# CAPE COD CANAL & SANDWICH BEACHES

# SHORE DAMAGE MITIGATION STUDY

## APPENDIX C

## WOODS HOLE GROUP
## COASTAL ENGINEERING REPORT

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# Cape Cod Canal Section 111 Feasibility Study
# Coastal Modeling Support Services
**Contract # W912BU-15-D-0004**

## Town Neck Beach
## Sandwich, Massachusetts

**September 23, 2019**

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# 1.0 INTRODUCTION

This report investigates the relationship between the Cape Cod Canal (CCC) and erosional problems experienced along the shorelines of the Town of Sandwich, Massachusetts, which lies directly downdrift to the southeast of the Cape Cod Canal. Due to the predominant northwest to southeast movement of sediment, the CCC has created an interruption in the natural transport of sediment to the Town of Sandwich beach system. As such, this report evaluates the influence the Cape Cod Canal and its associated structures may have on the adjacent shorelines, and then assesses potential alternatives to mitigate potential impacts attributed to the CCC Federal Navigation Project (FNP). Specifically, the tasks and evaluations presented in this report are focused on providing support to the United States Army Corps of Engineers (USACE), New England District to complete the CCC Section 111 Feasibility Study.

## 1.1 BACKGROUND

The study area is located on the north shore of Cape Cod, Massachusetts, facing Cape Cod Bay. The primary focus region includes nearly 2.5 miles of shoreline, extending from the south side of the Cape Cod Canal to Spring Hill Beach (Figure 1). Old Sandwich Harbor is located near the center of the project area and serves to connect an extensive salt marsh system with Cape Cod Bay. The upper reaches of this salt marsh system directly abut many areas of historic downtown Sandwich. A large portion of the beaches within the study site are owned by the Town of Sandwich, including Town Neck and portions of Spring Hill Beach located to the north and south of Old Sandwich Harbor, respectively. Town Beach extends from Sandwich Harbor north towards the Canal, and fronts the residential development known as Town Neck Hill (Figure 1). The southeastern end of Spring Hill Beach is privately owned and developed with several homes and cottages. Finally, the nearby power utility company owns the area immediately adjacent to the southern Cape Cod Canal jetty.

The beaches in the Town of Sandwich, including Town Neck Beach and Spring Hill Beach have a history of erosion (as presented in Chapter 2). It has long been assumed that construction of jetties at the east end of the Cape Cod Canal in 1906 has been the primary reason for this coastal erosion (Giese, 1980). The two Canal jetties cause an interruption in the natural alongshore sediment transport from northwest to southeast. In order to combat the erosion, subsequent alterations to the Sandwich barrier beach system (e.g., development on the barrier beaches, construction of jetties, and the construction of groins) attempted to stem some of the erosion that was occurring along Town Neck Beach but due to the lack of incoming sediment, had minimal impact.. Ultimately, the influence of the Cape Cod Canal has limited the sediment supply to the Town of Sandwich beaches such that the barrier beach system cannot maintain a healthy beach and dune complex. For example, during storm events, the sediment starved beach offers minimal energy dissipation and sediment is mobilized from the dunes as the beach attempts to protect itself. Through time, the dunes have been narrowed and now offer a minimal amount of remaining sediment such that the beach and dune system has a critical shortage of sediment.

Hundreds of thousands of cubic yards of sand destined for Sandwich's beaches have been trapped at the western jetty, or within the Cape Cod Canal, and subsequently dredged and disposed offshore, exacerbating natural erosional pressures arising from coastal storms and sea-level rise, increasing the potential for community-wide flooding, and reducing valuable habitat for threatened shorebirds.

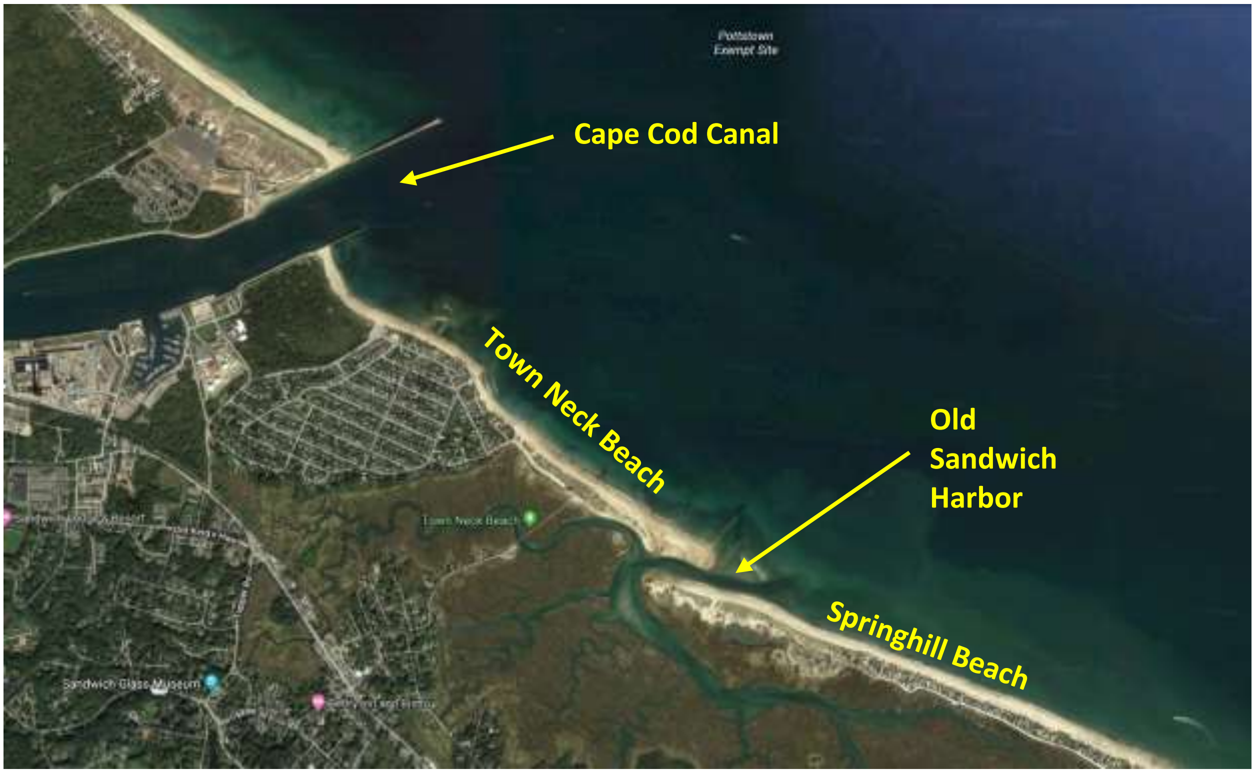


**Figure 1. Project Location Map.**

While the Town of Sandwich has made efforts to improve the dune systems through multiple dune restoration projects (between 1990 and 2016), storm events have increasingly taken their toll on the struggling barrier beach. For example, in October 1991 (during the "No Name" Storm), the Sandwich Harbor Inlet breached out of its existing jetties and the primary channel migrated to the southeast towards the homes on Spring Hill Beach. Multiple nor'easters in the last decade have caused breaches in the barrier beach system, flooded the great marsh system and landward areas, and resulted in loss of homes at Town Neck Hill.

### 1.2 GEOLOGICAL HISTORY

The geologic evolution of Cape Cod, including the project area, can be directly linked to the advance and retreat of continental glaciers, and the change in relative sea level that followed retreat of the last ice sheet. Uchupi et al. (1996) provides a comprehensive update of this complex, post-glacial evolution of Cape Cod that initially reflected a rapid fall in relative sea level, followed by a rise in relative sea level. During the most recent glacial stage that occurred between 75,000 and 21,000 years ago, more snow fell over the northern latitudes than melted each year. As the snow accumulated and compacted, it formed large ice sheets or glaciers. The Laurentide Ice Sheet (named after the Laurentian region of Canada where it first formed) spread into the United States. Its southeast advance extended from New York City, east to Long Island and Nantucket, covering all of New England (Figure 2). This maximum southern extent of the ice sheet occurred approximately 21,000 years ago.

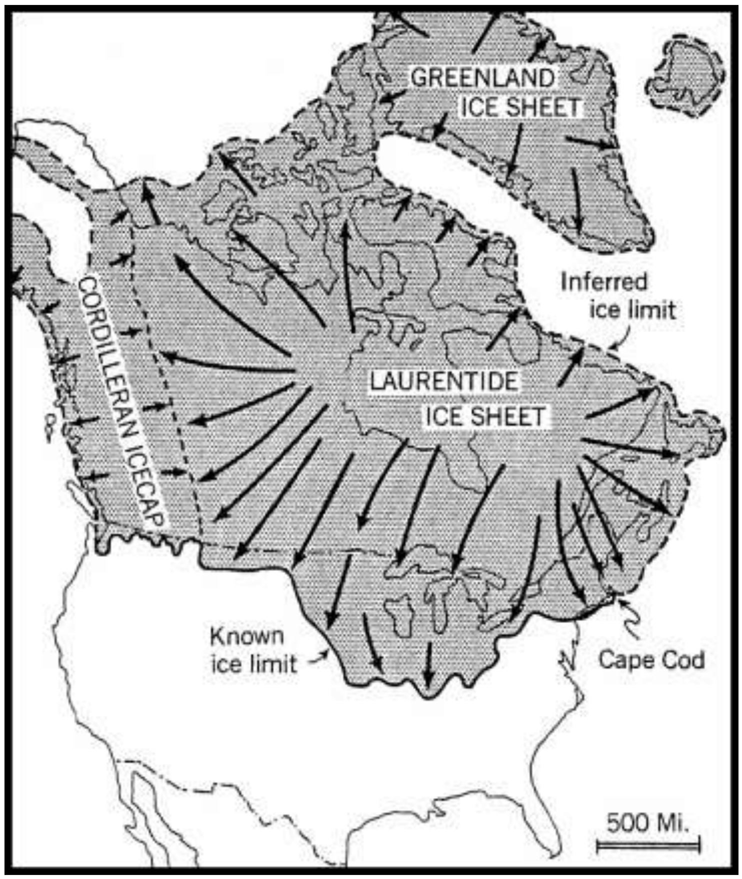


**Figure 2. The southward-most extent of the continental ice sheet during the most recent ice age. Directions of ice flow are indicated by arrows (Strahler, 1966).**

Three major lobes of the Laurentide Ice Sheet covered Massachusetts and the Cape Cod region: Buzzards Bay Lobe, Cape Cod Bay Lobe, and the South Channel Lobe. Figure 3 indicates the location and flow direction of these three lobes during their advance. At its point of farthest advance around 21,000 years ago, the ice sheet reached Martha's Vineyard, Nantucket Island, and the Elizabeth Islands (Figure 3). During its advance, the glacier carved the land underneath, tearing off large pieces of bedrock from the terrain, sculpting ridges and valleys, and grinding larger rocks into sand and silt-sized particles. The ice sheet held this maximum position for more than 1,000 years, until a rapid warming of the world's climate caused glacial melting. Evaporation rates exceeded snowfall rates, and the ice began to melt. As the ice receded, the sand, gravel, clay, and boulders that the glacier had accumulated were deposited in the form of moraines and outwash plains to form Martha's Vineyard and Nantucket, as well as many adjacent shoals.

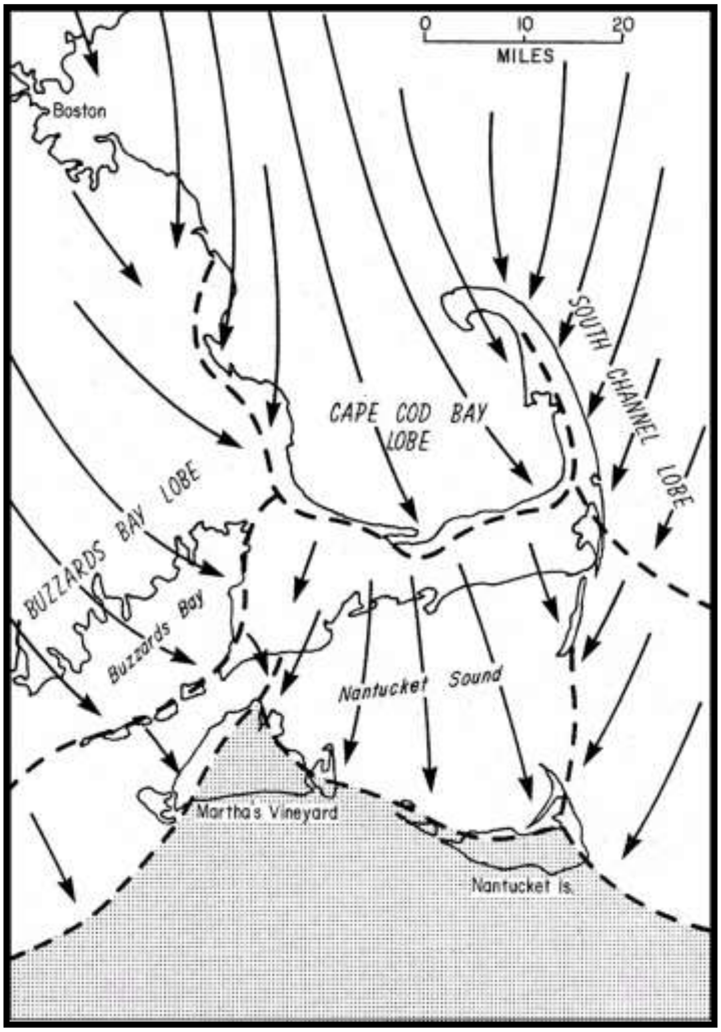


**Figure 3. Flow directions of the three major ice lobes in southeast Massachusetts (indicated by arrows) and positions of ice standstill (indicated by dashed lines) (Leatherman, 1988).**

Radiocarbon dating by Kaye (1964) suggests that sometime around 15,300 years before present; temperature elevations caused rapid retreat of the ice sheet to a second stationary position. The Buzzards Bay Lobe retreated to the position of the Elizabeth Islands and Western Cape Cod, the Cape Cod Bay Lobe retreated to the northern shores of present-day Cape Cod, and the South Channel Lobe formed most of outer Cape Cod (Larson, 1982; Figure 3). This second stationary position lasted for several thousands of years. During this long period, gravel, sand, clay, and boulders were deposited as moraines along the edges of the glacier, or as till sheets from the continual inflow and melting of ice (Strahler, 1966; Oldale, 1982). The Buzzards Bay and Sandwich moraines mark the positions of the glacial lobes during this second stationary period (Strahler, 1966; Figure 4). The Sandwich moraine extends from the Cape Cod Canal region, in an easterly direction towards Eastham.

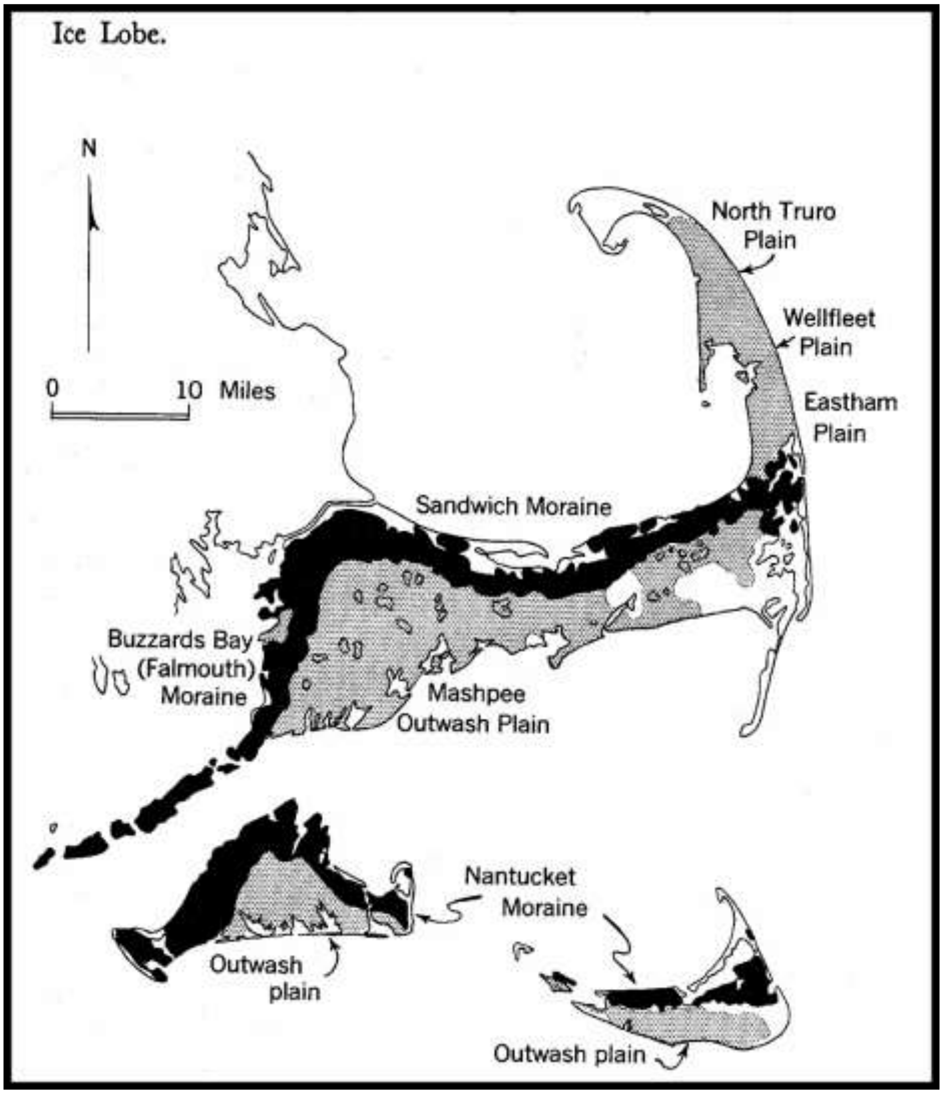


**Figure 4.** **Glacial geologic map showing the Buzzards Bay and Sandwich Moraine deposits marking the edges of the Buzzards Bay and Cape Cod Bay ice lobes (Strahler, 1966).**

As the glaciers continued to recede northward, the melt waters accumulated to form large glacial lakes, rimmed in part by the higher terminal moraines. One of the largest of these glacial lakes formed in Cape Cod Bay and has been named Glacial Lake Cape Cod. Earliest levels of the lake ranged between 80 and 50 ft above present sea level (Oldale, 2001). Drainage of Glacial Lake Cape Cod occurred across the Sandwich moraine in the vicinity of the Cape Cod Canal, into the lowland that eventually became Buzzards Bay. As the glaciers retreated farther northward, additional drainage outlets for Glacial Lake Cape Cod were developed. Also, at this time the earth's crust began to rebound (rise) at a rapid rate, resulting in emergence of the shoreline and a lowering of relative sea-level. In southeast Massachusetts, a lowstand of sea level occurred between 10,000 and 12,000 years ago (Leatherman, 1988). During this time Glacial Lake Cape Cod drained completely and the shoreline was much further seaward than at present (Figure 5).

The initial drainage outlet for Glacial Lake Cape Cod across the Sandwich moraine eroded a low divide forming a natural location for construction of the Cape Cod Canal.

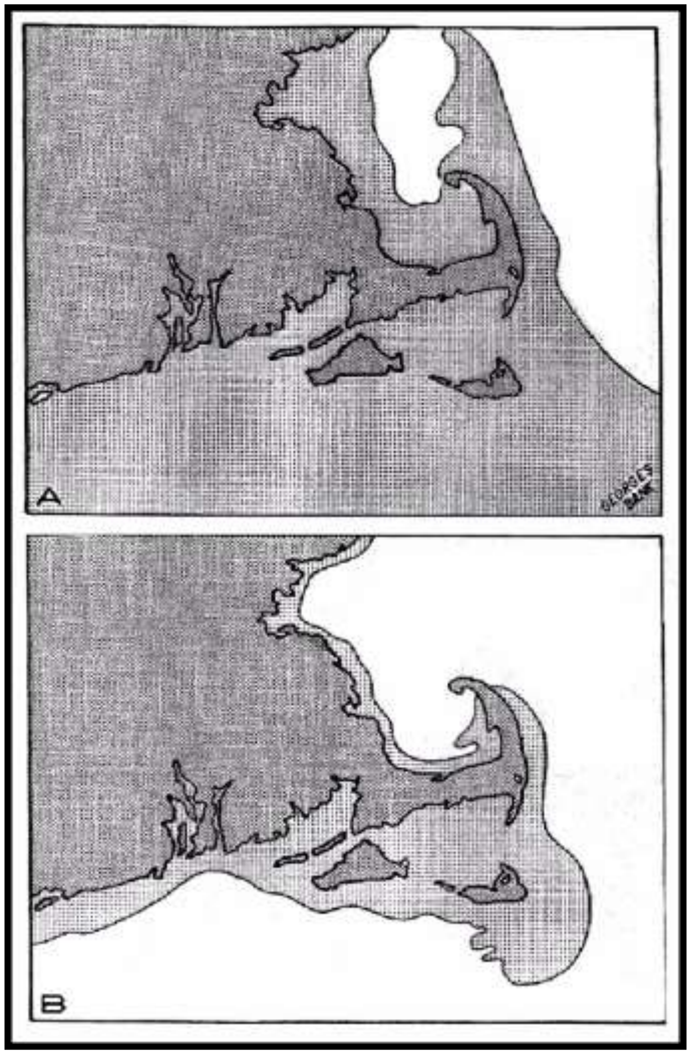


**Figure 5. Shoreline changes of southeastern Massachusetts: (A) 12,000 years ago and (B) 7,000 years ago (Leatherman, 1988).**

Between 12,000 years ago and the present, the rate of isostatic rebound decreased, while eustatic sea level (worldwide) continued to rise. This resulted in a rise of relative sea-level and a transgression of the shoreline. Emery and Aubrey (1991) report that during the last 18,000 to 10,000 years, ocean levels have risen between 60 and 120 m. Sea levels rose rapidly at first, causing local submergence of Buzzards Bay and Cape Cod Bay (FitzGerald et al., 1992). This rapid rise in sea level continued until approximately 3,500 years ago, at which point sea level rise slowed significantly (Redfield and Rubin, 1962). From this point on, sea levels gradually rose to the location of the present-day shoreline, as waves and currents reworked the pre-existing glacial features and sediments into the modern-day beaches, spits, dunes, bays, and marshes that comprise the shorelines of Cape Cod Bay today.

The existing shorelines of the study area are composed of modern-day beach and dune deposits. The area of Town Neck Beach was originally located along the edges of Glacial Lake Cape Cod, and the Old Harbor area was subsequently transformed into a quiet estuarine environment protected from the higher energy of Cape Cod Bay by the evolving barrier beach and dune system.

### 1.3 CONTEMPORARY HISTORY

This lack of natural sediment transport to Town Neck Beach, which has exacerbated erosion, has forced the Town of Sandwich and private homeowners to provide resiliency by artificially nourishing the beach. The barrier beach and dune system has been eroded to the point where even small to moderate storms can breach, damage, flood, and impact the Town of Sandwich. Over the last few decades, the Town has permitted numerous small projects using upland sand sources, as well as three large-scale projects where dredge material from the Cape Cod Canal were pumped onto the beach to restore the beach and dune resources (Chapter 3). Placement of the most recent large-scale nourishment (2016) was permitted through the Town of Sandwich Dune and Beach Reconstruction Project (EEA #15213), which was originally reviewed and permitted by the Executive Office of Energy and Environmental Affairs (EOEEA) in 2014. These larger-scale projects have been conducted in cooperation with the adjacent power utility company and the US Army Corps of Engineers (USACE). In 1990, the Town rebuilt the dunes at the eastern end of Town Neck Beach by placing approximately 122,000 cubic yards of sand dredged from the Cape Cod Canal in front of the public beach parking lot. In April 2004, the Town worked with Mirant Canal, LLC to beneficially reuse 50,000 to 65,000 cubic yards of sand dredged from Mirant's approach channel in the Cape Cod Canal as beach nourishment on Town Neck Beach. Several smaller post-storm restoration projects have been carried out at the eastern end of the beach to repair dune overwash areas created during hurricane Sandy in 2012 and winter storm Nemo in 2013. The most recent beach restoration was completed in January 2016, when approximately 110,000 to 120,000 cubic yards of sand was dredged from the Canal by the USACE for improved navigation, and placed on the eastern end of Town Neck Beach.

Clearly the demands for nourishment material at Town Neck Beach calls for a long-term sustainable solution that addresses the on-going erosion problems. There is also an immediate need to augment the recent beneficial reuse of 110,000 cy of Canal sand in the project template that was permitted in 2014 (EEA #15213). The continued need for a sustainable solution was the impetus for this study to address the long-term influence of the Cape Cod Canal. Beneficial reuse of sand dredged from the Cape Cod Canal as nourishment material is certainly welcome as part of the ongoing maintenance, but an additional source is needed to meet volumetric demand in the near-term and to continue future maintenance of Town Neck Beach.

## 2.0 SHORELINE CHANGE AND VOLUME ANALYSIS

The Town of Sandwich shoreline stretches east of the Cape Cod Canal for approximately 8.3 miles and includes Town Neck and Springhill Beaches, which are separated by Old Sandwich Harbor Inlet (the Inlet). As discussed, installation of the jetties at the Cape Cod Canal in the early 1900s created an interruption of the natural movement of sediment along the shoreline. A significant portion of sand was trapped updrift (northwest) of the northwest jetty. Sand not impounded by the jetty is either transported around, or through the jetty and ultimately is either deposited in shoals at the mouth of the Canal, makes its way deeper into the Canal, or is transported seaward of the canal into deeper waters, outside of the active littoral zone. As such, much of the sand destined for Sandwich's beaches has been trapped at the western jetty, or within the Cape Cod Canal. In order to determine the potential impacts of the Cape Cod Canal and its associated structures on the Town of Sandwich beaches, one key component is the assessment of the historic rates of shoreline change, and estimated volume losses. These data are also valuable to determine future shoreline projections if no mitigation actions are undertaken. Specifically, the goal of this technical task, which consists of shoreline change analysis and volume change assessment, is to:

1) Estimate the influence distance associated with the littoral interruption caused by the Cape Cod Canal
2) Assess the potential future shoreline conditions and erosion that may occur over a 50-year time horizon if no mitigation actions are undertaken
3) Determine the amount of volume loss that has occurred over the past 50 years, as well as the projected volume loss over the next 50 years

Ultimately, creation of a healthier beach and dune system that is more resilient to the effects of sea level rise and increased frequency of coastal storms requires an improved understanding of the historical and present-day changes taking place downdrift of the Canal. Historical and future rates of shoreline change and volumes of sediment lost from the beach system were assessed.

### 2.1 TOWN NECK AND SPRINGHILL BEACH SHORELINE CHANGE ANALYSIS

A computer-based shoreline mapping methodology within a Geographic Information System (GIS) framework was used to compile and analyze changes in historical shoreline position for Town Neck and Springhill Beach shorelines. The purpose was to quantify the spatial and temporal changes in past shoreline positions using the most accurate data sources and compilation procedures available, and to evaluate the long-term rates of change. Assuming that the historical trends continue at the same rate into the future, the information from the shoreline change analysis was also used to predict patterns of shoreline change over the next several decades.

Data from a variety of publicly available sources were used to document changes in shoreline position over the 66-year period between 1952 and 2018 (Table 1). Shoreline change data from MassCZM between 1952 to 2009 were obtained from the Massachusetts Shoreline Change Mapping and Analysis Project (USGS 2013). More recent data from 2014 to 2018 were added to the analysis and obtained by digitizing the mean high water (MHW) line from georeferenced orthoimagery available from MassGIS and Google Earth.

**Table 1. Data Sources for Shoreline Change Analysis**

| Year | Source |
|---|---|
| 1952 | MassCZM shoreline from U.S. Coast and Geodetic Survey |
| 2000 | USGS LiDAR, MassCZM |
| 2009 | USGS 30-cm Digital Orthophotography, MassCZM |
| 2014 | USGS Color Ortho Imagery via MassGIS |
| 2018 | Aerial Imagery TerraMetrics via Google Earth |

Once the shoreline data were compiled, spatial and temporal changes in the data were computed. This was accomplished by identifying a series of shore perpendicular transects (SLX transects) along the coastline where discrete measurements of change could be made through time, and where rates of change could be determined. A total of 139 SLX transects were established at 100 foot evenly-spaced intervals along the coastline; the entire study area spanned just over 3.2 miles. Figures 6 and 7 show the regional distribution of transects between Town Neck and Springhill Beach. A summary of the SLX transect locations is as follows:

- Town Neck Beach (defined as the beach between the CCC and Old Harbor Inlet) – Transects 1 to 74
- Spring Hill Beach (defined as the beach to the east of Old Harbor Inlet) – Transects 75 to 139

At each SLX transect, distances of shoreline movement and annual rates of change were determined. Data from 1952 to 2018 were used to compute long-term rates of change as seen in Figure 6, and data from 2000 to 2018 were used to compute contemporary, short-term rates of shoreline change, as seen in Figure 7. The rates of change were calculated using the linear regression method. In this method, an average rate of change is based on a best-fit line to a series of points representing the shoreline position over time. The linear regression method is most accurate when looking at long-term averages and is commonly used for planning purposes and management decisions.

