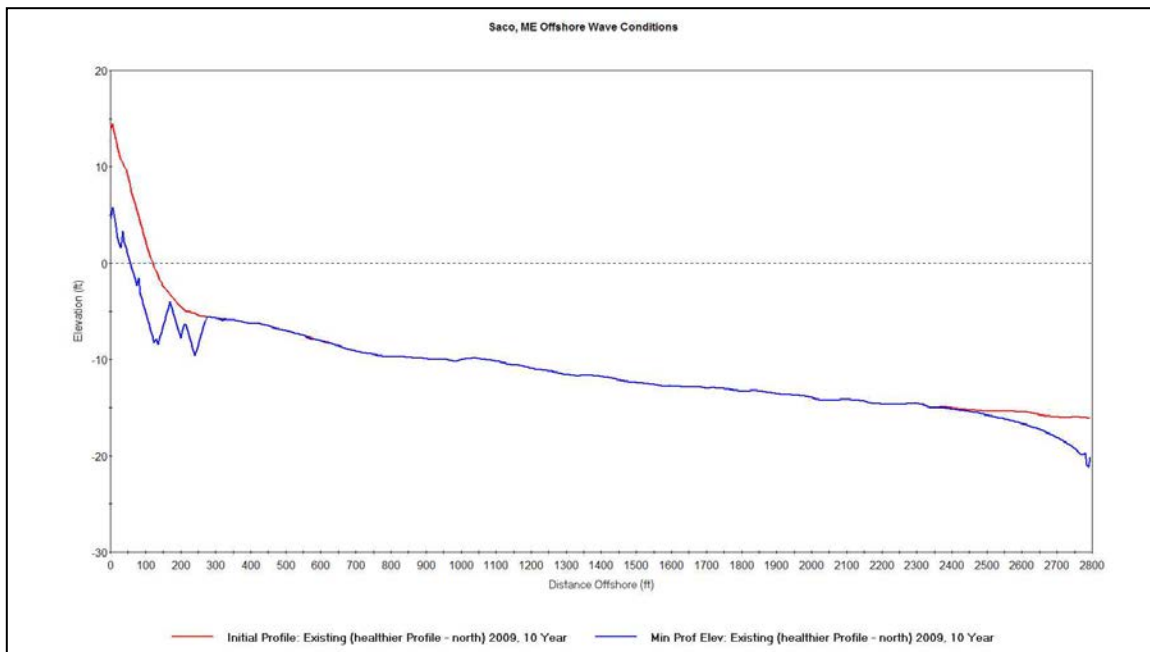


Figure 9-7. SBEACH model performance of healthier beach profile (elevations in ft-NAVD88) – minimum storm profile shown.

9.9.3.2 30 foot Berm Alternative Model Performance

The modeled storm performance of the full 10 yr storm time series was then run on the 30 foot berm beach fill alternative to make sure the performance was the same. As mentioned earlier, based on beach shape and volume, the 30 foot berm was considered to be a close match to the existing condition healthier beach profile. As can be seen in Figure 9-8, the starting and minimum beach profile during the storm is provided for each profile and the response is very similar.

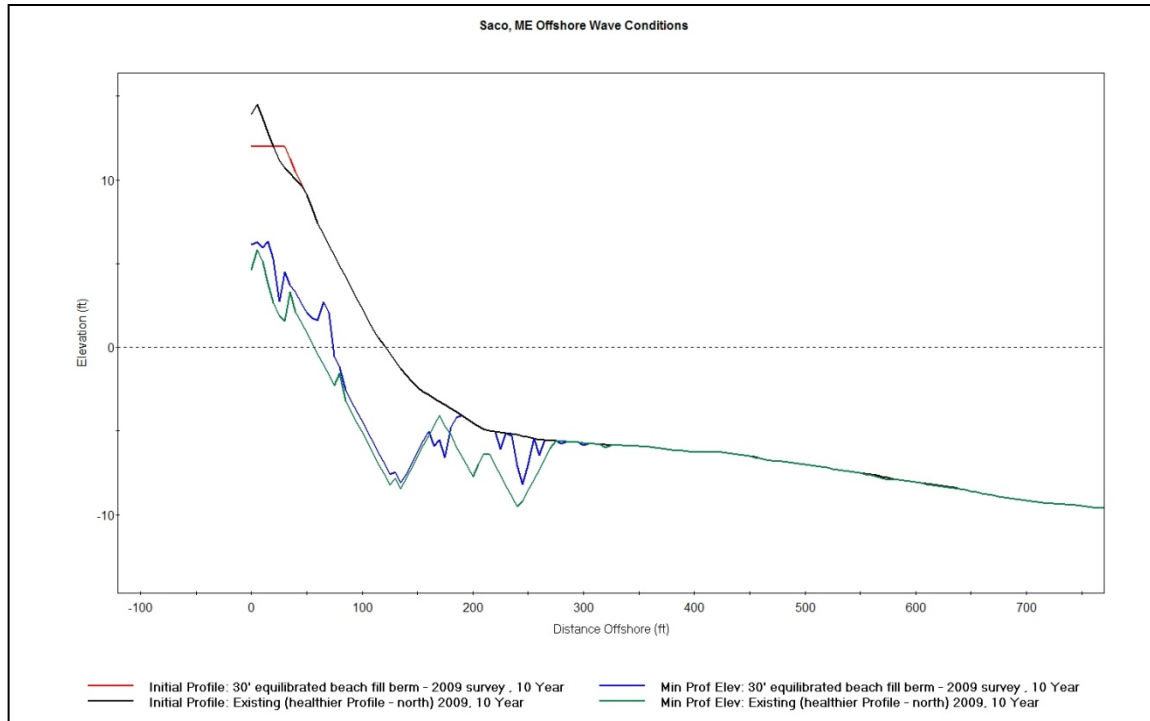


Figure 9-8. 30 foot alternative model performance.

9.9.3.3 Idealized Storm Generation and Validation

The purpose of this effort was to be able to model the beach with the alternatives in place and with the reduced wave energy behind the structural alternatives. Provided in Table 9-3 are the reduced energy/wave heights for each alternative based on zonal areas along the beach. This information was provided by WHG as part of the wave modeling effort and was taken from the WHG report.

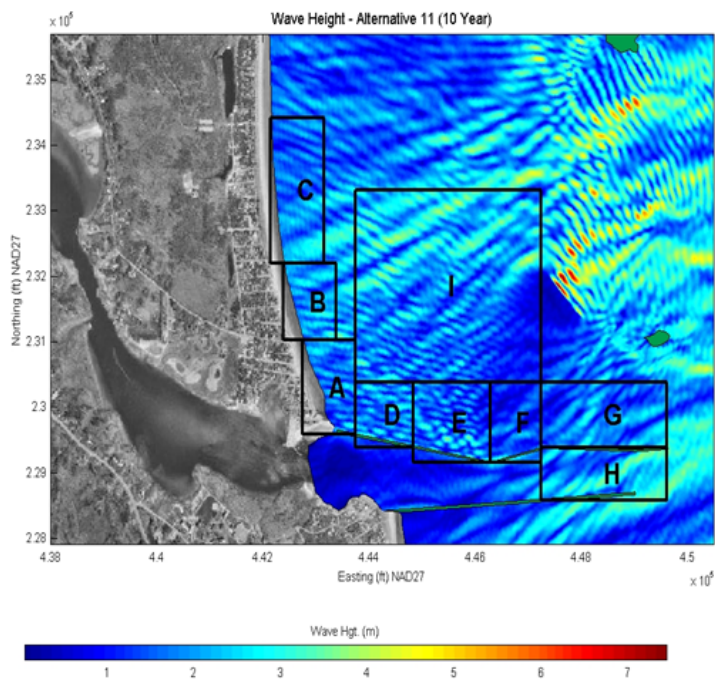
With the reduced wave height information it was decided to generate storm conditions that matched the WHG model output conditions shown in Table 9-3 for use in the SBEACH model. It was decided not to have WHG spend the time to rerun the wave models and output actual time series data at key locations for time and budgetary reasons. It was first attempted to use the actual 10 year storm time series shown in Figure 9-6 and only adjust the wave height by basically capping the wave heights in the time series to the values provided by WHG, as shown in Figure 9-9, but this lead to unrealistic output profiles in SBEACH.

The next approach was developed with the assistance of ERDC-CHL with the use of idealized storms constructed with uniform wave height, period and water levels for each of the nearshore storm conditions provided by WHG. The first idealized storm had a 7 ft wave height, 8 second period, and an 8 ft-NAVD88 water elevation (Figure 9-10). This configuration was meant to simulate the existing conditions since these conditions were the same or similar to the 10 year storm near shore conditions provided by WHG in Table 9-3. Since the variation in tide elevation was removed along with the increase and decrease in wave height and period, the variable that was altered in the idealized storm was the duration. As can be seen in Figures 9-6 and 9-9 the water level varies due to tidal forcing and

therefore the peak erosion impacts due not occur during the full time period of the storm. The idealized storm duration was adjusted until the modeled idealized storm on the 30 foot berm profile produced a similar minimum beach profile during the storm as the full 10 year storm time series. It was reasoned that both were simulating 10 year storms so both should have the same impact. Basically, this exercise was done to calibrate the idealized storm duration. Once again idealized storms were used because the artificially capped 10 year storm time series in Figure 9-9 was unrealistic in the model output. This approach was not the preferred method but was the only one available at the time.

Table 9-3. WHG Wave Model Output

| 10 Year Storm Parameters - Offshore and Nearshore. | | | |
|---|---------------|-----------------|---------------|
| Offshore Wave Information | $H_s = 6.2$ m | $T_p = 7.9$ sec | Surge = 2.4 m |
| Existing Conditions | Area A | Area B | Area C |
| H_s (m) (10-year Storm conditions) | 1.95 | 2.12 | 1.81 |
| H_s (ft) (10-year Storm conditions) | 6.4 | 7.0 | 5.9 |
| Case 6 - Spur Jetty | Area A | Area B | Area C |
| H_s (m) (10-year Storm conditions) | 1.86 | 2.05 | 1.78 |
| H_s (ft) (10-year Storm conditions) | 6.1 | 6.7 | 5.8 |
| Energy Reduction (Average conditions) | 20.9% | 6.6% | 9.5% |
| Energy Reduction (10-year Storm conditions) | 10.9% | 6.9% | 3.7% |
| Case 25A - Spur/Breakwater | Area A | Area B | Area C |
| H_s (m) (10-year Storm conditions) | 1.45 | 1.95 | 1.78 |
| H_s (ft) (10-year Storm conditions) | 4.8 | 6.4 | 5.8 |
| Energy Reduction (Average conditions) | 58.9% | 6.4% | 9.5% |
| Energy Reduction (10-year Storm conditions) | 52.6% | 18.1% | 3.7% |
| Case 25 - Spur/Breakwater | Area A | Area B | Area C |
| H_s (m) (10-year Storm conditions) | 1.45 | 1.36 | 1.78 |
| H_s (ft) (10-year Storm conditions) | 4.8 | 4.5 | 5.8 |
| Energy Reduction (Average conditions) | 58.5% | 61.2% | 9.5% |
| Energy Reduction (10-year Storm conditions) | 52.6% | 66.6% | 3.7% |



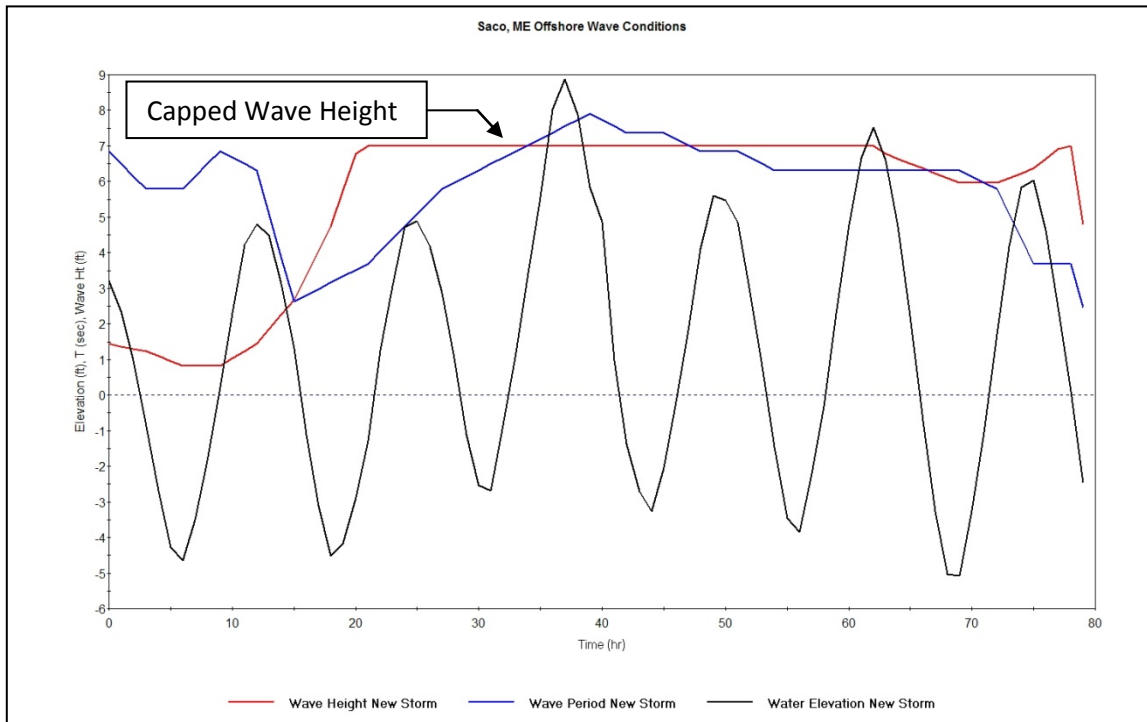


Figure 9-9. Idealized storm first attempt (failed) (elevations in ft-NAVD88)

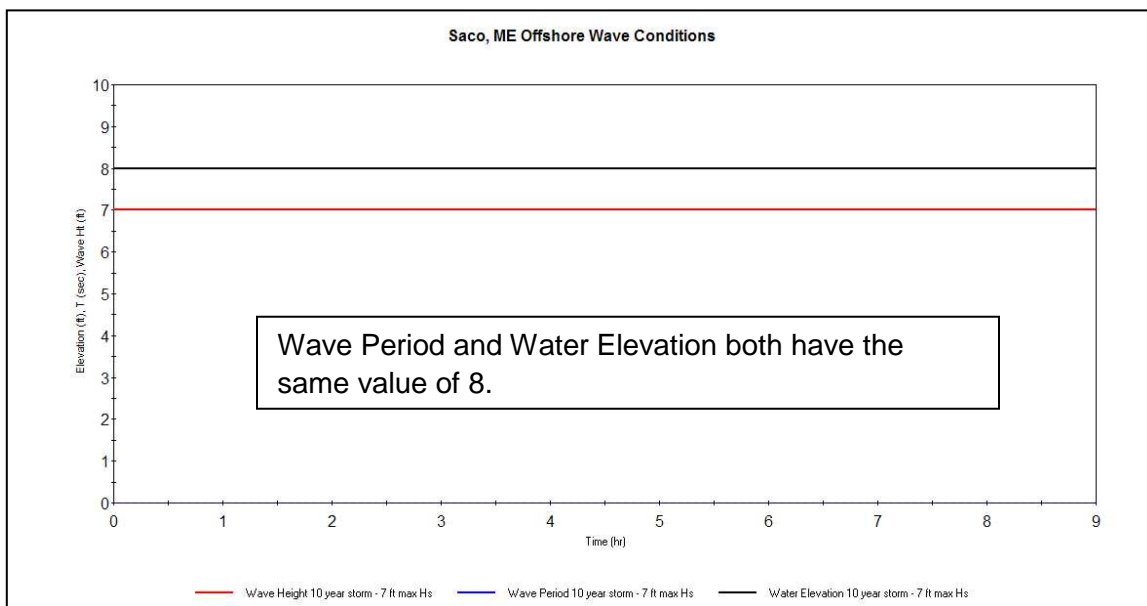


Figure 9-10. Idealized storm (used) (elevations in ft-NAVD88)

9.10 Alternative Modeling

As can be seen in Table 9-3, the model output conditions provided by WHG for the various alternatives covered a range of wave heights. Instead of developing an idealized storm with each exact wave height that resulted from the various alternatives, storms were developed with wave heights ranging from 4.5

feet to 7.0 feet in 0.5 foot increments. This led to six idealized storms. All storms had the same duration, water level, and period, with the only difference being wave height. Remember all were representing the same storm but the reduced wave heights were used to account for the wave height reduction provided by the nearshore alternative structures. The storm shown in Figure 9-10 is representative of these storms with the exception of wave height. These storms were then modeled in SBEACH for each of the beach berm alternatives shown in Figure 9-3 and the output profiles were recorded. Specifically, for each berm and each idealized storm combination, the minimum profile experienced during the storm was recorded. The minimum profile was used since the intent of the Section 111 project was to mitigate erosion impacts from the jetties.

In order to measure and plot the minimum profile results, significant consideration was given to a proper or meaningful benchmark to measure the profiles with. The first measurement that was used was simply the beach berm width at a particular contour. An example of this method has been provided in Figure 9-11.