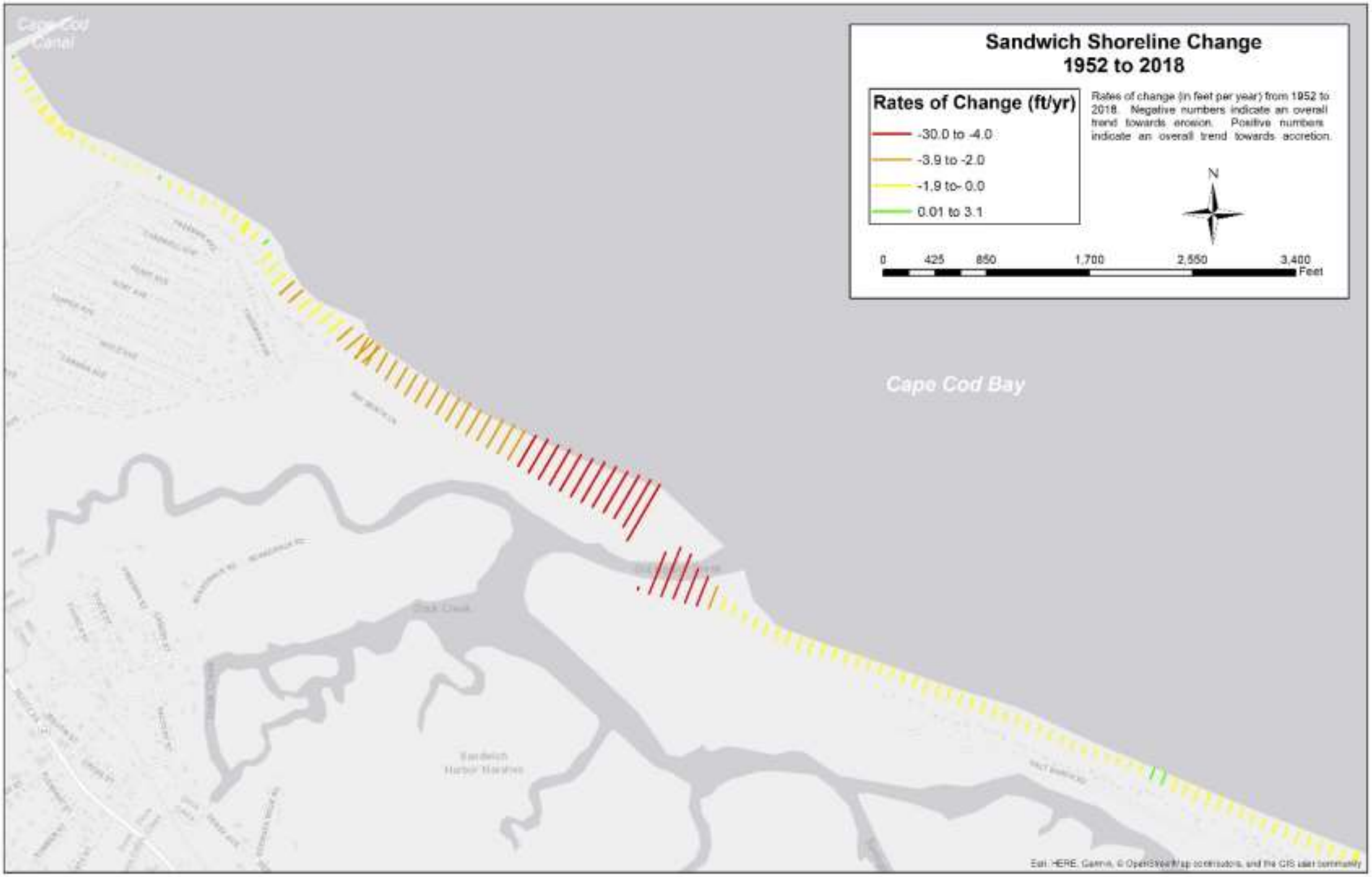


**Figure 6.** **Map of long-term (1952-2018) rates of shoreline change downdrift of the Cape Cod Canal to Springhill Beach. All values are in feet/year. Red, orange and yellow lines indicate erosion, while green lines indicate accretion.**

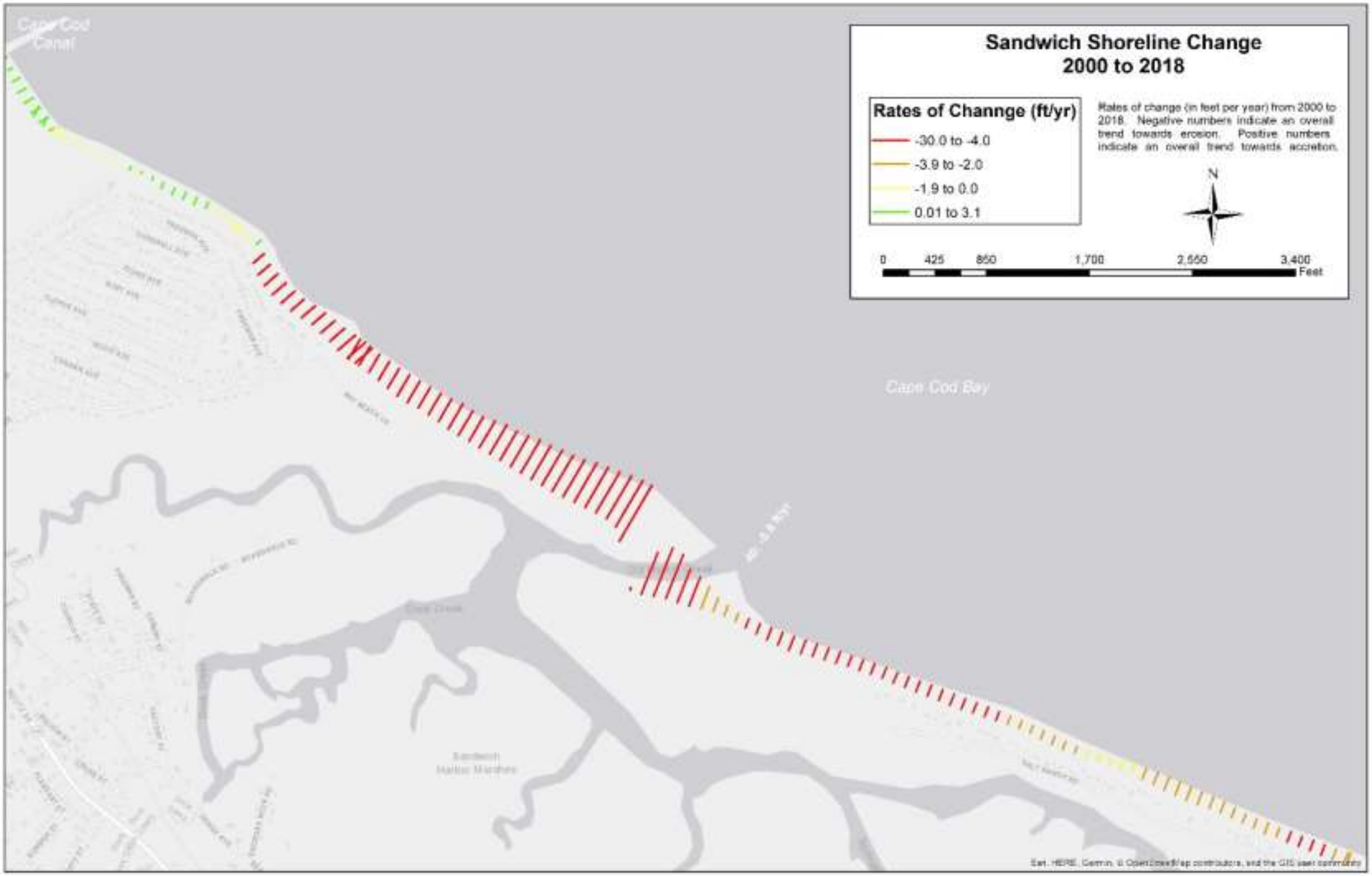


**Figure 7.** **Map of short-term (2000-2018) rates of shoreline change downdrift of the Cape Cod Canal to Springhill Beach. All values are in feet/year. Red, orange and yellow lines indicate erosion, while green lines indicate accretion.**

The shoreline change analysis revealed several trends extending downdrift from the Canal towards Spring Hill Beach. The linear regression rates of shoreline change for the short- (2000-2018) and long-term (1952 to 2018) time periods are presented in the graph shown in Figure 8. In general, the rates of change from 1952 to 2018 (long-term) and 2000 to 2018 (short-term) are similar; however (as expected), there is greater variability in the short-term rates as the shoreline movements are averaged over a much shorter time interval.

In the contemporary time frame (short-term) there is an area of slight accretion and mild erosion that extends from the area immediately east of the Cape Cod Canal to the longer groin located near the intersection of Dillingham Avenue and Freeman Avenue (approximately Transect 31). This stretch of shoreline has been relatively stable in both the short- and long-term, consisting of either smaller erosion rates (long-term) or areas of accretion (short-term). It is likely that this area, which lies in the shadow of the Cape Cod Canal jetties, experiences reduced wave energy afforded by the influence of the canal jetties on local wave transformations (see Chapter 4, wave transformation modeling). This is typical of an area immediately downdrift of large coastal inlet with jetties, where the area immediately downdrift of the structures may experience reduced erosion rates and a reversal in sediment transport since waves from the north do not directly impact this location. This energy reduction, coupled with the effects of the groins in this area and the slight reversal in sediment transport direction, has produced a more stable section of the shoreline relative to the areas further to the east.

The short-term shoreline change rates from the longer Town groin (approximately Transect 31) extending east to the Old Sandwich Harbor inlet, indicate a trend of increasing erosion. Much of this area has average contemporary erosion rates between -6 and -10 feet/year, while the 1,400 ft stretch of shoreline updrift of the inlet shows a dramatic increase in erosion up to -25 feet/year. These dramatically higher rates of erosion in the vicinity of the inlet are the result of inlet migration outside the jetties, which were originally constructed to constrain the inlet mouth for vessel transit to and from the Sandwich Glass Factory. Spring Hill Beach, east of Old Harbor Inlet, shows consistent erosion as well, with rates averaging approximately -3 feet/year. The short-term, contemporary time period shows more significant erosion than the long-term, perhaps due to the dwindling sediment supply that has had a major influence on the beach system.

The longer-term shoreline change rates follow a similar pattern as the short-term data; however, trends are more consistent and well defined. Generally, erosion occurs along most of Town Neck Beach and Spring Hill Beach. Some key findings include:

- The area immediately east of the Cape Cod Canal has experienced long-term erosion, but at a reduced rate compared to the shoreline further east. This is the area between the Cape Cod Canal to the longer groin located near the intersection of Dillingham Avenue and Freeman Avenue (approximately Transect 31) that has shown stability in both the long and short term. The Town of Sandwich has not nourished this section of the beach historically, nor is this area of the beach included in the permitted beach nourishment template (EEA #15213). Dune restoration only is proposed in this area, and only at the far eastern end of this section of the beach.

- From the large groin (Transect 31) to Old Sandwich Harbor inlet the shoreline change rates show a trend of increasing erosion. Immediately east of Old Sandwich Harbor inlet, the long-term data reveal variability in rates of erosion, due in large part to inlet migration. Rates of erosion range between -2 to -5 feet/year.

- Further to the east, beyond the influence of the Old Harbor inlet, the data show a trend of decreasing erosion. This trend continues to approximately Transect 108, where the rates of erosion level off to a consistent value and the shoreline is stable. This distance is approximately 10,800 feet downdrift of the Cape Cod Canal and is a reasonable estimate of the influence distance of the Canal. In other words, the disruption in the natural sediment transport caused by the Canal and its structures appears to extend approximately 10,800 feet downdrift.

Over the long-term, data show erosion along the entire 3.2 mile stretch of shoreline. The highest rates of erosion occur on both sides of the Old Sandwich Harbor inlet, and along Town Neck Beach. Lower rates of erosion occur along Springhill Beach and immediately downdrift of the Cape Cod Canal. Similar trends are seen over the short-term period between 2000 and 2018; however, the rates of erosion along Springhill Beach and updrift of Old Sandwich Harbor are higher (perhaps due to a dwindling sediment supply), and an area of shoreline accretion is shown downdrift of the Canal (in the shadow of the CCC jetties).

### 2.2 Town Neck and Springhill Beach future shoreline position

Information developed during the shoreline change analysis was also used to estimate a future shoreline position assuming (1) the rates of erosion determined from the long-term analysis remained constant over the next 50 years, and (2) the latest sea level rise projections that are consistent with those being applied across the Commonwealth of Massachusetts and published by Massachusetts Coastal Zone Management. Using these assumptions, a projected shoreline for the Town of Sandwich was generated 50 years from now (2068).

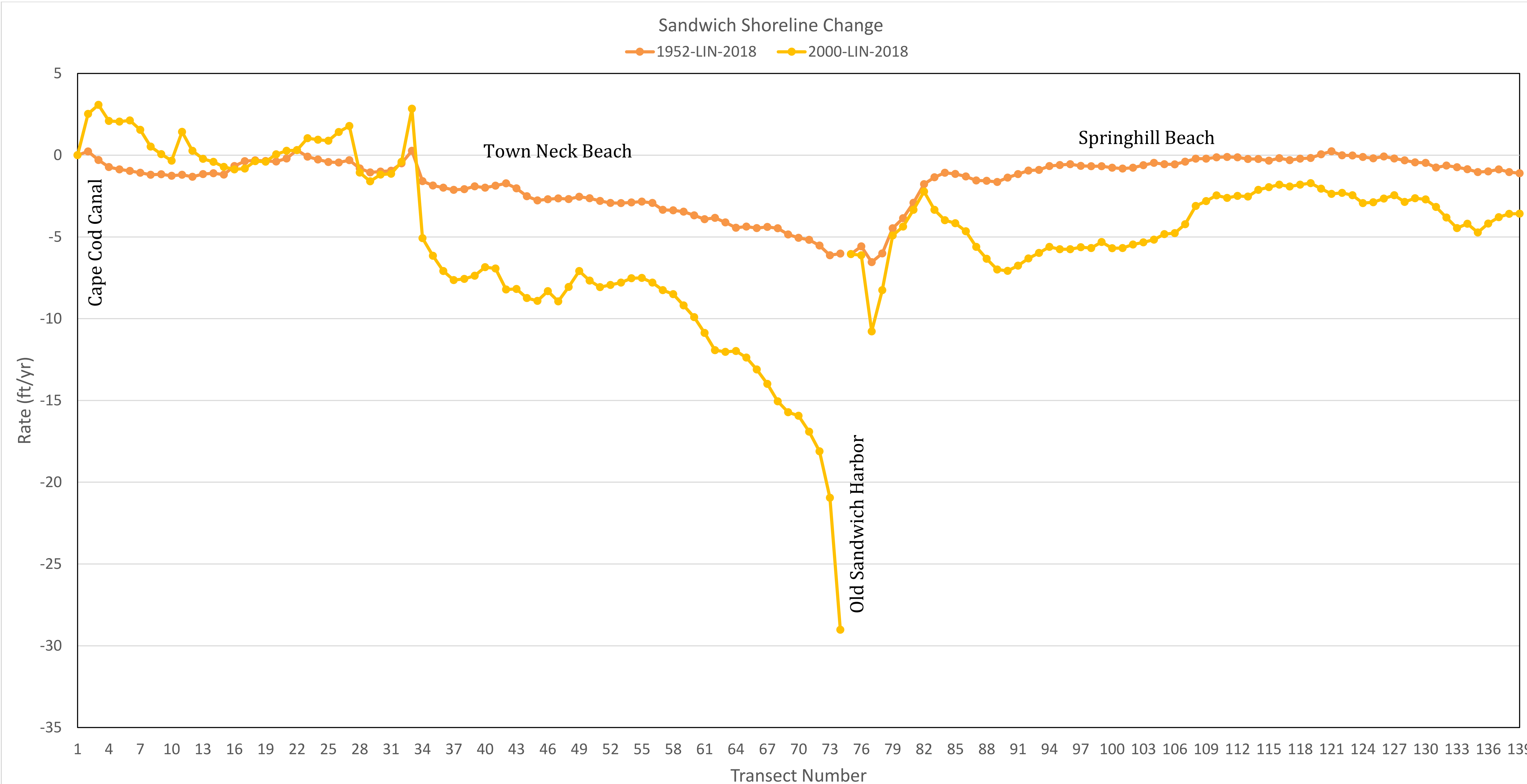


**Figure 8.** **Graph of long- and short-term shoreline change rates along each transect extending downdrift of the Cape Cod Canal to Springhill Beach. All values are in feet/year.**

First, a projected 2068 shoreline (without the influence of expected sea level rise) was generated using long-term rates of annual change for each transect. Then, the impacts of expected sea level over the next 50 years were combined with the projected shoreline change results to develop a more comprehensive picture of future shoreline position. Sea level rise projections are based on the Representative Concentration Pathways (RCP) greenhouse gas concentration trajectories developed as part of the Intergovernmental Panel on Climate Change (IPCC). These pathways describe a wide range of possible scenarios that may occur due to future anthropogenic greenhouse gas emissions. The RCP pathway utilized in this assessment (RCP 8.5) essentially assumes that no changes are made to human based emissions. The sea level rise produced under this scenario (RCP8.5) was developed specifically for the Commonwealth of Massachusetts (DeConto and Kopp, 2017) and is consistent with the projections being implemented for the statewide hazard mitigation assessments by MACZM and by MassDOT in the development of the Massachusetts Coast Flood Risk Model (MC-FRM). Therefore, this assessment aligns with the recommended projection values used for the coastlines in Massachusetts. Projections were developed for the Commonwealth of Massachusetts and take into consideration the regional considerations of the Northeast (DeConto and Kopp, 2017). As such, a predicted relative sea level rise for Boston, MA in the year 2070 of 4.29 feet was utilized for this analysis (MassCZM, 2018).

To estimate the predicted elevation of mean high water in 2068, the relative rise of 4.29 feet was added to the present day mean high water elevation to yield a projected elevation of 8.4 feet NAVD88 for mean high water in 2068. Topographic data from MassGIS collected in 2013 by USGS to study Hurricane Sandy impacts were used illustrate the combined effects of shoreline change and sea level rise in 2068. Figure 9 predicted inundation at mean high water along Town Neck and Springhill Beach in combination with the 2068 shoreline. All elevations below the 8.4 feet NAVD88 elevation and seaward of the projected 2068 shoreline are shown in blue, while areas remaining above the 8.4 feet NAVD88 mean high water line are displayed in green. As such, this figure represents the potential shoreline and beach condition in 2068 (approximately 50 years) if no mitigation actions are undertaken and sea level rise projections follow the RCP8.5 emissions scenario. Under these projections, there is almost a complete loss of the barrier beach at Town Neck. This condition would also result in significant ecological impacts to the expansive salt marsh system, as well as lead to impacts directly on the center of the Town during storm events.

### 2.3 Town Neck and Springhill Beach historic and projected volume losses

Information developed during the shoreline change and sea level rise analyses were also used to estimate the potential volume of sediment that would be lost along Town Neck and Springhill Beaches over the next 50 years. A backward-looking analysis was also performed to estimate the volume of sand that was lost from the system over the previous 50-year period (from approximately 1968 to 2018). For the purpose of this study, the total volume of sand lost from the beach, over the past and future 50 years, was calculated for areas above the present-day elevation of mean low water (-5.2 feet NAVD88). While some of the sediment lost may remain in the nearshore, or within the estuary system, the focus of this estimate was to predict loss along the beachfront downdrift of the Cape Cod Canal. This calculation is a rough first-order estimate to provide a general idea of the volumes of sediment that may not be arriving to the Sandwich shorelines.

To determine the volume losses, thirty, shore perpendicular volume change transects were generated approximately 500 feet apart along the shoreline to approximate the loss of sediment for different portions of the beach. Beach profiles were developed at each transect location to characterize the slope and elevation of the barrier beach. By comparing the beach profiles for present day conditions with similar profiles representing +/- 50 years, it was possible to develop estimates of the volume of sediment lost in the previous 50 years, as well as the volume expected to be lost over the next 50 years.

For present day conditions, recent topographic data were not available to depict the elevations throughout the entire area of interest, and as a result a combination of topographic datasets were utilized. LiDAR data from MassGIS recorded in 2013, aerial drone surveys from June of 2018, and a real-time kinematic GPS survey conducted by Woods Hole Group in November 2018 were utilized to create the beach profiles for present-day conditions. A map of the locations and corresponding data sources for each transect is depicted in Figure 10.

To generate the future profiles in 2068, the present-day topography was translated landward using the long-term shoreline retreat rates previously calculated. For example, in Figure 10, the graph displays the 2018 beach profile in blue and the predicted 2068 beach profile in red for volume change transect number 6. For example, in Figure 11, the 2018 profile was translated landward 43.25 ft to represent the predicted total retreat by 2068. At the intersection of the two profiles, at approximately 429 ft along the transect, the 2068 line was modified to reflect the shape of the profile in 2018. This manipulation of the data was an attempt to depict the erosion along the foreshore slope of the beach, while still maintaining the morphology of the more landward portion of the profile. By determining the change in area between the present day and 2068 profiles for each of the thirty transects, and accounting for distance along the shoreline, the predicted volume of sand lost downdrift of the Canal was estimated. The change in area for each transect was converted to a unit volume of erosion per linear foot of beach, and then multiplied by the representative length of shoreline for each transect. This provides a rough estimate of volume lost if the long-term shoreline erosion rates continued. Of course, there are numerous assumptions and simplifications that are applied in this analysis. For example, the assumption that the rate of change from 1952 to 2018 is representative of what may occur in the future.


**Figure 9.** **Projections of the MHW inundation in 2068 assuming long-term rates of shoreline erosion and an estimated 4.29 ft sea level rise. The projected MHW line in 2068 is represented in red.**


**Figure 10. Location of volume change transects and their corresponding data source.**

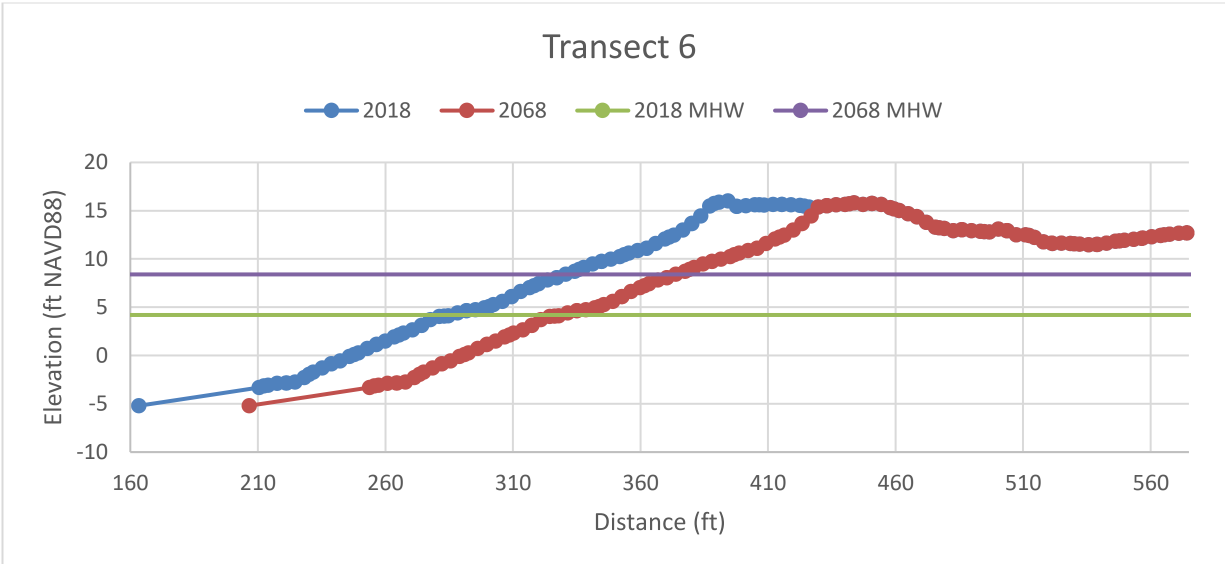


**Figure 11. Elevation profile in 2018 and 2068 at Transect 6 along shoreline.**

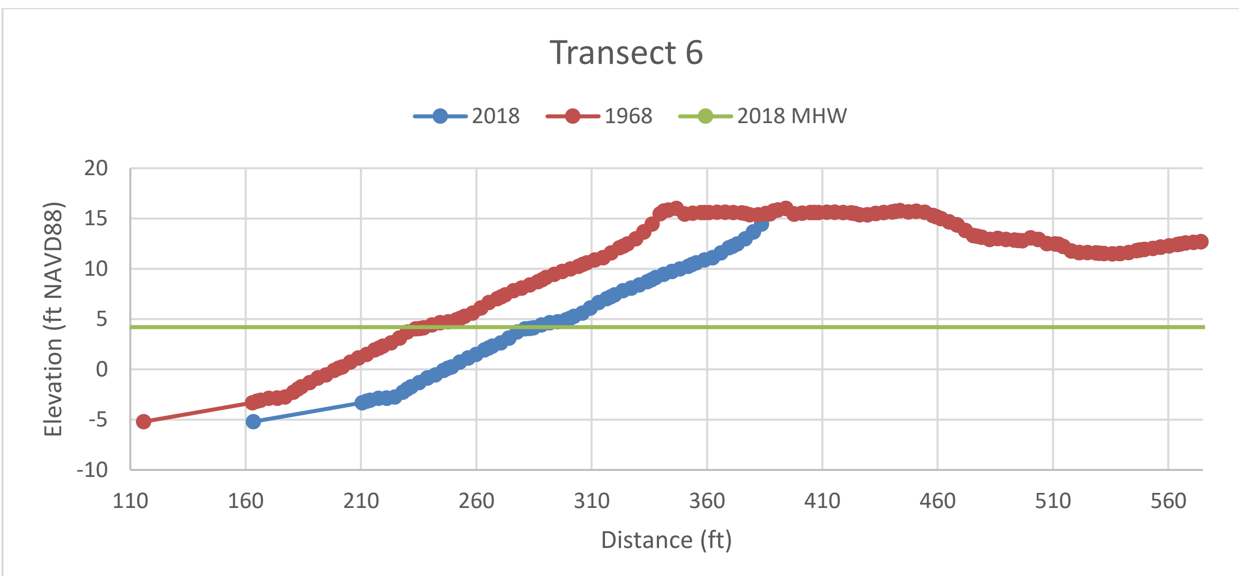


**Figure 12. Elevation profile in 2018 and 1968 at Transect 6 along shoreline.**

A similar process was used to determine the volume lost over the previous 50 years. The present-day profiles for each volume change transect were translated seaward using the same long-term shoreline retreat rates. For example, in Figure 12, the 2018 line was translated seaward 43.25 feet to represent the previous position of the foreshore of the beachfront in 1968. The 1968 profile was then modified at the intersection of the two profiles, at approximately 387 feet along the transect, to mirror the present-day morphology of the more landward portion of the beach. The volume of sediment lost between 1968 and 2018 was estimated using the change in area between these transects. The change in area for each transect was converted to a unit volume of erosion per linear foot of beach, and then multiplied by the representative length of shoreline for each transect.

Using the methodology presented above, predicted volume loss over the next 50 years was estimated at approximately 900,000 cubic yards (Table 2). The annual volume loss is approximately 1.66 cubic yards per year per linear foot alongshore. As also shown in Table 2, approximately 782,000 cubic yards of sediment was lost over the past fifty years, roughly 120,000 cubic yards less than that predicted to be lost in the next 50 years (between 2018 and 2068). The volumetric change analysis presented here was conducted over the entire length of Town Neck and Springhill Beaches (transects 1-139). Minimal volume loss occurs east of the area influenced by the Cape Cod Canal (east of transect 108). This corresponds with the influence distance determined from the shoreline change analysis. Therefore, the volumetric change influence assessment is consistent with the shoreline change influence assessment.

**Table 2. Volume Loss Estimates for Town of Sandwich Beaches over Canal influence distance.**

| Time Horizon | Total Volume Loss Estimate (cu yds) | Annual Volume Loss per Linear Foot of Beach (cu yds/ft/year) |
|---|---|---|
| 1968-2018 | 782,442.67 | 1.45 |
| 2018-2068 | 898,424.95 | 1.66 |

### 2.4 Scusset Beach Shoreline Change

More general information on historical shoreline change in the vicinity of Scusset Beach, upstream of the CCC and the Town of Sandwich's beaches, is publicly available from the Massachusetts Shoreline Change Mapping and Analysis Project (USGS 2013). The MSCP compiled relative positions of shorelines between 1860 and 2009 for all seaward facing coastal areas within the Commonwealth of Massachusetts. The MSCP included shoreline positions at Scusset Beach for the following years: 1861, 1909, 1952, 1978, 1994, 2000, 2001, and 2009. Original sources for the historical shorelines were NOAA/NOS topographic maps, hydrographic maps, FEMA topographic maps, orthophotos, and aerial photographs.

Figure 13 shows the historical shoreline positions for the area west of the Cape Cod Canal as delineated by the MSCP. Eight shoreline positions from years ranging from 1861 to 2009 are shown along with the MSCP shoreline change transect locations where shoreline change statistics were calculated. The long-term rates of shoreline change between calculated between 1861 and 2009 by the MSCP are shown in feet/year at the end of each transect. The data indicate long-term accretion rates on Scusset Beach as high as 9.0 feet/year near the center of the Scusset Beach Reservation. The rate of accretion generally decreases to the west as the distance from the Canal increases. The influence of the jetty construction in 1909 in trapping easterly moving littoral drift is clearly evident in these data.


**Figure 13. Long-term shoreline change rates along Scusset Beach from the CZM MSCP. Shoreline change rates are shown in feet/year.**

## 2.5 Sandwich Shoreline Change Prior to the Construction of the CCC

In order to assess the impact of the CCC on sediment transport patterns in the vicinity of the canal jetties, available shoreline position data from prior to the construction of the jetties were evaluated. Shoreline position data from prior to the construction of the jetties in approximately 1909 are relatively limited. The U.S. Coast and Geodetic Survey (USC&GS) created topographic and shoreline position surveys prior to the advent of aerial photography. One of these surveys was published for the CCC region in 1860 (USC&GS, 1860). Figure 14 shows a subsection of this survey focusing on the canal region.

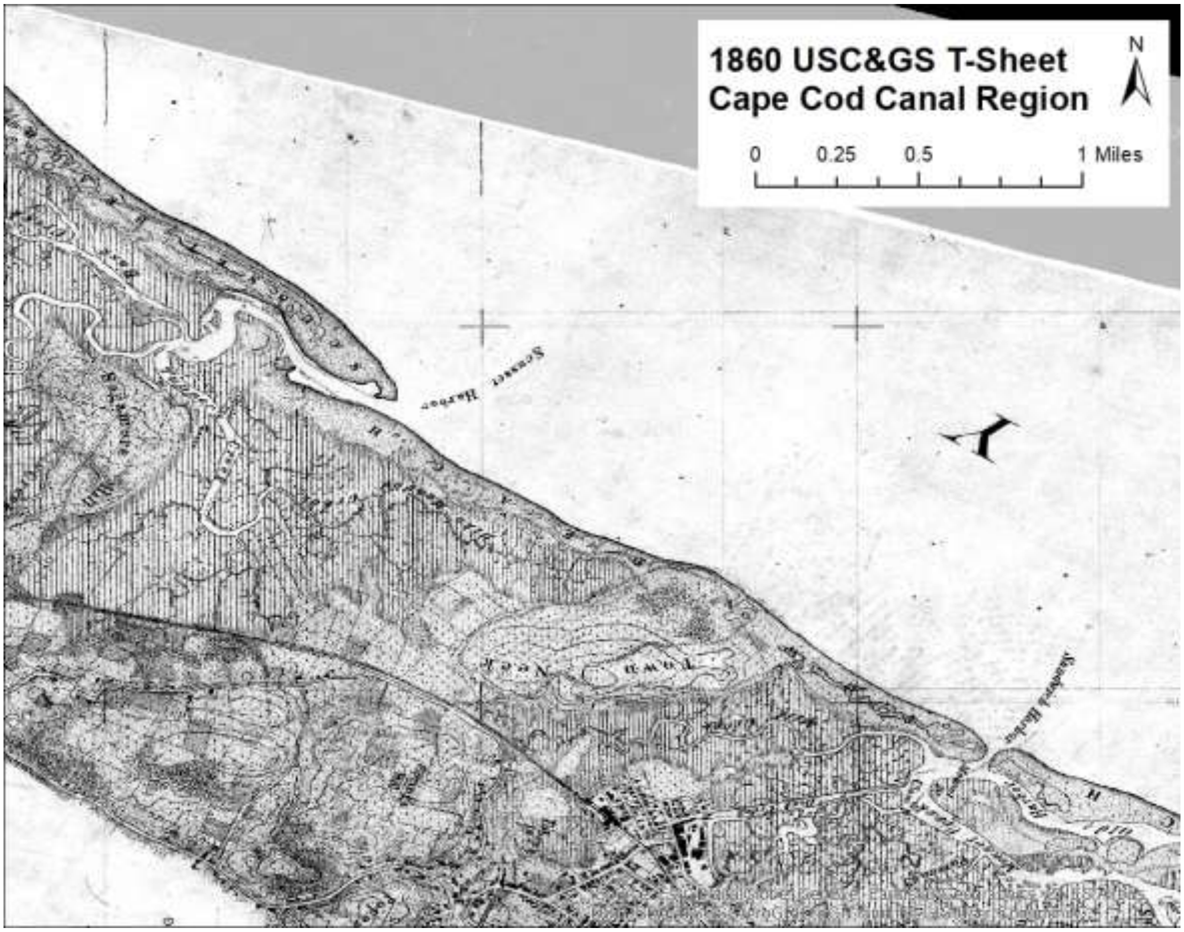


**Figure 14. Magnified area of U.S. Coast and Geodetic Survey 1860 T-sheet focusing on the canal region.**

In 1909, O.B. French of the USC&GS published a tracing showing changes in shoreline position from the 1860 T-Sheet (USC&GS, 1909). Interestingly, the tracing shows the position of the cut for the yet to be completed Cape Cod Canal, as well as containing a note that the "Jetties for this canal are now being built." The tracing shows the position of the 1860 shoreline (traced from the 1860 T-sheet), as well as the position of the shoreline in 1909. Figure 15 shows a subsection of the tracing focusing on the canal region.

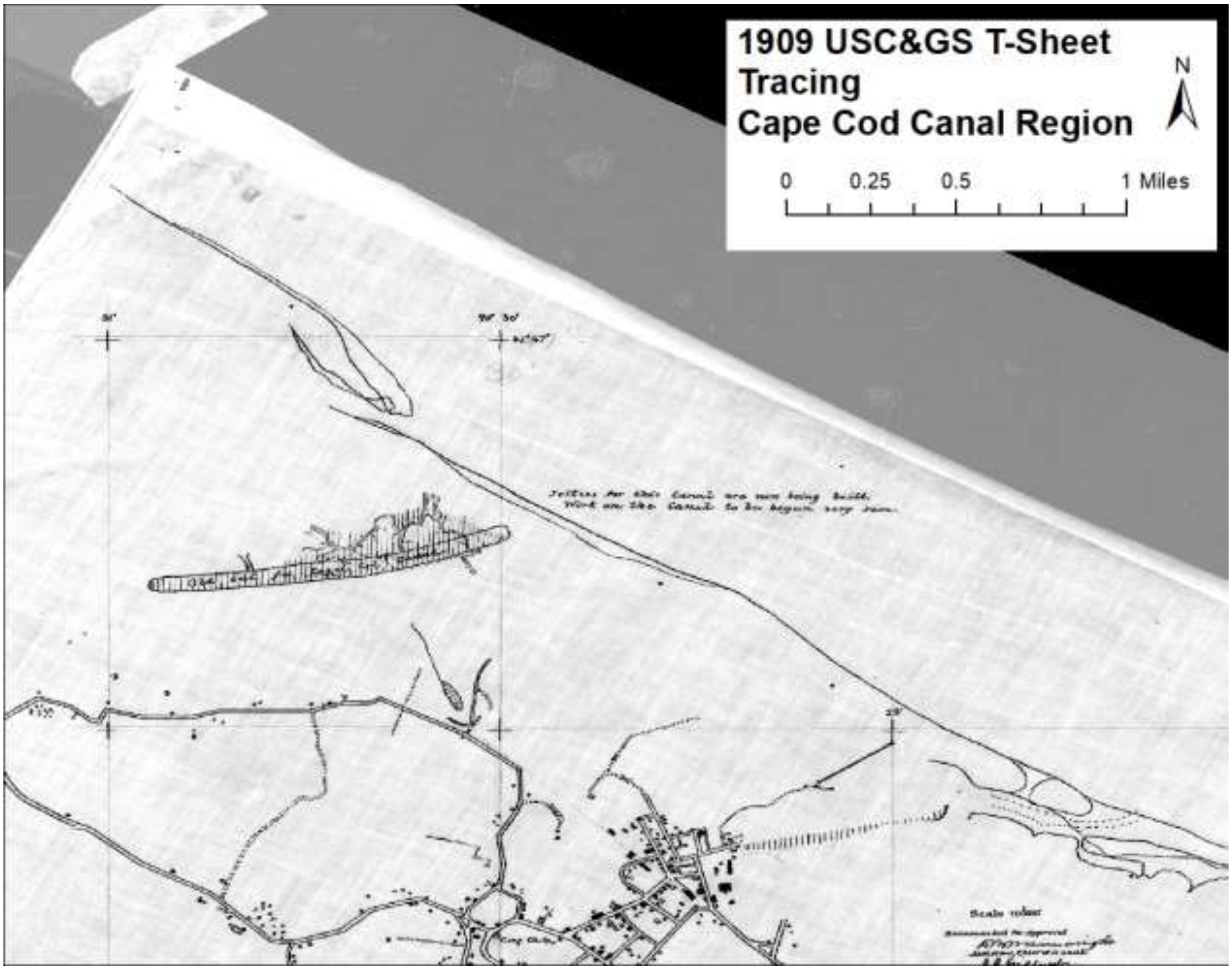


**Figure 15. Magnified area of U.S. Coast and Geodetic Survey 1909 T-sheet tracing focusing on the canal region. Shoreline positions for 1909 and 1860 are both shown on the tracing.**

To determine the change in shoreline position between 1860 and 1909, both the 1860 T-sheet and 1909 tracing were georeferenced using point positions that were included in the original T-sheet drawings. The georeferenced data shows that in the period between 1860 and 1909, the area of Town Neck Beach from south of the present-day northern Cape Cod Canal Jetty to Sandwich Harbor experienced an average accretion rate of approximately 0.97 feet/year (if analyzed from the shoreline downdrift of the eventual south Canal jetty) to 1.3 feet/year (if analyzed from the shoreline downdrift of the former Scusset Harbor). Figure 16 shows a transparent version of the georeferenced 1909 T-sheet tracing overlain on an aerial photograph of the region. From this figure it is evident that a large quantity of material has been impounded behind the northern canal jetty, as well as that the area south of the canal has experienced erosion since the construction of the jetties. Therefore, prior to the Canal being constructed, Sandwich Town Neck Beach was accretionary, at a rate of 0.97 to 1.3 feet/year, and has clearly been erosional after the construction of the Canal. Over the same area (Sandwich Town Neck Beach), the shoreline has retreated at a rate of approximately -1.36 feet/year between 1909 and

2009. As such, once the Cape Cod Canal and its associated jetties were constructed, the downdrift shoreline shifted from an accretionary shoreline to an erosional shoreline with a net rate of change was approximately 2.43 feet/year.

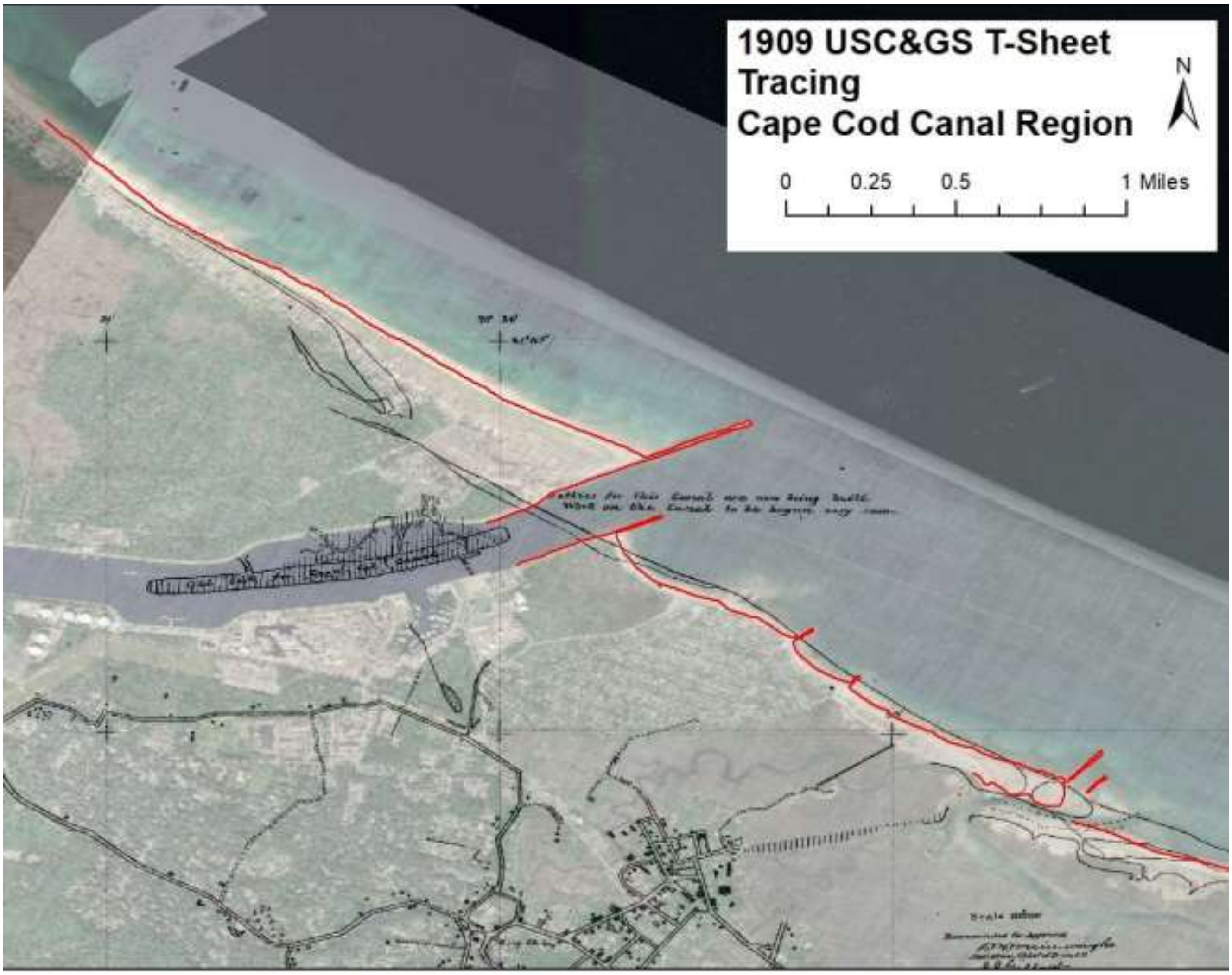


**Figure 16.** **Magnified area of U.S. Coast and Geodetic Survey 1909 T-sheet tracing overlain on an aerial image of the canal region. Red line shows 2009 shoreline position.**

## 3.0 SEDIMENT BUDGET

In order to further understand existing sediment transport patterns and the influence of the Cape Cod Canal on the adjacent shorelines, a sediment budget was developed for the current conditions at the Cape Cod Canal. The sediment budget was developed utilizing a variety of data sources including shoreline change analysis, historical records, analytical calculations, and numerical modeling results. A sediment budget provides a framework for understanding the complex patterns of sediment transport that occur at the engineered inlet at the eastern terminus of the Cape Cod Canal. The budget also provides a baseline to consider when evaluating potential engineering projects. A sediment budget represents an accounting of all sources and sinks of sediment within a specified series of connected cells, over a period of time. The USACE has produced a Coastal Engineering Technical Note (CETN IV-15) that provides guidance on the steps in producing a sediment budget (Rosati and Kraus, 1999). This technical note provides the basis for the development of the Cape Cod Canal sediment budget, and more information regarding the formulation of a sediment budget can be found in Rosati and Kraus (1999). In its simplest form, a sediment budget can be expressed by the equation:

$$\sum Q_{Source} - \sum Q_{Sink} - \Delta V + P - R = 0$$

Where $Q_{source}$ and $Q_{sink}$ represent sources and sinks out of the budget cell; $\Delta V$ is the change of volume within the cell; and P and R represent the amounts of sediment placed or removed from the cell. The cell budget is considered balanced when this equation is equal to zero. Figure 17, taken from CETN IV-15, shows a conceptual box model version of this equation with examples of the types of parameters considered.

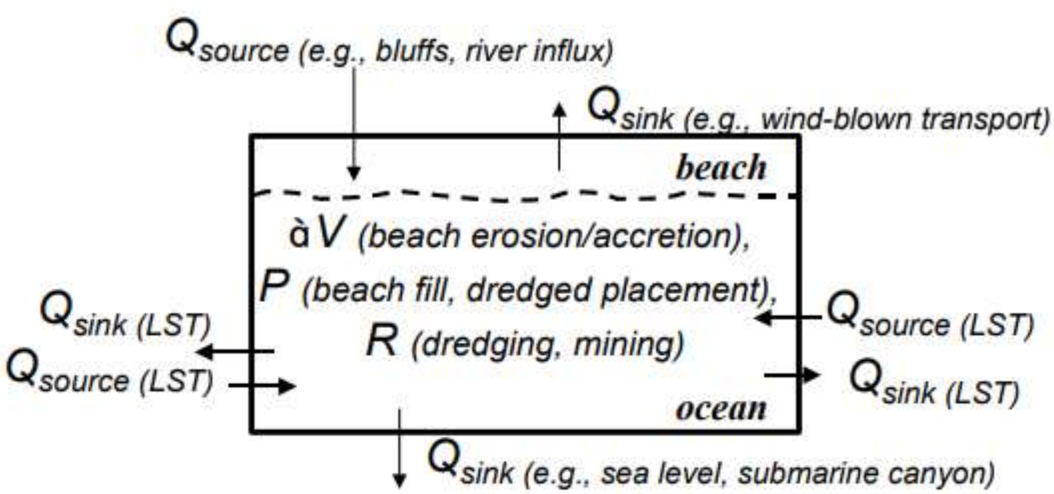


**Figure 17. Conceptual box model of sediment budget showing examples of parameters (from CETN IV-15).**

For developing the sediment budget for the Cape Cod Canal, three (3) sediment budget cells were established:

- Scusset Beach Cell - from the terminus of the cliffs north of Sagamore Beach (at approximately Starfish Lane, Bourne, MA) to the northern Cape Cod Canal Jetty on Scusset Beach
- Cape Cod Canal Cell – The area in-between and offshore of the Cape Cod Canal Jetties
- Town Neck Beach Cell – From the southeastern Cape Cod Canal Jetty to the terminus of the Town Neck Beach spit at approximately the old eastern Sandwich Harbor Jetty.

The components of the Cape Cod Canal sediment budget include: a) wave-induced alongshore transport into and out of each cell (**$Q_{LST}$**); b) offshore sediment volume losses due to a long-term increase in sea level (**$Q_{SLR}$**); c) nourishment placed on Town Neck Beach from 1975 to 2016 (**P**); d) Sediment transport from the surrounding beaches into the canal (**$Q_{canal}$**); e) material shoaling and dredged from the Cape Cod Canal (**R**); f) Volumetric changes on the up- and down-drift beaches ($\Delta V_{beach}$); g) Volumetric changes offshore of the canal ($\Delta V_{canal}$).

Alongshore sediment transport rates (**$Q_{LST}$**) were estimated using the process-based wave-induced sediment transport modeling described in Chapter 4. Transport rates in the sediment budget are presented as a likely range of transport rates into and out of the cells as calculated by the sediment transport model. Alongshore sediment transport rates were only calculated for Scusset Beach and Town Neck Beach cells to provide the requisite information for a sediment budget of the Cape Cod Canal. The alongshore transport rates determined for the cell boundaries are shown in Table 3.

**Table 3. Alongshore transport rates at sediment budget cell boundaries.**

| Transport Rate Location (see Figure 15) | Transport Rate (cy/year) |
|---|---|
| **$Q_{LST-SC}$** | 95,000 to 115,000 |
| **$Q_{LST-TN}$** | 35,000 to 45,000 |

Long-term offshore sediment losses due to sea level rise (**$Q_{SLR}$**) were estimated using the Bruun (1962) rule. The Bruun rule relates sediment losses due to sea level rise with the local closure depth and the distance to the depth of closure. The closure depth for the Cape Cod Canal region was defined using Wave Information Study (WIS) hindcast data calculated as part of CHETN-VI-45 (Brutsche, et al., 2016). CHETN-VI-45 calculated depth of closure utilizing the wave parameters simulated as part of the (WIS). For the purposes of this study, the depth of closure calculated using the full Birkemeier (1985) equation (using data from the closest WIS station to Sandwich) was utilized. Cross-shore profiles from the CoNED Topobathymetric Model (1887 - 2016): New England were used to calculate distance from the seaward most active berm to the depth of closure. Long-term sea level rise (not projected future sea level rise) was assumed to be 2.63 mm/year, consistent with the historic sea level rise trend at the Boston NOAA tide gauge. The estimated losses due to sea level rise for the sediment budget use the historic observations of sea level rise, rather than projections into the future, to match the other variables (taken historically) in the analysis. Total sediment volume losses due to sea level rise calculated using these parameters was found to be approximately 3,200 cy/year for the Town Neck Beach cell (**$Q_{SLR-TN}$**), and 5,700 for the Scusset Beach cell (**$Q_{SLR-SC}$**).

While the Town Neck Beach Cell has been nourished a number of times, there were three significant nourishments that occurred between 1990 to 2016. The volumes of these nourishments were quantified using historical records of nourishment from what was available. The volume of the individual Town Neck Beach nourishments is summarized in Table 4. For the contemporary time period, the average annual rate of nourishment using these individual nourishments was calculated as approximately 11,200 cy/year (**P**).

**Table 4. Nourishment of Town Neck Beach between 1990 and 2016.**

| Year | Volume (cy) |
|---|---|
| 1990 | 122,000 |
| 2004 | 50,000 to 65,000 |
| 2016 | 110,000 to 120,000 |

The volume of material dredged from the Cape Cod Canal was determined using historic dredge records from the United States Army Corps of Engineers. The CCC has shifting sand shoals that have historically formed in the eastern end of the Canal. The shoals present a navigation hazard to deep draft vessels and as such maintenance dredging has been performed numerous times in order to maintain the design channel. Over the past 30 plus years, the same areas within the channel have shoaled and have required dredging efforts. As such, for the purposes of this sediment budget, it is assumed that the volume of shoaling in the canal itself is approximately equal to the volume dredged from the Canal. The history of dredging in the Canal from 1975 to 2016 is summarized in Table 5. The average annual volume of material that shoals in the eastern end of the Canal was determined by assessing these dredge records. The average volume of material shoaling in the channel, as well as that removed from the channel, was calculated as approximately 28,100 cy/year.

**Table 5. Dredge history of the canal between 1975 and 2016.**

| Year | Volume (cy) |
|---|---|
| 1975 | 126,000 |
| 1977 | 73,000 |
| 1979 | 100,000 |
| 1986 | 177,000 |
| 1990 | 122,000 |
| 1998 – 2000 | 162,000 |
| 2002 | 117,000 |
| 2004 | 50,000 |
| Jan 2010 | 21,000 |
| March 2010 | 85,000 |
| 2016 | 120,000 |

Volumetric changes on Town Neck and Scusset Beaches were calculated based on shoreline change data. An updated shoreline change analysis for Town Neck beach was conducted as part of this study (Chapter 2). For Scusset Beach, shoreline change data was acquired from the Massachusetts Shoreline Change Project by the Massachusetts Office of Coastal Zone Management (MACZM) (Thieler et al., 2013).

Shoreline change rates for the Scusset Beach Cell were converted to a volumetric rate of change by multiplying the rate of change in cross-shore shoreline position over the given alongshore-shore length and assuming that the shoreline translates parallel to itself over a given active depth defined as the height from the depth of closure to the seaward most active berm. The equation for this calculation is:

$$\Delta V = \frac{\Delta Y \Delta X D_a}{\Delta t}$$

in which $\Delta Y$ is the change in cross-shore position, $\Delta X$ is the shoreline length, $D_a$ is the height of between the seaward most active berm and the depth of closure, and $\Delta t$ is the period of time being averaged over. This analysis results in a volumetric rate of change of +54,700 cy/year for the Scusset Beach cell ($\Delta \boldsymbol{V}_{SC}$).

For Town Neck Beach, the volume loss estimate calculated using cross-shore profiles described in Chapter 2 was utilized to estimate volumetric change rates. Utilizing the cross-shore profiles, a change rate of approximately -10,000 cy/year was calculated. However, the cross-shore analysis utilized for the rough estimate determined in Chapter 2 did not extend to the depth of closure, likely causing an underestimation of the volumetric change. In addition, the shoreline change analysis conducted as part of this study included four shoreline position datasets from 2000-2018, and only one from prior to 2000 (1952). This value must be considered with considerable uncertainty, as the conditions at Town Neck Beach have likely changed drastically from the historical period (prior to 2000), as well as there being significant uncertainty associated the 1952 shoreline position dataset. As such, an additional data set was considered in evaluating the volumetric rate of change on Town Neck Beach to verify the volume change in the Town Neck Beach cell. The Provincetown Center for Coastal Studies prepared a report (1980) summarizing an applied science study carried out for the Towns of Sandwich and Barnstable. As part of that study, a shoreline position change analysis was conducted. That study, which assessed shoreline positions in 1957 and 1972 found a volumetric rate of change of approximately -67,000 cubic yards per year out to a depth of -18 feet Mean Low Water. This value is significantly different than that calculated using the shoreline change analysis presented in Chapter 2, most likely due to the uncertainty with the 1952 information and the limited shoreline profile information (only going seaward to a depth of -5 feet NAVD88). Due to these uncertainties it was decided to average the two rates of change, and as such the volumetric rate of change for Town Neck Beach for the purposes of the sediment budget is 38,500 cy/year ($\Delta \boldsymbol{V}_{TN}$).

Additionally, Borelli et al. (2016), as part of a study evaluating sediment transport for the Sandwich and Barnstable coasts, noted that it appeared likely that sediment moving around the Cape Cod Canal jetties was likely "being deposited beyond the extent of lidar (~10 m water depth) and the commonly accepted wave base for a moderately energetic shoreline)." To assess this premonition, Borrelli et al., (2016) utilized a USC&GS 1933 T-sheet as well as collecting 2016 altimeter data to quantify the sediment movement offshore of the Canal. Figure 18 (from Borelli et al., 2016) shows the change calculated between 1933 and 2016 just offshore of the eastern end of the CCC. From the figure, it is clear that there is a significant scour hole directly offshore of the canal, with an ebb shoal surrounding the scour hole, and with some amount of material being transported even farther offshore. Borelli noted that this material that is transported far offshore is not available for movement back onshore through normal waves. To quantify the volumetric change in the offshore inlet cell, the Borelli figure was digitized and the volume contained in the shoals, as well as the volume removed from the scour hole was estimated. Approximately 2.5 million cubic yards was estimated to be contained in the offshore shoals, while 1.7 million cubic yards was estimated to have been removed from the scour hole. Averaged over an 83-year period (1933 – 2016) this equated to 30,400 cy/year being deposited offshore in the shoals, and -20,600 cy/year being removed from the scour hole. Summing these volumes together equates to a volumetric rate of change of approximately +9,800 cy/year in the area just offshore of the eastern end of the Canal ($\Delta \boldsymbol{V}_{\mathbf{canal}}$).

With the individual components of the sediment budget resolved (excluding $\mathbf{Q}_{\mathbf{canal}}$) the total budget for each of the three cells could be resolved according to the following equations:

Scusset Beach Cell

$$Q_{LST-SC} - Q_{Canal1} - Q_{SLR-SC} - \Delta V_{sc} = 0$$

Cape Cod Canal Cell

$$Q_{Canal1} + Q_{Canal2} - R - \Delta V_{canal} = 0$$

Town Neck Beach Cell

$$\sum Q_{Source} - Q_{LST-TN1} - Q_{Canal2} - Q_{SLR-TN} - \Delta V_{TN} + P = 0$$

These equations need to be solved such that the net sediment sums to zero. As such, the two unknowns, $Q_{Canal1}$ and $Q_{Canal2}$, need to be determined such that the sediment budget is balanced. Solving the Scusset Beach cell equation for $Q_{Canal1}$ yields a range of 34,600 to 54,600 cy/year, which corresponds to the amount of sediment entering the CCC cell from the Scusset Beach side.

Based on observations and Borelli et al. (2016), sediment being transported offshore from the canal is not available for transport, Therefore, for the Town Neck Beach Cell it is assumed that $\sum Q_{Source}$ the net transport coming into the cell from the canal sediment is equal to 0. Therefore, solving the Town Neck Beach Cell for $Q_{Canal2}$ yields a range of 1,500 to 11,500 cy/year entering the CCC cell from the Town Neck Beach side.

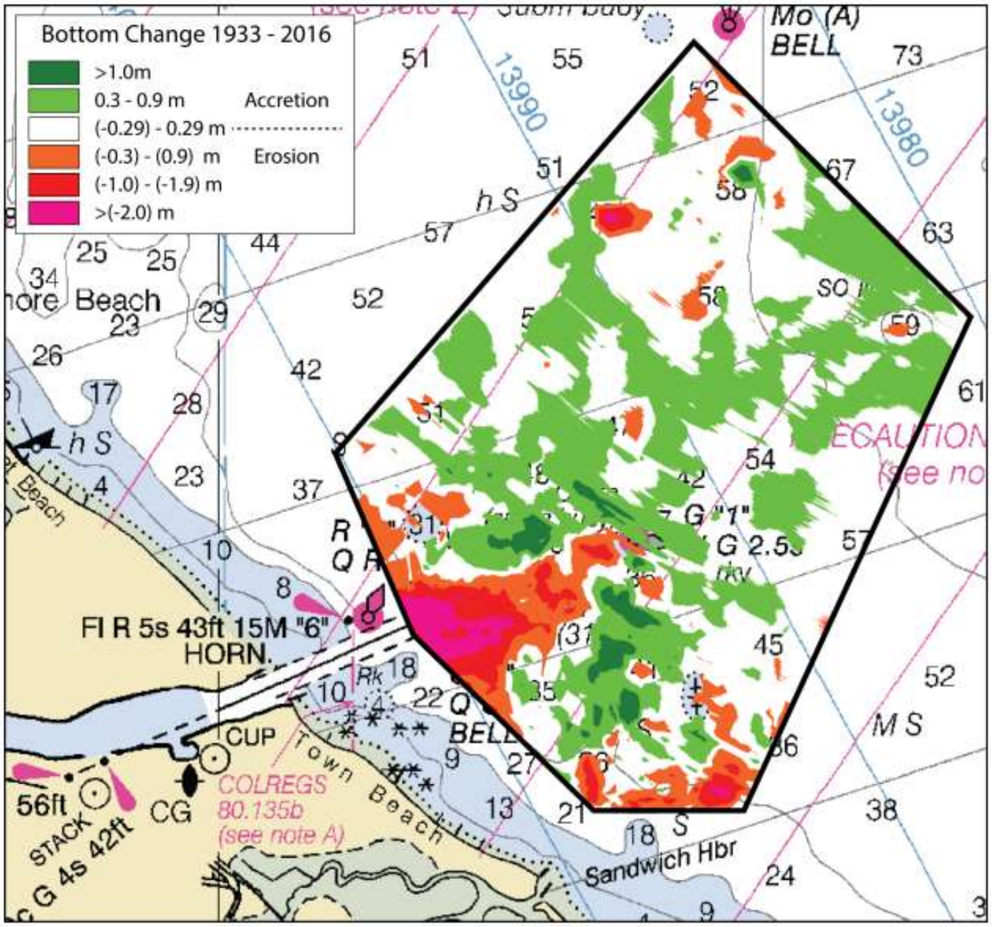


**Figure 18. Change along the seafloor offshore of the Cape Cod Canal from 1933 to 2016. Change is expressed in meters, while the underlying nautical chart shows depths in feet. Figure taken from Borrelli et al., 2016.**

Then applying the values of $Q_{Canal1}$ and $Q_{Canal2}$ in the Cape Cod Canal cell, the range of these variables is narrowed to:

$$Q_{Canal1} = 34{,}600 \text{ to } 36{,}400 \text{ cy/year}$$
$$Q_{Canal2} = 1{,}500 \text{ to } 3{,}300 \text{ cy/year}$$

The 1,500 cy/year to 3,300 cy/year calculated for $Q_{Canal2}$ is consistent with the wave-induced alongshore sediment transport calculated for the reversal area for Town Neck Beach (10,000 to 20,000 cy/year from the sediment transport modeling presented in Chapter 4), since a relatively significant portion of the sediment is expected to be impounded behind the existing southern jetty of the Cape Cod Canal.

Iterating these values for $Q_{Canal1}$ and $Q_{Canal2}$ back into the equations for the Scusset Beach and Town Neck Beach cells reduces the range of the alongshore sediment transport values to:

$$Q_{LST-SC} = 95{,}000 \text{ to } 96{,}000 \text{ cy/year}$$

$$Q_{LST-TN} = 43{,}200 \text{ to } 45{,}000 \text{ cy/year}$$

The specific processes that are transporting the sediment into the canal are not as well defined. For example, sediment could be entering the CCC a number of potential ways, including being transported over each of the jetties via either windblown transport or overwash, through the jetties themselves, or around the end of the jetties. The specific sources of sediment shoaling within the CCC are also undefined. In addition to the sediment arriving into the CCC from the adjacent beaches, other sediment sources may contribute to the overall shoaling in the CCC. This includes sediment arriving/moving via canal scour or channel adjustments. Sediment sourced from channel failure or scour within the canal itself have not been accounted for here. Additional uncertainties arise from the uncertain value of the volume of erosion on Town Neck Beach. This value may vary from historic values due to anthropogenic alterations of the beach, as well as due to a lack of sediment availability for erosion. However, the sediment budget determined as part of this study is a good overall, quantitative representation of the sediment movement and volumes at the eastern end of the Cape Cod Canal. Figure 19 shows a graphical representation of this sediment budget as determined for this study.

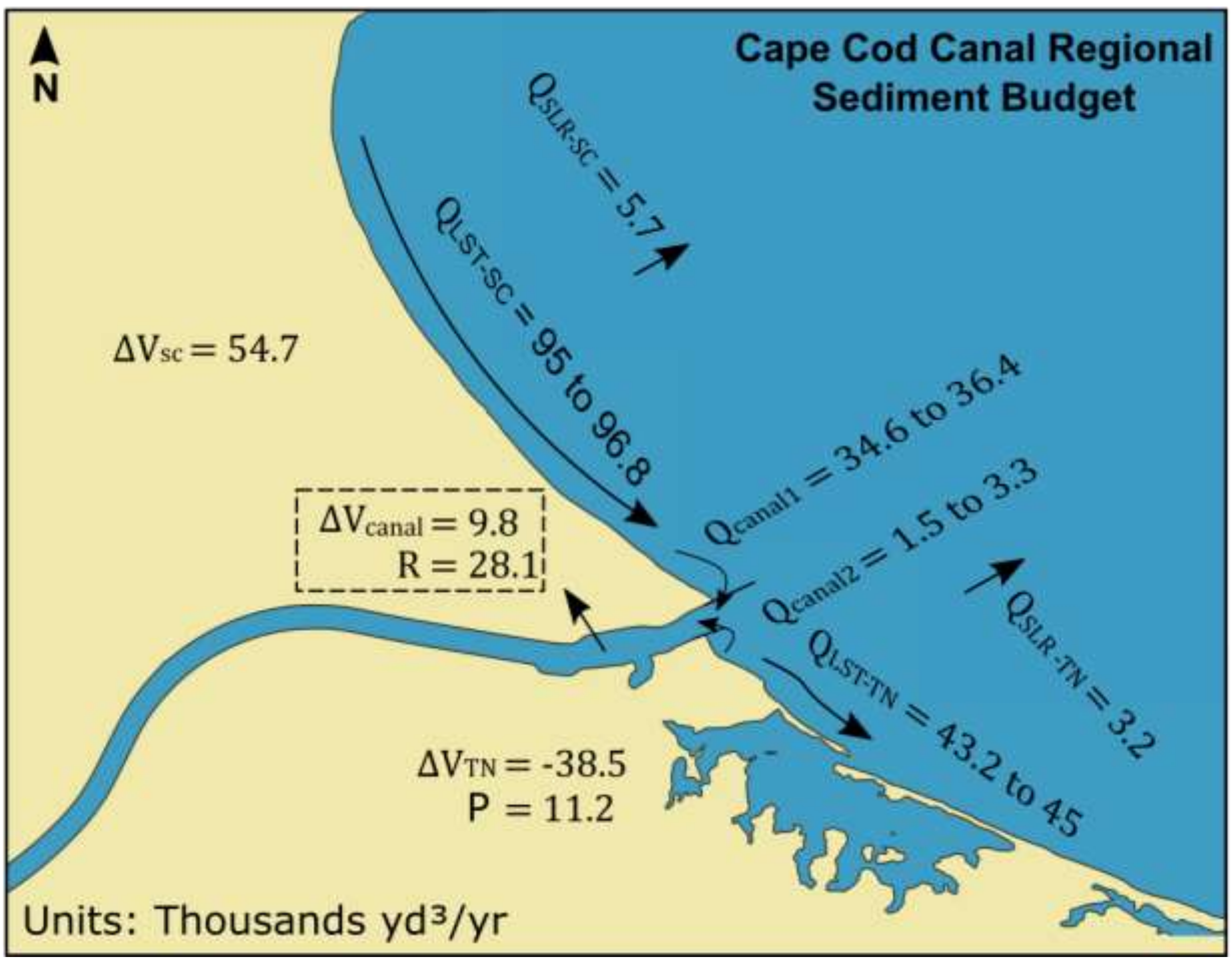


**Figure 19. Sediment budget for the Cape Cod Canal.**

# 4.0 EXISTING CONDITIONS

## 4.1 SEDIMENT SOURCE AND CHARACTERISTICS

The characterization of natural sediments, as well as the source of these sediments at sites surrounding the CCC is an important step in evaluating littoral processes and the movement of sediments along the shoreline. In addition, knowledge of the grain size of the beach sediments help to define the design grain size for any shore protection alternative involving beach nourishment.

Characteristics of sediments found in the Canal region are a result of two processes, their geological source and the active processes occurring near the CCC (e.g. winds, waves, and tidal currents). Sediment has been supplied to Scusset Beach, and historically to Town Neck Beach (the two beaches surrounding the Canal), from the glacial cliffs located to the north in Plymouth (Fitzgerald, 1993). These cliffs are made up of sand rich glacial outwash deposits and therefore represent an abundant source of sediment. Relative sea level has been rising since the last glacial maximum, which has eroded these cliffs and provided a steady source of sediment to beaches via alongshore transport (Fitzgerald et al., 1994). As determined later in this chapter, the prevailing alongshore transport is directed south from Plymouth, turning to a more southeasterly direction towards Sandwich. This pattern is driven primarily by the strong northeast winds and waves that dominate in the region.

The portion of the Plymouth shoreline that likely supplies the majority of sediment is located south of the Ellisville recessional moraine, near the Plymouth/Barnstable county line. This source area represents 2.5 km of shoreline dominated by 40 to 50-meter-high cliffs made up of sand-rich glacial outwash deposits. A review of georeferenced aerial photographs identified approximately 350 meters of shoreline armored with stone rip-rap, and another 250 meters protected by coir along these cliffs. Collectively, the armoring covers approximately 24% of the source area shoreline. South of these cliffs there are a series of 10 small groins located on Sagamore Beach. As these groins are almost completely buried, it is thought that these structures are not a significant trapping mechanism for sediment. This data indicates that there is abundant sediment available for alongshore transport towards Scusset Beach, sourced from the glacial cliffs south of Ellisville.

Sediment samples at Town Neck and Scusset Beaches were collected by Woods Hole Group in 2016 to physically characterize the sediments on the beaches surrounding the CCC. Samples were obtained as surface grabs using a stainless-steel shovel. Grab samples were collected during low tide on March 16 - 17, 2016 at Scusset Beach and Town Neck Beach, Sandwich. Twelve (12) samples were collected at each beach (Scusset and Town Neck, 24 samples total) along six shore normal transects. Each transect consisted of two samples: one sample collected on the beach above MHW and one collected on the intertidal beach. Figure 20 shows the locations of the samples taken and analyzed. Sediment samples were sent to GeoTesting Express Laboratories for grain size analysis using ASTM method D422. Results from the Scusset Beach samples are summarized in Table 6. The results characterize Scusset Beach with a homogenous matrix of medium-coarse grained sand, with a D50 of 0.61 mm. Results from the Town Neck Beach samples are summarized in Table 7. The results characterize Town Neck Beach samples ("TB" prefix) with a homogenous matrix of medium-coarse grained sand with some gravel, and a D50 of 0.86 mm. The D50 of Town Neck Beach samples decreases to 0.60 mm with the removal of sample TB1. This sample contained an anomalously high gravel content that skewed the calculation. These results are consistent with a single glacial source of sediment supplying both Scusset and Town Neck Beaches.


**Figure 20. Sediment Samples Collected on March 16th – 17th, 2016 at Town Neck and Scusset Beaches.**

## 4.2 WAVE TRANSFORMATION

### 4.2.1 Wave Climatology

The impact of waves on nearshore coastal processes and shoreline change is highly dependent on the offshore wave climate and the transformation of waves propagating to the shoreline. Subsequently, as the waves interact with the coastline, the wave-induced currents are a major component of sediment transport and shoreline change. Therefore, a key component of understanding the impacts of the CCC, as well as potential alternative designs is determining the nature of the wave field both offshore and in the nearshore region.

To quantify the potential impacts of waves on the nearshore and offshore areas surrounding the CCC site specific wave conditions were determined using available wind data, wave data, and a series of numerical and analytical models. A description of the procedures used to evaluate wave conditions within this portion of Cape Cod Bay is presented in this section. Wave characteristics were developed for average annual conditions, as well as for extreme storm events. These wave conditions were utilized to assess existing conditions and to aid in evaluation of impacts for the different potential design alternatives.