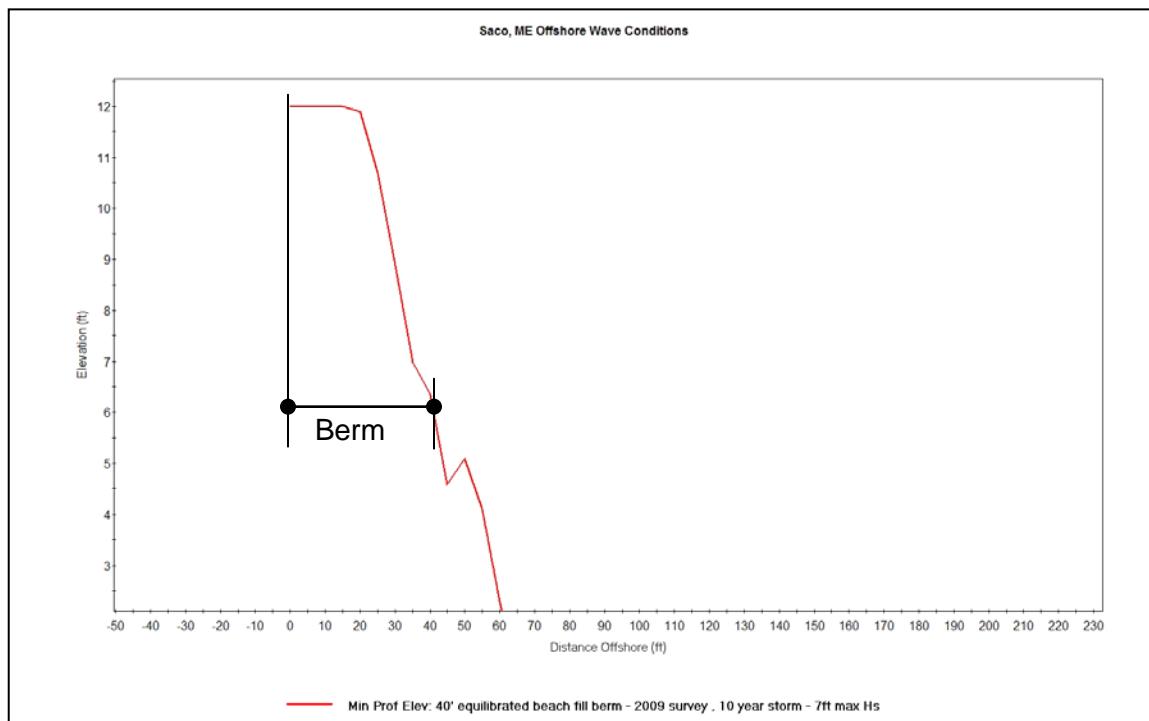


Figure 9-11. Eroded profile measurement - berm width (elevations in ft-NAVD88)

This information was taken from the output tables provided in SBEACH. This metric was found to be inconsistent in that smaller beach berms sometimes were shown to significantly outperform wider berms for the same wave height. This was typically due to anomalies in beach shape near the shoreline that caused the beach width to appear wider. It was then decided to look at remaining beach volume above a particular contour to measure performance. This method provided more consistent results that behaved in a fashion that one would expect. Wider beach berms typically performed better than narrower ones. The exact contour used did take some consideration and several were looked at. After looking at the results of using various contours and discussions with the Project Development Team

(PDT), the volume above the 6 ft-NAVD88 contour was chosen as the indicator that should be used. As noted, the peak water level for a 10 year storm is 8 feet. This would mean that even if the beach was just above the 6 feet contour the water depth would only be 2 feet. This would allow minimum wave energy to reach behind the beach and the waves would be completely broken at that point. This contour also corresponds to the elevation that the “healthier” profile was eroded to during the full 10 year storm run (shown in Figure 9-7). Also, as noted in Sections 2 and 4, the shoreline is armored with rip rap in many areas. While this rip rap was not designed using coastal engineering criteria it does provide protection, which is evident in the fact at the time of writing this report it was providing protection in a much more eroded beach conditions then would exist following a beach fill). An example of the volume measurement has been provided in Figure 9-12.

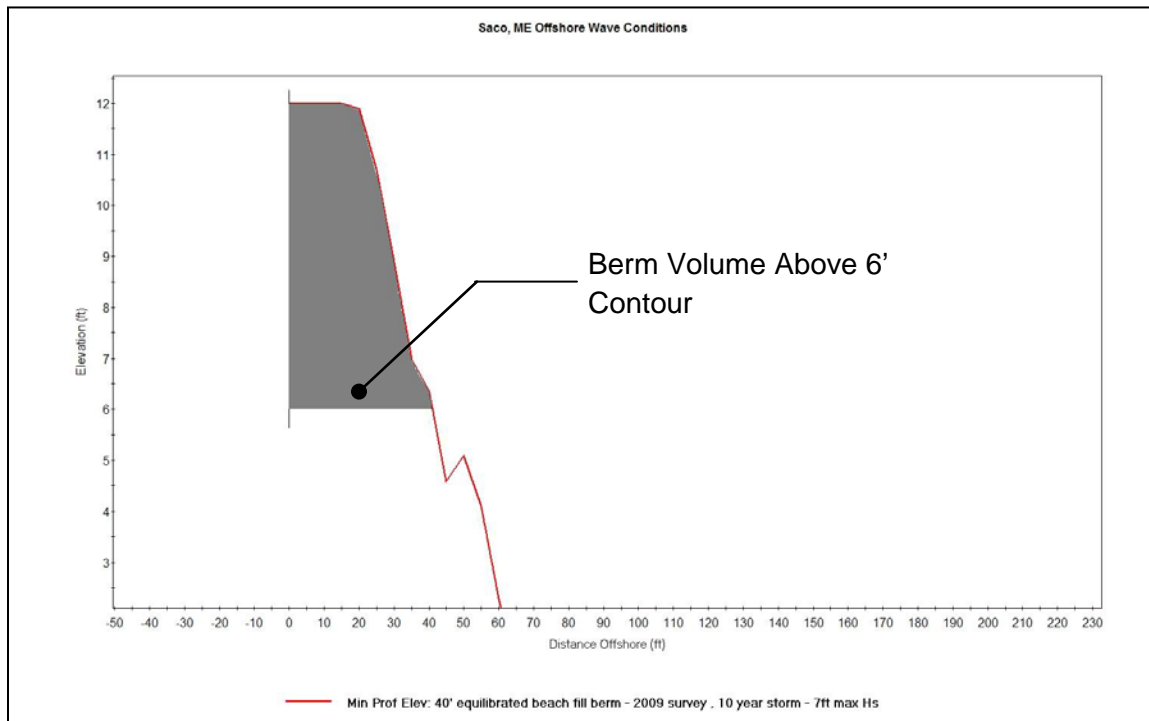


Figure 9-12. Eroded profile - berm volume (elevations in ft-NAVD88)

9.11 SBEACH Modeling Results

Each of the six idealized storms were run on the six beach fill alternatives, resulting in thirty six output volumes above the 6 ft contour from the minimum profiles experienced during each run. The resulting volume (yds³/ft of beach) from each model run for each storm and berm combination was plotted and has been provided as Figure 9-13. As an example of how to read the graph, if the performance for a 40 foot berm and for a 6.5 foot wave condition is desired the reader needs to find the 6.5 ft wave height on the X-axis, follow it until it intersects the 40 foot berm line on the graph, and then look to the left on the Y-axis for the remaining volume above the 6 ft contour. For this case the remaining volume above the 6 ft-NAVD88 contour is 2.6 yds³/ft of beach. As shown in Figure 9-13, best fit polynomial lines were plotted for the output data. This smoothed out some of the variation in the model output.

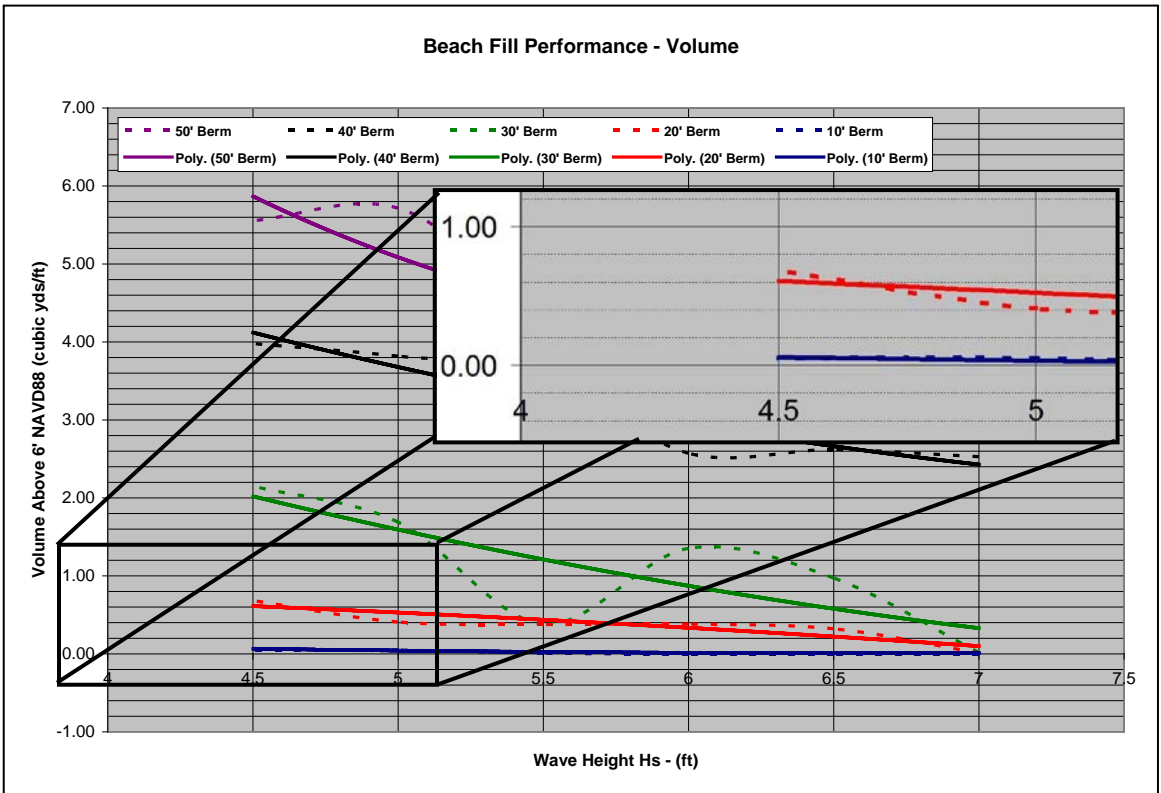
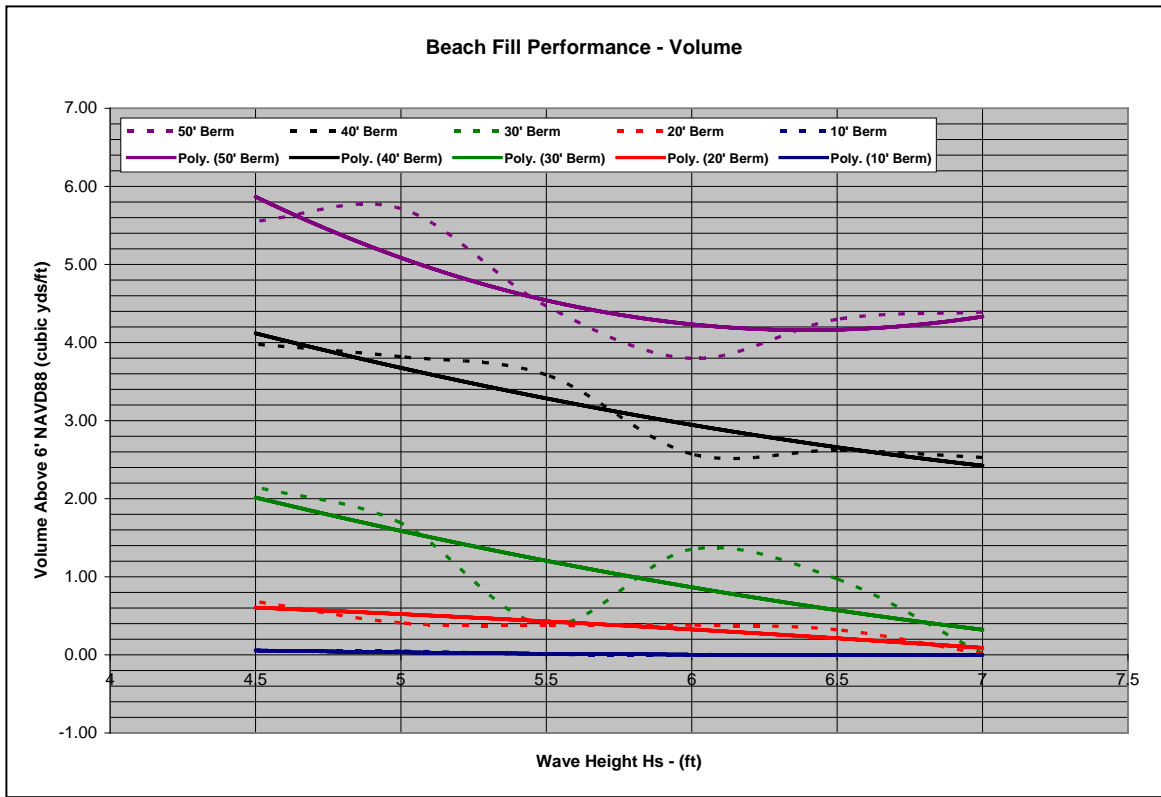


Figure 9-13 (a and b). Remaining volume above the 6 ft-NAVD88 contour for each alternative and zoom in.

9.12 Minimum Required Beach

9.12.1 Minimum Berm Widths

From the modeling results provided in Section 9.11 (Figure 9-13) and the modeled wave heights for each alternative provided in Table 9-3 it was possible to determine the minimum beach berm width that needed to be in place for each alternative to prevent shoreline erosion during a 10 year storm.

As shown in Table 9-3, the wave conditions are reduced near shore for each of the structural alternatives. For the wave heights provided by WHG that fall between the 0.5 foot intervals modeled, interpolation was used to determine the berm width. As an example:

From Table 9-3, for Alternative 25A, it can be seen that the wave heights in area A and B are 4.8 ft and 6.4 ft, respectively. Using these wave heights with Figure 9-13, it can be seen that for a 4.8 ft wave, and for beach to be present at the 6 foot contour a 10 ft berm width beach needs to be in place to protect area A. For the 6.4 ft wave it can be seen that there is zero beach at the 6 ft contour for a 10 ft berm so a jump up to the 20 ft berm is necessary to protect area B.

**As shown in the zoomed in area it can be seen that there is not a significant volume for 25A above the 6 foot contour but there is volume. Considering the level of protection provided by the breakwater structures this minimum volume was deemed acceptable.*

The method presented in the above example was used to determine the minimum beach width needed for each alternative presented in Table 9-3. The resulting minimum beach widths have been presented in Table 9-4.

As shown in the table, the beach fill only option, with no structural protection, needs a wider beach than the alternative 25A with structures. As the extent of structural protection increases, the size of the beach berm necessary for protection against a storm is reduced as can be expected. However, there was no difference in minimum beach width between the Beach Fill Only Alternative and Alternative 6. This is due to rounding during the analysis and the use of 10 foot beach berm increments. If a smaller increment of beach berm width was used, Alternative 6 would have shown that a smaller berm width is needed. It must also be remembered that even with the adjustments made to the SBEACH model input and the other accommodations made, the results from SBEACH must be used in the proper context. Since the Beach Fill Only Alternative does not address the reflected wave off of the North Jetty, storm performance is very likely overstated in the SBEACH modeling effort. While the wave heights used are taken from the WHG wave modeling effort, SBEACH does not account for wave angle, and therefore the increased sediment transport impacts, of the North Jetty reflected wave.

Table 9-4. Minimum required beach fill berm widths for each alternative

| Beach Fill Only | Area A³ | Area B³ | Area C |
|---|---------------------------|---------------------------|---------------|
| H _s (ft) (10-year Storm conditions) | 6.4 | 7.0 | 5.9 |
| Beach Berm Width Needed (ft)¹ | 20.0 | 30.0 | |
| | | | |
| | | | |
| Case 6 - Spur Jetty | Area A³ | Area B³ | Area C |
| H _s (ft) (10-year Storm conditions) | 6.1 | 6.7 | 5.8 |
| Energy Reduction (10-year Storm conditions) | 10.9% | 6.9% | 3.7% |
| Beach Berm Width Needed (ft)¹ | 20 | 30 | |
| | | | |
| | | | |
| Case 25A - Spur/2 Breakwaters | Area A³ | Area B³ | Area C |
| H _s (ft) (10-year Storm conditions) | 4.8 | 6.4 | 5.8 |
| Energy Reduction (10-year Storm conditions) | 52.6% | 18.1% | 3.7% |
| Beach Berm Width Needed (ft)¹ | 10 | 20 | |
| | | | |
| | | | |
| Case 25 - Spur/3 Breakwaters | Area A³ | Area B³ | Area C |
| H _s (ft) (10-year Storm conditions) | 4.8 | 4.5 | 5.8 |
| Energy Reduction (10-year Storm conditions) | 52.6% | 66.6% | 3.7% |
| Beach Berm Width Needed (ft)¹ | 10 | 10 | |
| | | | |
| | | | |

9.12.2 Minimum Berm Volumes

Once the minimum berm widths were determined in Section 9.12.1, the conversion was made to minimum beach fill volumes. As provided in Table 9-1, beach fill volumes were determined in RMAP for each beach fill berm width. The volumes for the various berm widths were used corresponding to the minimum beach widths that are summarized in Table 9-4. For determining the volumes for each alternative it was assumed that the project length of 3,250 feet was divided equally between the wave model analysis areas A and B. This was not exactly the case since the 3,250 feet extends slightly into area C, but for this level of calculation it was reasoned that this assumption was acceptable.