**Table 6. Grain Size Results from Scusset Beach Samples Collected on March 16, 2016.**

| Sample ID | Latitude Actual | Longitude Actual | Gravel % | Sand % | Silt&Clay % | $D_{50}$ (mm) | ASTM Classification |
|---|---|---|---|---|---|---|---|
| SB1 | N41 46.710 | W70 29.678 | 0 | 100 | 0 | 0.65 | Poorly graded sand |
| SB2 | N41 46.818 | W70 29.922 | 0 | 99.9 | 0.1 | 0.61 | Poorly graded sand |
| SB3 | N41 46.844 | W70 29.986 | 0 | 100 | 0 | 0.61 | Poorly graded sand |
| SB4 | N41 46.954 | W70 30.184 | 0 | 99.9 | 0.1 | 0.56 | Poorly graded sand |
| SB5 | N41 47.087 | W70 30.419 | 0 | 100 | 0 | 0.60 | Poorly graded sand |
| SB6 | N41 47.287 | W70 30.821 | 0 | 99.8 | 0.2 | 0.53 | Poorly graded sand |
| SI1 | N41 46.743 | W70 29.650 | 0.6 | 98.6 | 0.8 | 0.54 | Poorly graded sand |
| SI2 | N41 46.848 | W70 29.897 | 0.4 | 99 | 0.6 | 0.54 | Poorly graded sand |
| SI3 | N41 46.875 | W70 29.958 | 0 | 99.3 | 0.7 | 0.41 | Poorly graded sand |
| SI4 | N41 46.985 | W70 30.156 | 5.3 | 94.2 | 0.5 | 0.65 | Poorly graded sand |
| SI5 | N41 47.105 | W70 30.402 | 29 | 70.6 | 0.4 | 0.99 | Poorly graded sand with gravel |
| SI6 | N41 47.319 | W70 30.787 | 3.1 | 96.3 | 0.6 | 0.66 | Poorly graded sand |

**Table 7. Grain Size Results from Town Neck Beach Samples Collected on March 17, 2016.**

| Sample ID | Latitude Actual | Longitude Actual | Gravel % | Sand % | Silt&Clay % | $D_{50}$ (mm) | ASTM Classification |
|---|---|---|---|---|---|---|---|
| TB1 | N41 46.346 | W70 29.491 | 44.2 | 55.7 | 0.1 | 2.15 | Poorly graded sand with gravel |
| TB2 | N41 46.164 | W70 29.220 | 1 | 98.2 | 0.8 | 0.55 | Poorly graded sand |
| TB3 | N41 46.028 | W70 28.987 | 1 | 97.9 | 1.1 | 0.61 | Poorly graded sand |
| TB4 | N41 45.920 | W70 28.768 | 0 | 100 | 0 | 0.61 | Poorly graded sand |
| TB5 | N41 46.187 | W70 29.240 | 0.4 | 98.9 | 0.7 | 0.51 | Poorly graded sand |
| TB6 | N41 46.274 | W70 29.321 | 0.2 | 99.7 | 0.1 | 0.71 | Poorly graded sand |
| TI1 | N41 46.360 | W70 29.479 | 70.9 | 29.1 | 0 | 8.14 | Well-graded gravel with sand |
| TI2 | N41 46.175 | W70 29.193 | 1.1 | 98.2 | 0.7 | 0.35 | Poorly graded sand |
| TI3 | N41 46.044 | W70 28.965 | 16.7 | 82.9 | 0.4 | 1.10 | Poorly graded sand with gravel |
| TI4 | N41 45.942 | W70 28.748 | 25.2 | 74.4 | 0.4 | 1.44 | Poorly graded sand with gravel |
| TI5 | N41 46.196 | W70 29.210 | 0.2 | 98.9 | 0.9 | 0.35 | Poorly graded sand |
| TI6 | N41 46.289 | W70 29.304 | 44.5 | 55.5 | 0 | 4.19 | Poorly graded sand with gravel |

#### *4.2.1.1 Wave Data Analysis and Sources*

The wave climate at the CCC was assessed by considering locally generated wind waves, regional swell waves, and high energy storm waves. The area surrounding the CCC is influenced both by locally generated seas, produced within Cape Cod Bay, as well as swell waves generated in the Atlantic Ocean. This combination of wave sources produces a range of wave conditions at the shoreline that includes both high frequency seas and longer period waves. Figure 17 illustrates the distribution of wave types and approaches influencing the beaches surrounding the CCC. A sizable portion of Cape Cod Bay is sheltered from the Atlantic Ocean by the outer Cape (indicated by the blue region) and waves from this direction are therefore produced by local winds. However, the gap that exists between Rocky Point in Plymouth and Race Point in Provincetown (indicated by the yellow region) provides a potential entryway for long period wave energy from the Atlantic Ocean, as well as sea conditions due to both regional and local winds. Additionally, a small approach angle (indicated by the red region) consists of both locally generated wind waves and swell waves from the northeast that are able to enter Cape Cod Bay by wave refraction and diffraction transformations.

For this project, the USACE Wave Information Study (WIS) time series of wave and wind data were utilized to describe the wave climate offshore of the Cape Cod Bay side of the Cape Cod Canal entrance. The WIS, developed by the USACE, has met a critical need for wave information utilized in coastal engineering studies since the 1980s and is widely accepted for design purposes for United States shorelines by many coastal engineers and scientists. The WIS contains time series information of spectrally based, significant wave height, peak period, peak direction, wind speed and direction produced from a computer hindcast (prediction) model. The hindcast wave model, WISWAVE (Resio and Tracy, 1983) is simulated using wind information (speed and direction) at selected coastal locations around the United States. The model predicts wave climate based on local/regional wind conditions. Wave measurements made by NOAA during the 1980s made verification of the WIS results possible by comparing the statistics and the distributions of wave heights and periods from different time periods (Hubertz et al., 1993)

The WIS station (station 93) located offshore of Plymouth at the entrance to Cape Cod Bay (Figure 21) provides a 20-year time series of simulated wave hindcast data that was used to quantify the swell and regional sea conditions entering Cape Cod Bay. The sea and swell approach region in Figure 21 (yellow) were defined by the wave information from the WIS station, while the local sea region (blue) used wind information from the WIS station, and the local sea and swell region (red) used a combination of both the wave and wind information.

This WIS station (station 93) represents data from 1976 to 1995. While there is more recent WIS data sets available (1980-2014); however, data from station 93 was utilized for the wave transformation assessment for a number of different reasons, including:

1. The more recent data sets do not have the same spatial coverage as the previous WIS data sets. WIS station 93 (Figure 21) was located much closer to the Sandwich shoreline, and better represented ocean based swell conditions for Sandwich since it was located within Cape Cod Bay (and included the influence of the outer Cape). The more recent

data are located well outside of Cape Cod Bay and would require a much more complicated wave transformation model to determine the transformations that propagate towards Sandwich.

2. These data from WIS station 93 had been used extensively and successfully in prior wave modeling assessments for the Town of Sandwich shorelines (Woods Hole Group, 2004). This includes validation with sediment transport rates and involved local transformation to the Sandwich area. Due to all the previous studies completed using these data, it was logical to utilize these proven data sources.

3. These data from WIS station 93 had already been combined with locally wind generated waves that are formed in Cape Cod Bay (Woods Hole Group, 2004). This combined wave data set was successfully used in these previous studies and includes a combination of the various wave spectra from both ocean swell waves and locally generated wind waves. These data sets also include storm conditions, which are an important part of this overall assessment.

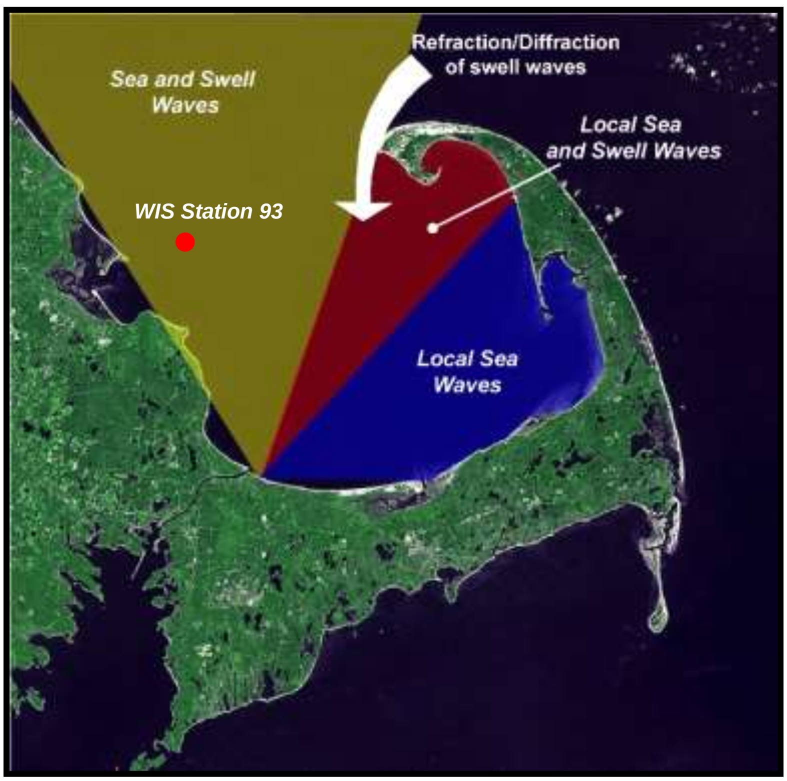


**Figure 21. Wave types and approach directions for the Town of Sandwich shoreline and WIS Station location used for wave and wind hindcast data.**

#### *4.2.1.2 Locally Generated Waves*

Locally generated waves are formed as a function of wind speed, wind duration, water depth, and fetch. Larger, longer waves are generated by sustained winds that blow toward the site across longer stretches of the Bay (north and northeast). Local, historic wind information from the WIS station was analyzed to determine the magnitude and direction of wind-generated waves in the area offshore of the CCC. Figure 22 shows a wind rose generated from the 20-year WIS time series. The gray-scale sidebar indicates the magnitude of wind speed, the circular axis represents the direction of wind 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 direction of wind approach is from the west.

Winds blowing offshore of the CCC region were excluded from the wind rose data since these conditions do not influence nearshore sediment transport in the area. Given the orientation of the Canal and surrounding shorelines, only winds from 295 degrees (west-northwest) clockwise to 115 degrees (east-southeast) were determined to affect the site. As such, locally generated wind waves were described by the data from between 25 degrees to 115 degrees (blue and red regions on Figure 21, while ocean generated waves were described by data from 295 degrees to 25 degrees (the yellow region). Table 8 provides the average wind speeds that were observed from each approach direction over the simulated 20-year time series from the WIS.

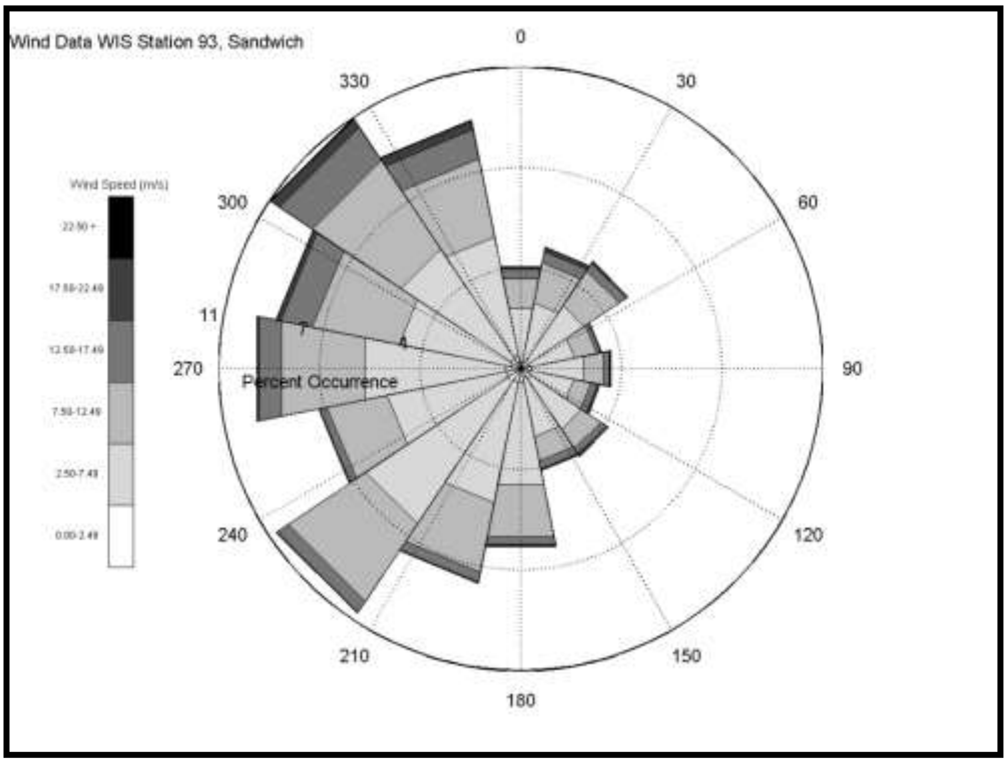


**Figure 22. Wind rose generated from WIS Station 93 data for the period Jan. 1, 1976 through Dec. 31, 1995.**

The average winds shown in Table 8 were used as input conditions for a wave-generation computer model developed by the USACE as part of the Automated Coastal Engineering System (ACES) to simulate the generation of local wave conditions caused by winds. For the ACES modeling Cape Cod Bay was divided into four (4) angle bands, each 22.5 degrees wide. Figure 23 shows the geometry of these bands and illustrates the semi-sheltered nature of the site. ACES was used to calculate a wave height, direction, and period for each angle band. The modeling considered the semi-restricted geometry, fetch length over water, and variations in wind speed in simulating resulting wave conditions. The resulting locally generated wave conditions predicted by the ACES model for each angle band are summarized in Table 9.

**Table 8. Average wind speeds from WIS Station 93 within each approach bin for the period Jan. 1, 1976 through Dec. 31, 1995.**

| Directional Bin Range (coming from, 0º = N) | Average Wind Speed (mph) |
|---|---|
| 25E – 47.5E* | 15.7 |
| 47.5E – 70E | 15.5 |
| 70E – 92.5E | 14.8 |
| 92.5E – 115E | 15.5 |

* This directional approach bin is combined with the regional swell waves.


**Figure 23. Angle bands describing the semi-restricted geometry of Cape Cod Bay affecting locally generated waves near the Cape Cod Canal.**

**Table 9. Locally generated wave conditions from ACES predictions in Cape Cod Bay given average wind conditions.**

| Wind Direction Band (coming from, 0°=N) | Average Water Depth (ft) | Sig. Wave Height (ft) | Peak Wave Period (sec) | Wave Direction (coming from, 0°=N) |
|---|---|---|---|---|
| 36.25E | 100.5 | 2.37 | 3.5 | 43E |
| 58.75E | 71.9 | 2.35 | 3.5 | 61E |
| 81.25E | 41.6 | 2.15 | 3.3 | 76E |
| 103.75E | 24.8 | 1.67 | 2.8 | 82E |

#### *4.2.1.3 Regional Swell Waves*

Although a portion of Cape Cod Bay is sheltered from the Atlantic Ocean, the northern opening provides a potential entryway for long-period wave energy from the Atlantic Ocean. The frequency and magnitude of wave energy advancing through this entry point has the potential to result in significant longer period waves at the area surrounding the CCC. The energy associated with these waves was determined using the 20 years of simulated wave hindcast data from the WIS station 93, located in a water depth of approximately 59 ft (Figure 17). Figure 24 shows a wave rose of the significant wave heights from the WIS study for station 93. Only waves that entered Cape Cod Bay thorough the northern opening (yellow and red region on Figure 21) were considered significant for processes affecting the CCC region. The data from the WIS station show both that the largest waves with heights greater than 6.5 ft (2 m) occur from the north through north-northeast approach directions, as well as that the most frequently occurring waves approach from the northeast.

Changes in wave conditions between the WIS station and the area offshore of the Canal were assessed using the wave transformation model WAVETRAN. Waves were transformed from a water depth of 59 ft at the WIS station to a water depth of 26 ft offshore of the Canal. Statistics of the transformed waves for the approach directions through the Cape Cod Bay opening are presented in Table 10. The largest transformed swell waves generated in the Atlantic Ocean that affect the Canal region enter from the northeast bin (25E to 47.5E).

**Table 10. Transformed wave heights, periods, and directions for waves entering Cape Cod Bay from the northern opening and approaching the Cape Cod Canal.**

| Wave Dir. Band (coming from, 0°=N) | Sig. Wave Height (ft) | Peak Wave Period (sec) | Wave Direction (coming from, 0°=N) |
|---|---|---|---|
| 295E – 317.5E | 2.40 | 4.0 | 306E |
| 317.5E – 340E | 2.82 | 4.2 | 329E |
| 340E – 2.5E | 3.08 | 4.9 | 351E |
| 2.5E – 25E | 3.61 | 5.4 | 10E |
| 25E – 47.5E* | 2.23 | 11.0 | 25E |

* This directional approach bin is combined with the locally generated wind waves.

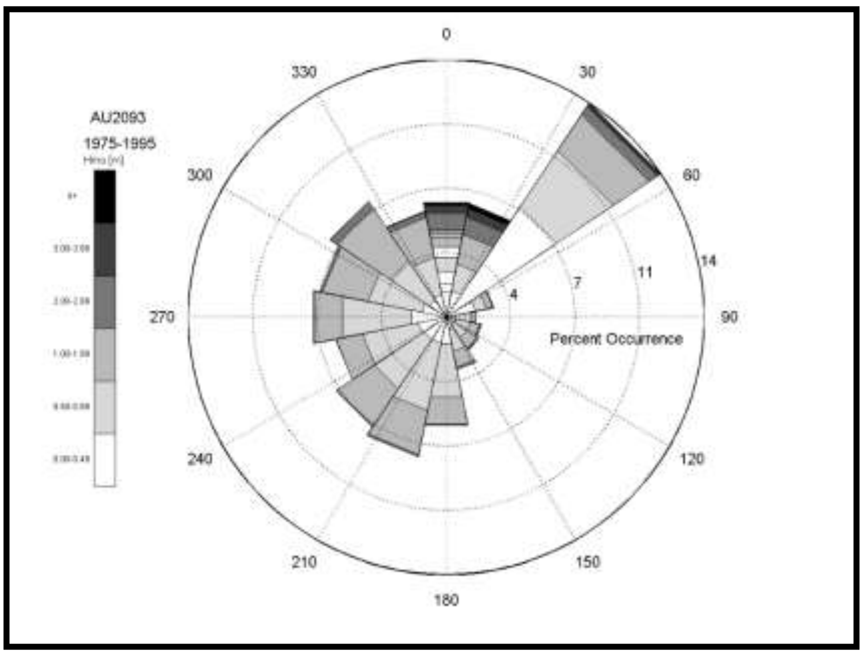


**Figure 24. Wave rose generated from WIS Station 93 data for the period Jan. 1, 1976 through Dec. 31, 1995.**

#### *4.2.1.4 Storm Waves*

In addition to the average conditions consisting of both local wind-generated and regional swell waves, a major component of the wave climate near the CCC consists of storm waves. The primary storm events that impact the region are extra tropical nor'easters. Nor'easters, which are large-scale, low pressure disturbances, often move slowly and are frequently of significant intensity, although wind speeds are generally less than those associated with a hurricane.

Historical quantified observations of extreme waves and storm surges can usually be achieved for a limited period of time (i.e. 20 years of wave information at the WIS station), most often less than the specified design life for engineered projects. In order to estimate design conditions for a specified project lifetime, a probability distribution must be derived from the available data. A return period or recurrence interval can then be estimated from the probability analysis. Estimates of wave heights associated with certain return period storm events were determined for the CCC region using this type of extremal analysis.

The return period can be thought of as the average period of waiting between events exceeding some specified value. For instance, a 25-year return value of 16 feet means that for any given year, there is a 1/25 chance that a wave height of 16 feet will be reached or exceeded. Table 11 presents the results of the extremal analysis performed on the transformed wave information. The longer-period storm waves are much larger than any potential wind generated wave in Cape Cod Bay, and these regional swell waves are more representative of larger scale storms produced in the Atlantic Ocean caused during nor'easters or hurricane events.

**Table 11. Extremal results for wave information from January 1, 1976 through December 31, 1995 at transformed location offshore of Sandwich.**

| Return Period (Years) | Wave Height (ft) |
|---|---|
| 1 | 15.42 |
| 10 | 15.75 |
| 25 | 16.14 |
| 50 | 16.40 |
| 100 | 16.70 |

### 4.2.2 Nearshore Wave Modeling

In order to evaluate nearshore wave transformation for the area surrounding the Cape Cod Canal, the CMS-Wave model was utilized (Lin et al., 2008). Nearshore wave modeling was conducted to estimate the existing effects of refraction, diffraction, shoaling, and breaking of waves in the project area. CMS-Wave (formerly known as WABED – Wave-Action Balance Equation Diffraction) is a two-dimensional (2-D) spectral wave transformation model available as part of the Coastal Modeling System (CMS) developed by the Coastal Inlets Research Program of the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL) in collaboration with two universities in Japan.

The model was formulated from a parabolic approximation equation (Mase et al., 2005) with energy dissipation and diffraction terms included. The model can simulate wave refraction and shoaling induced by changes in bathymetry, as well as wave interactions with currents. The model is regularly used and is widely accepted in coastal design studies. Many validations studies have been conducted showing its applicability for simulating the propagation of random waves over complex bathymetry and near inlets and structures where wave refraction, diffraction, reflection, shoaling and breaking are simultaneously occurring. Further information about CMS-Wave including validation cases and model theory can be found in the ERDC/CHL Technical Report-08-13 (Lin et al., 2008).

The CMS-Wave model is publicly available, and a graphical user interface (Surface Modeling System – SMS) for setting up the model is available from Aquaveo (ww.aquaveo.com). CMS-wave Version 3.2 (dated November 2015) distributed with SMS version 12.1 was utilized for this study.

#### *4.2.2.1 Wave Model Input Conditions*

To adequately assess wave propagation from the offshore observation locations to the nearshore region surrounding the CCC, a variable cell size grid was used within CMS-Wave to provide for an adjustable level of resolution (more detail in the study region). In CMS, a grid consists of a mesh of points with dimensions NI and NJ. At each point within the grid domain, bottom elevation can be specified. Reference points are separated by spacing DX (x-direction) and DY (y-direction). Grid resolution was varied from 25-meters$^2$ at the boundaries of the grid, to 10-meters$^2$ in the area of interest. The offshore boundary of the offshore grid was chosen at the location where the offshore wave data was transformed to (utilizing WAVETRAN), at a water depth deep enough that waves would not be significantly affected by ocean bottom friction. The orientation of the grids was selected to closely represent a shore-parallel contour line. CMS-Wave is a half-plane model meaning that waves can only propagate from the offshore boundary towards the shoreline.

Bathymetry and topographic data sets were compiled from existing available data sources for defining the grid. multiple bathymetry and topography data sources were utilized. Bathymetric data previously collected by Woods Hole Group in 2014 provided high resolution coverage in the nearshore areas surrounding Scusset and Town Neck beaches. To supplement these data and provide full bathymetric coverage for the local grid the 2013 USACE New England District topobathy LiDAR was obtained along with NOAA's NOS H11695 hydrographic survey collected in 2007. The sole topographic data source used was the 2013-2014 USGS Post-Sandy LiDAR. All data were converted to the NAVD88 vertical datum and then merged to create a seamless topobathy surface. Elevations were interpolated to the CMS-Wave grid. Figure 25 shows color contours of the bathymetric and topographic elevations for the local wave grid. Figure 26 shows a close-up view of the wave grid where the cells are shown.

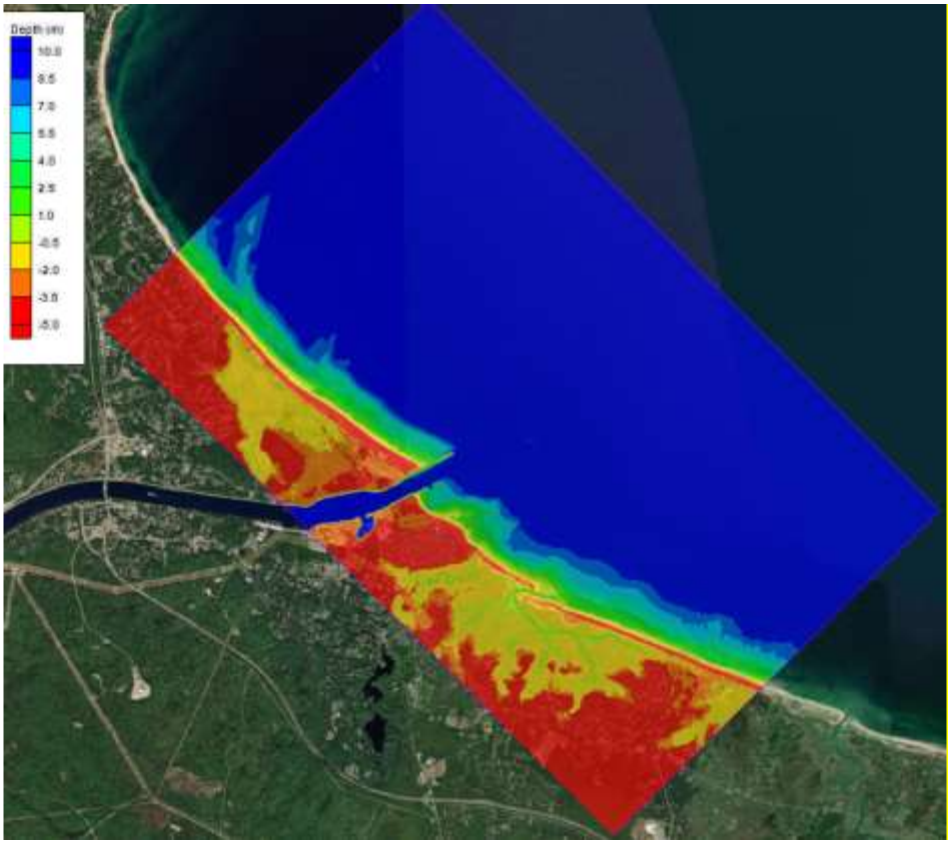

**Figure 25.** **CMS-Wave model grid showing topography/bathymetry for existing conditions. Depths are shown relative to NAVD88 (meters).**

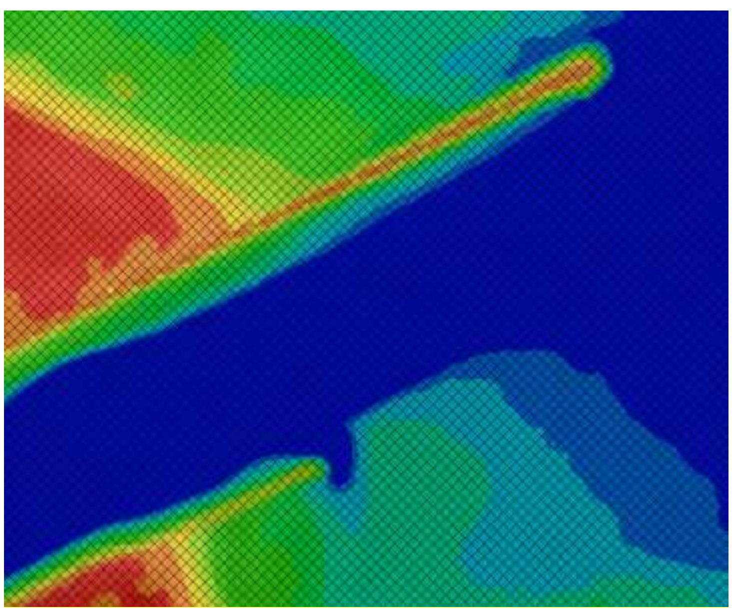

**Figure 26.** **Close up view of CMS-Wave model Grid showing 10-meter grid resolution in Canal region.**

CMS-Wave requires the input of a directional wave spectrum, which represents the distribution of wave energy in the frequency and directions domains. The two-dimensional wave spectrum is given as the product of the energy and directional spectra. The directional spreading function provides the relative magnitude of directional spreading of wave energy, while the frequency spectrum provides the absolute value of wave energy density. Table 12 presents the input conditions from the WIS station data utilized in creating the directional wave spectrum for the CMS-Wave simulations. These input conditions were obtained by analyzing the 20-year WIS simulated hindcast. Data were segregated by direction of approach and wave statistics were calculated for each directional bin. Extreme significant wave heights (Hs) were obtained by extremal analysis and peak wave periods (Tp) for each scenario was obtained by review of historical wave parameters correlation of specified wave heights.

**Table 12. Input conditions and scenarios for the wave transformation numerical modeling.**

<table>
<tr><th>Directional Bin<br>(0 deg.= N)</th><th>Type</th><th>Occurrence<br>(%)</th><th>Hsig<br>(m)</th><th>Hsig<br>(ft)</th><th>Tp<br>(sec)</th><th>Peak Dir<br>(0 deg.= N)</th></tr>
<tr><td>From 295 to 317.5</td><td>Sea and Swell</td><td>8.80</td><td>0.73</td><td>2.40</td><td>4.01</td><td>306</td></tr>
<tr><td>From 317.5 to 340</td><td>Sea and Swell</td><td>6.80</td><td>0.86</td><td>2.82</td><td>4.16</td><td>329</td></tr>
<tr><td>From 340 to 2.5</td><td>Sea and Swell</td><td>8.30</td><td>0.94</td><td>3.08</td><td>4.85</td><td>351</td></tr>
<tr><td>From 2.5 to 25</td><td>Sea and Swell</td><td>8.30</td><td>1.10</td><td>3.61</td><td>5.44</td><td>10</td></tr>
<tr><td rowspan="2">Combination From 25 to 47.5</td><td>Local Sea</td><td rowspan="2">14.60</td><td>0.72</td><td>2.36</td><td>3.50</td><td rowspan="2">43</td></tr>
<tr><td>Sea and Swell</td><td>0.68</td><td>2.23</td><td>11.00</td></tr>
<tr><td>From 47.5 to 70</td><td>Local Sea</td><td>2.80</td><td>0.72</td><td>2.35</td><td>3.46</td><td>25</td></tr>
<tr><td>From 70 to 92.5</td><td>Local Sea</td><td>2.90</td><td>0.66</td><td>2.15</td><td>3.28</td><td>61</td></tr>
<tr><td>From 92.5 to 115</td><td>Local Sea</td><td>2.50</td><td>0.51</td><td>1.67</td><td>2.82</td><td>76</td></tr>
<tr><td>Calm</td><td>Offshore Winds</td><td>45.0</td><td></td><td></td><td></td><td>82</td></tr>
<tr><td>10-year Storm</td><td colspan="2">Surge 8.9 ft above MTL</td><td>4.8</td><td>15.75</td><td>11.3</td><td>20</td></tr>
<tr><td>50-year Storm</td><td colspan="2">Surge 9.9 ft above MTL</td><td>5.0</td><td>16.40</td><td>13.3</td><td>20</td></tr>
</table>

Results from each of the directional cases developed were assessed to assess the existing wave climate. The results from each directional case was combined with the percentage of occurrence for wave from that direction to create a long-term (20-year) evaluation of wave impacts at the shoreline. This long-term evaluation was usedto assess the existing sediment transport patterns around the CCC region (described in the following section on sediment transport). In addition, results from the extreme event cases were utilized to assess the potential extreme waves experienced at the site, as well as transport during extreme events.

#### 4.2.2.2 Wave Model Results

Model simulations were performed for the typical wave conditions represented by the directional bin spectra presented in Table 12. As an example of the results, Figure 27 illustrates the wave results for the CMS wave grid simulation for waves approaching from the north/northeast (2.5 to 25-degree bin). The color map corresponds to the distribution of significant wave height (meters) throughout the modeling domain. Reds and yellows represent higher wave heights, while blues indicate smaller waves. Arrows on the figure represent the modeled wave directions as they propagate and approach the shoreline. The directions become more shore-normal as the waves get closer to the coastline and are affected by the irregular bottom bathymetry.

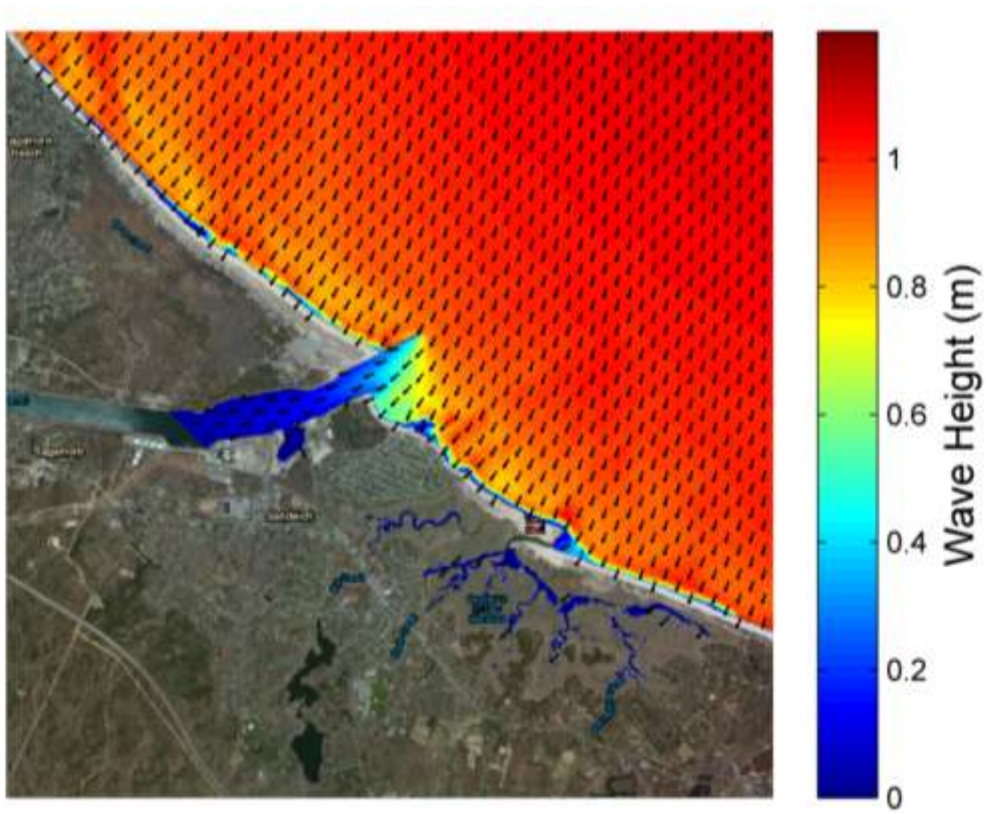


**Figure 27. Spectral wave modeling results for a northeast approach direction (2.5- to 25-degree bin). Wave height is represented in meters.**

Figure 27 shows how the bathymetric features near the Cape Cod Canal affect wave energy for this specific approach direction. For example, wave shadowing is shown to occur in the lee of the Scusset Beach Canal Jetty. In order to understand the overall dynamics of the region surrounding the CCC, all approach directions must be considered. The variability in the wave climate is clearly indicated by the differences in nearshore wave patterns arising from the various input spectra approach directions. 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 analysis, which is described in the next section, includes waves coming from all approach directions.

The wave transformation model was also used to simulate high energy events as shown in Table 12. The simulation of extreme high energy events was important to quantify the short-term impacts that occur during these energetic scenarios. Figures 28 and 29 show the spectral wave model results for the 10-, and 50-year return period events, respectively. Wave heights are significantly higher in these cases than in the annual average directional cases, as the offshore wave heights are more energetic. Overall, the storm simulations show that the region surrounding the CCC can become a high-energy environment conductive to large wave events in the event of extreme storms. These large wave events, although short-lived can potentially have a significant impact on the mobilization of sediments on the beaches surrounding the CCC.

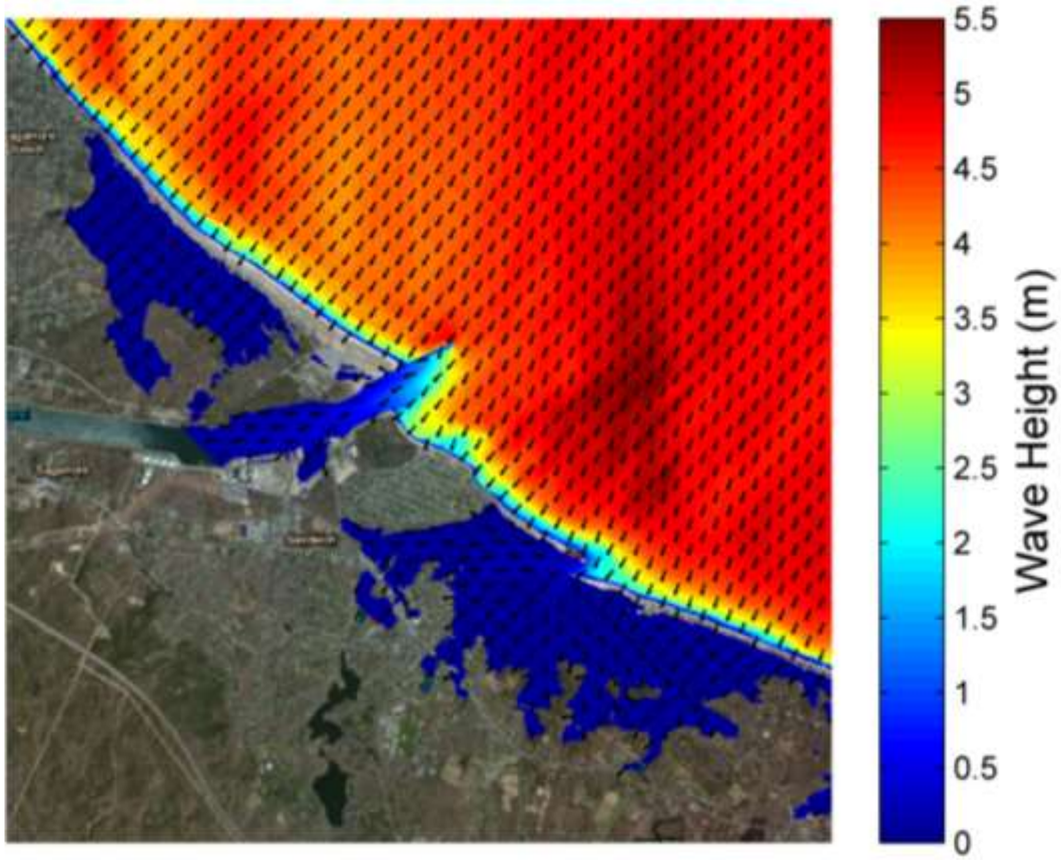


**Figure 28. Spectral wave modeling results for a 10-year return period storm event. Wave height is represented in meters.**

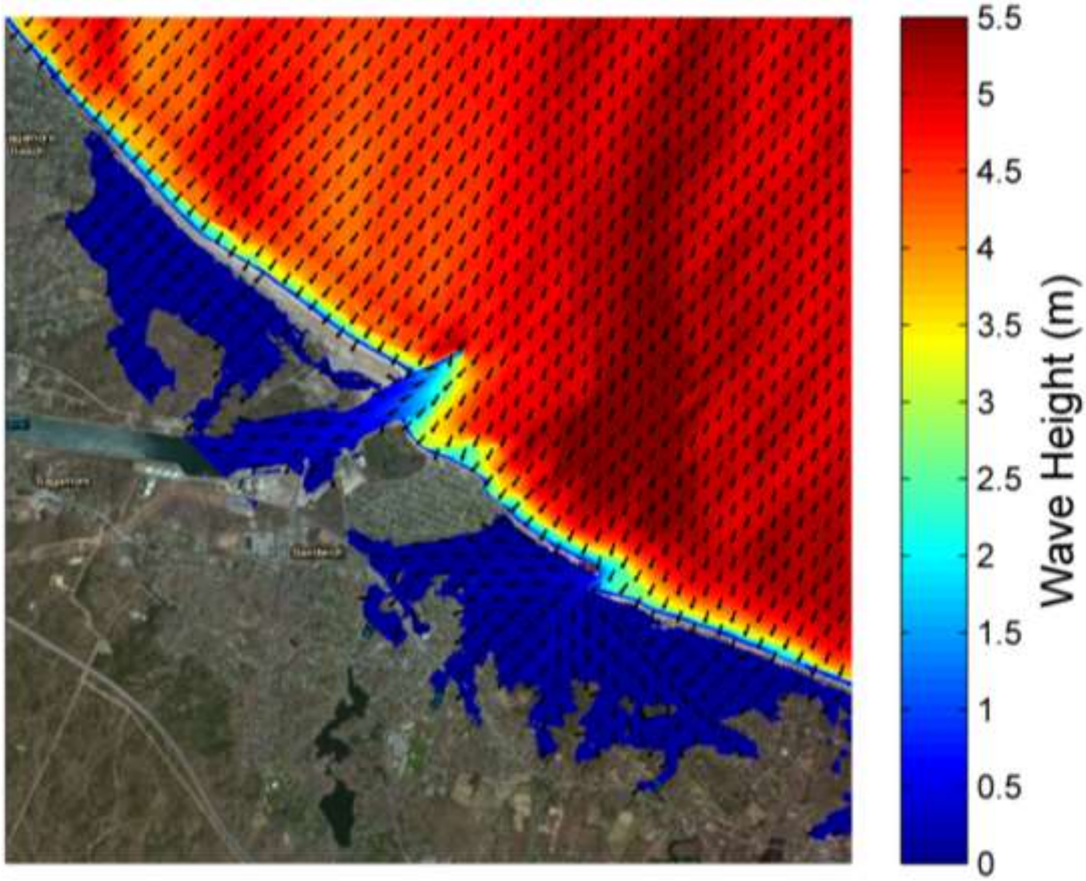


**Figure 29. Spectral wave modeling results for a 50-year return period storm event. Wave height is represented in meters.**

## 4.3 SEDIMENT TRANSPORT

In order to evaluate the existing conditions surrounding the CCC, as well as to assess any alternative that may be considered in the coastal region, the sediment transport dynamics for the region must be understood. In engineering practice, it is typical to evaluate alongshore and cross-shore sediment transport separately due to the different processes that govern the two. This section describes the sediment characteristics for the CCC region, the methodology utilized to estimate the alongshore sediment flux integrated across the surf zone, as well as the methodology used to estimate across-shore sediment transport for the beaches (specifically Town Neck Beach) adjacent to the CCC. The sediment transport analysis described here was also used for assessing potential design alternatives (Chapter 5).

### 4.3.1 Alongshore Sediment Transport Analysis Approach

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 detail, in degree of representation of the physics of the problem, in complexity, and in other manners. Process-based sediment transport models are those that directly address the fundamental physics of waves and sediment transport. These models which focus on those essential physics are able to encompass a variable wave field. Such sediment transport models may not represent all of the details exactly, but they can be used to demonstrate regional sediment transport trends and the spatial influence of coastal structures on adjacent shorelines. The sediment transport model utilized for the analysis present herein is a process-based model which determines regional sediment transport trends in the presence of time-variable (in direction and height) waves.

The regional sediment transport model requires the results of the near-shore wave field analysis presented in the previous chapter. The sediment transport model consists of a hydrodynamic component to determine the wave-induced currents, and a sediment transport component to quantify the amount of sediment moved by those 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 representation of the alongshore current. The sediment transport component is based on peer-reviewed and published formulation by Haas and Hanes (2004), which has been shown to be consistent with complex formulae for wave-driven sediment transport and with the Coastal Engineering Research Center (CERC) formula (USACE, 2002) for the total (laterally-integrated) alongshore sediment flux.

The grid for the sediment transport model was the same based on the same high-resolution grid used for the CMS wave transformation model (as described in the previous section). The bathymetry and results from the CMS-Wave transformation modeling was interpreted to a regular 5-meter cell size grid at the same orientation as the CMS-wave grid. The results from the CMS-Wave simulations for the average annual wave conditions summarized in Tables 12 were applied as input into the sediment transport model.

### 4.3.2 Alongshore Model Description

As stated above, the sediment transport model used for the analysis is a process-based model that uses standard steady-state, depth-averaged mass and momentum equations for the hydrodynamics, in conjunction with calculations of alongshore sediment transport based on a methodology by Haas and Hanes (2004). The following subsections present in detail the model theory and formulation of the various model components, but it is not critical that the reader becomes familiar with the concepts presented below to understand the results of the modeling.

#### *4.3.2.1 Hydrodynamic Component*

The wave-averaged, depth-integrated, mass-conservation equation for a constant-density fluid with a rigid lid is

$$\frac{\partial(Hu)}{\partial x} + \frac{\partial(Hv)}{\partial y} = 0,$$

and the wave-averaged, depth-averaged momentum equations for a non-rotating system are

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}=-g\frac{\partial \eta}{\partial x}-\frac{ru}{H}+\frac{\tau_x}{(\rho H)}$$

and

$$\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}=-g\frac{\partial \eta}{\partial y}-\frac{rv}{H}+\frac{\tau_y}{(\rho H)}.$$

Here $x$ and $y$ are the horizontal coordinates, $t$ is time, $u$ and $v$ are the $x$ and $y$ components of the wave-averaged and depth-averaged horizontal velocity, $g$ is the gravitational acceleration, $\eta$ is the surface displacement, $r$ is the bottom resistance coefficient, $H$ is the water depth, $\rho$ is the fluid density, and $\tau_x$ and $\tau_y$ are $-(1/H)\partial S_{xx}/\partial x$ - $(1/H)\partial S_{xy}/\partial y$ and $-(1/H)\partial S_{xy}/\partial x$ - $(1/H)\partial S_{yy}/\partial y$, respectively, where $S_{xx}$, $S_{xy}$, and $S_{yy}$ are the components of the wave-induced radiation stress tensor (Mei, 1989).

A stream function ($\psi$), which defines the two-dimensional flow, can be defined by

(A) $$(u,v)=H^{-1}\left(\frac{\partial \psi}{\partial y},-\frac{\partial \psi}{\partial x}\right),$$

and an equation for the wave-averaged potential vorticity ξ, defined by

(B) $$\xi=\frac{1}{H}\left(\frac{\partial v}{\partial x}-\frac{\partial u}{\partial y}\right)=-\frac{1}{H}\frac{\partial}{\partial x}\left(\frac{1}{H}\frac{\partial \psi}{\partial x}\right)-\frac{1}{H}\frac{\partial}{\partial y}\left(\frac{1}{H}\frac{\partial \psi}{\partial y}\right),$$

is obtained by taking the curl of the equations above and dividing the result by H, which yields

(C) $$\frac{\partial \xi}{\partial t}+u\frac{\partial \xi}{\partial x}+v\frac{\partial \xi}{\partial y}+\lambda\xi=\lambda\xi_0+\frac{v_0-v}{H}\frac{\partial \lambda}{\partial x}-\frac{u_0-u}{H}\frac{\partial \lambda}{\partial y}$$

where $\lambda=r/H$, $u_0=\tau_x/(\rho r)$, $v_0=\tau_y/(\rho r)$, and $\xi_0=H^{-1}(\partial v_0/\partial x - \partial u_0/\partial y)$.

In the present application, H is known, r is assumed to be given in the linear long wave approximation by cd[Hs/(4H)](gH)1/2 (e.g., Mei, 1983), and τx and τy are output from the wave transformation model. Here cd = 0.003 is the drag coefficient for the surf zone under breaking waves (Feddersen et al., 1998) and H is the significant wave height, defined to be four times the standard deviation of the wave-induced oscillatory surface displacements, which is also given by the wave model. With this information, equations A, B, and C, shown above, determine the coupled evolution of ξ, ψ, u and v.

The coordinate system is defined so that x is positive onshore, x = 0 defines the offshore boundary of the computational domain, y = 0 and y = Ly denote the alongshore boundaries of the computational domain, and the shoreline is a potentially irregular boundary in x > 0. In the present application, there can be only one shoreline, and H is restricted to be positive and nonzero everywhere in the domain. Boundary conditions are required for ψ on all boundaries and for ξ on inflow boundaries. The following boundary conditions are intended for applications in which the offshore boundary is well seaward of the surf zone and the shoreline at the alongshore boundaries is approximately straight and parallel to the y axis.

At the offshore boundary, the forcing and velocity fields are assumed to be weak, so that the alongshore velocity and potential vorticity are negligibly small and the offshore boundary conditions become

$$\frac{\partial \psi}{\partial x} = 0 \text{ and } \xi = 0 \text{ at } x = 0.$$

At the alongshore boundaries, the velocity field is assumed to be approximately confined to the y direction and approximately independent of y, so that the alongshore boundary conditions become

$$\frac{\partial \psi}{\partial y} = 0 \text{ and } \frac{\partial \xi}{\partial y} = 0 \text{ at } y = 0, L_y.$$

The shoreline is a streamline, so that ψ on the shoreline must be a constant, which may be set to zero, without loss of generality:

$$\psi = 0 \text{ on the shoreline.}$$

The shoreline is not an inflow boundary, so that the shoreline potential vorticity does not affect the solution.

These equations are solved by means of a standard numerical procedure described, for example, by Roache (1998). Spatial derivatives are represented using finite differences on a rectangular grid with equal spacing dx in the x and y directions. The representation of the spatial derivatives is second-order-accurate except that the advective terms are represented by a first-order upwind scheme. The time derivative is represented by an explicit first-order scheme with time step dt. The solution for each application begins from rest and advances in time until it reaches an asymptotic steady state. At each time step, the potential vorticity ξ is advanced and the elliptic equation is then solved for the stream function ψ using Jacobi iteration (e.g., Lynch 2004), and finally the velocities u and v are calculated. Attainment of an approximate steady state requires that the solution advance until t is approximately equal to 3 times the maximum value of λ.

Stability requires that the Courant number (u2+v2)1/2dt/dx based on the maximum flow speed be less than approximately unity.

#### *4.3.2.2 Sediment Transport Component*

Haas and Hanes (2004) proposed a simple formula for the alongshore sediment flux, which is, in the present notation,

$$q_s = \left(\frac{2c_1 c_d}{g}\right) < |\boldsymbol{u}|^2 > u_s$$

where qs is the alongshore component of the sediment flux, c1 is an empirical constant approximately equal to 1.3, brackets denote an average over many wave periods, u is the instantaneous velocity vector (including both the wave-induced oscillatory velocity and the current), and us is the alongshore component of the current velocity.

In the present application, u is assumed to be dominated by wave-induced oscillatory velocities and to be related to wave-induced surface displacement by linear long wave theory, so that <|u|2> approximates [Hs/(4H)]2gH. In addition, a right-handed coordinate system (s,n,z) is defined so that s is locally alongshore, n is locally shore-normal, and z is vertical and positive upward. In this coordinate system, Hus = ∂ψ/∂n. The equation above can therefore be written as:

$$q_s = 2c_1 c_d \left[\frac{H_s}{4H}\right]^2 \frac{\partial \psi}{\partial n}$$

In the surf zone, Hs/H is approximately constant (Hs/H < 0.63 is explicitly assumed by STWAVE), so that (5-11) can be integrated with respect to n across the surf zone to yield

$$Q = \frac{c_1 c_d}{8} \left[\frac{H_{sb}}{H_b}\right]^2 \psi_b$$

where Q is the alongshore sediment flux integrated across the surf zone and subscript b denotes evaluation at the break point, (i.e., at the seaward edge of the surf zone). In the present application, this equation is used to determine the sediment flux integrated across the surf zone after the stream function has been computed from the hydrodynamic component.

In determining sediment mobility, the threshold for mobility was established using the criterion parameter θcr, defined by Soulsby (1997) as:

$$\theta_{cr} = \frac{0.30}{1+1.2D_*} + 0.055\left[1 - \exp\left(-0.02D_*\right)\right]$$

where $D_*$ is the dimensionless grain size given by:

$$D_* = \left[\frac{g(s-1)}{\nu^2}\right]^{1/3} d_{50}$$

with $g$ being the acceleration due to gravity, $\nu$ is the kinematic viscosity of water, $d_{50}$ is the median grain size, and $s = \rho_s/\rho$.

The computation of the maximum bed shear stress due to the combined waves and currents, employed the algebraic expression by Soulsby (1997), which best fits the analytical model of Grant and Madsen (1979). The drag coefficient $c_d$ of steady current in absence of waves and the wave friction factor $f_w$ for waves in absence of current were determined as:

$$f_w = 1.39\left(\frac{A}{z_o}\right)^{-0.52}$$

$$c_d = \left[\frac{\kappa}{1+\ln\left(z_o / h\right)}\right]^2$$

where $A=U_wT/2\pi$, the bed roughness length $z_o=d_{50}/12$, $\kappa=0.40$ is von Karman's constant, and $h$ is the water depth.

### 4.3.3 Alongshore Average Annual Sediment Transport Results

Wave results from each of the average annual directional spectra bin simulations were used to develop a complete summary of sediment transport for various wave conditions. Simulations of sediment transport were conducted using a grain size distribution with a median of 0.61 mm ($D_{50}$=0.61mm) and the results were assessed to define the average annual sediment transport regime through the CCC region.

Model simulations were performed for the wave conditions represented by the directional bin spectra presented in Table 12. To accurately represent sediment transport over an average year, the various wave scenarios were combined to represent an average year of wave climate. Using the percent occurrence of wave approach, the average annual approach directions were normalized and combined to determine the net alongshore transport rate. Figure 30 presents the average yearly sediment flux determined using the process-based sediment transport model for the CCC region. The arrows on the figure indicate direction of transport while colors of arrows indicate magnitude. The figure also includes the shoreline change transects for the beaches surrounding the CCC for comparison purposes. The shoreline change analysis and results for

Town Neck Beach are based on data from 1952 to 2018 and are described in further detail in Chapter 2, while the Scusset Beach results are from based on the CZM Massachusetts Shoreline Change Project and represent the long-term change rates from 1861 to 2009. The colors of shoreline change transects indicate the rate of change, while the length of the transect indicate the distance of maximum change (most landward to most seaward) over the period of change.

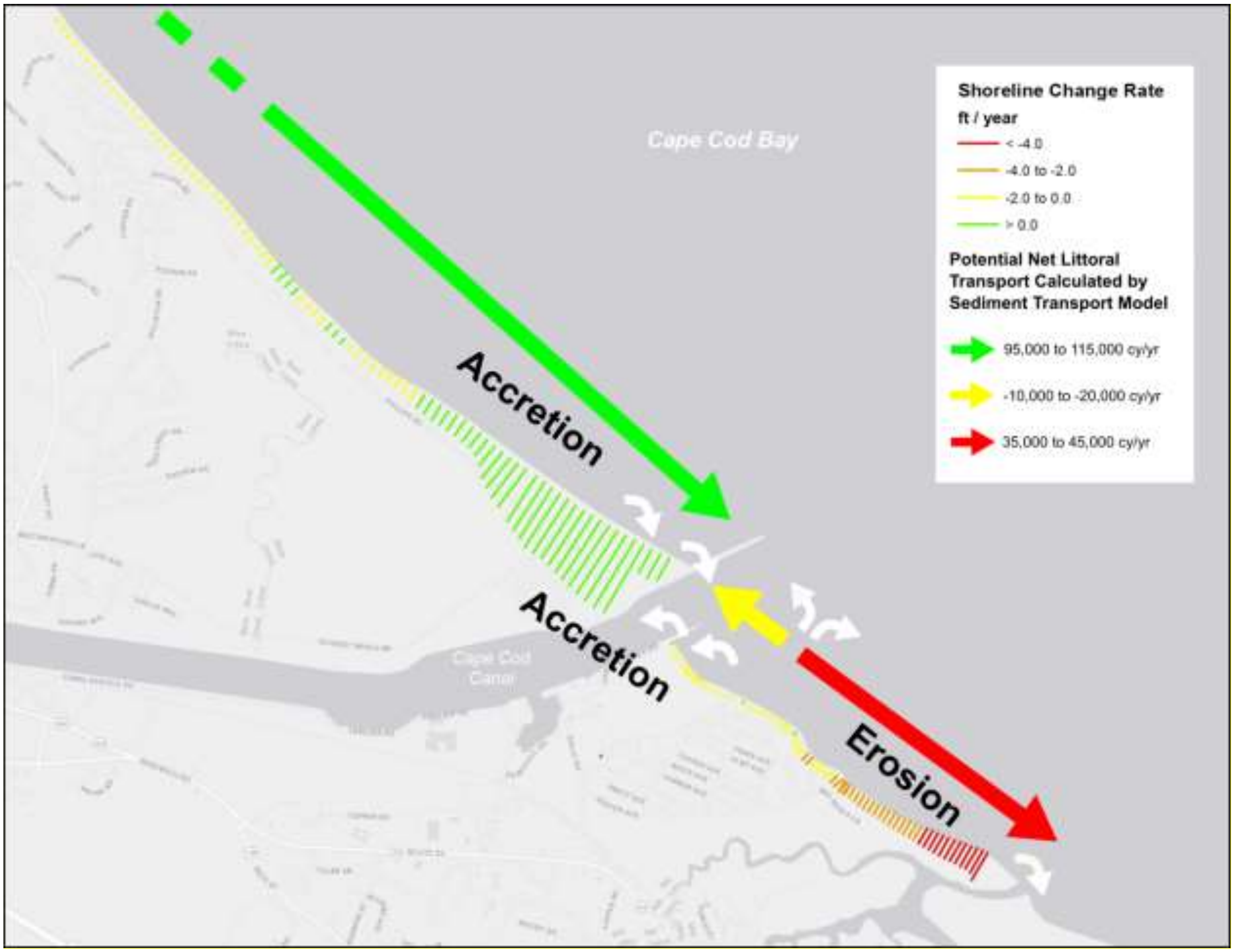


**Figure 30. Results from the physics based alongshore sediment transport model for average annual conditions plotted alongside shoreline change results.**

The sediment flux presented in Figure 30 represents the potential rate of sediment moving along the coast. The rates are presented in units of yards$^3$/year and represent a theoretical rate of transport of sediment caused by wave-induced currents. The calculations used in this analysis assume that sediment is available on the beach and in the surf zone for transport (e.g. transport potential). If the shoreline does not have a sediment source available, or transport is interrupted by structures (such as groins or jetties), then the sediment transport rates may vary compared to the values presented here. These rates represent an approximation of the potential sediment that could be transported along Scusset and Town Neck beaches based on wave-induced currents.

The sediment flux data indicates that there is a strong net alongshore transport in the CCC Region from northwest to the southeast, consistent with the prevalent northeast wave approach direction. Along Scusset Beach, north of the CCC, the average annual alongshore transport is directed to the southeast at an average rate of approximately 95,000 to 115,000 cy/year, ending at the western Cape Cod Canal jetty. This pattern and range of net alongshore transport rates are consistent with those presented by Berman (2011), FitzGerald (1993), and Borrelli et al., (2016). Southeast of the CCC and ending approximately at Knott Avenue there is a small zone of transport reversal, located in the shadow of the Canal jetties, which limits the wave energy from the northeast, yet allow energy from the less predominant eastern directions. Net transport at this reversal ranges from approximately 10,000 to 20,000 cy/year toward the northwest. Southeast of the reversal, net alongshore sediment transport patterns continue to be directed towards the southeast, where transport rates range from approximately 35,000 to 45,000 cy/year until reaching Old Harbor Inlet.

These sediment transport rates were compared to the historical shoreline change rates to assess the performance of the sediment transport model. In addition to presenting the net overall transport results, Figure 30 also overlays the model sediment flux results against the historic rates of shoreline change. The transect colors represent the historic rates of shoreline change (in terms of feet/year). Negative values of shoreline change indicate erosion, while positive values indicate accretion. The areas of erosion and accretion shown in Figure 30 generally match the expected patterns of alongshore transport based on the modeled results. For example, sand moving from Town Neck Beach to the southeast would be expected to result in a loss of sediment from the region, resulting in the observed erosion. Similarly, sediment transport from the north-west Plymouth region towards the northern Canal Jetty results in accretion to the north of the Jetty where sediment is being trapped. Additionally, the area directly southeast of the CCC has shown to be more stable through time. This area corresponds to the area of sediment reversal in the model, where a low rate of sediment transport is expected back towards the CCC. There also should be an area of potential increased erosion located approximately near the Knott Ave. extension, where there is a divergence in flux. However, there does not appear to be any significant increase in erosion over this area. This area is also the location of a large rocky-intertidal tide pool. It is likely that when this was originally a sand rich system, this area would be prone to erosion; however, now, due to lack of sediment availability, all that remains is larger grain size material that cannot be easily transported and is more stable Additionally, the influence of the coastal groins throughout this area result in numerous small reversals and transport influences that change sediment transport directions and magnitudes based on slight changes in the incoming wave energy.

### 4.3.4 Cross-shore Sediment Transport Analysis Approach

In addition to alongshore sediment transport, the physical processes of cross-shore sediment transport were also evaluated at Town Neck Beach in Sandwich utilizing the XBeach numerical model. XBeach is an open-source numerical model developed to simulate wave, hydrodynamic, and morphodynamic processes. The model has been developed with support of various agencies including the USACE, Rijkswaterstaat and the EU, together with a consortium of UNESCO's institute for Water Education, Deltares (formerly WL Delft Hydraulics), Delft University of Technology, and the University of Miami.

XBeach includes the hydrodynamic processes of short-wave transformation (refraction, shoaling, and breaking), long-wave (infragravity wave) transformation (generation, propagation, and dissipation), wave-induced setup and unsteady currents, as well as overwash and inundation. The morphodynamic processes include bedload and suspended sediment transport, dune face avalanching, bed update, and breaching. The model has been validated with a series of analytical, laboratory, and field test cases using a standard set of parameter settings. Further details of the XBeach model and its theory can be found in the XBeach Technical Reference (Deltares, 2015).

Cross-shore sediment transport was evaluated using XBeach along a representative 1-diminsional cross-shore transect located at the southeastern end of the parking lot on Town Neck Beach; a location that still has a fairly healthy dune in place from previous nourishment efforts. Elevations along the transect were specified at a resolution of approximately 10m (32.8 ft) offshore, with more refined model cell spacing (down to 1 m or 3.3 ft) closer to shore. Topography was defined for the transect using an April, 2019 topographic survey of the beach conducted by Woods Hole Group. Offshore bathymetry was defined using the CoNED Topobathymetric Model (1887 - 2016): New England (OCM Partners, 2018). This dataset was created after Hurricane Sandy to identify inundation hazard zones and utilizes the most recently available bathymetric and topographic data to create a continuous DEM. The model transect extends, perpendicular to the orientation of the shoreline, from approximately 70 ft at the offshore end of the transect to the marsh on the far side of Town Neck Beach. A sediment grain size distribution with a D50 of 0.66 mm and a D90 of 1 mm were defined for all model simulations.

The model was used to simulate cross-shore sediment transport for the existing profile under three different storm conditions. Table 13 includes the input boundary parameters that were defined for each of the three cases. Synthetic surge hydrographs were developed for input into XBeach as a water level boundary using the peak water levels for each storm event listed in Table 13. Each storm event was created from 144 hours of normal tidal data, with water levels ramping up and down to/from the storm tide value for each storm, starting with, and ending with normal tidal conditions to allow conditions to equilibrate after the storm.

**Table 13. Boundary Conditions for XBeach Simulations.**

| Storm Event | Significant Wave Height (ft) | Peak Wave Period (sec) | Storm Tide (ft, NAVD88) |
|---|---|---|---|
| 1-year | 15.42 | 10.9 | 6.2 |
| 10-year | 15.75 | 11.3 | 8.1 |
| 50-year | 16.40 | 13.3 | 9.1 |

Two options are possible in XBeach for the specification of wave conditions: (1) wave spectra, and (2) non-spectra, such as stationary wave conditions or time series. For this study, the wave spectra boundary condition was applied and a JONSWAP parametric spectrum was utilized. The spectrum shape is defined through the specification of wave height, wave period, wave angle, and other parameters, which XBeach then uses to generate a random wave time series. A conservative shore normal direction was specified for all waves. Similar to the water levels, wave heights and periods were ramped up and down corresponding to the peak of the storm event.

The model output consists of wave height, water surface elevation, and velocity along the profile for each output timestep, along with changes in the bottom profile showing areas of erosion and deposition. The final profile for each case was extracted from the model simulations for comparisons with the initial profile to determine possible impacts to the beach from storm conditions under existing conditions and for comparison with potential alternatives.

Results from the three storm cases for existing conditions are shown in Figure 31. The figure shows the cross-shore profile under existing conditions, as well as the eroded profiles under the three different storm cases. The profiles are plotted with elevation in terms of feet, NAVD88 on the y-axis, and distance along the model transect on the x-axis.