The information from Table 9-1 was used with the berm width information in Table 9-4 to determine the minimum beach fill volumes or non sacrificial beach fill volumes. A simple multiplication of beach fill length and the beach fill volume/ft of beach provided in Table 9-1 was performed. As explained, these are the volumes that need to be in place to prevent erosion during a 10 year storm. The volumes

calculated are provided in Table 9-5 and can be seen under the columns labeled “Without Profile Cutoff from Structures”.

Table 9-5. Minimum required beach fill volumes for each alternative

| Beach Fill Only | | | | | | |
|--|--|----------------------------|--|-----------------------------|-------------------------------|---------------------------|
| | Area A³ | | Area B³ | | | |
| Beach Berm Width Needed (ft) ¹ | 20.0 | | 30.0 | | | |
| Beach Berm Volume (yds ³) ² | 146,733 | | 163,724 | | | |
| Total for Areas A and B | 310,456 | | | | | |
| Case 6 - Spur Jetty | | | | | | |
| | Without Profile Cutoff From Structures | | With Profile Cutoff From Structures | | | |
| | | | Area A³ | | Area B³ | |
| | Area A³ | Area B³ | Unprotected (875 ft) | Protected (750 ft) | Unprotected (1,625 ft) | Protected (0 ft) |
| Beach Berm Width Needed (ft) ¹ | 20 | 30 | 20 | 20 | 30 | 30 |
| Beach Berm Volume (yds ³) ² | 146,733 | 163,724 | 79,010 | 57,970 | 163,724 | 0 |
| Total for Areas A and B | 310,456 | | 300,703 | | | |
| Case 25A - Spur/2 Breakwaters | | | | | | |
| | Without Profile Cutoff From Structures | | With Profile Cutoff From Structures | | | |
| | | | Area A³ | | Area B³ | |
| | Area A³ | Area B³ | Unprotected (575 ft) | Protected (1,050 ft) | Unprotected (1,375 ft) | Protected (250 ft) |
| Beach Berm Width Needed (ft) ¹ | 10 | 20 | 10 | 10 | 20 | 20 |
| Beach Berm Volume (yds ³) ² | 129,686 | 146,733 | 45,889 | 63,261 | 124,158 | 17,094 |
| Total for Areas A and B | 276,419 | | 250,403 | | | |
| Water Management Beach Fill Design Volumes - Non Sacrificial Fill Volumes | | | | | | |
| Alternative | Non Sacrificial Fill Volume (yds³) | | Profile Cutoff Volume Reduction | | | |
| | Without Profile Cutoff | With Profile Cutoff | Volume (yds³) | Percent Reduction | | |
| Beach Fill Only | 310,456 | 310,456 | 0 | 0.00% | | |
| Case 6 - Spur Jetty | 310,456 | 300,703 | 9,753 | 3.14% | | |
| Case 25A - Spur/2 Breakwaters | 276,419 | 250,403 | 26,016 | 9.41% | | |

The second iteration of the beach fill design was to take into account the benefits of the spur groins and breakwaters in Alternatives 6 and 25A. As discussed in Section 9.4.2, breakwaters or spur groins typically cutoff a beach profile and therefore limit the cross shore equilibration of the beach profile following a fill placement. Provided in Table 9-2 are the reduced volumes required per foot of beach for the various structural alternatives. The formulation for determining the fill volume was made by multiplying the reduced beach fill volume density for the associated minimum beach berm width in Table 9-1 times the appropriate length of beach. This resulted in a slight reduction in the non sacrificial beach fill volume which can be seen in Table 9-5 under the columns labeled “With Profile Cutoff from Structures”. As seen at the bottom of Table 9-5, the non sacrificial beach volumes are summarized. It was shown that by considering the profile cutoff, only a 3% reduction in fill volume was calculated for alternative 6 and less than a 10% reduction in volume was realized for Alternative 25A. Since these fill volume reductions were not very significant and fall within the error of the calculations being performed, the volumes adjusted for the profile cutoffs were not used.

The final iteration of the beach fill volume determination was estimated using 3-D surfaces in the Microstation software. As discussed, the values determined previously were made using single profile and did not take into account the non uniform shape of the shoreline, and that the beach fill would be tapered into the natural beach shape at the north end of the project. This can be seen in Figure 9-13.

Using Micro Station and the full X,Y,Z point data set available, more exact volumes were calculated. As expected, the minimum beach fill volumes or non sacrificial beach fill volumes found were less than the simplified, single profile, volumes calculated. A comparison has been provided in Table 9-6. On average the actual design values are 25% less than the preliminary design values. The volumes calculated using Micro Station and the 3-D surfaces should be considered the more accurate of the two and should be used during further analysis and design.



Figure 9-13. Irregular shoreline and beach taper

Table 9-6. Minimum Beach Fill Volume Comparison – Water Management vs. Civil Design

| Alternative | Water Management Volumes (yds ³) | Civil Design Volumes ¹ (yds ³) | Volume Difference (yds ³) | % Difference |
|---|--|---|---------------------------------------|--------------|
| Case 6 - Spur Jetty | 310,456 | 233,589 | 76,867 | 24.76% |
| Case 25A - Spur/2 Breakwaters | 276,419 | 198,040 | 78,379 | 28.36% |
| Case 25 - Spur/3 Breakwaters | 259,373 | 169,621 | 89,752 | 34.60% |
| ¹ Civil Design volumes should be used for final numbers and final analysis | | | | |

As discussed earlier, the volumes shown in Table 9-6 are the volumes that the beach should not be allowed to go below, or should be considered the trigger point for when a beach should be re-nourished as soon as possible. The volumes provided are what was calculated to be necessary to protect against a 10 year storm. Perhaps the proper perspective, as discussed previously, is to consider the volumes in Table 9-6 as beach nourishment to restore the beach to a minimum protection level and that the actual

beach fills and re-nourishments will be placed on top of this fill. This will be discussed further in Section 10.

10.0 BEACH FILL RENOURISHMENT ANALYSIS

As discussed in Section 9, the minimum beach berm widths and volumes needed to prevent shoreline erosion during a 10 year storm were determined. Also discussed, was the view that these should be considered non sacrificial beach fills. In order to prevent the non sacrificial fill from being eroded, additional beach fill needs to be placed that can be sacrificed to beach erosion. As the sacrificial fill or nourishment fill erodes, and the beach approaches the minimum volumes provided in Table 9-6, a re-nourishment project would need to be scheduled. This will require forecasting and forethought since it often takes several years to plan for and construct a beach re-nourishment project. This will be an essential component to this project and must be addressed during plan formulation.

10.1 Beach Fill Longevity Calculation Methodology

The optimum nourishment volume and re-nourishment volume/schedule is dependent on economic formulation and will not be provided in this appendix. However, information provided by WHG regarding beach fill longevity has been provided below as Figure 10-1 and as Table 10-1. Within the figure and table are the anticipated beach fill longevity associated with each alternative. The methodologies for developing this information have been provided in the WHG report, Section 12.5.5, Appendix C.

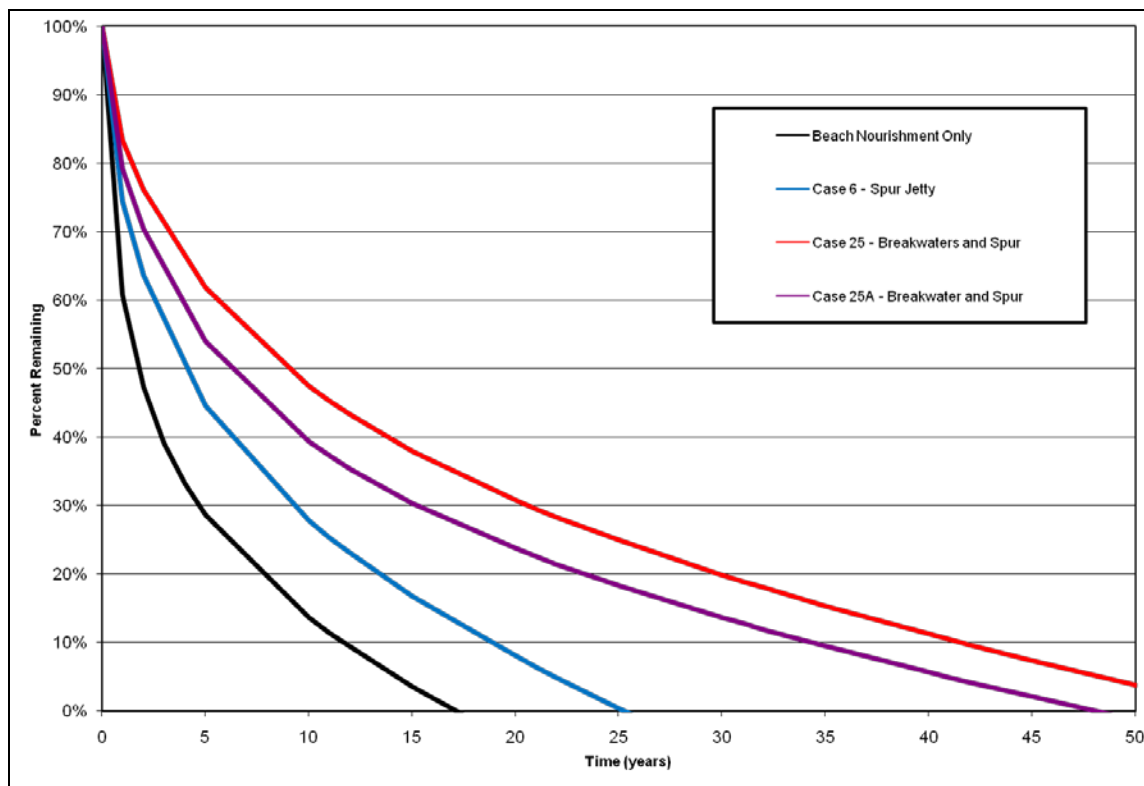


Figure 10-1. Beach fill longevity (graphical presentation)

Table 10-1. Beach fill longevity (table presentation)

| Beach Nourishment Only | | Case 6 – Spur Jetty | | Case 25 – Breakwaters and Spur | | Case 25A – Breakwaters and Spur | |
|------------------------|--------------------------|---------------------|--------------------------|--------------------------------|--------------------------|---------------------------------|--------------------------|
| Time (yrs) | % Remaining in fill area | Time (yrs) | % Remaining in fill area | Time (yrs) | % Remaining in fill area | Time (yrs) | % Remaining in fill area |
| 1 | 60.8% | 1 | 74.4% | 1 | 83.3% | 1 | 79.2% |
| 2 | 47.3% | 2 | 63.6% | 2 | 76.2% | 2 | 70.3% |
| 3 | 39.1% | 5 | 44.6% | 5 | 61.9% | 5 | 54.0% |
| 4 | 33.2% | 10 | 27.8% | 10 | 47.5% | 10 | 39.4% |
| 5 | 28.6% | 11 | 25.3% | 11 | 45.3% | 11 | 37.3% |
| 10 | 13.6% | 12 | 23.0% | 12 | 43.3% | 12 | 35.4% |
| 11 | 11.4% | 15 | 16.8% | 15 | 38.0% | 15 | 30.4% |
| 12 | 9.3% | 20 | 8.0% | 20 | 30.8% | 20 | 23.7% |
| 15 | 3.6% | 21 | 6.4% | 21 | 29.6% | 21 | 22.6% |
| 20 | | 22 | 4.8% | 22 | 28.3% | 22 | 21.4% |
| 21 | | 25 | 0.4% | 25 | 25.0% | 25 | 18.3% |
| 22 | | 30 | | 30 | 19.9% | 30 | 13.7% |
| 25 | | 31 | | 31 | 19.0% | 31 | 12.8% |
| 30 | | 32 | | 32 | 18.0% | 32 | 11.9% |
| 31 | | 35 | | 35 | 15.4% | 35 | 9.5% |
| 32 | | 40 | | 40 | 11.3% | 40 | 5.7% |
| 35 | | 41 | | 41 | 10.5% | 41 | 4.9% |
| 40 | | 42 | | 42 | 9.7% | 42 | 4.2% |
| 41 | | 45 | | 45 | 7.5% | 45 | 2.1% |
| 42 | | 50 | | 50 | 3.8% | 50 | |
| 45 | | | | | | | |
| 50 | | | | | | | |

Based upon discussions with WHG it was concluded that when utilizing Figure 10-1 and Table 10-1, the user should not assume that the beach can erode down to 0% volume because that would be an incorrect use of this information. Instead a cutoff volume should be used. This also makes sense due to the uncertainty within the beach fill longevity information. It was decided, based upon engineering

judgment, that 10 yds³/ft of beach “cushion” should be assumed to account for modeling errors and uncertainty. For the 3,250 foot long beach fill, this equates to 32,500 cubic yards of fill or if 100,000 yds³ of sacrificial beach fill were initially placed a re-nourishment should be constructed when 33% of the fill is remaining. If 200,000 yds³ were placed then a re-nourishment should be completed when 16% of the initial fill is remaining. Once again this “cushion” is based on judgment and no modeling or analysis was done. One could make the argument that this cushion should be greater for the beach fill only option since this is a “riskier” alternative than the structural alternatives, but choosing these cushion levels for each alternative would even be more arbitrary. It should be assumed, as will be discussed in Section 12 that structural alternatives will contain less risk than beach fill alone.

10.2 Beach Fill Volume Calculation – 20, 30, and 40 FT Berm Widths

As mentioned, the optimum nourishment and re-nourishment volume/schedule will not be provided in this appendix. Three re-nourishment volumes/schedules have been provided in Tables 10-2, 10-3 and 10-4 using the longevity information and the cushion information discussed in the above paragraph. The results in Table 10-2 provide the actual calculated volumes. As a reminder, and as shown in Section 9.12.2, the sand fill volume savings associated with the beach profile being cut off by the nearshore breakwaters and spur groin were not significant with a savings of only a 3% reduction in fill volume for alternative 6 and less than a 10% reduction in volume for Alternative 25A. Since these fill volume reductions were not very significant and fall within the error of the calculations being performed, the beach fill volumes do not take into account the profile cutoff from the structures.

Table 10-2. Beach fill lifetime volumes and re-nourishments (non-adjusted model results)

| 20 ft beach nourishment berm - sacrificial fill (65,000 yds ³ nourishment) | | | | | | | |
|--|---|---|---|---------------------|----------------|--------------------------|------------------------------------|
| Alternative | Initial Fill (non sacrificial) yds ³ | Initial Fill (sacrificial) yds ³ | Total Initial Fill (non- sacrificial + sacrificial fill) yds ³ | Renourishment Fills | | Number of Renourishments | Total Sand Volume yds ³ |
| | | | | yds ³ | interval (yrs) | | |
| Beach Fill | 233,589 | 65,000 | 298,589 | 32,500 | 1.5 | 33.3 | 1,381,922 |
| 6 | 233,589 | 65,000 | 298,589 | 32,500 | 4.5 | 11.1 | 659,700 |
| 25A | 198,040 | 65,000 | 263,040 | 32,500 | 6.4 | 7.8 | 516,946 |
| 30 ft beach nourishment berm - sacrificial fill (97,500 yds ³ nourishment) | | | | | | | |
| Alternative | Initial Fill (non sacrificial) yds ³ | Initial Fill (sacrificial) yds ³ | Total Initial Fill (non- sacrificial + sacrificial fill) yds ³ | Renourishment Fills | | Number of Renourishments | Total Sand Volume yds ³ |
| | | | | yds ³ | interval (yrs) | | |
| Beach Fill | 233,589 | 97,500 | 331,089 | 65,000 | 4.1 | 12.2 | 1,123,772 |
| 6 | 233,589 | 97,500 | 331,089 | 65,000 | 8.4 | 6.0 | 720,311 |
| 25A | 198,040 | 97,500 | 295,540 | 65,000 | 13.2 | 3.8 | 541,752 |
| 40 ft beach nourishment berm - sacrificial fill (130,000 yds ³ nourishment) | | | | | | | |
| Alternative | Initial Fill (non sacrificial) yds ³ | Initial Fill (sacrificial) yds ³ | Total Initial Fill (non- sacrificial + sacrificial fill) yds ³ | Renourishment Fills | | Number of Renourishments | Total Sand Volume yds ³ |
| | | | | yds ³ | interval (yrs) | | |
| Beach Fill | 233,589 | 130,000 | 363,589 | 97,500 | 6.2 | 8.1 | 1,149,879 |
| 6 | 233,589 | 130,000 | 363,589 | 97,500 | 11.6 | 4.3 | 783,848 |
| 25A | 198,040 | 130,000 | 328,040 | 97,500 | 19.0 | 2.6 | 584,619 |

10.3 Beach Fill Calculation – Beach Fill Only Re-nourishment Interval

Following discussions with the PDT regarding the results shown in Table 10-2, it was concluded the beach fill alternatives that required short renourishment intervals were not feasible. This essentially removed the 20 ft beach berm alternative from further consideration. Additionally, as shown in Table 10-2, the Beach Fill Only alternative requires at least double the number of renourishments compared to

Alternatives 6 and 25A for the 30 ft and 40 ft beach fill berm sizes. After discussions with the PDT, it was decided that to get a more accurate comparison between the alternatives, the beach fill volume for the Beach Fill Only Alternative should be increased to a level that would result in a renourishment interval that more closely approximated Alternatives 6 and 25A. Based on the renourishment intervals shown in Table 10-2 for Alternatives 6 and 25A, for the 30 ft and 40 ft beach berm widths, a 10 year re-nourishment interval was chosen for the Beach Fill Only Alternative. The calculations to determine the initial fill volume and the subsequent renourishment volumes were calculated using Table 10-1 and were essentially the same used to generate Table 10-2, but performed in reverse. The results of the 10 year renourishment interval for the Beach Fill Only Alternative have been provided in Table 10-3, which is Table 10-2 with updated values. The updates have been highlighted in yellow.