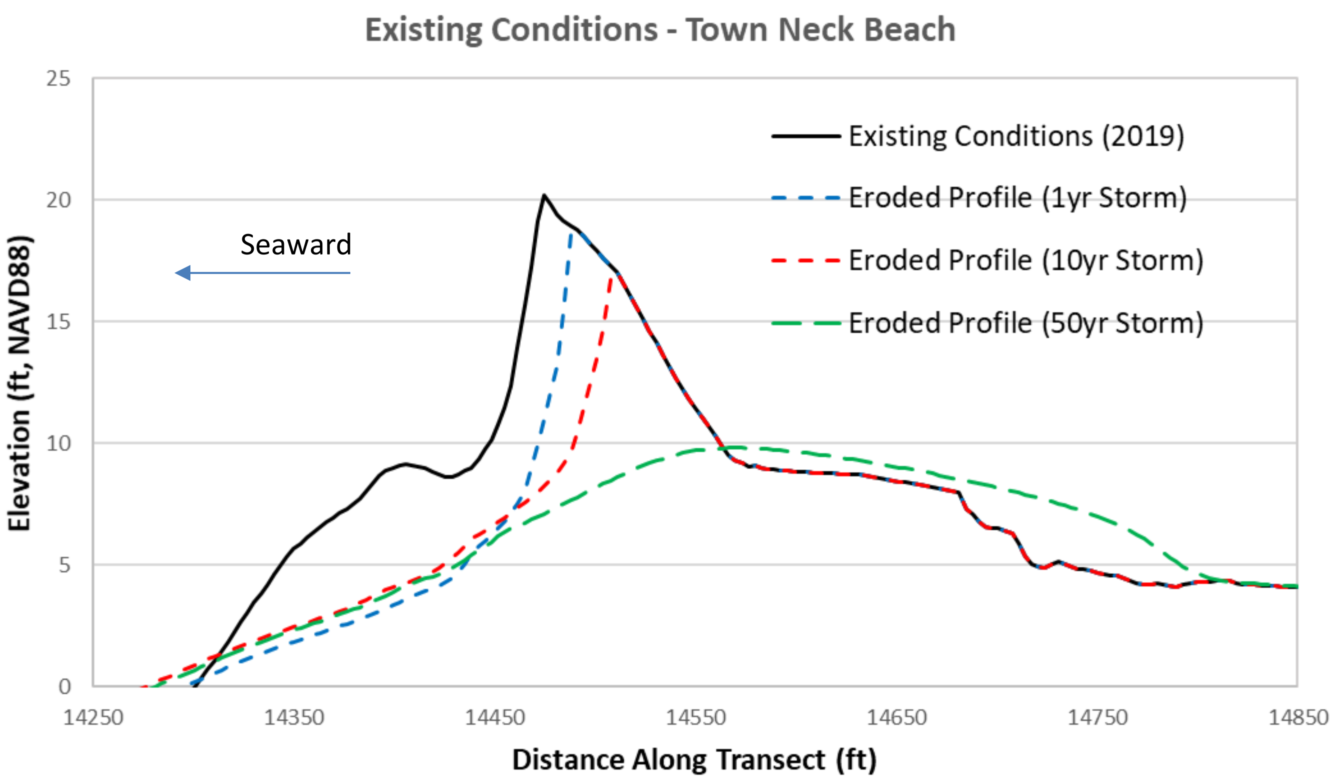


**Figure 31. Results from the XBeach cross-shore sediment transport model for three storm cases.**

Results from the storm condition scenarios show how the existing profile might be expected to perform during storm events of various sizes (return periods). The three storm events all result in erosion of the existing berm, and retreat of the existing dune scarp. The dune would also be overtopped (as indicated by the lowering of the crest of the dune) during all storm events, indicating the relative frailty of the existing barrier beach and dune system. Additionally, the 1-year and 10-year storm cases show retreat of the dune scarp, while the 50-year storm case shows complete failure of the dune. All three cases resulted in milder slopes of the beach following the event. It should be noted that the eroded profile represents a possible profile immediately following an event, the beach profile would be expected to change and come to a new equilibrium profile after the event and some period of regular tides.

# 5.0 ALTERNATIVE DEVELOPMENT AND ASSESSMENT

An alternatives analysis is the basis for determining the optimal solution and assessing potential impacts, both physical and environmental. A variety of factors are considered when evaluating the various alternatives (e.g., cost, feasibility, performance, environmental impacts, constructability, etc.), with the overall objective focused on selecting the optimal solution. As such, the goal of the assessment is to evaluate reasonable, practicable, and feasible alternatives that will achieve the goals and objectives of the project, while minimizing the short and long-term adverse effects, if any. The alternatives analysis procedure developed for the Cape Cod Canal Section 111 Feasibility Study, as well as a comprehensive list of the alternatives evaluated, is presented in this chapter.

Ocean waves, currents, tides, storm surges, and relative sea-level rise contribute to the erosion of sandy shorelines and the destruction of coastal property. Traditionally, attempts to combat these erosional pressures consisted of hard structures, such as groins, breakwaters, seawalls and revetments, and/or soft solutions such as artificial beach fills. Each of these established erosion mitigation measures has proven effective when used under favorable conditions; yet, none is suitable for every location, and implementation under the wrong conditions may have severe negative impacts on a coastal community.

Decisions regarding management of shoreline erosion at Sandwich Town beaches can only be made after a thorough evaluation of available erosion mitigation alternatives. The following chapter describes a variety of established coastal engineering methods for erosion mitigation, as well as several less traditional approaches. The ideas upon which these methods were developed are explained, and their particular application at Town Neck Beach and Springhill Beach is discussed.

The study alternatives were chosen at a meeting on November 19, 2018, during which all viable long-term solutions were discussed and considered. Careful consideration was given to all factors associated with each alternative. For example, potential impacts on the neighboring shoreline, engineering feasibility, likelihood of success, cost, etc. were all considered in the selection process. The alternatives that were viewed as the most highly advantageous were jointly selected for further analysis, both qualitative and quantitative. Some of the alternatives considered included optimization of the design (e.g., such as number of or types of groins). All members of the alternative development team (United States Army Corps of Engineers, Town of Sandwich, and Woods Hole Group) agreed upon the final alternatives that were selected for consideration.

## 5.1 ALTERNATIVES CONSIDERED

Types of alternatives that were considered included:

- No action
- Non-structural "soft" solutions (beach and dune restoration)
- Structural "hard" solutions (groins, jetty modifications, breakwaters, revetments and seawalls)

- Alternative technologies (beach dewatering, nearshore berms, submerged offshore reefs, and other alternative technologies)
- Sand bypassing plants (fixed or mobile)

Table 14 presents a list of the alternatives considered, if they are standard, established shore protection methods or alternative technologies (non-standard), hard or soft, and if they include a beach nourishment component. It is assumed that beach nourishment would be a component of all potential alternatives given the long-term sediment starvation of the system. Therefore, the value of the alternatives beyond just beach nourishment would need to enhance the benefit or performance of the beach nourishment alone alternative.

**Table 14. Alternatives considered in the alternative analysis procedure.**

| Alternative | Method | Hard/Soft |
|---|---|---|
| No action | N/A | N/A |
| Beach nourishment | Established | Soft |
| Jetty modifications | Established | Hard |
| Perched beach | Alternative | Hard |
| Dune Reconstruction | Established | Soft |
| Revetments and seawalls | Established | Hard |
| Groins / Groin Modifications | Established | Hard |
| Breakwaters | Established | Hard |
| Beach dewatering | Alternative | Hard |
| Nearshore berms | Alternative | Soft |
| Offshore reefs | Alternative | Hard |
| Other alternative technologies | Alternative | Hard |

## 5.2 NO ACTION

The no action alternative implies there would be no change to the present conditions at the Cape Cod Canal and Sandwich town beaches. Projected shoreline erosion presented in Chapter 2 indicates that the barrier beach is likely to be gone in approximately 50 years, and the usable beach gone much sooner. The erosion and eventual loss of the barrier beach also has significant impacts on the viability of the salt marsh system and produces increased flooding risk for the downtown Sandwich area. This alternative is considered unacceptable by the Town of Sandwich as the existing shorefront would continue to be eroded, a sustainable beach would not exist, no protective action would be taken, and the landward structures would face increased flood risk. The current water-dependent recreational function of the beaches, as well as the valuable habitat areas, would no longer be supported since the beach would not be maintained. Therefore, the "no action" alternative is not recommended for further consideration in the feasibility evaluation.

## 5.3 BEACH NOURISHMENT

One of the primary causes of coastal erosion is a deficit of sediment within the coastal littoral cell. To offset this deficit, nourishing the beach with compatible sediment placement is a logical means for improving the longevity of the shoreline where such a project is economically feasible.

Beach nourishment does not stop erosion. Rather, the damage to landward areas is postponed by extending the shoreline toward the ocean. As such, periodic renourishment must be anticipated, especially considering the lack of alongshore sediment supply that is inhibited by the Cape Cod Canal. At a site like Town Neck Beach, the beach also provides a major recreational benefit.

Beach nourishment is typically the most non-intrusive technique for coastal protection and involves placing sand, from an offshore or upland source, in a designed template on an eroding beach. Figures 32 and 33 present examples of beach nourishment projects being constructed. Beach nourishment is intended to widen the beach, as well as provide added storm protection, increased recreational area, and in some cases, added habitat area. Although nourished sand is eventually displaced alongshore or transported offshore, the nourished sand that is eroded takes the place of the upland area that would normally have been lost or eroded during a storm event. Therefore, beach nourishment serves a significant role in storm protection. In addition, beach nourishment is the only alternative that introduces additional sand into the system. For coastlines with a dwindling sediment supply, such as Town Neck Beach, this is critical for long-term success. Solutions that do not involve beach nourishment typically involve rearranging the existing sand in a manner that will only benefit a portion of the beach.

Environmental concerns with beach nourishment projects include the potential for decreased water quality when sediments are dredged and deposited, and disturbing natural habitat when removing or depositing the dredged material. These concerns can be addressed by adhering to dredging time windows that avoid periods of shellfish, finfish, and shorebird activity. Grain size compatibility between the borrowed and native beach sediments should be maximized in order to avoid disturbance of offshore resources such as shellfish and submerged aquatic vegetation, as well as to increase the lifespan of the nourished beach. For example, large differences in grain size between the native and borrow material may lead to changes in beach slope through natural adjustment of the new grain size introduced to the beach. This change in beach slope, as well as the change in grain size directly, may negatively influence the offshore resources.

The many benefits of beach nourishment, and the ability to control negative environmental impacts with careful design and planning, make beach nourishment a viable alternative for the Town Neck Beach area. A beach fill project for this area would mitigate the on-going erosion, improve storm damage prevention and flood protection to infrastructure, and improve the recreational resource of the public beach.

**Figure 32. Beach nourishment project under construction.**

**Figure 33. Beach nourishment project under construction in Virginia Beach, VA (photo courtesy of Virginia Beach).**

### 5.3.1 Beach Nourishment Design

A successful beach nourishment project consists of more than simply placing sediment on a beach. Beach nourishment projects are engineered. A beach nourishment template, which consists of numerous design parameters, is based on the characteristics of the site and the needs of a project. Every beach nourishment design is unique, since different beaches in different areas have different physical, geologic, environmental, and economic characteristics, as well as different levels of required protection. The design must consider climatology, the shape of the beach, type of native sand, volume and rates of sediment transport, erosion patterns and causes, waves and water levels, historical data and previous storms, probability of certain beach behaviors at the site, existing structures and infrastructure, and past engineering activities in the area. As such, beach nourishment design must identify the coastal processes at the site. Typically, computer models (Chapter 4) are used to help design the nourishment template.

The structure of a nourishment template is designed to yield a protective barrier that also provides material to the beach. A higher and wider beach berm is designed to absorb wave energy. Dunes may need to be constructed or existing dunes improved to reduce damage, including potential upland flooding, from storms. Figure 34 depicts a beach berm and dune on a typical beach profile. Nourishment length, berm height and width, dune height, and offshore slope are critical elements of a beach nourishment design. Periodic renourishment intervals are also usually a part of the nourishment design. If renourishment is required in less than 5 years, then the nourishment is probably not cost-effective. If renourishment is required between 7-10 years, then a nourishment project is likely cost-effective. The renourishment interval will vary based on the initial design, wave climate, sand used, number and types of storms, and project age. In addition, beach nourishment is not an exact science; variables and uncertainties exist. Actual periodic renourishment intervals may differ from planned intervals based on conditions at the nourished beach and the frequency and intensity of storms from year to year.

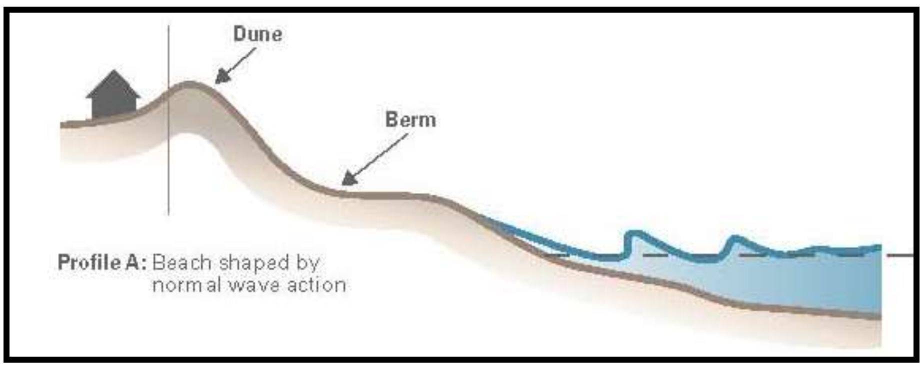


**Figure 34. Typical beach profile and features (USACE, 2002).**

This alternative consists of a combination of beach nourishment and dune creation, with no proposed jetty or groin work. This proposed beach nourishment has already been designed and permitted as part of a previous project developed by the Town of Sandwich (EEA #15213). As part of that design development, a dune and beach restoration template was developed that offered a holistic approach by encompassing the entire Town Neck beach and dune system. The existing jetty structures around Old Sandwich Harbor Inlet and the existing groins would be left in place. Beach nourishment and dune creation in this alternative required approximately 388,000 cy of clean beach compatible sediment. The nourishment would primarily be used to stabilize, strengthen and rebuild weak and eroded beach and dune reaches throughout the Town Neck Beach system. This nourishment would also serve as a feeder system for eroding downdrift beaches (Springhill). The creation of additional beach and dune resources would expand critical habitat area, and serve the protectable interests of storm damage prevention and flood control.

Figure 35 illustrates the beach nourishment alternative considered for this study. The material would be placed along approximately 5,000 linear feet of shoreline, beginning 1,000 feet southeast of the Cape Cod Canal in the west, and extending to within 600 feet of the Old Sandwich Harbor Inlet in the east, covering an approximate area 41.1 acres (1,792,300 $ft^2$). The crest of the newly created dune will be at an elevation of approximately 15 to 21 ft (NAVD88), with a width ranging from 50 to 150 ft (depending upon location). For the eastern barrier beach portion of the project, the beach berm would be increased in width by at least 100 ft at an elevation of 6 ft (NAVD88), and then extend seaward at a slope of 1V:20H to approximately –4 ft to –10 ft NAVD (depending upon existing grade). Dunes would have a slope of 1V:10H to 1V:15H to meet habitat requirements for endangered and threatened shorebirds and would be graded to match existing slopes. At the western end of the project area, the design was constrained by the presence of Rocky Intertidal Shore and complex hard bottom resources. Dunes at this end of the project would have a slope of 1V:5H, and the beach would slope seaward from the toe of dune at a slope of 1V:10H. At both ends of the project, the sand would be graded to feather in with the existing grades of the Coastal Beach and Dune.

This alternative will restore the Sandwich beaches as buffers to storm waters and flooding, restore sediments to eroding beach and dune resources, be a source of additional dune and beach sediments, and increase the surface area of bird habitat. The placement of this material increases the jurisdictional shoreline resources of Coastal Beach and Coastal Dune and enhances their associated functions, values and interests. However, since erosion will continue on this sediment-starved coast, in order to provide long-term benefits, regular maintenance of additional nourishment material will be necessary. As such, location of sediment sources for long-term maintenance will be crucial to the long-term success of this alternative. Without maintenance, the shoreline will return to its destabilized state. While this alternative considers the beach and dune nourishment alone, each subsequent alternative also could be, and perhaps needs to be, combined with this alternative.

### 5.3.2 Beach Nourishment Performance

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. A more detailed description of the derivation of the equations and their applications can be found in Dean (2002).

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

where M(t) is the proportion of sand remaining in the placed location, G is the alongshore diffusivity parameter, t is time, and l is the project (nourishment) length.


**Figure 35. Dune and beach restoration alternative at Town Neck Beach, Sandwich, MA.**

The alongshore diffusivity is presented by Pelnard-Considére (1956) as:

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

where K is the sediment transport coefficient (a function of sediment size), B is the berm elevation, $H_b$ is the breaking wave height, $h_*$ is the depth of closure, p is the *in-situ* sediment porosity (approximately 0.35 to 0.40), s is the sediment specific gravity (approximately 2.65), and $\kappa$ is the ratio of wave height to water depth within the surf zone (approximately 0.78).

The Pelnard-Considére equation can be applied to determine the performance of a beach nourishment project. For example, Figure 36 presents the spreading of an idealized, rectangular nourishment. Although simplified, this example illustrates the planform view of nourishment dispersion. Figure 36 contains a series of lines depicting the temporal planform evolution of a rectangular nourishment. The resulting planform is symmetrical about the centerline of the nourishment. Therefore, only one-half of the resulting planform is shown in Figure 36. The solid black line indicates the initial fill template, and subsequent lines indicate the temporal progression of the nourishment. The vertical axis indicates the nourishment width (or distance seaward from the original shoreline), while the horizontal axis indicates the alongshore distance from the center of the nourishment. Within 1-year of placement of the nourishment, the shoreline excursion at the center of the project has already retreated over 100 ft, as sand has been transported in both directions due to the perturbation that is created on the shoreline. However, as shown by the lines corresponding to temporal changes in fill, the material diffuses onto the adjacent properties and is not lost from the local system immediately.

The Pelnard-Considére equation can be applied to many different scenarios by adjusting the boundary conditions. Dean (2002) has adapted the equations to evaluate sand movement in regions with inlets and/or structural influences. In this case, the Pelnard-Considére equation is modified to include the spreading of the nourishment, and the influence of coastal structures (e.g., groins). This was applied for Town Neck Beach to evaluate not only the performance of the nourishment, but also the implementation of structures used in concert with nourishment.

Since the material diffuses (spreads) over time, it is possible to evaluate the longevity of the nourishment by looking at the amount of material (by percent) left in the Town Neck Beach project area. The lifetime of the beach nourishment is based upon the percent of the initial beach nourishment left within the boundary of the initial fill. The percentage remaining will decrease with time, but that material is not necessarily lost from the system, it has just spread to regions outside of the original nourishment template. For example, sediment may have been transported offshore or along the beach to the southeast. Therefore, although the sediment no longer falls within the initial nourishment template, it has not disappeared from the system as a whole. The project lifetime was calculated using the wave model results, the sediment transport results, sediment grain size analysis (fill material was chosen to match the native grain size), and the historical shoreline change rates.

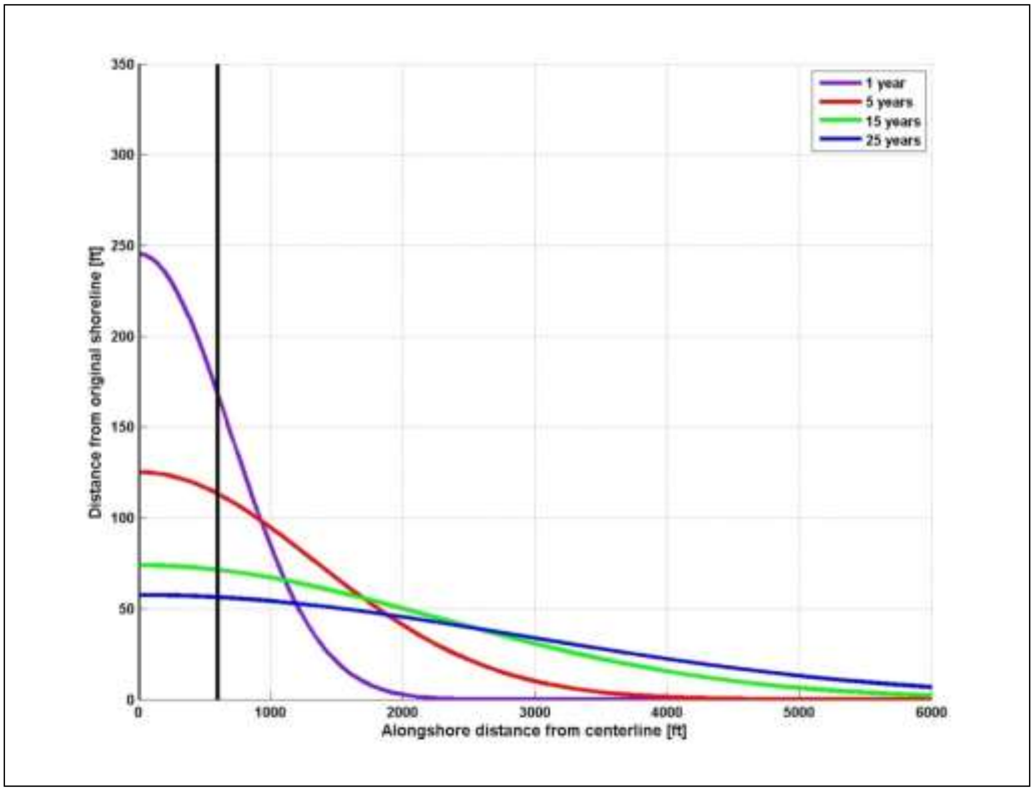


**Figure 36. Temporal evolution of an example nourishment. Since the nourishment spreading is symmetrical in this simple case, only half the fill distance is presented.**

Figure 37 presents the performance of the beach nourishment alone alternative at Town Neck Beach. The performance is expressed in terms of amount of material remaining in the initial template region, as a function of time. All results include a background erosion rate corresponding to -1.1 ft/year, which corresponds to the long-term rate of erosion. That is, in addition to the dispersion that is occurring, an additional -1.1 ft/year is eroded due to the natural erosion of the beach. This background erosion rate is already an "unnatural" rate that is influenced by the Cape Cod Canal structures. This rate is needed to assess the performance of a nourishment project that would be placed on the Sandwich beaches with the current system configuration (Canal and jetties in place). This background rate should not be confused with the erosion rate or shoreline influences that would have naturally occurred without the construction of the Cape Cod Canal (as presented in Section 2.5). The percent of initial material remaining is presented along the left-hand axis, while the time (in years) is presented along the bottom axis. For example, after 5 years, approximately 43% of the initial fill volume is remaining. Additionally, Figure 37 shows that 50% of the nourishment remains in the initial template region after approximately 3 years; however, it should be noted the reduction in fill does not mean the sand has been lost entirely from the system.

To verify that the performance modeling for Town Neck Beach was reasonable, the performance curves were compared to the measured performance of monitored beach nourishment projects in Massachusetts, as well as some in Florida. Figure 38 presents a comparison of the alternative scenario (red line) with monitored nourishment performances for Gulf Shores and Bonita Beach, Florida, as well as Dead Neck and Long Beach, Massachusetts. The modeled nourishment performances for Town Neck Beach appear reasonable, and perhaps somewhat conservative, when compared to actual nourishment performance. The modeled performance curve for Town Neck Beach compares well to those nourishments in the northeast, where a similar wave climate would be expected.

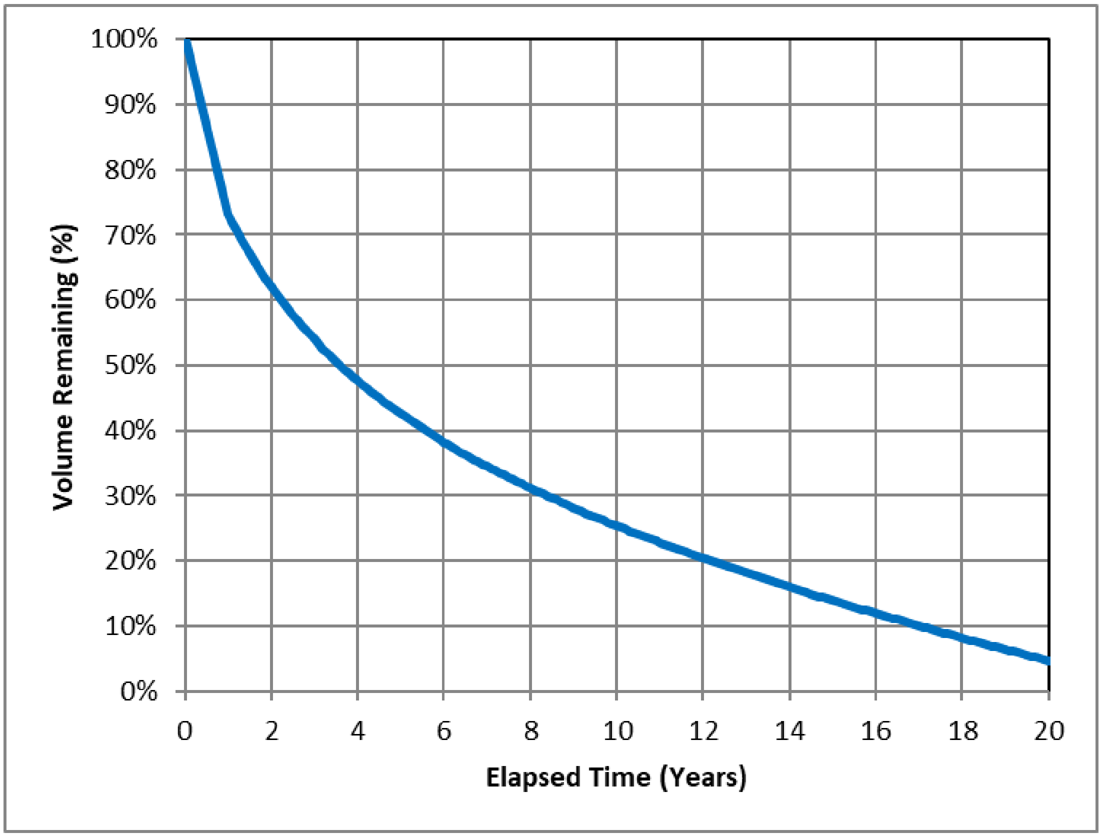


**Figure 37. Beach nourishment performance at Town Neck Beach. The vertical axis represents the percent of fill remaining in the initial template area.**

### 5.3.3 Beach Nourishment Critical Width

Beach nourishment projects are designed to optimize storm damage reduction benefits relative to costs. Designing a project to protect against any and all storms is not economically feasible. Extreme conditions and severe storms could exceed the capacity of a beach nourishment project to protect property. Therefore, a reasonable storm damage protection goal is typically established, defined here as the critical width. For this assessment, the critical width is defined as the minimum beach width remaining after nourishment before which a 10-year storm event would jeopardize upland infrastructure (e.g., homes, parking lot, buildings, boardwalk). It assumes that once the beach width reaches the critical width, a maintenance nourishment would be required to provide protection against a 10-year storm event, even though a substantial amount of the existing nourishment may still be remaining. To assess critical width, a cross-shore profile adjustment model was used to evaluate the storm protection provided by the design nourishment template. Once the beach reaches this critical width, there is a reasonable chance that damage may occur during a moderate to large storm event. This signifies when a renourishment project should be planned. This doesn't mean that the beach and dune system couldn't survive a 10-year storm event after this time, nor does it mean that the beach and dune system will not be eroded completely during a storm that occurs before this critical time. Rather, this analysis provides a general recommendation on the expected service life of the beach nourishment alternative that can be used for planning purposes.

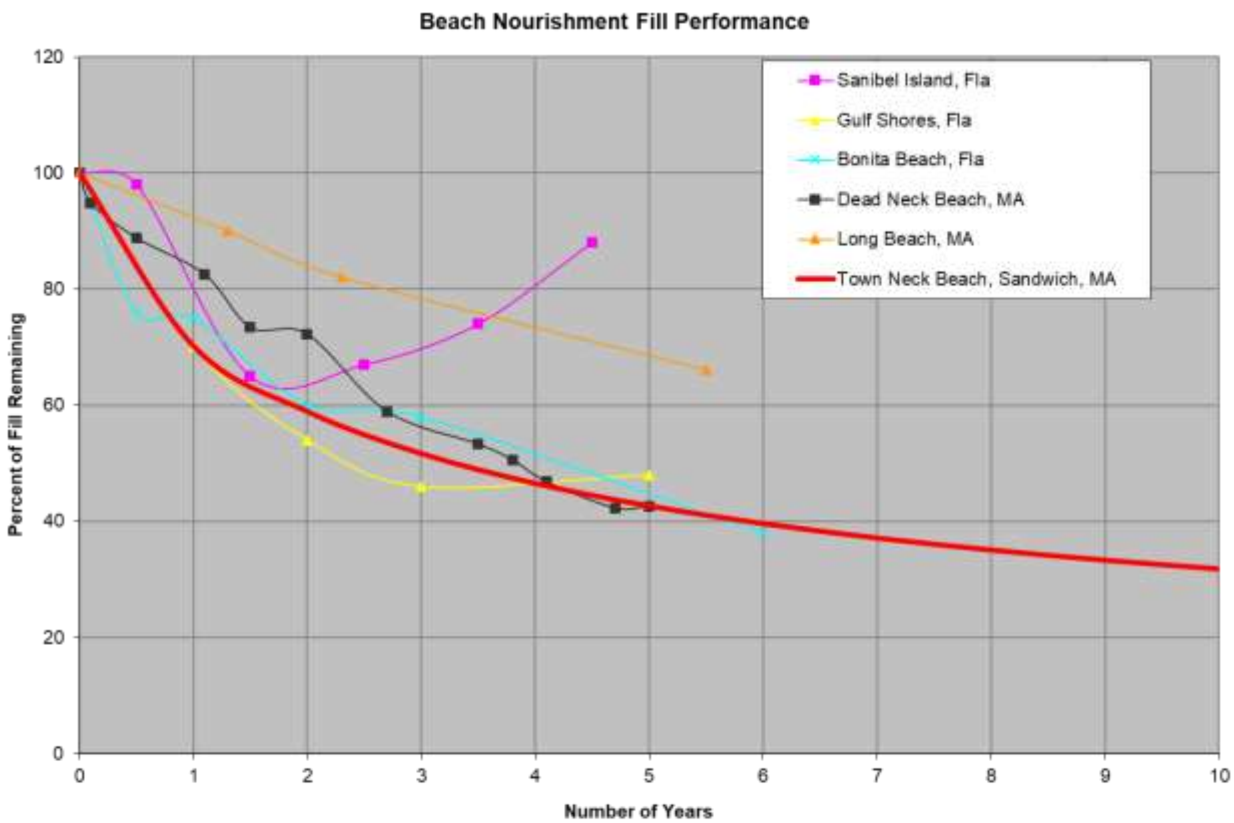


**Figure 38. Comparison of projected Town Neck Beach nourishment performance to monitored beach nourishment performance.**

The computer model chosen to perform the beach cross-shore evolution was XBEACH, as described in Chapter 4. The assessment indicated that once the initial nourishment has decayed to width of approximately 30 feet, a 10-year storm event could cause significant overtopping of the dune system and potential upland damage. The critical width threshold was then applied to the performance curves to determine the estimated renourishment interval. Figure 39 presents the beach berm width remaining in years after the nourishment with an initial berm width of 100 feet (which is an overall average of the proposed berm with in the permitting beach template). A horizontal dashed line at the critical width value of 30 feet and a vertical line at the 9-year lifetime (a reasonably cost-effective renourishment level) are also presented on the figure.

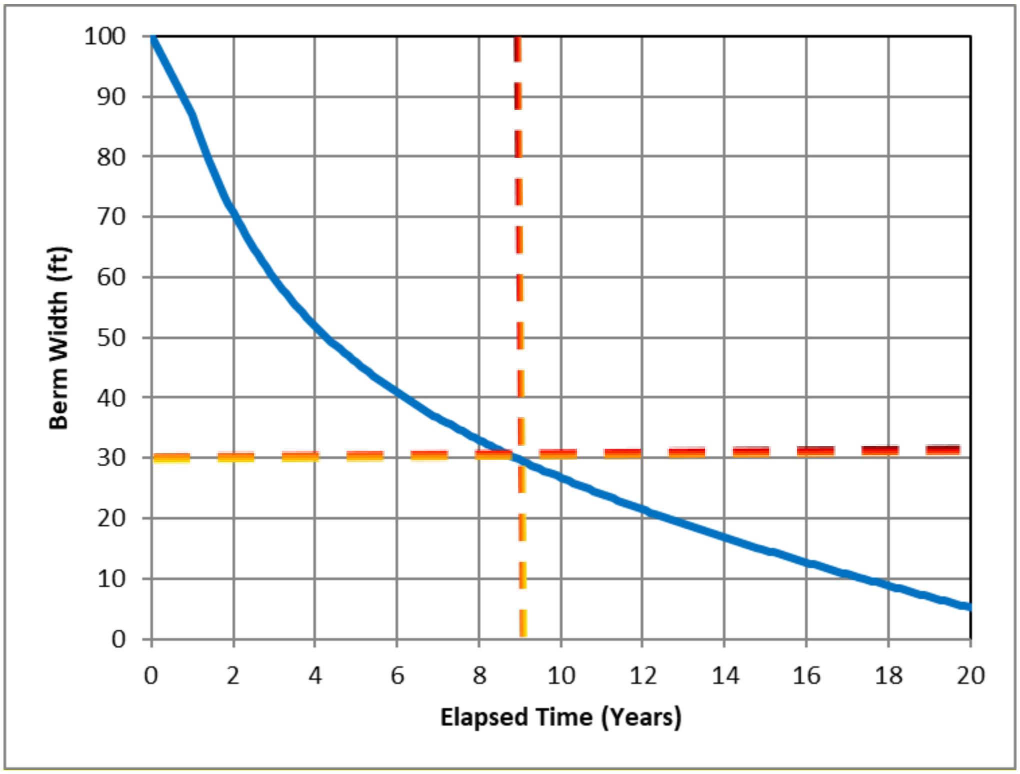


**Figure 39. Beach berm width (in feet) for an initial 100-foot berm width at Town Neck Beach. The horizontal axis presents years after nourishment. The horizontal line presents the critical berm width level (~30 feet), while the vertical line presents the lifetime expectation requirement (~9 years).**

### 5.4 BEACH NOURISHMENT WITH DUNE COIR ENVELOPES

The alternative is similar to the beach nourishment only alternative, in that it proposes beach nourishment and dune restoration along the entire 5,000-foot length of Town Neck Beach. However, this alternative also includes biodegradable sand filled coir envelopes within the dune to act as a last line of defense and dune enhancement (Figure 40). These coir envelopes would provide additional storm damage protection and could either be incorporated into the core of the dune, or simply as toe protection along the seaward toe of dune. For this assessment, the coir envelopes are assumed to be utilized as a core to the dune.

With the presence of coir envelopes within the dune, the dune may not erode as quickly once the dune erodes back to the location of the coir envelopes. However, when erosion does occur, these envelopes would likely be exposed. Once exposed, the envelopes would impede foraging habitat for threatened and endangered shorebirds present at Town Neck Beach, would breakdown fairly quickly due to UV light and salt water exposure, and require ongoing dune maintenance. Examples from a similar biodegradable erosion control project in Massachusetts are shown in Figures 41 and 42. Figure 41 shows a close-up of the coir envelopes during construction, prior to being covered with sand. Figure 42 shows the same area after a storm. Although the bank remains largely intact due to the presence of the envelopes, the overlying sand was eroded away, leaving the envelopes exposed.

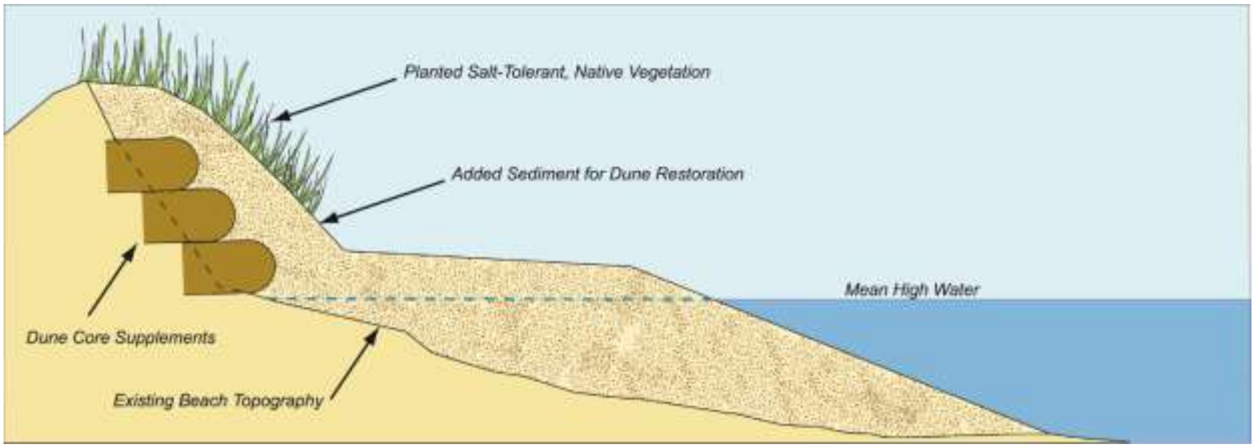


**Figure 40. Example schematic of potential coir envelopes acting as dune core supplements.**

**Figure 41. Biodegradable coir sand filled bag during construction.**

This alternative would result in similar performance as presented for the beach nourishment alone, but would also place biodegradable sand filled coir envelopes within the dune for the length of the project area. Although this would increase erosion protection along Town Neck Beach, it would be at the expense of potential long-term adverse impacts to bird habitat, and could also further disrupt the natural sediment transport along this coastline. Because the sand within the envelopes does not erode during a storm, downdrift beaches such as Springhill Beach could also be deprived of sediment normally transported alongshore. Additionally, the coir material typically has a lifetime of 5-7 years before they start biodegrading (less if the coir is regularly exposed), at that point the system needs to be replaced or maintained. In most situations, the coir envelopes function to provide protection during one moderate storm condition, but would require repairs prior to a subsequent storm event.

**Figure 42. Example of coir envelopes exposed after storm.**

The performance lifetime of the beach nourishment project with coir envelopes remains essentially the same as the beach nourishment alone since the coir envelopes are not integrated in the beach nourishment, but act as the last line of defense within the coastal dunes. As such, the results of the beach nourishment performance, as presented in Figure 37, do not change when including coir envelopes. However, assuming the coir envelopes are robust enough to survive a moderate level storm, they do slightly improve the critical width requirements associated with the overall beach and dune system restoration. Based on cross-shore modeling, the coir envelopes were shown to provide some small incremental improvement to the critical width level, indicating approximately 8-10 feet less width is required. This would indicate that there is a potential improvement of approximately 2 years (to a total of 11) for the service life beach and dune restoration project prior to required maintenance of the beach system. Additionally, the actual biodegradable coir envelopes components likely need to be serviced every 5-7 years. Costs associated with the installation of Coir envelopes can range from approximately $1,000 to $2,500 per linear foot of installation. After a significant storm event, if the envelopes were to fail, total replacement would be required with costs of the same scale as the initial installation.

## 5.5 NORTH JETTY MODIFICATION

The Cape Cod Canal Jetties, constructed at the terminus of the CCC to maintain the federal navigation channel, cause significant impacts to the sediment transport patterns on the adjacent beaches. The northern jetty situated on Scusset Beach impounds considerable quantities of sand along the beach, which has resulted in accretion along this portion of Scusset Beach since the construction of the jetties (Figure 1). The considerable length of the northern jetty causes most littoral drift to be impounded and accreted on Scusset Beach. Any sediment that does make it over, around, or though this northwestern jetty is likely deposited in the Cape Cod Canal. Therefore, there is an insignificant amount of sediment that actually is transported further downdrift (i.e. Town Neck Beach and Springhill Beach). To alleviate this and enhance natural bypassing of sediment around the Cape Cod Canal, this alternative evaluates the potential shortening of the northern jetty. Shortening of this jetty may allow additional sediment to be transported past the jetty, while decreasing the amount of sediment impounded behind the jetty. However, this potential increase in bypassing also must be weighed against the fact that modification of the northern jetty would also likely increase sediment deposition in the CCC, increase navigational dredge requirements, and increase wave energy at the entrance of the east end of the CCC. This alternative would require removal of a portion of the seaward end of the existing jetty, including all armor units and underlayers. A number of shortening lengths were considered and evaluated at a semi-quantitative level, but based on the existing shoreline conditions and specifically the impoundment at Scusset Beach, a shortening distance of 550 feet was utilized in this assessment (Figure 43).

In order to assess the potential impacts of shortening the northern jetty, the existing conditions wave model was altered to represent the alternative conceptual design. The topography of the wave model grid was altered to that of the new reduced-length jetty so as to provide a representation of how wave conditions may be altered by this alternative. Typical wave conditions, as well as storm conditions, were simulated using the updated model and compared to existing condition wave model results. Results from the model simulations were then utilized to assess the performance of the alternative relative to existing conditions. Figures 44 and 45 show example comparisons between the results for the annual wave approach bin from a northeast direction. Figure 44 shows the difference between wave height for the alternative and existing conditions with positive wave height change indicates larger wave energy with the jetty modification (i.e., red areas show areas of increased wave energy for the jetty modification alternative). Figure 45 shows a similar figure with a comparison of wave direction. Greater changes in absolute wave direction are shown as larger differences (reds). Wave vectors for both simulations in this figure allow for determination of the direction of change.

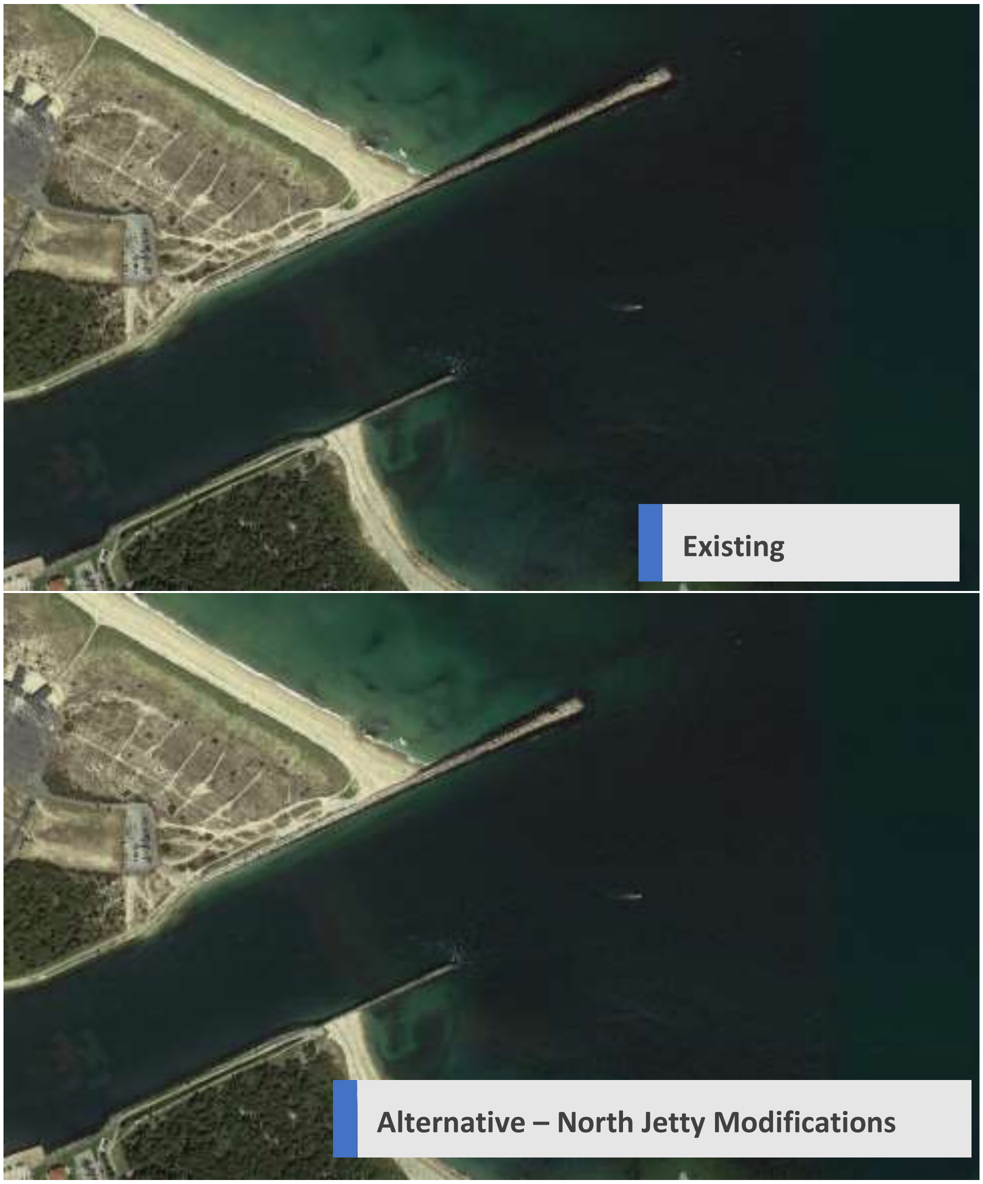


**Figure 43. North jetty modifications alternative.**

Wave model results show that by shortening the northern jetty, wave activity will increase in the CCC entrance primarily for waves approaching form a northeasterly direction, as well as due to storms (such as nor'easters). Figures 44 and 45 show that increased wave energy is expected both in the canal, as well as near to Town Neck Beach, directed at the long-term stable region between the CCC and the longer existing groin seaward of Dillingham Avenue. Qualitatively, these model simulations show that while shortening the jetty may cause increased littoral transport from offshore of Scusset Beach; this alternative will also potentially result in increased erosion at the western end of Town Neck Beach. Additionally, this alternative will also increase the wave energy at the entrance to the Canal, which may pose an increased navigation hazard.

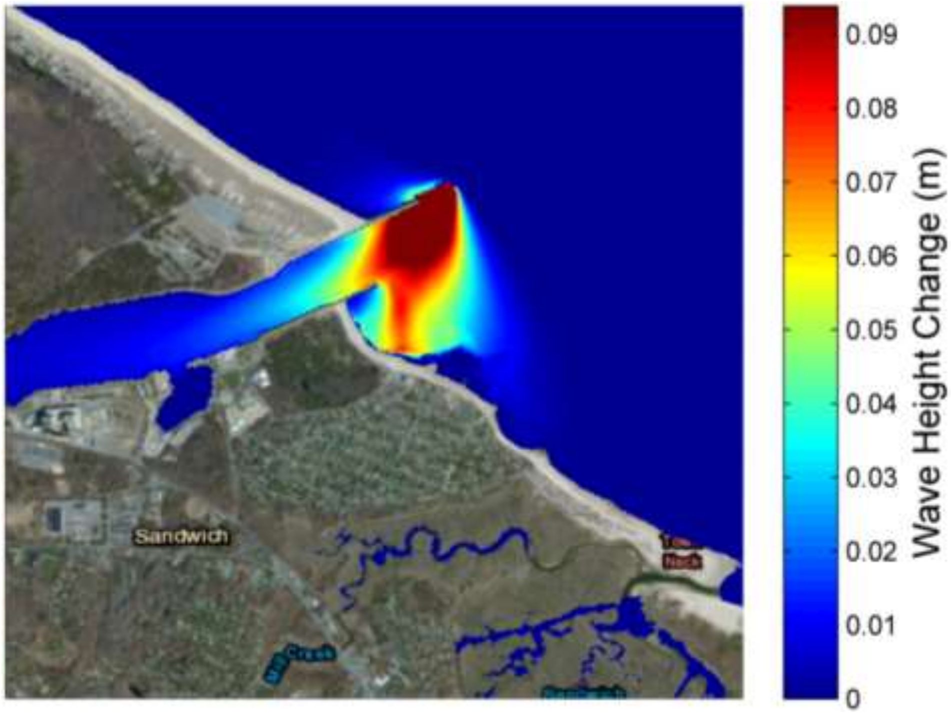


**Figure 44. Comparison of wave heights for the north jetty modification alternative. Positive wave height change means higher waves with the jetty modification.**

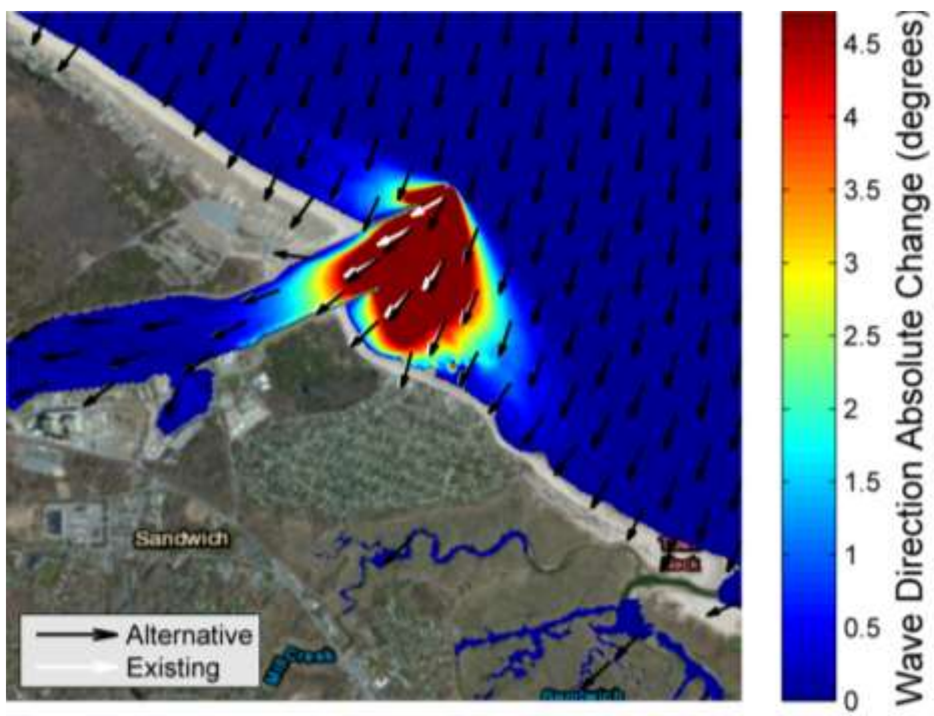


**Figure 45. Comparison of wave direction results for the north jetty modification alternative. Vectors show the direction of the waves, while the color variation shows the directional change magnitude.**

To provide a preliminary evaluation of the potential increase in littoral transport that may be expected by shortening the northern jetty, an analysis of the cross-shore distribution of the alongshore transport was performed using relationships proposed by Longuet-Higgins (1970, 1970a). Using the cross-shore distribution, the effect of a shore-perpendicular structure on reducing or increasing the alongshore sediment transport can be estimated.

The cross-shore velocity distribution is dependent on horizontal mixing, which is the result of waves breaking at different locations and wave-induced eddies varying the profile of the cross-shore velocity distribution. To account for this mixing, a quadratic equation is used to create a typical cross-shore flow profile. The shape of this profile is dependent on the known variability of the wave conditions and a horizontal eddy parameter which was experimentally derived by Longuet-Higgins (1970). Figure 46 presents a schematic representation of the long-shore velocity profile as a function of the normalized offshore distance to the wave-breaking line. The broken line represents the distribution without mixing. After applying the mixing coefficient calculated by Longuet-Higgins, the distribution looks like the solid line. The area under both lines equals to the volume flux rate, Q.

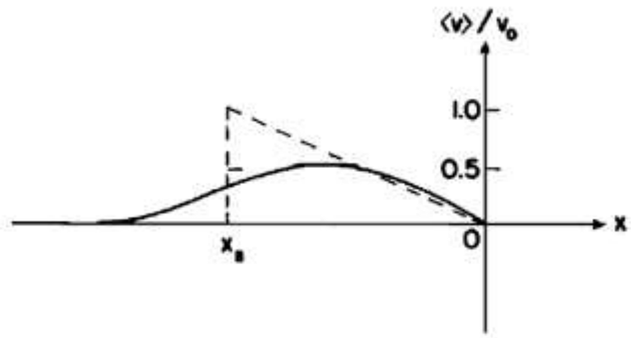


**Figure 46. Example cross-shore distribution of alongshore velocities.**

The alongshore current on Scusset Beach, which was calculated using a process-based sediment transport model (detailed in Chapter 4), indicated a net, potential sediment flux of approximately 95,000 to 115,000 cy/year. Of this volume, approximately 54,700 cy/year is deposited on the beaches along that stretch of shoreline (see Chapter 3), impounded behind the northern CCC jetty. The purpose of evaluating the cross-shore distribution of alongshore current for this alternative is to determine what portion of the sediment currently being impounded behind the jetty would be free for transport if the jetty were decreased in length. Figure 47 shows the Longuet-Higgins quadratic alongshore distribution to the sediment flux approaching the northern jetty of the CCC.

The distribution is calculated based on site-specific physical processes data (e.g. WIS hindcast information, cross-shore topography profiles, numerical modeling results, etc.). The distribution can be applied to assess the amount of littoral transport that may be intercepted by structures of varying lengths. Using this distribution, only about 74 cy/year is theoretically able to bypass the seaward end of the existing jetty. As discussed in Chapter 3, additional material is transported through and over the jetty and makes its way onto the shoals in the CCC.

Decreasing the length of the jetty by 550 ft (from its current length of approximately 1400 ft), theoretically only adds 160 cy/year around the seaward end (for a total of approximately 234 cy/year). This is not unexpected since a majority of the alongshore flux (both current and sediment) occurs within the nearshore portion of the surf zone, rather than at the depths of the seaward end of the jetty (even with a reduced length). This analysis suggests that the majority of the sediment being transported past the northern jetty under existing conditions (approximately 34,600 cy/year to 36,400 cy/year according to the sediment budget) is not going around the jetty, but rather is being transported through the porous jetty or being transported by wind and wave overtopping processes over the landward portion of the jetty. Both of these processes would not be expected to be enhanced significantly by shortening the jetty.

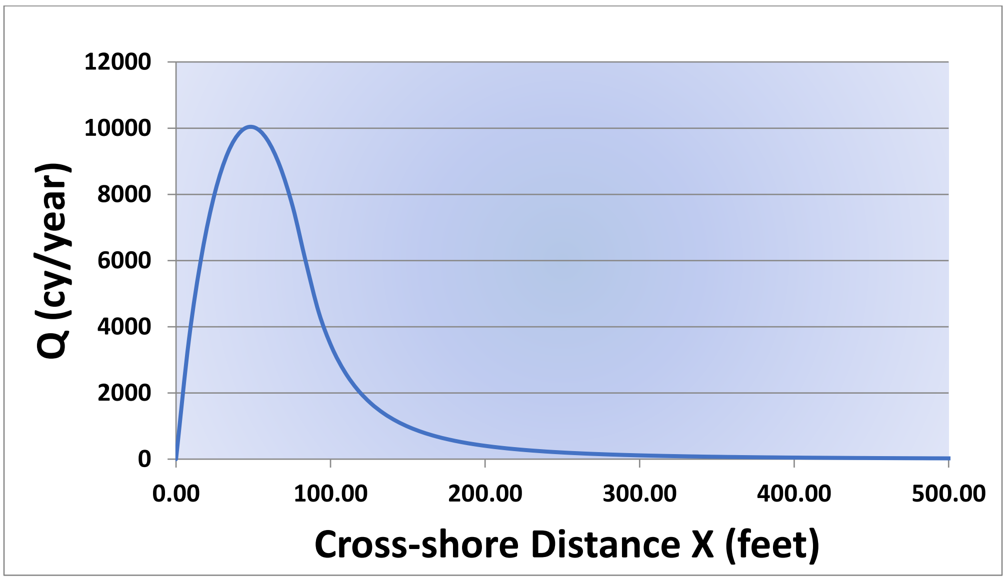


**Figure 47. Cross-shore distribution of the alongshore flux for Scusset Beach.**

Although a decrease in the length of the northern jetty may slightly increase littoral transport past the jetty, it may also produce negative effects that should be considered. Based on this analysis, the maximum gain in bypassing that could be expected under this north jetty modification alternative is approximately 160 cy/year. Additionally, Borelli et al. (2016) noted that most of the sediment bypassing the northern jetty was likely "being deposited beyond the extent of the lidar (~10 m water depth) and the commonly accepted wave base for a moderately energetic shoreline." This observation suggests that any sediment bypassing the existing, or shortened jetty would most likely be deposited in the deep channel, or in the CCC itself, and not available for transport on the downdrift beaches. The required maintenance depths of the CCC limit the ability of sediment to bypass even without the influence of the jetties. As such, any sediment being mobilized due to the north jetty modification would likely just cause increased shoaling, and thus increase dredging costs.

### 5.6 SOUTH JETTY MODIFICATION

Similar to the northern jetty, the southern CCC jetty causes significant changes to patterns of sediment transport on the surrounding beaches. The southern jetty (in conjunction with the northern jetty) causes a local sediment transport reversal (net transport to the northwest on Town Neck Beach due to the wave energy shadow caused by the CCC structures. This reversal, which is relatively small in magnitude due to the lower wave energy from other non-dominant directions and the lack of sediment availability, extends approximately from the CCC to Knott Avenue in Sandwich, MA. This wave shadowing effect is visible in the wave transformation modeling, and the reversal is identified in the sediment transport modeling.

Presently the southern jetty impounds some portion of this net reversal of sediment drift. Evidence of this impoundment can be seen in the shoreline change data discussed in Chapter 2, as the portion of the beach directly downdrift of the jetty has experienced less erosion than the rest of Town Neck Beach. Additionally, review of aerial photographs of the site show the formation of a small fillet to the east of the southern jetty. The portion of the reversal littoral drift that is not impounded behind the jetty is likely deposited within the canal, as discussed in the sediment budget analysis (Chapter 3). The motivation behind this alternative is to attempt to capture all of the sediment that may be transported back into the CCC from the eastern side of the Canal. As such, the southern jetty could be lengthened to capture a larger portion of the littoral drift; thereby allowing more sediment to remain on Town Neck Beach. Again, a number of jetty lengthening scenarios were considered, with a final lengthening of approximately 900 feet assessed for this alternative (Figure 48).

In order to assess the potential positive and/or negative impacts of lengthening the southern CCC jetty, the wave transformation model created for existing conditions was altered to represent this alternative. The topography of the existing conditions wave model grid was altered to include the new extended southern jetty. Typical wave conditions, as well as storm conditions, were simulated using the updated model grid to allow for direct comparison. Results from the alternative model simulations were compared to existing conditions model simulations to qualify the impact of the alternative design. Figures 49 and 50 show a comparison of the results for an annual approach direction bin with waves arriving from the northeast direction. Figure 49 shows the difference between wave height for the alternative and existing conditions with positive wave height change indicating larger wave energy with the jetty modification (i.e., blue areas show areas of increased wave energy associated with existing conditions). Figure 50 shows a similar figure with a comparison of wave direction. Greater changes in absolute wave direction are shown as larger differences (reds). Wave vectors for both simulations in this figure allow for determination of the direction of change.

Wave results show that by lengthening the southern jetty, wave activity caused by waves from a northeasterly direction, as well as due to storms with strong northeasterly winds (such as nor'easters) will decrease in the shadow of the jetty. Figure 49 and 50 show that decreased wave energy is expected in the shadow of the canal.

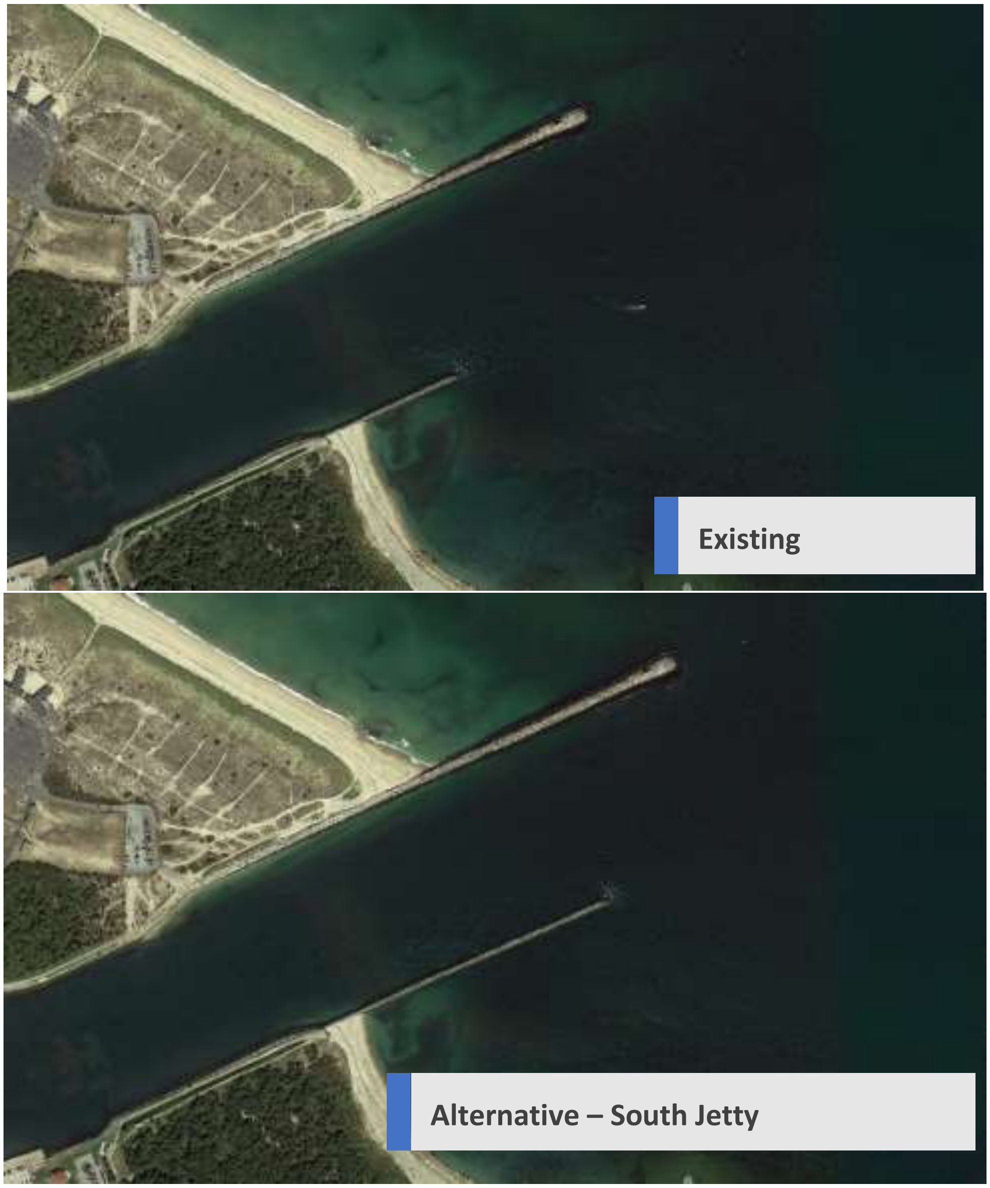


**Figure 48. South jetty modification alternative.**

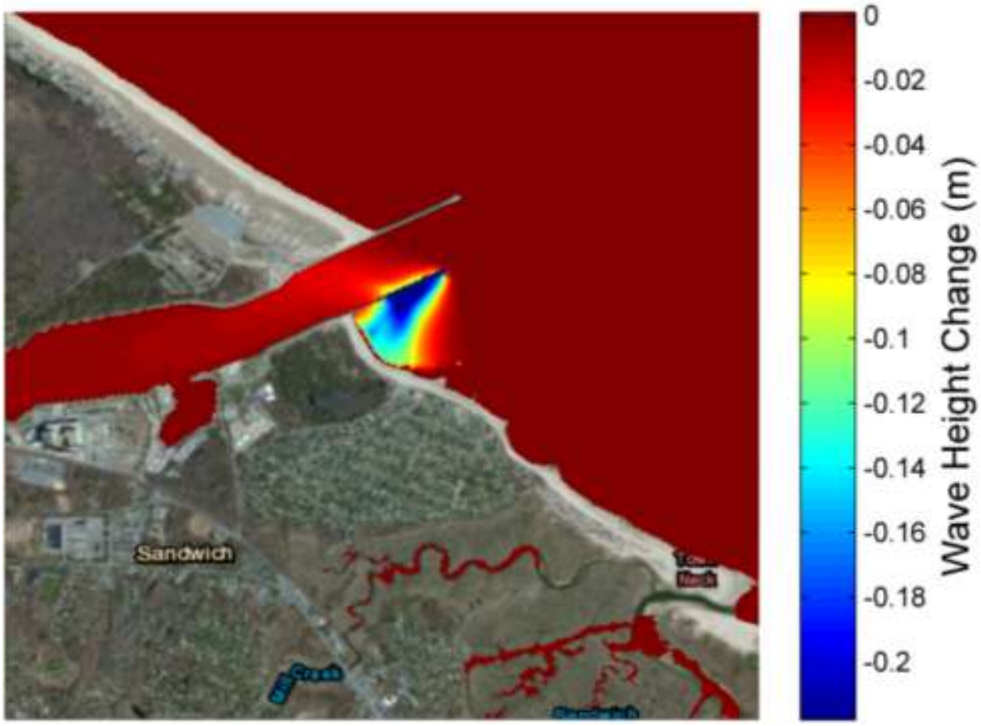


**Figure 49.** **Comparison of wave heights for the south jetty modification alternative. Positive wave height change means higher waves with the jetty modification.**

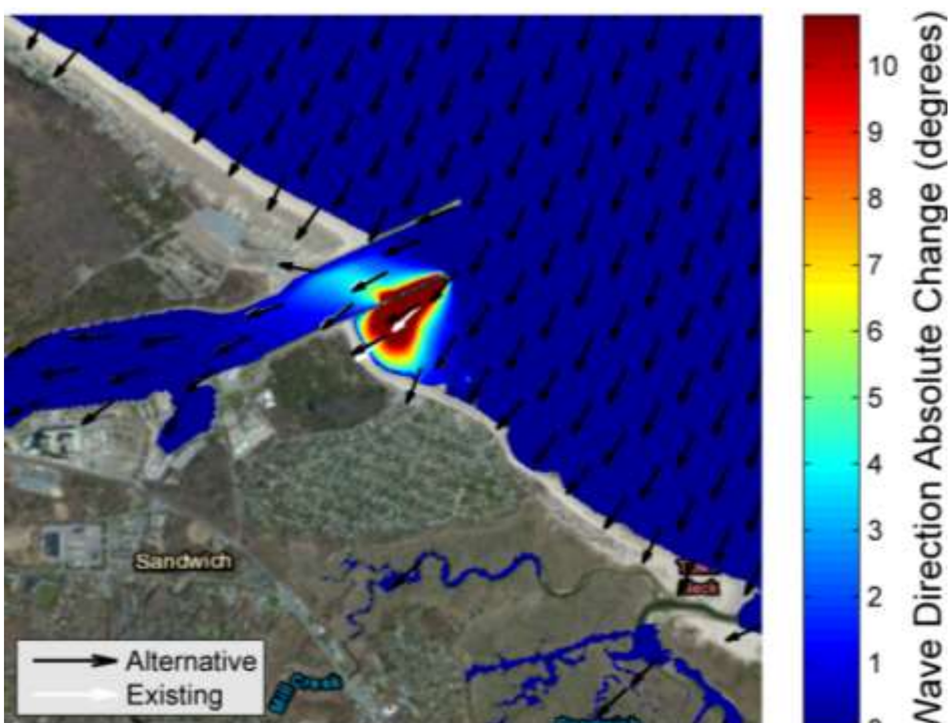


**Figure 50.** **Comparison of wave direction results for the south jetty modification alternative. Vectors show the direction of the waves, while the color variation shows the directional change magnitude.**

To provide a quantification of the potential volume of material that would be impounded behind the larger southern jetty, the cross-shore distribution of the alongshore transport was evaluated using relationships proposed by Longuet-Higgins (1970, 1970a).