Table 10-3. Beach fill lifetime volumes and re-nourishments (10 year re-nourishment for Beach Fill Only)

| 30 ft beach nourishment berm - sacrificial fill (97,500 yds ³ nourishment) ¹ | | | | | | | |
|---|-----------------------------------|-------------------------------|---|---------------------|----------------|-----------------------------|---------------------------------------|
| Alternative | Initial Fill (non sacrificial) | Initial Fill (sacrificial) | Total Initial Fill (non- sacrificial + sacrificial fill) | Renourishment Fills | | Number of Renourishments | Total Sand Volume yds ³ |
| | yds ³ | yds ³ | yds ³ | yds ³ | interval (yrs) | | |
| Beach Fill | 233,589 | 239,000 | 472,589 | 207,000 | 10.0 | 5.0 | 1,507,589 |
| 6 | 233,589 | 97,500 | 331,089 | 65,000 | 8.4 | 6.0 | 720,311 |
| 25A | 198,040 | 97,500 | 295,540 | 65,000 | 13.2 | 3.8 | 541,752 |
| 40 ft beach nourishment berm - sacrificial fill (130,000 yds ³ nourishment) ¹ | | | | | | | |
| Alternative | Initial Fill (non sacrificial) | Initial Fill (sacrificial) | Total Initial Fill (non- sacrificial + sacrificial fill) | Renourishment Fills | | Number of Renourishments | Total Sand Volume yds ³ |
| | yds ³ | yds ³ | yds ³ | yds ³ | interval (yrs) | | |
| Beach Fill | 233,589 | 239,000 | 472,589 | 207,000 | 10.0 | 5.0 | 1,507,589 |
| 6 | 233,589 | 130,000 | 363,589 | 97,500 | 11.6 | 4.3 | 783,848 |
| 25A | 198,040 | 130,000 | 328,040 | 97,500 | 19.0 | 2.6 | 584,619 |

¹. For this table the Beach Fill Only volume was calculated for a 10 year re-nourishment interval, while the renourishment volumes and schedule for Alternatives 6 and 25A were calculated based on volume loss rates as previously discussed.

10.4 Beach Fill Volume Calculation – Uncertainty Consideration

As discussed in Section 9.9.1, it was expected that the analysis results of the Beach Fill Only Alternative were likely underestimating the non sacrificial beach fill requirements and for many of the same reasons it was concluded that the beach fill re-nourishment rates reported in the WHG report were likely to be under estimated. To address this issue, and as discussed in Section 9.9.1, the renourishment volume for the Beach Fill Only Alternative was doubled. This was based on engineering judgment of the significant nature of the forcing from the reflected wave conditions and Mach stem currents that exist and also the past performance of beach nourishments at the project site. The volume of the initial sacrificial fill volume for the Beach Fill Only Alternative in Table 10-3 was doubled as well as the renourishment fill volume. No adjustments were made to Alternative 6 since it was reasoned the spur groin would reduce or eliminate most of the impacts of the jetties related to increased wave climate and circulation issues. This was also the case for Alternative 25A. The altered values are provided in Table 10-4. The increases in sacrificial fill volumes for the Beach Fill Only Alternative increase the effectiveness of the plan to the point where it could be favorably compared to Alternatives 6 and 25A. However, from a coastal engineering perspective, the Beach Fill Only Alternative did not fully address the issues caused by the

federal navigation project. In addition, the costs were greater than Alternative 6 (discussed in Main Report).

Table 10-4. Beach fill lifetime volumes and re-nourishments (engineering judgment considered)

| 30 ft beach nourishment berm - sacrificial fill (97,500 yds ³ nourishment) ¹ | | | | | | | |
|---|-----------------------------------|-------------------------------|---|---------------------|----------------|-----------------------------|---------------------------------------|
| Alternative | Initial Fill (non sacrificial) | Initial Fill (sacrificial) | Total Initial Fill (non- sacrificial + sacrificial fill) | Renourishment Fills | | Number of Renourishments | Total Sand Volume yds ³ |
| | yds ³ | yds ³ | yds ³ | yds ³ | interval (yrs) | | |
| Beach Fill | 233,589 | 478,000 | 711,589 | 414,000 | 10.0 | 5.0 | 2,781,589 |
| 6 | 233,589 | 97,500 | 331,089 | 65,000 | 8.4 | 6.0 | 720,311 |
| 25A | 198,040 | 97,500 | 295,540 | 65,000 | 13.2 | 3.8 | 541,752 |
| 40 ft beach nourishment berm - sacrificial fill (130,000 yds ³ nourishment) ¹ | | | | | | | |
| Alternative | Initial Fill (non sacrificial) | Initial Fill (sacrificial) | Total Initial Fill (non- sacrificial + sacrificial fill) | Renourishment Fills | | Number of Renourishments | Total Sand Volume yds ³ |
| | yds ³ | yds ³ | yds ³ | yds ³ | interval (yrs) | | |
| Beach Fill | 233,589 | 478,000 | 711,589 | 414,000 | 10.0 | 5.0 | 2,781,589 |
| 6 | 233,589 | 130,000 | 363,589 | 97,500 | 11.6 | 4.3 | 783,848 |
| 25A | 198,040 | 130,000 | 328,040 | 97,500 | 19.0 | 2.6 | 584,619 |

¹. For this table the Beach Fill Only volume was calculated for a 10 year re-nourishment interval, while the renourishment volumes and schedule for Alternatives 6 and 25A were calculated based on volume loss rates as previously discussed.

10.5 Beach Fill Volume “Size” Recommendation

Based on the findings in the coastal engineering effort of this study and the relatively high risk of this alternative (as discussed in Section 12.0), it was the “coastal engineering opinion” that the Beach Fill Only Alternative has the highest potential for failure. Although the volume of beach fill was increased to the point where it can be considered effective, mach stem, reflected wave energy and other wave forces make the fill section inherently unstable and subject to severe erosion. It is also recommended that the larger (40 ft) renourishment berm be selected for the two remaining alternatives (6 and 25A) since larger sacrificial berms result in less risk exposure to the project through the life of the project. The smaller renourishment volumes and more frequent renourishment schedule results in higher risk in two ways. The first is that the funding for re-nourishment may not be available. This is a known issue with beach fill projects. This often results in missed re-nourishment schedules. The more scheduled re-nourishments there are, the higher the probability that some re-nourishments will be missed. This inherently exposes the project to more risk since the beach will not be at an adequate width to withstand storms without shoreline erosion. The second is that even if the re-nourishments are completed on schedule the sacrificial fill beach width will reside near the minimum allowable width for more of the project life. Stated differently the sacrificial beach will be near the trigger point more often through the project life. Larger sacrificial berms have more resiliency built in and will be more resilient for greater periods of time through the life of the project. Once again, based upon the above discussion it is recommended to use the larger sacrificial berm width of 40 feet (equilibrated).

11.0 SEA LEVEL CHANGE IMPLICATIONS

11.1 Sea Level Change – Project History

During the 1990s, when this Section 111 project was initiated the issue of Sea Level Change was addressed in a 1986 memo from USACE Headquarters in which sea level change was addressed by assuming that the historical rate of sea level change would persist during the anticipated project life. In most cases the change was small and was within the calculation errors for a typical 50 year analysis period. For example, at this study site, the annual rate of sea level change has been 1.82 mm per year, or converted, 0.60 feet per 100 years, or 0.30 feet per 50 years. This means if the historic rate was used for analysis, the change that would have to be considered would be less than 4 inches. As mentioned, this small increase in water level would not be noticed in most coastal engineering analysis, and would be insignificant when compared to the variability of storm surge, tide range, localized wind setup, storm frequency, etc. Therefore during the study period that occurred in the 1990s. However, in 2000 the guidance related to sea level change was updated in ER 1105-2-100 (Planning Guidance Notebook). In the update, the analysis related to SLC was significantly increased and required USACE studies to investigate the impacts of higher future rates of SLC. The higher rates of SLC were taken from the National Research Council (NRC) 1987 report - *Changes in Sea Level: Engineering Implications, 1987*. Below is the key language taken from Planning Guidance Notebook.

k. Sea Level Rise. The National Research Council (NRC) study on sea level change (Responding to Changes in Sea Level: Engineering Implications, 1987) is a practical and rational review of data on relative sea level changes and the resulting impact on engineering structures. The study should be used by the Corps for technical guidance until more definitive data are available. The NRC study recommended that feasibility studies for coastal projects should consider the high probability of accelerated sea level rise. Since precise estimates of future sea level rise are unknown, the risks associated with a substantial rise should be addressed. Feasibility studies should consider which designs are most appropriate for a range of possible future rates of rise. Strategies that would be appropriate for the entire range of uncertainty should receive preference over those that would be optimal for a particular rate of rise but unsuccessful for other possible outcomes.

Given the release of the Planning Guidance Notebook in 2000 and that this Section 111 study was initiated well before that, and coupled with the attempt to fast track this project by pushing it directly into the Final Design phase (skipping the feasibility phase), it was decided that the guidance from 2000 should not be incorporated into the design. However, this project has experienced numerous delays, produced significant work, and is now just wrapping up 11 years of effort following the 2000 guidance. Additionally, updated SLC guidance was released by USACE in 2009 as EC 1165-2-211, which is very similar to the requirements of the Planning Guidance Notebook. The new EC requires a multiple scenario approach for three sea level rise scenarios, with each being considered equally plausible.

Given the length of time that has passed since the 2000 language release, and the release of the updated language, it was determined that SLC must be addressed in this study in a more robust manner than simply following the previous guidance of assuming historical rates for the future. However, it was also decided that for the PDT to go back and follow the extensive requirements outlined in the new EC, would not be possible do to the level of analysis that would need to be done for the various SLC scenarios. This would include all the wave modeling for the final alternatives, the associated sediment transport, the associated storm performance modeling, the associated beach fill longevity, and the structure design evaluation. The cost and time associated with this analysis would simply be too great this late in the project analysis, especially considering the long project history. However, given the strong move towards SLC, the prevailing science, and the need to determine if the proposed project could be impacted by higher SLC rates, it was decided the issue needed to be investigate to a sufficient level to address the potential issues related to SLC. Provided in the following sections is a discussion of SLC for the various project components, the performance of the project, and maintenance impacts.

11.2 Sea Level Change – Project Application

The approach was to address each component of the project and analyze impacts using engineering judgment, available science, experience with other USACE studies looking at SLC, and reaching conclusions based on this information. Lengthy discussions were conducted with the PDT regarding this matter, with two specific questions in mind, how would the SLC scenarios impact the project physically and how would the SLC scenarios impact the project economically.

11.3 Sea Level Change Projection

As described in the SLC EC 1165-2-211, USACE is required to use three projected SLC curves for a project area. These curves are; the historic rate of SLC at the project area, an intermediate SLC curve (modified NRC Curve I), and a high SLC curve (modified NRC Curve III). Formulation of the NRC curves from a defined starting date, and for localized subsidence was also provided in the EC which allows for SLC to be calculated for specific project time frames and for specific geographic areas. This is critical since SLC along the coast varies due to local subsidence, uplift, water body movement, etc. Using Equation 11-1 below, which is equation 3 from the EC, Figure 11-1 was developed for Portland, ME. Portland, ME was used since it is only 15 miles north along open coast, and the available historic SLC information from NOAA. Provided in Figure 11-2 is the long term sea level trend for Portland, ME taken from NOAA.

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_{22} - t_{12}) \quad (11-1)$$

where

t1 = is the time between the project’s construction date and 1986

t2 = is the time between a future date at which one wants an estimate for sea-level rise and 1986 .

b = 2.36E-5 for modified NRC Curve I

1.005E-4 for modified NRC Curve III

**Equation 11-1 is adjusted to include the historic global mean sea-level change rate of 1.7 mm*

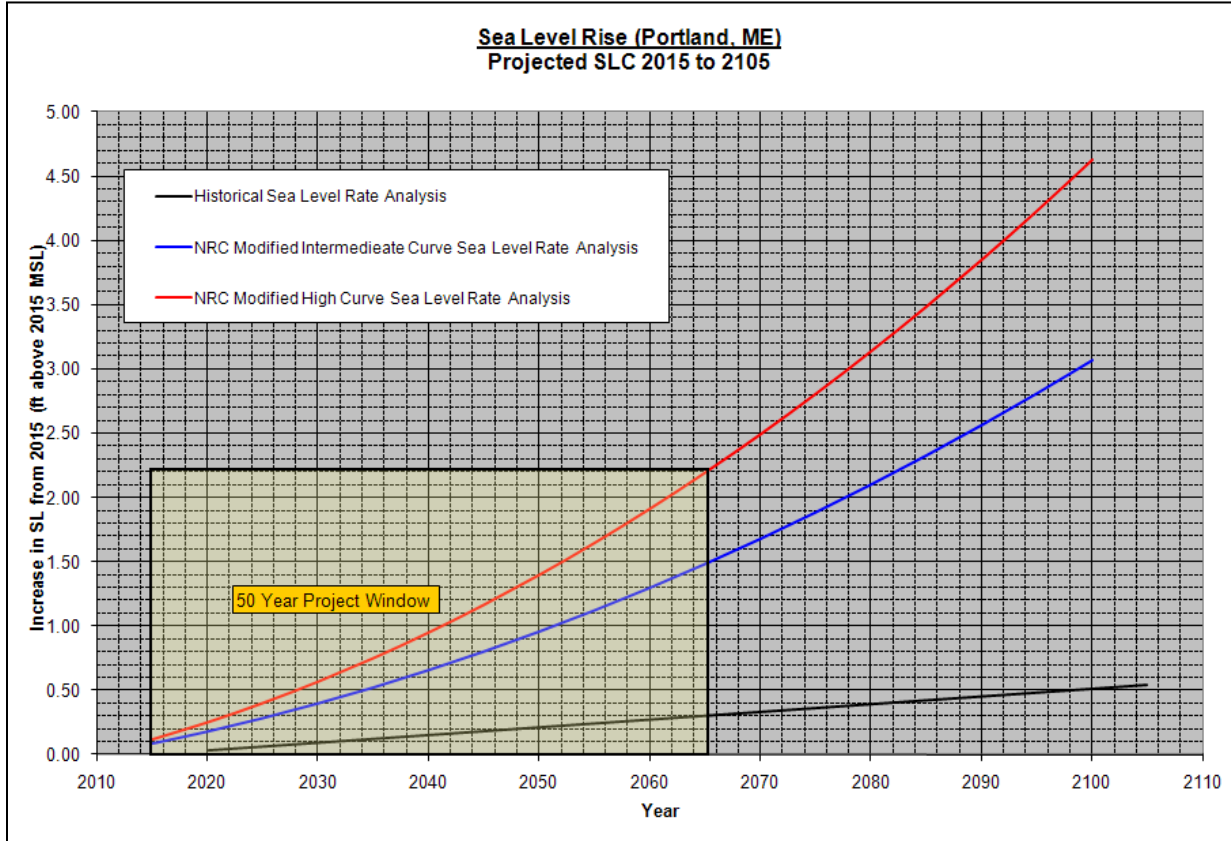


Figure 11-1. Sea level curves based upon USACE EC 1165-2-211 – start year of 2015

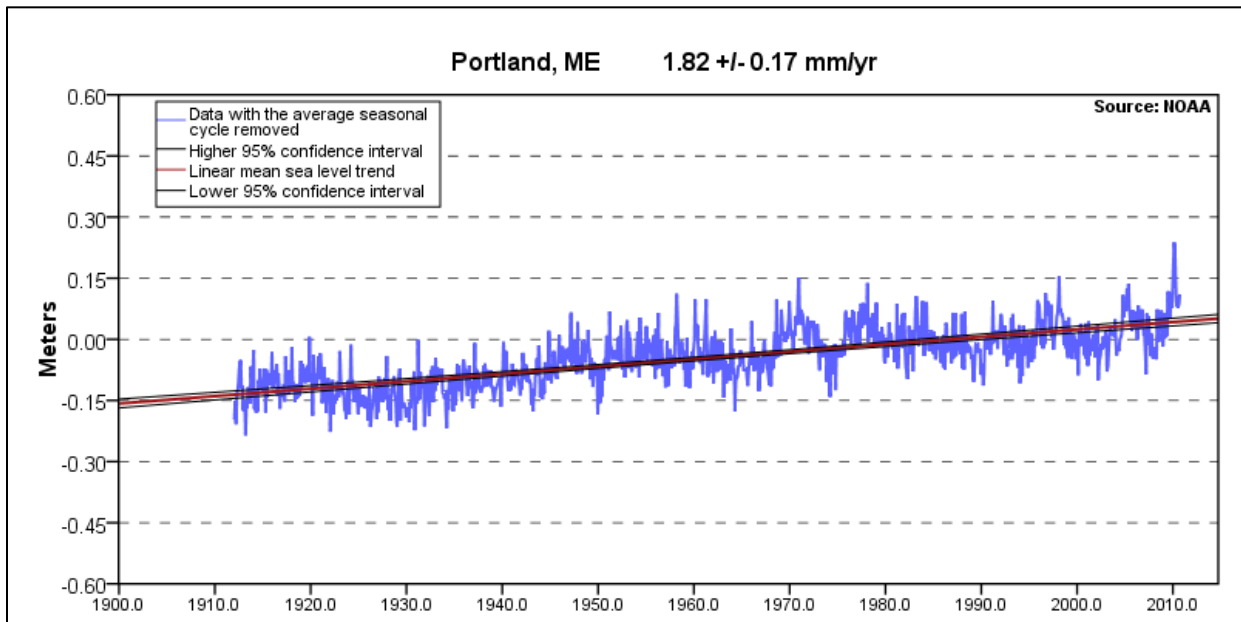


Figure 11-2. Historical sea level change trend for Portland, ME – provided by NOAA.

As can be seen in Figures 11-1, the historical rate would result in a rise of only 0.15 feet or 1.8 inches over the first 25 years of the project or 0.3 feet or 3.6 inches over 50 years. However, the level of SLR

increases for the intermediate and higher curves specified for use in the EC. As shown in Figure 11-1, for the intermediate curve, the increase in SL after 25 years is 0.65 feet or 7.8 inches and after 50 years the increase is 1.5 feet or 18 inches. For the high curve in Figure 11-1, the increase in SL during the first 25 years of project life is 0.95 feet or 11.4 inches and over 50 years is 2.2 feet or 26.4 inches. Considering these numbers, if the intermediate and high rates of sea level change do occur there may be project impacts and alterations to the performance. These higher rates of change were evaluated to better understand any potential impacts to the project alternatives and determine if alterations to the alternatives designs was warranted.

11.4 Sea Level Change Impact Discussion

As discussed in Section 11.3, the historical trend line in Figure 11-1 results in a SL increase that is small and is not expected to cause any changes in design, expected performance, project longevity or change to the system over the 50 year project evaluation. However the potential impacts for the intermediate and high curve sea level change rates could be significant to the project with the most likely impacts to the beach fill portion of the project.

11.4.1 Sea Level Change Impact – Without Project

The first project component considered for increased SLC is the without project condition. As discussed in the main report and the Economics Appendix, this project was not required to show a plus one benefit cost ratio, and as discussed previously this project is not a storm damage reduction project that is offering a defined level of protection, or risk reduction. It is in actuality an erosion reduction project. However, an economic analysis was performed for the without project condition based on anticipated erosion rates/shoreline erosion rates. Basically as the shoreline erodes at unnaturally high levels, houses and infrastructure will be lost, and eventually several businesses and a state pier will be isolated. If the rate of SLR increases, the rate of loss will also increase since erosion will increase. This would only increase the severity of the erosion and increase the need for mitigation.

As provided later in Section 11.4.5, it was calculated using the Bruun rule that for without project conditions the historic rate of SLR will cause an additional 28 feet of beach erosion over 50 years or 0.6 ft/year. For the intermediate curve the increased SLR will cause 140 feet of additional beach erosion over 50 years or 2.8 ft/yr. For the high curve the increased SLR will cause 205 feet of additional beach erosion over 50 years or 4.1 ft/yr. However, following further thought on these calculations, it was concluded that the impacts of historic SLC were already factored into the analysis since historic shoreline erosion rates were being used for the without project condition analysis. These historic shoreline erosion rates already included the historic SLR impacts on shoreline erosion. Basically the 28 feet of additional erosion was already built into the formulation. Knowing this number was useful though since it could be used to reduce the impacts of the intermediate and high curves for SLR on beach erosion. For the analysis the extra erosion from the higher SLR scenarios was added to the historic shoreline retreat rates. To avoid double counting the erosion caused by the historic rate of SLR, the 28 feet of erosion was subtracted from the intermediate and high erosion rates from SLR. This resulted in an additional loss of 112 feet of beach under the intermediate curve scenario versus 140 feet and 177 feet of additional loss under the high curve scenario versus 205 feet.

11.4.2 Sea Level Change Impact – With Project - Structure Crest Elevations

As discussed in Sections 8.6.3 and 8.6.4, the design crest elevation was fairly subjective for the existing conditions and was based upon a compilation of information, geotechnical foundation conditions, and engineering judgment. The crest elevation was not a calculated, exact value that was chosen based on tightly defined criteria nor was a particular storm event designed for. The breakwaters and groin features simply reduce the wave energy hitting the beach. With this consideration, it could be strongly argued that even for the high curve of SLR with 2.2 feet of rise over 50 years, the change is not significant. As shown in Figure 8-6, the breakwater performance is based upon submerged depth of the crest at peak water level. Even with the 2.2 foot SLR, the structure crest will only be submerged by approximately 1 foot at the peak of the storm, as shown in Figure 11-3. It is important to remember, with a 9 foot mean tide range, the structure crest will be above water for most of the tidal cycle. It is also important to keep in mind that this is the condition 50 years out, which is the end of the economic evaluation period. Looking at the midpoint at 25 years, the SLR projected under the high curve is 1 foot. Once again, given the level of design uncertainty in the crest elevation it is strongly suspected that 1 foot of rise will not be significant to the project performance.

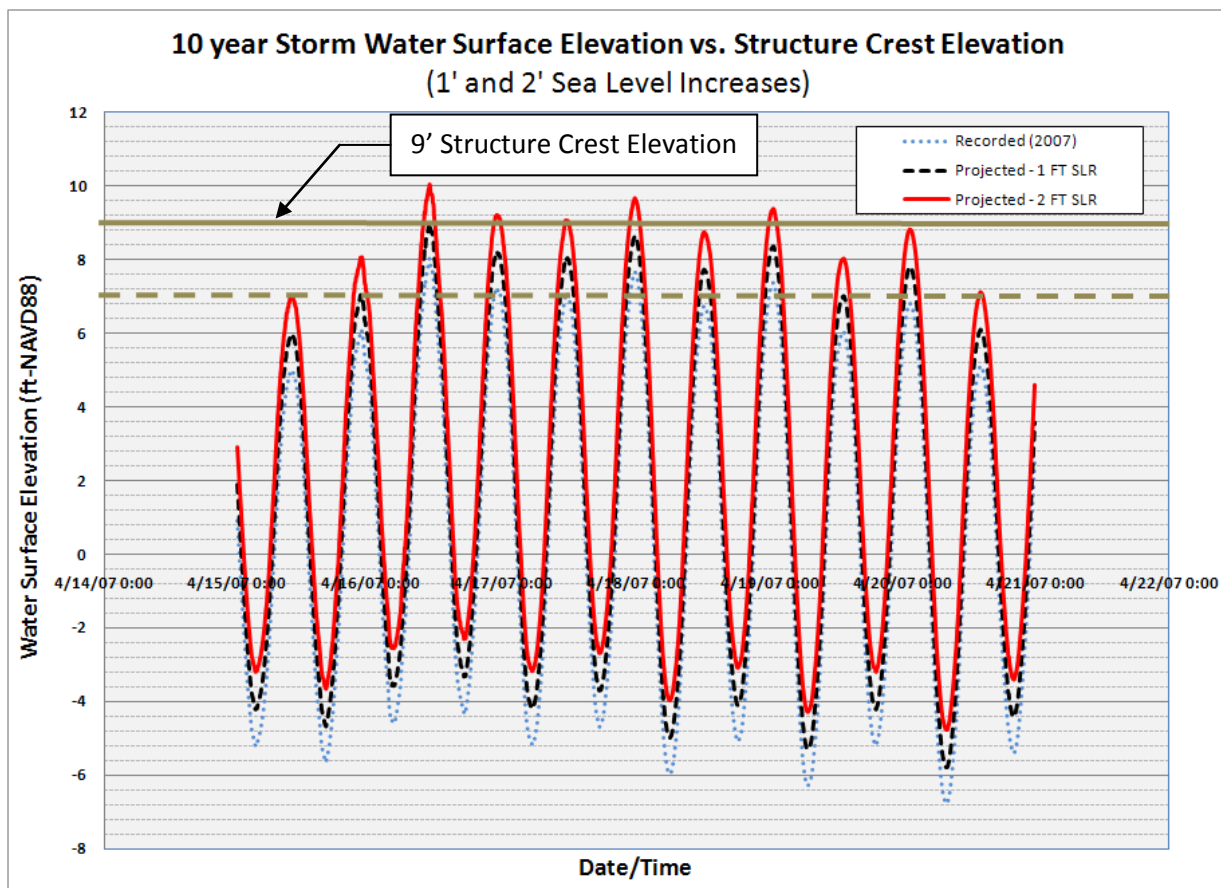


Figure 11-3. 10 year storm water elevation plot vs. crest elevation with SLR.

11.4.3 Sea Level Change Impact – Structure Maintenance

Additional consideration for the impacts of SLR to the structure is the anticipated repair schedule and potential for adaptive management. As with most coastal structures, these rubble mound alternatives will need periodic maintenance and most likely a major repair or two through the life of the project. The exact year this will occur is not possible to predict but for arguments sake, if 20 years is assumed, the SLR situation can be reassessed at that time and any necessary adjustments to the structure design could be made then. This will likely happen since past performance of the structures can be used as a check against the initial construction design. Considering further that the maintenance will likely occur multiple times through the project life reassessments can be made each time. Future repairs can be made with structural adjustments based on structure performance and based on measured SLC trends. No preemptive measures are recommended to account for the potential of SLC.

11.4.4 Sea Level Change Impact – With Project Stone Size

Sea level could impact stone size through increased wave heights reaching the structures. As discussed in Section 8.2, and shown in Equations 8-1, wave height is a cubed term in determining stone weight. Therefore, small increases in wave height can result in notable increases in stone weight. With increased sea level it is possible that incident wave heights could increase since water depth near shore has significant influence on wave transformation processes. As can be seen in Table 8-1, which summarizes design wave heights for each alternative and for various storm conditions, the design wave height is greater than the 50 year wave height for the various structural component locations by nearly 1 foot. This is due to the use of the maximum wave height experienced in the modeling efforts for sizing the stone. This means there is conservatism already built into the stone size determination.

Additionally, as discussed in the previous section related to crest elevation, before or at the time SLR becomes an issue it is likely the structure will be due for significant repairs. At that time stone size could be reassessed based upon past structure performance, new coastal processes information, and reevaluated SLC projections. Additionally, the Hudson armor stone equation is meant to result in 0% to 5% damage when design wave conditions impact the structure. If the wave height is increased due to SLC and damage is increased, the impacts will not be structure failure but an increase in damage. This could be addressed through future maintenance if the higher SLC scenarios develop and if larger wave heights result at the project.

11.4.5 Sea Level Change Impact – Beach Fill (Bruun Rule)

With a rise in sea level the beach berm will rise along with the beach profile down to the depth of closure. The volume of sand needed for the profile adjustment will cause the beach to erode (Bruun Rule). As mentioned in EC 1165-2-211, the Bruun Rule can be used to estimate the increase in horizontal beach erosion. Additionally, there is a possibility that the storm surge elevations and storm waves impacting the shore may increase due to reduced bottom friction impacts (deeper water) which could result in higher storm induced erosion rates. As shown below in Figure 11-4, the Bruun Rule is illustrated.

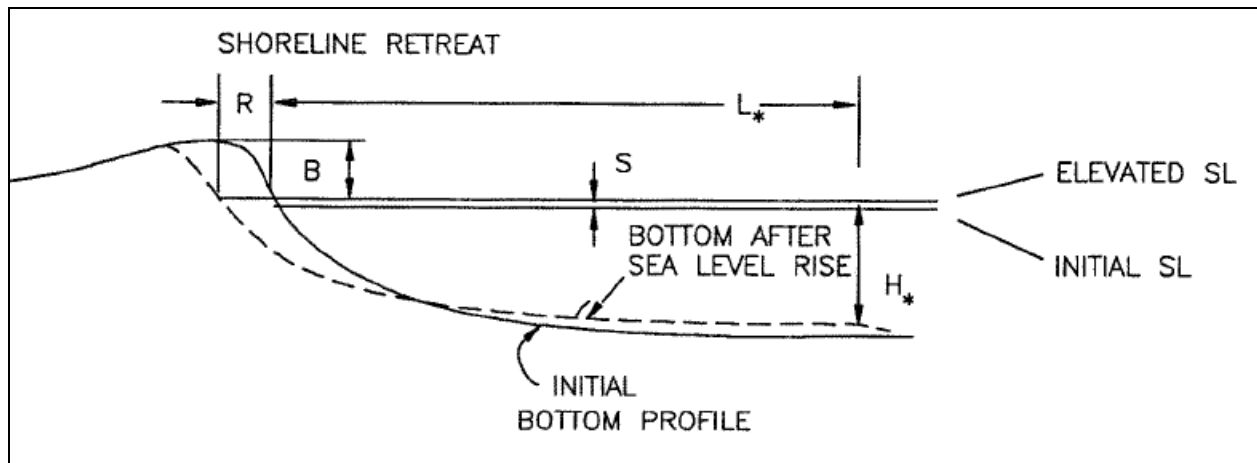


Figure 11-4. Shoreline response to SLR - Bruun Rule

As stated by Bruun, with a rise in sea level the storm berm will rise, and erosion of the shoreline will occur to make up sand volume needed for the higher storm berm. The quantity of material required to re-establish the nearshore slope must come from erosion of the shore (assuming longshore littoral transport in and out of the shoreline is equal). Bruun developed an equation to estimate shoreline recession, R , resulting from SLR.

$$R = S L / (H^* + B) \quad \text{Equation 12-1}$$

Where:

R = shoreline retreat

S = sea level rise

L = the width of the active profile (distance from depth of closure (H^*) to top of berm (B))

B = elevation of berm.

For Saco Beach, as discussed in Section 9, $L = 2,800$ ft, $B = 7$ ft, and $H^* = 23$ ft. The resulting shoreline recession varies from 0.3 ft/yr, to 0.6 ft/yr, to 1.6 ft/yr for historical, modified-NRC Curve 1, and modified-NRC Curve 3 rates, respectively.

11.4.5.1 Beach Impacts – Bruun Rule

As discussed in Section 11.4.1, increased sea level rise rates will accelerate beach erosion. This is largely due to the cross shore adjustment of the beach profile to the higher water level conditions. The Bruun Rule formulation was entered into an Excel spread sheet and the impacts of the three sea level rise scenarios were calculated. For the historical rate, one could argue that the impacts are already built into the future beach conditions since is inherently included in the long term erosion rate of the beach. WHG included this erosion rate, or background erosion rate, in their determinations of long term beach erosion and beach fill longevity. However, it was included in the spreadsheet formulations to provide

the reader with a relative sense of how much the increased sea level rise rates impact the beach compared to the historical rate. The historical rate impacts could also be added for conservatism. The spreadsheet has been provided as Table 11-1.

As shown in the table, there are three major columns which correspond to three different values of L (length of active profile offshore), which is the width of the active profile. For the without project and beach fill only conditions the value of L was selected as 2,800 feet. As discussed in Section 9.5 determining the depth of closure was not straight forward and was based upon the best information available. If the value of L should actually be greater the impacts to the beach from SLC will increase and if the value is less, the impacts of SLC to the beach from SLC will be less. Two other values of L were used as well to account for the influence of the structures cutting off the profile. The distance offshore for the two structural alternative components were used with Alternative 6 having an L value of 1,500 feet landward of the spur groin and an L value of 1,000 feet was used for Alternative 25A landward of the spur and two breakwaters.

The shorter L values have a definite impact on the beach loss due SLC. Looking at the very last row of Table 11-1, it can be seen that the L value of 1,000 feet versus the 2,800 feet cuts down the annual volume loss due to SLC by nearly a factor of three. This means that for the areas protected by the structures in Alternative 25A the impact of SLC related to beach loss, or the increase in beach renourishment requirements will be reduced by nearly 1/3 compared to a beach fill only alternative or the without project condition. However it must be remembered that the structural alternatives only cover a portion of the 3,250 foot beach within the project area. With the structures in place there will still be portions of the beach experiencing the full impact of SLC. To determine this, the information from Table 11-1 was used to determine the overall volume loss or the increase in renourishment requirements. The overall volume for the entire beach was determined by using the proportional distance protected by the structure. For example for Alternative 6, only 750 feet out of the 3,250 feet of beach length is protected by structure and the L value for that structure (distanced from shore) is 1,500 feet. This means that 23% of the beach experiences the lower impacts of SLC due to structure protection. This proportion was 40% for Alternative 25A and the L value was lower since the structures were closer. This means 25A provides more protection, or lessens the impact of SLC more than Alternative 6. The calculated volume losses or renourishment needs with consideration of structure protection have been provided as Table 11-2.

As shown in Table 2 the impacts of higher rates of SLC are considerable, especially for the Beach Fill Only alternative. In order to demonstrate the impacts, the annualized rate of increased erosion/required renourishment for each alternative was added to the renourishment volumes provided in Table 10-3 from the beach fill longevity discussion in Section 10.0. This reformulated information has been provided as Table 11-3. A comparison between Table 10-3 and 11-3 has been provided as Table 11-4, in which the project's total sand volumes have been compared.

It can be seen that the Beach Fill Only alternative requires the most additional sand to accommodate the increase in sea level. For the historic rate of SLR an additional 91,00 yds³ will be needed which really is not significant over a 50 year project life but for the high curve of SLR an additional 667,000 yds³ will be

required which is a significant volume of sand. With the structural alternatives, the volumes of extra fill required to address SLR are less than for the Beach Fill Only Alternative but still significant for the intermediate and high SLR scenarios.

As discussed, the structural alternatives will be more resilient against increased sea level rise rates. For the historic SLR the difference between Alternative 25A and the Beach Fill Only is just over 23,000 yds³ but for the high rate of SLR the difference is nearly 172,000 yds³. This further demonstrates that the Beach Fill Only alternative should not be the recommended plan.

Table 11-1. Bruun Rule formulation and results.

| Bruun Rule Analysis - Saco Alternatives - SLC | | | |
|---|---------------------------------------|-------------------------|---------------------------|
| | | | |
| SLC Rates | | | |
| Historical | | 0.3 | ft/50 yrs |
| Intermediate | | 1.5 | ft/50 yrs |
| High | | 2.2 | ft/50 yrs |
| Input Parameters | | | |
| | Alternative (profile cutoff location) | | |
| | Beach Fill Only or Without Project | Alternative 6 Structure | Alternative 25A Structure |
| L = | 2,800 | 1,500 | 1,000 |
| H* | 23 | 23 | 23 |
| B = | 7 | 7 | 7 |
| SLC Impacts - Calculated using 3 Different L Values (structures cut off profile and therefore L) | | | |
| | Beach Fill Only or Without Project | Alternative 6 Structure | Alternative 25A Structure |
| | L = 2,800 ft | L=1,500 ft | L=1,000 |
| Historical SLC Rate | | | |
| Increased Erosion Distance (ft/50 yrs) | 28 | 15 | 10 |
| Increased Annual Erosion Rate (ft/yr) | 0.6 | 0.3 | 0.2 |
| Increased Volume Loss (yds ³ /50 yrs) | 91,000 | 48,750 | 32,500 |
| Increased Annual Volume Loss (yds ³ /yr) | 1,820 | 975 | 650 |
| Intermediate SLC Rate | | | |
| Increased Erosion Distance (ft/50 yrs) | 140 | 75 | 50 |
| Increased Annual Erosion Rate (ft/yr) | 2.8 | 1.5 | 1.0 |
| Increased Volume Loss (yds ³ /50 yrs) | 455,000 | 243,750 | 162,500 |
| Increased Annual Volume Loss (yds ³ /yr) | 9,100 | 4,875 | 3,250 |
| High SLC Rate | | | |
| Increased Erosion Distance (ft/50 yrs) | 205 | 110 | 73 |
| Increased Annual Erosion Rate (ft/yr) | 4.1 | 2.2 | 1.5 |
| Increased Volume Loss (yds ³ /50 yrs) | 667,333 | 357,500 | 238,333 |
| Increased Annual Volume Loss (yds ³ /yr) | 13,347 | 7,150 | 4,767 |

Table 11-2. Bruun Rule results for project area (benefits of structures included)

| Project Totals (accounting for profile or L parameter cutoff) | | | |
|---|------------------------------------|-------------------------|---------------------------|
| Historical SLC Rate | Beach Fill Only or Without Project | Alternative 6 Structure | Alternative 25A Structure |
| Increased Volume Loss (yds ³ /50 yrs) | 91,000 | 81,250 | 67,600 |
| Increased Annual Volume Loss (yds ³ /yr) | 1,820 | 1,625 | 1,352 |
| Intermediate SLC Rate | | | |
| Increased Volume Loss (yds ³ /50 yrs) | 455,000 | 406,250 | 338,000 |
| Increased Annual Volume Loss (yds ³ /yr) | 9,100 | 8,125 | 6,760 |
| High SLC Rate | | | |
| Increased Volume Loss (yds ³ /50 yrs) | 667,333 | 595,833 | 495,733 |
| Increased Annual Volume Loss (yds ³ /yr) | 13,347 | 11,917 | 9,915 |

Table 11-3. Beach fill renourishment volumes and project total volumes (with SLC adjustments).
Updated Table 10-4.

| Historic Sea Level Rise Rate - 40 ft beach nourishment berm | | | | | | | |
|---|---|---|---|---------------------|----------------|--------------------------|------------------------------------|
| Alternative | Initial Fill (non sacrificial) yds ³ | Initial Fill (sacrificial) yds ³ | Total Initial Fill (non- sacrificial + sacrificial fill) yds ³ | Renourishment Fills | | Number of Renourishments | Total Sand Volume yds ³ |
| | | | | yds ³ | interval (yrs) | | |
| Beach Fill | 233,589 | 478,000 | 711,589 | 432,200 | 10.0 | 5.0 | 2,872,589 |
| 6 | 233,589 | 130,000 | 363,589 | 116,350 | 11.6 | 4.3 | 865,098 |
| 25A | 198,040 | 130,000 | 328,040 | 123,188 | 19.0 | 2.6 | 652,219 |
| Intermediate Sea Level Rise Rate - 40 ft beach nourishment berm | | | | | | | |
| Alternative | Initial Fill (non sacrificial) yds ³ | Initial Fill (sacrificial) yds ³ | Total Initial Fill (non- sacrificial + sacrificial fill) yds ³ | Renourishment Fills | | Number of Renourishments | Total Sand Volume yds ³ |
| | | | | yds ³ | interval (yrs) | | |
| Beach Fill | 233,589 | 478,000 | 711,589 | 505,000 | 10.0 | 5.0 | 3,236,589 |
| 6 | 233,589 | 130,000 | 363,589 | 191,750 | 11.6 | 4.3 | 1,190,098 |
| 25A | 198,040 | 130,000 | 328,040 | 225,940 | 19.0 | 2.6 | 922,619 |
| High Sea Level Rise Rate - 40 ft beach nourishment berm | | | | | | | |
| Alternative | Initial Fill (non sacrificial) yds ³ | Initial Fill (sacrificial) yds ³ | Total Initial Fill (non- sacrificial + sacrificial fill) yds ³ | Renourishment Fills | | Number of Renourishments | Total Sand Volume yds ³ |
| | | | | yds ³ | interval (yrs) | | |
| Beach Fill | 233,589 | 478,000 | 711,589 | 547,467 | 10.0 | 5.0 | 3,448,922 |
| 6 | 233,589 | 130,000 | 363,589 | 235,733 | 11.6 | 4.3 | 1,379,681 |
| 25A | 198,040 | 130,000 | 328,040 | 285,879 | 19.0 | 2.6 | 1,080,352 |

Table 11-4. Project life total beach fill requirement (without and with SLC comparison)

| Total Beach Fill Volume - 40 Berm (yds ³) | | | | | | | |
|---|-------------|--------------|----------|------------------|----------|-----------|----------|
| Alternative | Without SLR | Historic SLR | | Intermediate SLR | | High SLR | |
| | | Volume | Increase | Volume | Increase | volume | Increase |
| Beach Fill | 2,781,589 | 2,872,589 | 91,000 | 3,236,589 | 455,000 | 3,448,922 | 667,333 |
| 6 | 783,848 | 865,098 | 81,250 | 1,190,098 | 406,250 | 1,379,681 | 595,833 |
| 25A | 584,619 | 652,219 | 67,600 | 922,619 | 338,000 | 1,080,352 | 495,733 |

11.4.6 *Sea Level Change Impact – With Project Beach Fill Storm Protection*

As discussed several times in this appendix and the main report, the purpose of this project and the various alternatives is not to protect against a certain design storm, or reduce the risk of flooding, but instead to address the increased erosion levels caused by the Saco River Jetties. The beach fill designs and the cross shore performance of the beach in various storm conditions were tested with the SBEACH model, to ensure adequate performance. However, although not quantified, it cannot be denied that there will be storm damage reduction benefits in addition to the prevention of damages through shoreline erosion. It is believed that SLR will not impact those benefits that do occur for storm damage reduction and shoreline erosion as long as the beach fill is maintained. While the higher curves for projected SLR are significantly higher than historical SLC, the rates are still low enough to allow more than adequate time for the beach profile to adjust to the higher water levels. As SLR occurs the beach profile will adjust with the berm elevation increasing to match the new water level. Beaches adjust to more dramatic changes in water level through the monthly tidal cycles and short term storm events. The real issue will be the potential impacts to beach fill renourishments as previously discussed. Obviously if the beach fill is not maintained to keep pace with increased SLR (if it does occur) there would be negative implications to the project performance.

11.4.7 *Sea Level Change – Should it Be Addressed?*

The intent of this project is to mitigate against the impacts of the USACE jetties, which reduce the volume of sand entering into the littoral system and increase wave energy hitting the beach. With the current array of alternatives it has been concluded that we can address those issues and mitigate for the negative impact caused by the Federal jetties. With that said it can be argued that it is not the responsibility of USACE to address the impacts of SLR and meet the extra sand demands that would be needed under the increased SLR scenarios. SLR increase is not caused by USACE and the negative impacts of SLR will affect everyone on the coast. It could be argued that if the beach fill portions of the project are maintained to address SLR that this goes beyond the Section 111 authority since it would be providing residents within the project area a protection level that goes beyond what would have been there without any federal navigation projects (channel and jetties). However, the structural elements of the project would still have to be maintained since they are directly addressing the wave energy coming off of the north jetty structure. If increased SLR results in the need for higher crest elevations or stone size that would need to be addressed.

11.5 SLC Discussion Conclusions

The impacts of SLC were investigated for the various alternatives following the intent of the relatively new SLC EC. It was concluded that even under the higher estimates of SLC that the spur and breakwaters in Alternatives 6 and 25A do not need to be altered and that the original design should be adequate to handle the higher water levels and any increased in wave energy. If this conclusion is incorrect the increased damage and alterations to the structural design can be made during future repair efforts.

It was found using the Bruun Rule that the required beach renourishment volumes do increase substantially under the higher rates of SLC, with the impacts of SLC being lower for the structural alternatives. However, it must be determined if the increased fill requirements due to SLC are the responsibility of USACE under the Section 111 authority. Even in a pristine system with no federal project, the shoreline of Saco Bay would be dramatically impacted by increased SLC rates. If USACE is addressing the issues caused by the federal projects and addressing the historical rate of SLC than anything beyond that should not be considered a federal responsibility.

12.0 ALTERNATIVE RISK ANALYSIS

Based upon past experience, numerous discussions with subject matter experts within USACE, and literature (USACE and external), it was concluded that a beach fill only option is not feasible and does not address the negative impacts caused by the federal navigation project.

In many cases where there is a persistent and high erosion rate down drift of an inlet or terminal type structure, it is not feasible to address the problem with sand fill only. It is believed for this study that is the case. As described in this appendix and the WHG report (Appendix C), the project area is impacted by wave energy that is focused by bathymetry into the corner created by the beach and the North Jetty. Also, the reflected wave energy coming off of the North Jetty into the beach is at a very shallow angle and has been shown to be significant in the wave modeling results. This will cause strong potential for long shore transport to the north during storms and cause significant erosion that was not quantified in the SBEACH numerical model. A beach fill only option will not diminish this strong reflected wave and will likely erode quickly.

Based on available literature from several Corps sources, and external journals, there is significant support for using structures to help address these acute down drift impacts. Where these situations exist it is often difficult to keep up with excessive fill volume requirements caused by high erosion rates. This is especially true given the sand source for this project is likely upland sand. Texts consulted included.

Hanson, H., and Kraus, N.C. "Chronic Beach Erosion Adjacent to Inlets and Remediation by Composite (T-Head) Groins" ERDC/CHL CHETN IV-36, U.S. Army Engineer Research and Development Center, Vicksburg, MS

Bodge, K.R., "Design Aspects of Groins and Jetties" ASCE/COPRI Special Report on Advances in Coastal Structures Design. American Society of Civil Engineers, 2002

Bodge, K.R., "Beach Fill Stabilization with Tuned Structures: Experience in the Southeastern USA and Caribbean" Olsen Assoc. Inc.

Hanson, H., and Kraus, N.C. "T-Head Groin Advancements in One-Line Modeling (GENESIS/T)"

Also, during the late 1990s dredged sand from the federal navigation project within the Saco River was placed directly on the beach just north of the north jetty. Approximately 80,000 yds³ were placed in that effort. During the placement, it was witnessed that the sand was eroding nearly as fast as it was placed and no appreciable beach was ever created.

During this study a request was sent out by email to the CWG regarding analysis techniques that could be used to help quantify the performance of beach fill only options in areas such as this project. Within several responses it became evident that it is difficult to quantify the performance of beach fill in such

locations, and that beach fill requirements are typically increased to address these “hotspot” erosion issues. One specific project from California had proposed structures to address this anticipated high erosion down drift of an inlet but in the end a beach fill only project was constructed. The project is now required to re-nourish every 5 to 6 years with 1.75 million cubic yards of sand. It must be kept in mind these are two different projects and a direct comparison of the processes at the CA project were not made to Saco, but it further supports that the erosion rates for a beach fill only project will likely be high and perhaps not sustainable, especially with the likely source being upland sand.

12.1 Beach Fill Risk Based Alternative Analysis

To help understand the results of the various modeling efforts and the qualitative analysis, two risk matrices were developed to qualitatively describe the risk associated with the beach fill modeling and performance for each alternative. These risks were determined based on the technical expertise and opinion and were not developed using quantitative risk models or methods.

12.2 SBEACH Risk Discussion

As mentioned, a color coded matrix summarizing risk and analysis issues was created for the SBEACH modeling effort. Table 12-1 is color coded with red being a non addressed issue that will make the results less conservative (more likely for the project to fail), with yellow meaning a non addressed issue but neutral in its impacts (not more or less likely to fail), and green meaning a non addressed issue but conservative (more likely if those processes were included the project would be more likely to succeed). In essence this matrix should provide a confidence level for the SBEACH modeling results related to each alternative.

The verbal description of the color scheme can be found below the risk matrix. In this case higher risk can be equated to a greater chance of error or lower confidence in the modeling effort that would lead to increased risk of project failure. As an example, for the beach fill only alternative, SBEACH does not address the significant angle of the reflected waves coming from the North Jetty, and it is very likely that this is an important process in the beach erosion during a storm. This deficiency likely makes the SBEACH model over predict the beach performance during a storm. This over prediction or overly optimistic storm performance, likely means the constructed project will likely perform below what has been reported herein and therefore will be more likely to fail. This means for this alternative the reflected wave risk is red.

Table 12-1. SBEACH Modeling Confidence Matrix (Risk)

| Alternative | Beach Process/Model Impact | | | | |
|---|---|---|--|--|--|
| | Mach Stem Current | Reflected Wave | Hydraulic Processes | Cross Shore Sand Movement | Structure/Beach Shape |
| Beach Fill | Model does not account for this forcing | Model does not account for the reflected wave direction | Model does not account for hydraulic impacts from north jetty and corner shape | | |
| 6 | Spur knocks down Current | Spur knockd down waves | Spur reduces near structure current | | |
| 25a | Spur knocks down Current | Spur and breakwaters knock down waves | Spur and breakwaters reduce near shore current | Structures will reduce cross shore losses during storm and beach equilibration | Sailients and beach shape will reduce the long shore current |
| Red - Model likely over predicting beach performance (increased risk of beach failure) | | | | | |
| Yellow - Model is likely to be predicting performance of beach (no impact on risk factor for beach failure) | | | | | |
| Green - Model is likely under predicting beach performance (reduced risk of beach failure) | | | | | |

Overall, it can be seen for each of the factors considered the Beach Fill Only Alternative runs a higher risk of project failure than both Alternative 6 and 25A. Alternative 25A is more likely to perform better than the modeling analysis since several benefits from this alternative were not able to be incorporated into a quantitative analysis.

As a brief summary, the SBEACH modeling should be considered as over predicting the storm performance of the beach fill only option since there is nothing to address the severe angle of the reflected wave from the North Jetty impacting the beach, the mach stem wave/current along the North Jetty that impacts the beach, and the complicated hydraulic flow regime that is likely to exist in the project area due to the corner created by the beach and North Jetty. For the spur and beach fill alternative, Alternative 6, SBEACH modeling results are most likely the most representative of the actual results. This is because the sizable spur feature was shown to significantly reduce the reflected waves from the North Jetty, eliminate the Mach stem effect, and while not modeled, likely reduces the complex hydraulic flow in the corner of the project area. For these reasons, the remaining process of shore normal waves impacting a beach, with limited outside impacts, most closely resembles the conditions SBEACH was developed to model. The SBEACH modeling for the spur and breakwater option, Alternative 25A, are likely under predicting the beach fill performance due to the extra protection provided by the beach shape behind each breakwater structure (shown in Figure 9-5), the reduced long shore currents resulting from the breaking wave direction behind the structures and from the modified beach shape.

12.3 Alternative Risk Discussion

The second risk based matrix that was developed was for the overall beach fill alternative analysis effort and performance. This discussion is not solely related to beach modeling but it is heavily influenced by it. This risk matrix takes into account the SBEACH modeling risk (from Table 12-1), the modeling output from WHG, beach process analysis, and other project factors. For the alternative risk matrix, the colors of red, yellow, and green were used. The verbal description of the color scheme can be found below the risk matrix in Table 12-2. In this case higher risk can be equated to a greater chance of error, lower confidence, and lesser performance leading to a higher risk project failure. For example, for the beach

fill only alternative, performance in high erosion down drift areas, such as this project, have been shown to underperform. Also, beach fills that have been placed in this area have essentially disappeared with no appreciable longevity. Therefore, the risk is higher that the Beach Fill Only Alternative will fail when considering these parameters. Rating each alternative for each parameter demonstrates that the beach fill only option has the greatest risk of failure and alternative 25A has the greatest chance of success.

Table 12-2. Alternative Analysis Risk

| Alternative | Risk Factor | | | | | |
|--|--|--|---|---|--|---|
| | Historical Performance For Alternative | Analysis Uncertainty | Performance in Larger Storm | Ability to Maintain | Performance in Unmaintained State | Equilibrium Beach Profile Uncertainty (Depth of Closure) |
| beach fill | Beach fills placed on the project beach have not had any longevity. In the late 1990s, approximately 80,000 yds ³ of sand was placed just north of the jetty and no berm was ever constructed. The sand was leaving as fast as it was placed. | As discussed in several areas, the analysis related to the beach fill option has the most uncertainty associated with the numerous modeling components and formulations. | During a lower return period event than the 10 year storm, the beach fill only option is much more prone to failure | The beach fill source is unknown. If an upland source is used it may be difficult to truck significant volumes of fill. | If the beach is not maintained there is no protection. | Noted in Section 2.5 there is a definite possibility the beach fill will equilibrate to a point further offshore due to the depth of closure. |
| 6 | Spur structures have been used with jetty structures on numerous projects. When designed correctly they have been shown as an effective element for reducing wave reflection, Mach Stem effects, redirecting near shore currents, and reducing erosion adjacent to jettied inlets. | The wave modeling demonstrated that the spur removed the wave driven impacts of the north jetty. Therefore this alternative is likely to be the most accurately modeled alternative regarding beach performance. | The performance will be improved by the 750 feet of structure protecting the beach. Larger storms are anticipated to cause less erosion due to the protection offered by the spur groin. | As shown in Section 10 the beach in this alternative will require significantly less beach renourishment than the beach fill only alternative. This will make this alternative easier to maintain and will increase the chances of the necessary beach being in place during a storm event. | If the beach is not maintained the spur structure will still provide some protection to the project area and will still significantly reduce the wave energy hitting the beach since the reflected waves from the north jetty will be blocked. The Mach Stem effect waves/currents will also be blocked. | The 750 ft spur will help to reduce the cross shore profile within its extent but 2,500 feet of beach fill will not be prevented from equilibrating fully. This will help to retain sand closer to the beach within the spurs extent and improve storm performance. |
| 25A | Structures have not been used at this location, but near shore breakwaters and t-head groins have been shown at other locations to be very effective at reducing and reversing beach erosion in high erosion areas such as down drift of inlets. | As discussed there are definite unknowns and uncertainty within the analysis for the beach in alternative 25A. However, these unknowns are beneficial impacts that are not being accounted for. This means the analysis is likely conservative, or would likely perform better once constructed. | Given the nature of the structures they will continue to break wave energy in larger, lower event storms. The beach may erode more than during a 10 year storm but due to the protection offered by the structures will perform better than the beach fill only option. | Because the beach will have greater longevity and will require fewer renourishments it will be easier to maintain and will therefore will more likely be in place and providing protection during storm events. | If the beach is not maintained in this alternative the breakwaters will still provide significant protection during storm events. | The breakwaters and spur will help to cut off the profile and will retain sand closer to shore (shown in Figure 4). This should contribute to a wider beach than predicted in the analysis behind the spur. |
| Red - Beach fill component has highest risk of failure and negative impacts. | | | | | | |
| Yellow - The beach fill is expected to most closely perform as the analysis has shown. Risk of failure or better performance are considered equal. | | | | | | |
| Green - Risk of beach fill failure or resulting negative impacts are reduced (likely a negative risk). | | | | | | |

13.0 CONCLUSIONS AND RECOMMENDATIONS

The Saco-Camp Ellis Beach area, located adjacent to the Saco River federal jetties, represents a complex coastal setting that has not been well understood. Camp Ellis Beach has undergone significant shoreline changes over the past 150 years, including significant erosion over the past several decades. The purpose of this study was to evaluate potential alternatives that may be viable solutions to the ongoing erosion at Camp Ellis Beach. The study focused on evaluating the physical processes (concentrating on the wave environment) occurring within the vicinity of Saco Bay, and specifically the Camp Ellis Beach area, to assess potential alternatives that may be used to mitigate the erosion along the shoreline.

There were two main components of the study, a field data collection component, and a numerical modeling component. The field data collection component consists of observing the existing site-specific conditions (e.g., waves, currents, tides, bathymetry, etc.) and the historic environment (e.g., shoreline change, offshore wave data, existing studies, etc.) to develop an initial understanding of the ongoing coastal processes that shape the Camp Ellis shoreline. The field data also serve to provide the required data for developing predictive models of the Camp Ellis region. The numerical modeling component of the study consists of accurately simulating the existing conditions within the vicinity of Camp Ellis, verifying the models performance with observed data, and subsequently, utilizing the verified models to simulate various alternatives for shoreline protection. The numerical modeling portion of the study ultimately evaluates the performance of each of the alternatives and the ability to sustain a beach at Camp Ellis, while focusing on the ability of the alternatives to rectify the potential causes of erosion at Camp Ellis caused by the federally maintained structures. Specifically, the paucity of sediment supplied to the region by the direction of the sediment laden Saco River discharge a significant distance offshore and the structural impact producing increased wave energy exerted on the Camp Ellis coastline.

13.1 Historical shoreline change

The shoreline adjacent to the northern jetty (Camp Ellis Beach) experienced significant erosion. The shoreline between 1864 and 1998 eroded at rates between -3.41 ft/yr (at transect 41) and -0.16 ft/yr (at transect 53). The more contemporary time period (1944-1998) shows continued erosion, but at a reduced rate (approximately 1.0 ft/yr and less). This stabilization is likely the result man-made intervention in the form of heavy structural stabilization (seawalls, revetments, etc.). With the lack of sediment supply that currently exists at the Camp Ellis shoreline, erosion has begun to occur to the shorelines north of the historical erosion area as less sediment is available for northerly transport into these once stable regions.

13.2 Bathymetry

The nearshore bathymetry is complex, with rock outcrops, linear shoals, and irregular contours. The data collected was used extensively in the modeling of the Camp Ellis region.

13.3 Tides and Currents

Tide and current information was collected in the vicinity of Camp Ellis Beach. During a spring tide, the tidal range is approximately 11.5 to 12.0 feet (ft). However, during a neap tide the tidal range is reduced to approximately 7.0 ft. Only minor tidal damping occurs as water propagates upstream. The magnitude of the tidal currents in the nearshore vicinity north of the jetties is minimal. This lack of significant tide-induced currents offshore of Camp Ellis Beach suggests that wave processes are the primary driver of sediment movement in the nearshore vicinity of Camp Ellis Beach.

13.4 Wave Climate

Wave data were gathered from existing buoys, existing hindcast model nodes, and data observations associated with this study. Offshore data sources included, NOAA buoys, GOMOOS Buoys, and USACE WIS hindcast data. Wave data were also collected at target locations seaward and landward of the Eagle/Ram Island complex in order to accurately assess the wave transformations that occur due to the complicated physical nature of the islands and bathymetry in this region. The wave ADCP data were compared to existing NOAA buoy data to provide a level of confidence in the observed data. The wave data were a necessity for calibrating and verifying the numerical wave transformation models.

13.5 Sediments

A total of seven (7) surface grab samples were collected within the Saco Bay region. A general fining of sediment is observed from south to north. As such, the median grain size is finer at the northern end of the Bay (Scarborough) than it is at the southern end (Camp Ellis). Armed with the data observations and historical information, the modeling effort was calibrated in order to ensure that it was appropriately simulating reality within reasonable error bounds. The models were then used to simulate existing conditions, and finally, to assess a wide range of alternatives.

13.6 Generation Scale Wave Modeling

Because of the lack of temporal and spatial similitude between locally observed wave information and available data sources, a generation scale wave model was used to develop input into the detailed, shallow-water transformation-scale wave model. The generation scale numerical model used satellite observed wind fields as input and was calibrated and verified using local point wave observations. A nested grid of water depths was used to define the model domain. The calibration between the measured and modeled wave heights was visually successful and quantifiable error statistics were within acceptable bounds. In addition, assessment of specific higher energy events during the deployment time period indicated energy transfer across frequency bands during the passage of an event. Subsequently, spectral wave data were extracted from the generation scale model at the location coinciding with the offshore boundary of the regional wave model for the entire deployment time period. This spectral information were used as input conditions for validation of the regional scale modeling effort.

13.7 Transformation-Scale Wave Modeling

A nearshore, transformation-scale (regional) wave model was used to propagate the offshore wave climate into the Saco Bay region and evaluates the transformations waves experience as they propagate towards the coastline. The model was verified using spectral output from the generation scale modeling results, and the error estimates were within acceptable bounds. Once validated, the transformation-scale (regional) model was used to simulate average annual directional cases (developed from WIS data), specific historic storms events, and return-period storms. Regional wave transformation models provide predictive tools for evaluating various forces governing wave climate and sediment transport processes. The transformation-scale (regional) model identified transformation effects that produced an uneven distribution of wave energy along the coast that affects sediment transport in the region, revealing areas of increased erosion (“hot spots”) or areas of increased energy. The transformation mechanisms also result in changes in the offshore wave direction that may significantly influence the rate and direction of sand movement. Therefore, the quantitative information provided from the numerical model(s) can be used to explain the physical processes that dominate a region, provide the required input into higher resolution models, and to potentially furnish appropriate recommendations/solutions for each stretch of coast. Results of the transformation-scale (regional) model are used to develop regional sediment transport fluxes and divergence, while providing spectral input for the local wave modeling effort.

13.8 Local Scale Wave Modeling

The regional model only represents an intermediate step in the wave modeling system and although is useful for identifying regional sediment transport trends, cannot be used for local sediment transport calculations for the Camp Ellis Beach area. Therefore, it was important to advance to higher-resolution models that embodied the reflection processes and could more accurately determine the nearshore structural interactions. The nearshore (local) wave model was calibrated using two simulated time periods to quantify the overall accuracy. Evaluation of the sea surface results for the existing conditions revealed: (1) the significant wave reflection off of the northern jetty indicating the beach is impacted not only by the incident wave energy, but also by the reflected wave energy, (2) independent of offshore direction of approach, the nearshore waves propagated directly towards the Camp Ellis Beach area and the northern jetty, (3) Mach-Stem waves propagating shoreward along the northern jetty can be seen in most cases, (4) waves are refracted towards the northern jetty due to the jetty-parallel bottom contours, and (5) variations between annual average approach directions are important to understand the processes occurring at Camp Ellis Beach.

13.9 Sediment Transport Assessment

The sediment flux indicates an average annual longshore transport rate to the north. However, the magnitude of the transport varies throughout the domain. A region extending from just north of the navigational structures to approximately 3 km to the north, averages approximately 25,000 to 50,000 m³/yr (32,700 to 65,000 cubic yards) towards the north. In the center of the bay, extending approximately 3 km to the north, the average annual sediment flux rate is small. This is a region that gross transport direction shifts depending on the angle of the incoming wave field, and generally are

equivalent. There is a small net northward transport rate of 10,000 to 20,000 m³/yr (13,000 to 27,000 cubic yards) and the flux divergence indicates a stable stretch of coastline. The northernmost region of the bay is strongly influenced by the Bluff and Stratton Island complex and there are major fluctuations in both the sediment flux and divergence. The average sediment transport rate in this region is 40,000 m³/yr (52,000 cubic yards) towards the north.

13.10 Initial Alternative Screening

A variety of alternatives were considered for addressing the chronic erosion at Camp Ellis Beach. Over twenty-five (25) potential solutions, including both structural and non-structural alternatives were determined jointly between Woods Hole Group, the USACE New England District, Maine Geological Survey (MGS), and members of the Saco Bay Implementation Team (SBIT). The nearshore (local) wave model was used as the initial screening tool through evaluation of results, wave height changes, wave energy reduction, and assessment of potential impacts. Potential adverse impacts to neighboring beaches, navigation, and the Camp Ellis region were also evaluated. The initial screening process identified six (6) alternatives that warranted further evaluation in terms of sediment transport changes and beach performance. These alternatives include beach nourishment to restore the beach to a useable (and more importantly protective) width.

13.11 Final Alternative Screening

The data, modeling, and analysis indicate that the federal structures at Saco River have created increased wave energy to the north of the Saco River inlet and significantly reduced the amount of sediment supplied to the beach by the Saco River by channeling the discharge a greater distance offshore. Therefore, the preferred alternative should (1) mitigate the increased wave energy caused by wave reflection off of the northern jetty and (2) replenish, to a feasible level, the sediment that would naturally be supplied to the system by the Saco River discharge.

- Beach nourishment alone is not a viable solution without the inclusion of a significant maintenance component. Lacking any additional wave energy reduction, a standalone beach would be quickly eroded. The beach nourishment alone alternative also does not mitigate any of the reflected wave energy that impacts Camp Ellis Beach. Therefore, it would not be possible for beach nourishment alone to meet the project purpose of mitigating the erosive effect of the jetty although it could prevent further retreat of the Camp Ellis shoreline with placement of large amounts of renourishment fill.
- Each of the final alternatives, coupled with beach nourishment, performs with relative levels of improvement. Therefore, each alternative can be effective depending on the level of maintenance expected and success criteria.
- The geology at the offshore breakwater location (Alternatives 11a and 18) is not suitable to support the weight of the structure. There is considerable concern related to settlement at this location, as such these final alternatives are not recommended.
- Alternative 6 provides a reduction in the reflected wave energy caused by the northern jetty, and coupled with beach nourishment provides a reasonable solution for consideration.

However, increased maintenance, as compared to the segmented breakwater and spur jetty combinations would be expected.

- From a purely performance perspective, Alternatives 25 and 26 are unmatched. There are subtle differences between the two alternatives in performance, but overall there is no measurable distinction. However, as subsurface explorations determined that soft marine clay underlies the northernmost breakwater for both alternatives, constructing a stable structure at this location would be costly.
- Alternatives 25 and 26 do show some potential for salient formation in terms of amplitude at the northernmost breakwaters, but this would not significantly impact the shorelines to the north due to the increased sediment supply provided by the beach nourishment. Alternative 25A provides an alternative segmented breakwater solution that removes the northernmost breakwater and still provides reasonable performance.
- It is critical that the final alternative selected includes a beach nourishment component. Shoreline erosion is continuing to occur north of the structure and erosion is extending a further distance north of the structure due to the lack of sediment supply available for transport. However, a significant nourishment will replenish the sediment supply to the coastline and this additional sediment can only benefit the regions to the north. Even at a reduced sediment transport rate, the net sand moving to the northern beaches will be far greater than currently exists.

13.12 Structure Design

Once the final array of alternatives was selected a more detailed design was conducted for each alternative. For the alternatives that included structures, rubble mound breakwaters and spurs were designed. This included the determination of the armor stone weight, structure slopes, crest elevation, toe protection, existing structure reinforcement, and cross sectional design.

13.13 Beach Fill Design

As discussed, a key component to address the shoreline erosion issues caused by the federal navigation project was beach fill. This component will help to protect the shoreline from further erosion, restore a natural buffer that has been lost, and restore a supply of sediment to the system that has been removed by the federal navigation project.

The beach design analysis provided the minimum fill requirements needed to be in place to withstand a 10 year storm and the longevity of various beach fill alternatives in conjunction with structural alternatives. As expected the minimum fill needed to be in place for the beach fill only option was greater than for the spur and breakwater alternatives with beach fill.

The lesser performance of the beach fill only alternative during a storm, and the shorter beach fill life, were expected. In several literature sources, and in personal communications, it was concluded that the high erosion rate down drift of the Saco River Inlet Jetties would likely require structural components, and that beach fill alone would not be feasible due to the high nourishment rates.

The effort to determine beach fill performance and to provide a more detailed design was hampered by the lack of available models that could handle the complex wave climate in the project area. Simplification and estimation were applied to the available models but the accuracy of the beach performance modeling results must be understood so the results of the analysis can be used in the proper context.

13.14 Sea Level Change Impacts

The impacts of sea level change (SLC) were investigated late in the study. Instead of running the full analysis over for each of the SLC curves required in EC (), each component of the project was evaluated separately regarding SLC. For each component, it was concluded that SLC would not have a significant impact, with the exception of beach fill renourishment. IF the higher rates of SLC occur as predicted the renourishment volumes or schedule would have to be increased if erosion of the Camp Ellis Beach shoreline is to be prevented. However, it must be decided if it is a federal responsibility to protect against SLC since the federal government is not responsible for increased SLC and the intent of the project is to mitigate for negative impacts caused by the federal navigation project.

13.5 Risk Analysis

To help better understand the risks associated with the beach fill alternative and the structural alternatives, two color coded risk matrices were developed and both demonstrated that the beach fill alternative has the highest risk of failure, with 25A (spur and break water structures) having the lowest risk. Based on the findings of this appendix it is recommended that the beach fill only alternative not be considered as a feasible alternative. The uncertainty in this alternative's performance is significant, resulting in higher risk of project failure.

14.0 REFERENCES

- Ahrens, J.P. and Cox, J. 1990. Design and Performance of Reef Breakwaters. *Journal of Coastal Research*. p. 61-75.
- Anders, F.J. and M.R. Byrnes. 1991. Accuracy of Shoreline Change Rates as Determined from Maps and Aerial Photographs. *Shore and Beach*, 59(1): p. 17-26.
- Barber, D.C. 1995. Holocene Depositional History and Modern Sand Budget of Inner Saco Bay, Maine: Unpub. M.S. Thesis, University, Maine, Orono, 178 pp.
- Barnhardt, W.A., W.R. Gehrels, D.F. Belknap, and J.T. Kelley. 1995. Late Quaternary Relative Sea-Level change in the Western Gulf of Maine; Evidence for a Migrating Glacial Forebulge. *Geology*, v 23, no. 4, p. 317-320.
- Belknap, D.F., R.C. Shipp, J.T. Kelley, and D. Schnitker. 1989. Depositional Sequence Modeling of the Late Quaternary Geologic History, West Central Maine Coast, In Tucker, R.D., and Marvinney, R.G., eds., *Studies in Maine Geology, Volume 5: Quaternary Geology*: Augusta, Maine Geol. Survey, Dept. of Conservation, p. 29-46.
- Belknap, D.F. and R.C. Shipp. 1991. Seismic Stratigraphy of Glacial Marine Units, Maine Inner Shelf, in Anderson, J.B. and Ashley, G.M. (editors), *Glacial Marine Sedimentation; Paleoclimatic Significance*: Geological Society of America, Special Paper 261, p. 137-157.
- Bloom, A.L. 1963. Late-Pleistocene Fluctuations of Sea Level and Postglacial and Crustal Rebound in Coastal Maine: *American Journal of Science*, v. 261, p. 862-879.
- Briggs, M.J., B.P. Donnell, Z. Demirbilek, and R.D. Carver. 2003. Tediuous Creek Small Craft Harbor: CGWAVE Model Comparisons Between Existing and Authorized Breakwater Configurations. U.S. Army Corps of Engineers ERDC/CHL CHETN-I-67.
- Brooks, R.M. and W.D. Corson. 1984. Summary of Archived Study Pressure, Wind, Wave, and Water Level Data. U.S. Army Engineer Waterways Experiment Station, Wave Information Study, Vicksburg, MS, WIS Report 13.
- Brothers, L., in press, Nearshore sedimentary pathways and their social implications, Saco Bay, Maine. MS degree, University of Maine.
- Byrnes, M.R. and M.W. Hiland. 1994. Compilation and Analysis of Shoreline and Bathymetry Data (Appendix B). In: N.C. Kraus, L.T. Gorman, and J. Pope (editors), *Kings Bay Coastal and Estuarine Monitoring and Evaluation Program: Coastal Studies*. Technical Report CERC-94-09, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS, p. B1-B89.
- Byrnes, M.R., R.M. Hammer, B.A. Vittor, J.S. Ramsey, D.B. Snyder, K.F. Bosma, J.D. Wood, T.D. Thibaut, and N.W. Phillips. 1999. Environmental Study of Identified Sand Resource Areas Offshore Alabama: Volume I: Main Text, Volume II: Appendices. U.S. Department of the Interior, Minerals Management Service, International Activities and Marine Minerals Division (INTERMAR), Herndon, VA. OCS Report MMS 99-0052, 326 pp. + 132 pp. appendices.
- Byrnes, M.R., R.M. Hammer, B.A. Vittor, J.S. Ramsey, D.B. Snyder, J.D. Wood, K.F. Bosma, T.D. Thibaut, N.W. Phillips. 2000. Environmental Survey of Potential Sand Resource Sites: Offshore New Jersey. U.S. Department of the Interior, Minerals Management Service, International Activities and Marine Minerals Division (INTERMAR), Herndon, VA. OCS Report MMS 2000-052, Volume I: Main Test 380 pp. + Volume II: appendices 291 pp.

- Brooks, R.M. and W.D. Corson. 1984. Summary of Archived Study Pressure, wind, Wave, and Water Level Data. U.S. Army Engineer Waterways Experiment Station, Wave Information Study, Vicksburg, MS, WIS Report 13.
- Coastal and Marine Geology Program of the United States Geological Survey
<http://woodshole.er.usgs.gov/project-pages/oracle/gomaine/bathy/>
- Collins, J. 1972. Prediction of Shallow-Water Spectra. *Journal of Geophysical Research*, Vol. 77, p. 2693-2707.
- Crowell, Mark, Stephen P., Leatherman, and Michael K. Buckley. 1991. Historical Shoreline Change: Error Analysis and mapping Accuracy. *Journal of Coastal Research*, 7(3): p. 839-852.
- Dalrymple, R. A., J.T. Kirby, and P.A. Hwang. 1984. Wave Diffraction Due to Areas of High Energy Dissipation. *J. Waterway, Port, Coastal and Ocean Eng.*,110, p. 67-79.
- Dean, Robert G. 2002. Beach Nourishment theory and Practice. *Advanced Series on Ocean Engineering – Volume 18*. World Scientific.
- Demirbilek, Z. and V.Panchang. 1998. CGWAVE: A Coastal Surface Water Wave Model of the Mild Slope Equation. U.S. Army Corps of Engineers technical Report CHL-98-xx.
- Dickson, S.K., J.T. Kelley, D.F. Belknap, L.K. Fink, D.C. Barber, D.M. FitzGerald, S. van Heteren, and P.A. Manthorp. 1993a. A Sediment Budget for Saco Bay, Maine, and an Evaluation of the Long- and Short-Term Geologic and Oceanographic Processes: In List, J.H., ed., *Large-Scale Coastal Behavior '93*: U.S. Geol. Survey Open-file Rept. 93-381, p. 48-51.
- Ellis M.Y. 1978. *Coastal Mapping Handbook*. U.S. Department of the Interior, Geological Survey, U.S. Department of Commerce, National Ocean Service, U.S. Government Printing Office, Washington, D.C., 199 pp.
- EPA (U.S. Environmental Protection Agency). 2000. *Global Warming – climate, Sea Level*.
<http://yosemite.epa.gov/OAR/globalwarming.nsf/content/ClimateTrendsSeaLevel.html>
- Farrell, S.C. 1972. *Present Coastal Processes, Recorded Changes, and the Post-Pleistocene Geologic Record of Saco Bay, Maine*: Unpub. Ph.D. Dissertation, Univ. Massachusetts, Amherst, 296 pp.
- Feddersen, F., R. T. Guza, S. Elgar, and T. H. C. Herbers. 1998. Alongshore Momentum Balances in the Nearshore. *J. Geophys. Res.*, v. 103, p. 15,667-15,676.
- FitzGerald, D.M., P.A. Manthorp, C. Kuo, and S. van Heteren. 1993. Estuarine Circulation and Sediment Transport Trends in the Lower Saco River. *Abstr. Programs, Northeast Sect. Meet., Geol. Soc. Am.*, 25.
- FitzGerald, D.M., S. van Heteren and R.M. Montello. 1994. Shoreline Processes and Damage Resulting from the Halloween Eve Storm of 1991 Along the North and South Shores of Massachusetts Bay. *U.S.A. Journal of Coastal Research*, 10, p. 113-132.
- FitzGerald, D.M., I.V. Buynevich, R.A. Davis Jr., and M.S. Fenser. 2002. New England Tidal Inlets with Special Reference to Riverine-Associated Inlet Systems. *Geomorphology* 48, p. 179-208.
- Forristall, G.Z. 1981. Measurements of a Saturated Range in Ocean Wave Spectra. *Journal of Geophysical Research*, v. 86, p. 8075-8094.
- Haas, K. A. and Hanes, D. M. 2004. Process Based Modeling of Total Longshore Sediment Transport. *J. Coastal Res.* v. 20, p. 853-861.

- Hakeem Johnson, Jort Wilkens, Andy Parsons, Tim Chesher. 2010. Guidance for Outline Design of Near Shore Detached Breakwaters on Sandy Macro Tidal Coasts. Department for Environment Food and Rural Affairs (DEFRA) Project SC060026/RI 2010
- Hayes, M.O. 1975. Morphology of Sand Accumulations in Estuaries. In: Cronin, L.E. (Eds.), *Estuarine Research*, Vol. 2. Academic Press, New York, p. 3-22.
- Heinze, Heather W. 2001. Anthropogenic Influences and Meteorological Effects: How They are changing the Sand Beaches in Southern Maine. Unpublished MA Thesis, The Graduate School, the University of Maine. 167 pp.
- Hsu, J.R.C. and R. Silvester. 1990. Accretion Behind Single Offshore Breakwater. *Journal Waterway, Port, Coastal, and Ocean Eng.*, ASCE, Vol. 116, No. 3, p. 362-380.
- Hubertz, J.M., R.M. Brooks, W.A. Brandon, and B.A. Tracy. 1993. Hindcast Wave Information for the U.S. Atlantic Coast. WIS Report 30, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Wave Information Study, Vicksburg, MS.
- IPPC. 2003. http://www.ipcc.ch/publications_and_data/publications_ipcc_third_assessment_report_synthesis_report.htm.
- Irish, J.L. and W.J. Lillycrop. 1999. Scanning Laser Mapping of the Coastal Zone: The SHOALS System: *ISPRS Journal of Photogrammetry and Remote Sensing*, v. 54, p. 123-129.
- Jensen, X.E. 1983. Atlantic Coast Hindcasting Shallow Water Significant Wave Information. WIS Report, 8, Vicksburg, MS, 75 pp.
- Jet Propulsion Laboratory. 2002. SeaWinds Science Data Product: User's Manual. Ed. Ted Lungu, California Institute of Technology.
- Jonsson, I.G. 1990. Wave-Current Interactions. *The Sea*. Vol. 9, Part A, B. LeMehaute and D.M. Hanes, ed., John Wiley & Sons, Inc., New York.
- Kahma, K.K. 1981. A Study of the Growth of the Wave Spectrum with Fetch. *Journal of Physical Oceanography*, v. 11, p. 1503-1515.
- Kelley, J.T. and W.A. Anderson. 2000. The Maine Shore and the Army Corps; A Tale of Two Harbors, Camp Ellis and Wells, Maine. *Maine Policy Review*, 9,p. 20-35.
- Kelley, J.T., A.R. Kelley. D.F. Belknap, and R.C. Shipp. 1986. Variability in the Evolution of Two Adjacent Bedrock-Framed Estuaries in Maine, In Wolfe, D.A. (Editor), *Estuarine Variability*: Academic Press, Orlando, p. 21-42.
- Kelley, J.T., A.R. Kelley and O.H. Pilkey. 1989. *Living with the Coast of Maine*. Durham, NC: Duke University Press, 174 pp.
- Kelley, J.T., S.M. Dickson, D.F. Belknap, and R. Stuckenrath. 1992. Sea-Level Change and Late Quaternary Sediment Accumulation on the Southern Maine Inner Continental Shelf, in Fletcher, C.H., III, and Wehmler, J.F. (Editors), *Quaternary Coastal Systems of the United States*: Society of Economic Paleontologists and Mineralogists, Special Publication 48, p. 23-34.
- Kelley, J.T., D.M. FitzGerald, P.A. Manthorp, and D.C. Barber. 1994. Seasonal Variation in the Direction of Estuarine Sediment and Water Movement, Saco Bay, ME. *Geol. Soc. Am, Abstr. Programs*, 1994 Annual Meeting 26, A68.

- Kelley, J.T., D.F. Belknap, D.M. FitzGerald, D.C. Barber, S.M. Dickson, S. van Heteren, P.A. Manthorp, and L.K. Fink. 1995. A Sand Budget for Saco Bay, Maine. Maine Geological survey Open-file Report 95-1, Maine Geological Survey, Augusta, ME., 59 pp.
- Kitaigorodskii, S.A. 1983. On the Theory of the Equilibrium Range in the Spectrum of Wind-Generated Gravity Waves. *Journal of Physical Oceanography*, v. 13, p. 816-827.
- Kraus, S.D., A.R. Knowlton, and J.H. Prescott. 1988. Surveys for Wintering Right Whales (*Eubalaena glacialis*) Along the Southeastern United States, 1984-1988. Final Report to the Department of the Interior, Minerals Management Service, Branch of Environmental Studies, Washington, DC, 19 pp. + app.
- Lynch, D. R. 2004. Numerical Partial Differential Equations for Environmental Scientists and Engineers. Springer.
- Maine State Planning Office. 1979. A Study of Beach Processes and Management Alternatives for Saco Bay: Maine State Planning Office Report, Augusta, Maine, 82 pp.
- Maine State Planning Office. 1998. Improving Maine's Beaches: Recommendations of the Southern Maine Beach Stakeholder Group, Maine State Planning Office Report, Augusta, Maine.
- Manthorp, P.A., D.M. FitzGerald, P.A. McKinlay, S. van Heteren, and J.T. Kelley. 1994. The Effects of Spring Freshets in the Lower Saco River, Maine: Geological Society of America, Abstracts with Programs, v. 26, no 3, p. 58.
- Manthorp, P.A. 1995. Estuarine Circulation and Sediment Transport in the Saco River Estuary, ME. Unpublished MA Thesis, Department of Geology, Boston University, Boston, MA. 230 pp.
- Mei, C. C. 1983. The Applied Dynamics of Ocean Surface Waves. Wylie.
- Merchant, D.C. 1987. Spatial Accuracy Specification for Large-Scale Topographic Maps. *Photogrammetry Eng. Remote Sensing*, 53: p. 958-961.
- Noble, R.M. 1978. Coastal Structures' Effects on Shorelines, Proc 17th Intl. Conf. Coastal Eng. ASCE, Sydney, p. 2069-2085.
- Osberg, P.H., A.M. Hussey II, and G.M. Boone. 1985. Bedrock Geologic Map of Maine: Augusta, Maine Geol. Survey, Scale 1:500,000.
- Pelnard-Considère, R. 1956. Essai de Théorie de l'Évolution des Formes de Rivage en Plages de Sable et de Galets, 4th Journées de l'Hydraulique Les Énergies de la Mer, Question III, Rapport No. 1.
- Resio, D.T. 1981. The Estimation of Wind Wave Generation in a Discrete Model. *Journal of Physical Oceanography*, v. 11, p. 510-525.
- Resio, D. 1987. Shallow-Water Waves-Part I: Theory. *Journal of Waterway, Port, Coastal and Ocean Engineering*, Vol. 113, p. 266-283.
- Resio, D. 1988. Shallow-Water Waves-Part II: Data Comparison. *Journal of Waterways, Port, Coastal and Ocean Engineering*, Vol. 114, p. 50-65.
- Resio, D.T. 1989. EXTRM2 Extremes Program User's Guide. Offshore & Coastal Technologies, Inc. (OCTI).
- Resio, D.T. 1990. Program WAVAD: Global/Regional Wave Model for Wave Prediction in Deep and/or Shallow Water, Offshore & Coastal Technologies, Inc.

- Resio, D.T. and B.A. Tracy. 1983. A Numerical Model for Wind-Wave Prediction in Deepwater, WIS Report 12, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Wave Information Study, Vicksburg, MS.
- Roache, P. J. 1998. Fundamentals of Computational Fluid Dynamics. Hermosa Publishers.
- Saco Bay Regional Beach Management Plan. 2000.
<http://smrpc.org/landuse/Coastal/Saco%20Bay%20Plan%20Final.pdf>
- Seiji, M., T. Uda, and S. Tanaka. 1987. Statistical Study on the Effect and Stability of Detached Breakwaters, Coastal Eng. In Japan, Vol. 30, No. 1, p. 131-141.
- Shalowitz, A.L. 1964. Shoreline and Sea Boundaries, Volume 2. U.S. Department of Commerce Publication 10-1, U.S. Government Printing Office, Washington, DC, 420 pp.
- Slovinsky, Peter A. and Stephen M. Dickson. 2003. Variation of Beach Morphology Along the Saco Bay Littoral Cell. An Analysis of Recent Trends and Management Alternatives. Open-File Report 03-78, Maine Geological Survey Department of Conservation.
- Smith, J.M. and M.B. Gravens. 2002. Incident Boundary Conditions for Wave Transformation. 7th International Workshop on Wave Hindcasting and Forecasting, Banff, Alberta.
- Smith, J.M. and M.B. Gravens. 2003. Incident Boundary Conditions for Wave Transformation. 7th International Workshop on Wave Hindcasting and Forecasting, Banff, Alberta.
- Smith, J.M, A.R. Sherlock, and D.T. Resio. 2001. STWAVE: Steady-State Spectral Wave Model User's Manual for STWAVE, Version 3.0. U.S. Army Corps of Engineers, Vicksburg, MS.
- Stafford, D.B. 1971. An Aerial Photographic Technique for Beach Erosion Surveys in North Carolina, CERC Technical Memo No. 36, 115 pp.
- Suh, K. and R.A. Dalrymple. 1987. Offshore Breakwaters in Laboratory and Field. Journal Waterway, Port, Coastal, and Ocean Eng., ASCE, Vol. 113, No. 2, p. 105-121.
- Tanner, W.F. 1978. Standards for Measuring Shoreline Changes: A Study of the Precision Obtainable and Needed in making Measurements of Changes (Erosion and Accretion). Proceedings of a Workshop, Florida State University, Tallahassee, FL. 89 pp.
- Toba, Y. 1978. Stochastic Form of the Growth of Wind Waves in a Single-Parameter Representation with Physical Implications. Journal of Physical Oceanography, v. 8, p. 494-507.
- U.S. Army Corps of Engineers. 2002. Coastal Engineering Manual. Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 volumes).
- USACE. 1886. Report of the Acting Chief of the Engineers of a Survey of the Saco River. Executive Document 37, 49th Congress, p. 5.
- USACE. 1910. Reports of the Examination and Survey of the Saco River, Maine. Document 752, 61st Congress, p.3.
- USACE. 1955. Beach Erosion Control Report on Cooperative Study of Saco, Maine. Waltham.
- USACE. 1984. Shore Protection Manual Volume II.
- USACE. 1988. Tidal Flood Profile #11, Station 240, New England Coastline.

- USACE. 1991. Assessment of Coastal Processes in Saco Bay, Maine, with Emphasis on Camp Ellis Beach: U.S. Army Corps of Engineers, Coastal Engineering Research Center, Vicksburg, MS, 39 pp. and Appendices.
- USACE. 1995. Camp Ellis Beach, Saco Bay, Maine, Model Study of Beach Erosion. Coastal Model Investigation. Technical Report CERC-95-11.
- USACE. 2001. Saco River and Camp Ellis Beach, Saco Maine Beach Erosion Study: Section 111 Shoreline Damage Mitigation Study, Initial Appraisal (DRAFT), U.S. Army Corps of Engineers, New England District, Waltham, MA, 15 pp.
- van Heteren, S., D.M. FitzGerald, D.C. Barber, J.T. Kelley, and D.F. Belknap. 1994a. Bedrock Control on the Development of the Saco Bay, Maine, Barrier System: Geological Society of America, Abstracts with Programs, v. 26, no. 3, p.77.
- van Heteren, S., D.M. FitzGerald, D.C. Barber, J.T. Kelley, and D.F. Belknap. 1996. Volumetric Analysis of a New England Barrier System Using Ground-Penetrating –Radar and Coring Techniques: Journal of Geology, v. 104, p. 471-483.
- Woods Hole Group. 2004. Final Report Analysis of shoreline Change for Western Beach, Saco, Maine, 16 pp.
- Xu, Bingyi, Vijay Panchang and Zeki Demirebilek. 1996. Exterior Reflections in Elliptic Harbor Wave Models. Journal of Waterway, Port, Coastal, and Ocean Engineering, May/June, p. 118-126.