The alongshore current caused by the reversal on Town Neck Beach was calculated using the process-based sediment transport model explained in chapter 4, which indicated potential transport rates of 10,000 to 20,000 cy/year. As calculated in the sediment budget portion of this report, approximately 1,500 cy/year to 3,300 cy/year is transported by the reversal into the canal. There are two likely causes of this smaller volume. One, is that while the reversal is capable of transporting 10,000 to 20,000 cy/year of sediment, this volume of sediment may not be available for transport on the sediment starved beach. Two, the remaining volume of sediment is already being captured by the existing jetty and is being retained on the beach at some level. Figure 51 shows the Longuet-Higgins quadratic alongshore distribution for the sediment flux to the southern jetty calculating using a conservative volume of 20,000 cy/year (which assumes this whole volume is available for transport).

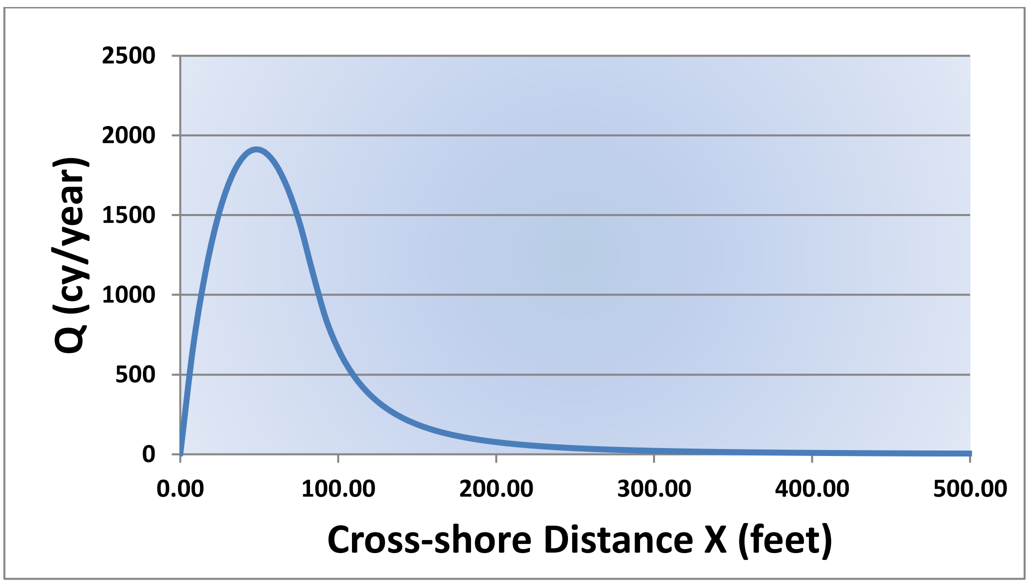


**Figure 51. Cross-shore distribution of the alongshore flux on the Town Neck Beach side of the CCC.**

Using this distribution (Figure 51), it was determined that approximately 19,920 cy/year is captured by the existing jetty length (80 cy/year bypasses the end of the southern jetty and deposits in the CCC). This calculation suggests that the majority of the littoral drift is already captured by the existing jetty length and most likely is why this area directly adjacent to the south jetty has remained somewhat stable. Therefore, lengthening the jetty would not significantly alter the sediment dynamics on the south side of the CCC. It is unlikely that significantly more sediment would be captured by a longer jetty. Even if all the sediment that may be moving back into the CCC from the downdrift side is retained on Town Neck Beach, this is a minimal amount of sediment that would not justify the extension of the south jetty.

### 5.7 TOWN NECK BEACH GROIN AND OLD HARBOR INLET JETTY MODIFICATIONS

Town Neck Beach is situated between two inlets that include jetties. The western end of the beach is adjacent to the Cape Cod Canal, while the eastern terminus of the beach is bounded by the Old Harbor Inlet (formerly contained between two jetties). Town Neck Beach also includes 9 groins along the shoreline, in various conditions, lengths, and sizes. Some of the groins are completely detached from the shoreline and have little impact on the transport of sediment, while others have a significant influence on the processes along the beach. Modification to these structures, such as the removal or shortening of the Old Harbor jetty or modifying the groins, may result in increased beach nourishment performance, as well as improved natural transfer to downdrift shorelines (e.g., Springhill Beach).

This alternative investigates the possibility of supplementing the beach nourishment with integration of coastal engineering structures (specifically enhanced groins) along the shoreline. The beach nourishment and dune restoration would still span the 5,000-foot length of Town Neck Beach, but this alternative also includes repairing and enhancing the existing groin structures and the Old Sandwich Harbor jetties. The intent would be to extend the service life of the beach and dune restoration at Town Neck Beach, while still allowing or improving natural transport towards Springhill Beach. For example, the potential removal of the Old Sandwich Harbor jetties would allow more sediment to make its way naturally towards Springhill Beach, while optimization of the existing groins may help improve beach nourishment performance and improve natural transport (e.g., through lower profiles, notches, etc.).

The existing groins and jetties were evaluated for their integrity, condition, and functionality. A number of potential alternatives were considered, including but not limited to:

- removing all the groins and jetties
- removing some of the groins
- adding or replacing individual armor units
- groin tightening
- constructing more groins
- rebuilding the groins at a lower profile
- constructing new groins with notches

In the process of assessing a number of various configurations, the intent of the structural manipulations was focused on (1) optimizing the groins ability to retain sediment at an improved capacity, and thus increasing the service life of the beach and dune restoration project such that the renourishment interval would be extended, and (2) improving or enhancing the ability of sediment to migrate downdrift to Springhill Beach.

After a number of model iterations, the optimized configuration involved removing two existing non-functional jetties at Old Sandwich Harbor, as well as deconstructing the five groins along the eastern portion of Town Neck Beach. Beneficially reusing this material, four (4) new notched groins were constructed on the eastern portion of Town Neck Beach. They would be approximately 250 ft in length (across shore) and include low profile notches along the shoreline in the intertidal zone. The position of the new notched groins is shown in Figure 52.

As a rough estimate, there is approximately 1,800 linear feet of existing groin/jetty material that would be available to repurpose into a new groin structure. Assuming a retention/reuse rate of approximately 33% (i.e., only 33% of the existing material is of adequate size and condition), there would be approximately 600 linear feet of adequate material to construct the new notched groins along the beach. Therefore, additional material would be required to construct the new notched groins.

The 4 groins on the western part of Town Neck Beach would remain in place. These groins lie updrift (west) of the largest groin that has stabilized the western portion of Town Neck Beach. As discussed throughout this report (Chapter 2), this area has been historically stable over the long-term. As such, it is assumed that removal of these groins would destabilize the entire western section of Town Neck Beach.

Based on a study by Donohue, et al. (2004), groin notching appears to be optimized if the notch length is approximately 1/5 of the overall length of the groins themselves. Therefore, the notches in the groins were designed to be approximately 50 feet in the cross-shore direction. There is limited information on where in the overall groin length a notch should be placed. Donohue et al. (2004) indicates that a notch placed closer to shore performs slightly better. For this alternative the groin notches were placed between 75 to 125 feet from the landward portion of the groin. Figure 52 presents the proposed layout for this alternative. The proposed groin field should not be constructed without the nourishment. The groin field is intended to be a supplement to the beach nourishment.

In order to assess the potential positive and/or negative impacts of constructing new groins, the wave transformation model was altered to represent the alternative conceptual design for the new groins. The topography of the existing conditions wave model grid was altered to include the new groins. Typical wave conditions as well as storm conditions were simulated using the updated model grid to allow for direct comparison. Results from the model simulations were compared to existing conditions model simulations to qualify the impact of the alternative design. Figure 53 shows an example of the comparison of the results for a simulation of a 50-year storm event (waves from a northeast direction). The difference in the wave heights are shown in the color distribution, with positive wave height change indicating larger waves for the alternative scenario.


**Figure 52. Old Sandwich Harbor jetty removal and groin notching alternative.**

Wave transformation modeling results show that by removing the existing groins, and building new groins, wave energy increases in some locations and decreases in others. Some of the increases correspond to where groins have been removed, and not replaced. The figure shows that where the groins have been built, there is an increase in wave energy near the seaward terminus of the groins. This wave transformation data were then used to develop the physics-based sediment transport model results, such that the alternative scenario results could be used to determine the improved performance and bypassing influence of the notched groins.

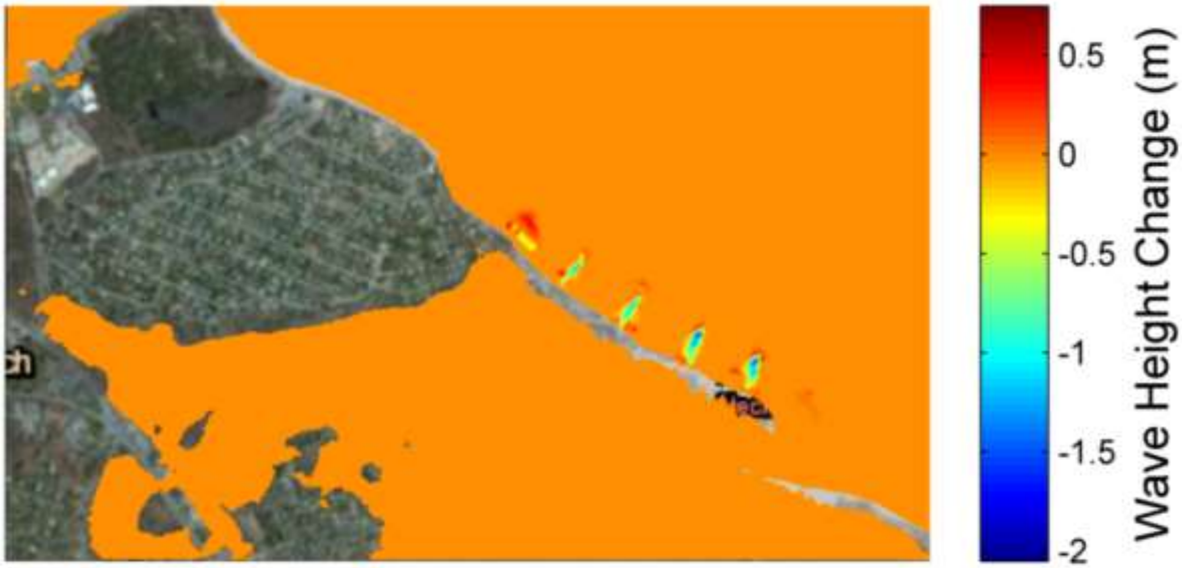


**Figure 53. Wave simulation wave height results comparison for. Positive wave height change means higher waves with the alternative in place.**

The notched groin field is incorporated into the performance analysis by determining their influence on the sediment transport rate. Wave and sediment transport model simulations were performed with the modified structures in the modeling grid. Figure 54 presents the beach performance improvements resulting from the structural influences combined with the beach and dune nourishment alternative. The vertical axis presents the percent of the nourishment volume remaining in the original template area, while the horizontal axis shows time (in years). This does not indicate that the sand placed is lost from the system, rather that the material has dispersed out of the original template area. The blue line shows the performance of the beach and dune restoration project alone (same as in Figure 37), while the green line shows the performance of the beach and dune restoration combined with the 4 notched groins. The structural influence shows some gains in beach nourishment performance. For example, the additional modified notched groins increased service life by approximately 4 years when 30% of the initial fill remains. Using the critical width criteria developed for the beach nourishment, the notched groins extend the renourishment interval by approximately 4.5 years. Therefore, instead of replenishments being required every 9 years (nourishment alone), replenishments would be required every 13.5 years.

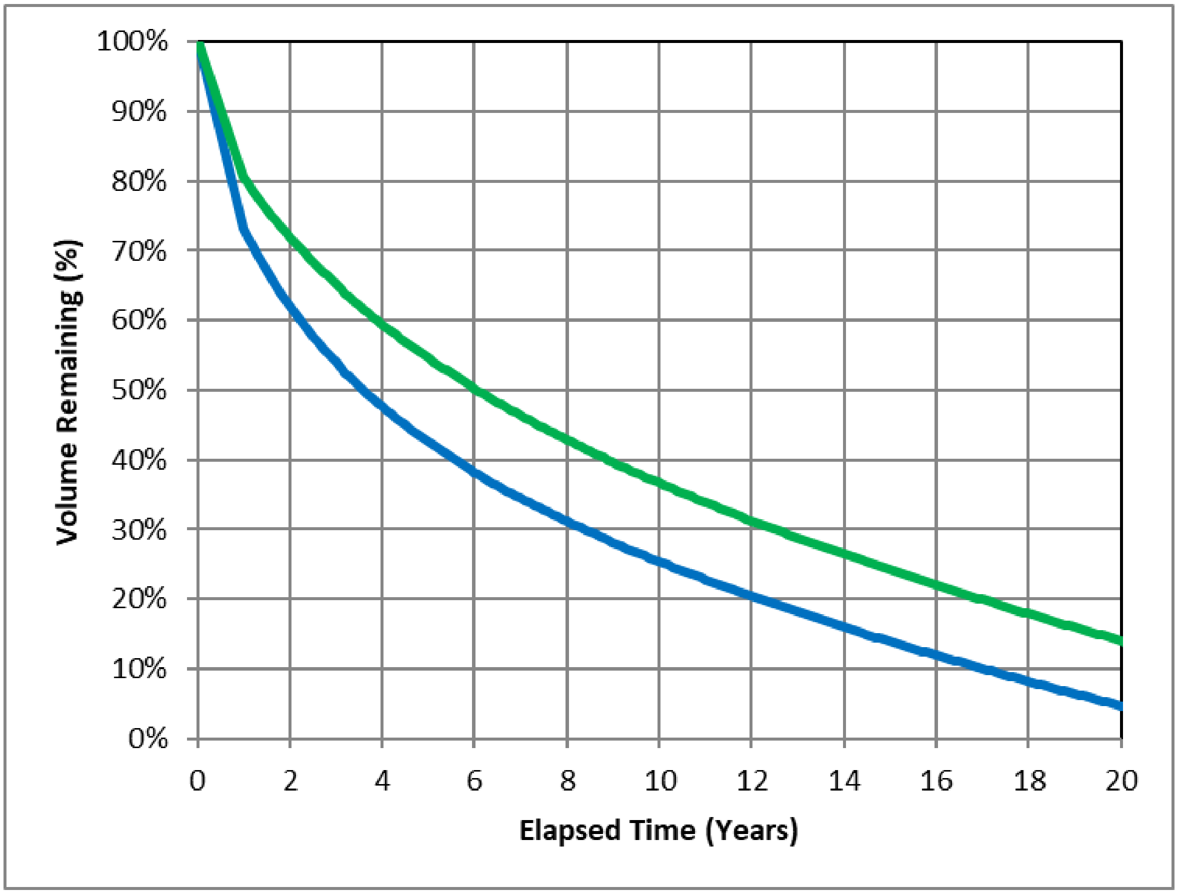


**Figure 54. Performance of the beach nourishment alternative (blue line) and the beach nourishment with notched groin field (green line) at Town Neck Beach. The vertical axis represents the percent of fill remaining in the initial template area.**

### 5.8 REVETMENT/SEAWALL

Seawalls and revetments separate land from water, with the primary function of protecting the upland from the erosional pressures of waves and currents. Seawalls are typically vertical structures (Figure 55), constructed with steel sheets or concrete. Revetments are sloping (Figure 56), constructed of concrete or quarry stones. Higher energy environments generally dictate the use of a seawall instead of a revetment; however, these two types of structures interact with the nearshore littoral processes in a similar fashion.

**Figure 55. Example of a typical seawall.**

Unlike groins and breakwaters, which may protect adjacent updrift beaches or improve the longevity of a beach fill, seawalls and revetments only protect the land directly behind them. If there is no beach fronting the structure, or if the beach is overtopped by storm flooding and wave action, a continual lowering of the profile in front of the structure will generally occur. This is due to the magnified erosional forces of the waves as they reflect from the structure, and to the loss of bank or dune sediments protected by the wall, that otherwise could help replenish the fronting beach. In an erosive environment, a seawall or revetment may actually accelerate the recession rates of adjacent beaches. In addition, toe scour and flanking at the ends of the wall may threaten the structure itself as erosion continues. Additional forces threatening the structure may be induced if the structure is overtopped, as soil becomes saturated and soil pressure is increased.

**Figure 56. Example of a typical revetment.**

Considering these complications, a seawall or revetment can be a successful protection measure when set back far enough from the ocean, and if the elevation of the structure is sufficiently high to prevent regular wave overtopping. Under these circumstances, a seawall or revetment provides protection from rare severe erosive forces. Installing a seawall back from the ocean often requires additional measures to build and maintain a beach in front of the structure. However, for Sandwich beaches, combined costs of beach maintenance and seawall construction would be economically prohibitive. Additionally, on the barrier beach of Sandwich Town Neck, there is limited to no room to construct and maintain a seawall or revetment structure. For a recreational beach setting like Sandwich Town Neck Beach, where maintaining a beach is important, a seawall or revetment would not be a preferred solution. Finally, given the valuable bird habitat area that exists on the Town Neck and Springhill beaches, seawalls and revetments would not be permittable. Therefore, seawalls and revetments are not recommended for further assessment.

### 5.9 BREAKWATERS

Breakwaters are designed to reduce wave action in the area leeward of the structure, and in turn reduce beach erosion. Typically, this type of shore protection is provided from a single large offshore rubble mound (rock) structure, or a series of shorter segmented breakwaters oriented parallel to the shoreline. A segmented breakwater (Figure 57) dissipates wave energy in its lee, and each breakwater allows for sediments to be deposited on the adjacent shoreline, forming a bulge in the beach defined as a salient. The wave climate and distance between the shoreline and the breakwater govern the salient growth. If the accreted sand makes contact with the breakwater, the formation is termed a tombolo.

The sources of the trapped sediment behind each breakwater are derived from the ambient littoral drift and the sediment transport induced by the diffraction pattern of the waves around the ends of the breakwater, which forces sediment toward the shadow zone behind the structures. Trapping the natural littoral drift is a concern because erosion of the downdrift beaches may result. Artificially filling the salients to an equilibrium planform (adding extra sediment seaward of the shoreline and landward of the breakwater) may prevent downdrift erosion for some finite period of time (until more nourishment is required), and the alongshore transport may continue, unaffected by the breakwater.

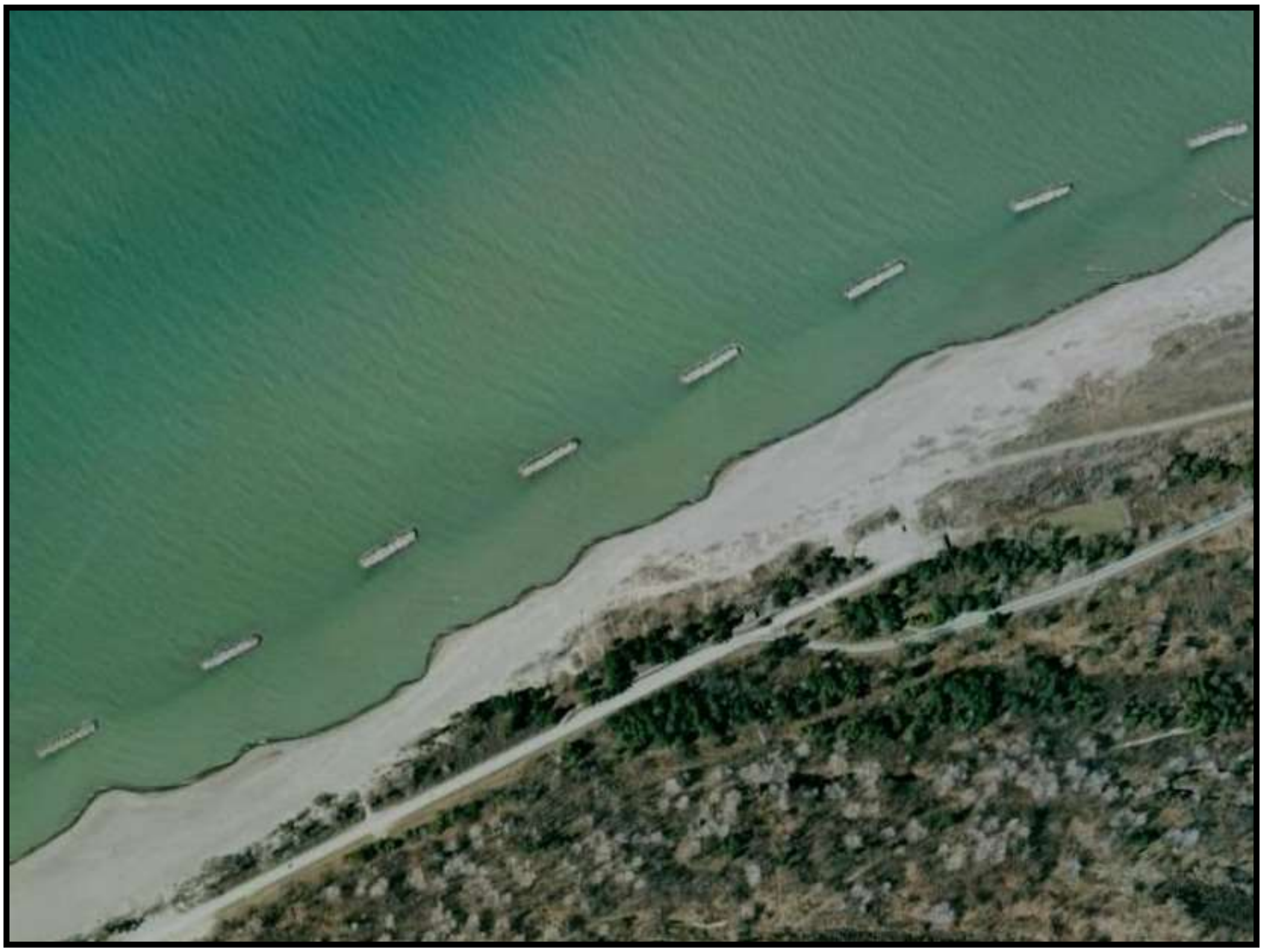

**Figure 57. Segmented breakwaters offshore of Presque Isle State Park, Erie, PA (Image from Google Earth).**

Determination of this equilibrium planform requires an accurate prediction of the salient growth behind a breakwater. A myriad of variables, spanning the natural littoral processes and wave conditions, as well as the properties of the structure, govern the shoreline response. For a single detached offshore breakwater, the reduction in sediment transport from the wave shadowing effect of the breakwater, the transport induced by the diffracted wave pattern, and the effects of wave energy transmitted through the structure must be weighed against the ambient sediment transport conditions to determine the shoreline response. A further consideration for a series of segmented breakwaters is the design geometry. The interrelated effects of each structure's length, distance from shore, and the gap between each structure relative to the incident wavelength determine the post-construction shape of the shoreline.

As with groins, breakwaters are a viable means of stabilizing the shoreline; however, there are typically significant adverse effects. Physically, there is the potential for downdrift erosion, which may be aggravated by the formation of tombolos that cut off alongshore sediment transport completely. Environmentally, alteration of bottom habitat and aesthetic beauty are also drawbacks. However, a properly designed system of breakwaters, where no tombolos form, may not inhibit alongshore transport as much as groins.

By understanding the environmental drawbacks of detached offshore breakwaters and designing them to mitigate these concerns, they may be a viable option to control coastal erosion. Unfortunately, the cost of breakwater construction in an open coastal region can be prohibitively expensive. To ensure that tombolos will not form, the offshore distance must be increased. It is also mandatory to construct the breakwaters far enough offshore to prevent impacts on the natural seasonal cross-shore transport of sand. This increase in offshore distance and water depth will directly impact the structure cost and environmental impact, since a breakwater constructed in deeper water will require more material. For example, for a typical trapezoidal-shaped cross-section rock breakwater, the construction costs are tripled (or more) when the depth is doubled. In deeper water, the footprint of the breakwater increases (at least 50% increase in footprint with doubled depth), and potential adverse environmental impacts are also increased. At Town Neck Beach, the high construction costs, interference with recreational uses, and difficulty with likely permitting outweigh the anticipated benefits, making detached offshore breakwaters infeasible. Therefore, although technically feasible, offshore breakwaters are not recommended for further assessment.

## 5.10 ALTERNATIVE TECHNOLOGIES

During the past several years, new shoreline erosion mitigation measures have been developed; these measures are often referred to as alternative technologies. In the context of this analysis, the term alternative technology refers to any erosion control measure that has not been extensively used in the northeastern United States. Dozens of alternative technologies have been implemented throughout the United States during the past several years, however, only a few have proven to be effective. Many of these technologies are based on principles similar to more accepted engineering methods. Some alternative technologies are based on sound scientific principles, and for certain conditions will induce accretion along a beach face. However, care should be exercised in applying these methodologies since each stretch of shoreline is unique.

### 5.10.1 Beach Dewatering

The primary goal of beach dewatering is to stabilize the shoreline by lowering the groundwater table. A beach dewatering system contains a series of pipes buried in the beach face through which water from the wave uprush is pumped from the beach. On a typical beach, the water table is governed by tidal fluctuations, groundwater flow from land, and the uprush of water in the swash zone (the zone of wave action on the beach, which moves as water levels vary, extending from the limit of run-down to the limit of run-up). Lowering the water table through beach dewatering at the shoreline theoretically may mitigate erosion problems in several ways. The process is analogous to the dewatering process used when excavating saturated soils, where the slopes are stabilized as a result of reducing the upward buoyancy force in the sand grains and through slight compaction as water percolates down through the soil. The decreased gradient between the lowered water table and sea level effectively decreases the outflow of water from the beach face, further stabilizing the berm and inhibiting offshore movement of sediment. Additionally, as sediment laden swash zone water is pumped into the beach face, erosion is prevented and small amounts of sediment may be accreted.

Beach dewatering projects using both gravity drainage and vacuum pumping systems have been designed and implemented at a number of sites. The most significant finding of these early cases is that dewatering systems may stabilize the beach, thereby providing an alternative for beach protection. However, the observed success of dewatering systems is limited to areas where an abundance of sediment is available. In the absence of a significant sediment supply, the effectiveness of beach dewatering is in question, and the technique cannot be expected to build a beach. The over-steepening of the beach due to the dewatering process indicates a change in the equilibrium profile shape meaning sand is captured on the upper portion of the profile. If the pumping process is discontinued, the beach profile can be expected to revert to its original equilibrium shape rather rapidly and transport this material offshore. Therefore, the beach that may have been built due to this temporary steepening of the profile, would be quickly lost during a readjustment of the profile. It is likely that this oversteepening may account for much of the accreted volume exhibited at the test sites.

In addition, the idea of beach dewatering raises a number of environmental concerns. First, the available literature does not adequately discuss the effects of dewatering systems on downdrift beaches. In a natural beach system, waves will tend to transport sand in the alongshore direction depending on the offshore wave angle with respect to the shoreline. Since beach face dewatering systems accrete sand by interrupting a portion of the natural littoral drift, downdrift erosion should be anticipated. Other concerns include high maintenance costs, and the potential for complete destruction of the system during major storm events.

A large beach dewatering project was initiated in the Siasconset area of Nantucket Island in 1994. This project has undergone at least one major redesign effort since its inception, and is still in the evaluation process. The construction and operation costs for this project have been significant, and to date the success of this technology at this site is unknown.

Due to the possible negative environmental impacts, the relatively high cost with respect to the potential benefits, and the unproven performance, this technology is considered experimental and is not recommended for Sandwich Town Neck Beach. The overall sediment deficit within the Sandwich region also argues against the use of beach dewatering.

### 5.10.2 Nearshore Berms

As an alternative to beach nourishment, sand may be deposited in the form of an offshore berm to act as a sediment source, or feeder berm for the beach. Although the best use of dredged material for shore protection is directly on the beach face in the form of nourishment, nearshore berms have been designed and implemented to make use of incompatible sediments that would normally have been transported to an offshore disposal site. Theoretically, the feeder berm serves as an offshore supply of sediment and a wave break that moves onshore during periods of low wave steepness, typically during the summer months. Depth of placement and grain size are important parameters for determining the behavior of the berm after placement. Wave forces cannot transport coarse material as readily as fine material. In addition, near-bottom velocities caused by waves are smaller in deeper water; therefore, the berm must be placed in depths where wave forces can transport the sediment.

The advantage of utilizing nearshore berms is their low construction cost. Dredged material can be easily dumped offshore to form a berm; however, the deposition of sediments must be within designed disposal area limits to assure shoreward transport. The deposition depth is also typically limited by the draw of the fully loaded barges delivering the material.

At this time, monitoring data from nearshore berm projects show that they have little measurable effect on beach stability. In many cases, poor results have occurred due to placing the berm too far from shore to facilitate onshore movement. Although placement of sand in nearshore berms is a better use of incompatible sediments than deep-water disposal, littoral transport of this material does not appear to affect beach erosion rates. Typically, incompatible sediments are too fine, and placement in the nearshore may introduce environmental problems associated with water clarity. For example, water quality may be temporarily reduced, and benthic organisms may be covered as the sediments settle. In cases where the nearshore berm sediments are too coarse, the wave climate is not able to move the sediments into the littoral system. Instead, the berm sediments remain offshore and have little influence over the nearshore sediment transport.

For the Sandwich area, nearshore berms most likely will not be beneficial as the deficit of sediment on the beach itself is so severe. Whenever possible, available beach sediments should be placed within the littoral system as beneficial reuse, and directly on the beach for cases where increased beach width is required for recreational purposes.

### 5.10.3 Submerged Offshore Reefs

Submerged offshore reefs and breakwaters are a variation of the breakwaters discussed above. In these instances, breakwaters are submerged to eliminate perceived aesthetic impacts. Various types of submerged breakwaters, such as rock structures, artificial reefs, and beach cones, have been developed to reduce erosional forces on the beach and/or prevent the loss of sediment from the nearshore. The theory behind these structures is to reduce the height of incoming waves by reflecting and dissipating energy as the waves propagate over the submerged structure. For sediment trapping purposes, the breakwater acts as a physical barrier blocking sediments from moving offshore.

Submerged offshore breakwaters are often rubble mound rock structures oriented parallel to the shoreline. Other designs include concrete shapes such as the Beachsaver (Creter, 1994) or Prefabricated Erosion Prevention (PEP; Mitchell, 1994) reefs that have been implemented on the Atlantic coast of the United States. Both of these reefs are constructed of prefabricated concrete modules, which can be interlocked to protect large sections of a shoreline. Beachsaver modules have a triangular profile shape, a saw-toothed bottom, and rough "stepped" seaward and landward slopes. Beach cones have been developed for more localized protection in low wave energy environments (Davis and Law, 1994), and consists of concrete cones arranged in pyramidal clusters, interlocked with interstitial wave blocks and anchored to the sea floor with PVC pipes.

A great deal has been learned about submerged breakwaters through laboratory and field testing. Major deficiencies include excessive settlement of the structures and an inability to achieve expected wave height reductions. The latter problem is exacerbated in storms because surge levels increase the water depth above the structure, allowing for higher than normal waves to break on the exposed beach. During storms, as much as 95 percent of wave energy may be transmitted past a submerged breakwater. In addition, laboratory experiments have indicated significant alongshore currents in the lee of the breakwaters (Browder, 1994). Although details of how this current might affect sediment transport are still being studied, initial indications show a net loss of sediment behind the structure with accretion at either end.

Submerged breakwaters can provide protection for beaches by dissipating wave energy during normal wave conditions, and combined with the advantage of their invisibility, these structures can potentially serve to mitigate beach erosion problems in a way that satisfies community interests. However, issues regarding environmental impacts remain unresolved. Locating "hard" engineering structures within the nearshore zone disturbs bottom habitat, inhibits recreation swimming and water use, and creates a potential navigational hazard. This alternative is not recommended for Town Neck Beach.

### 5.10.4 Additional Alternative Technologies

Over the last few decades, numerous other devices have been patented to prevent beach erosion. The types of alternative technology devices span a wide range of ideas, including beach cones (Davis and Law, 1994), ultra-low profile geotextiles injected with concrete (Janis and Holmberg, 1994), fishnets, stabilizers, and artificial seaweed (Stephen, 1994), and a host of additional innovative approaches. These alternative methods often employ nontraditional shapes or materials; however, they are positioned in traditional ways (e.g., to replicate a nearshore breakwater or groin). Ultimately, their potential success depends on their ability to resist storm impacts and maintain durability over a design life.

Many of these devices claim to have solved the coastal erosion problem through creation of a beach or capturing sand. In cases, some of these devices can be effective in capturing sediment, and test cases utilizing these alternative technologies have shown beach growth. However, these test cases lack corresponding long-term data documenting the source of the deposited sand. In order for sand to be built up along one stretch of beach, it must have been taken from somewhere else in the system (if sand is not supplied via beach nourishment). Without adding sand to the system, these devices are simply impacting adjacent beaches or the offshore environment by rearranging the existing sand in the active sediment transport zone, similar to groins, jetties, and breakwaters.

In order to compare alternative technologies to standard coastal engineering solutions, the alternative technologies must be thoroughly assessed to ensure that their performance is adequate from a technical standpoint. Technical assessment should include, at minimum:

- The alternative technology should be shown to maintain continued performance through the seasonal changes at a beach. For example, if a technology is put in place in the winter or spring, following the erosive storm season, the evaluation should consider the natural summer recovery of the beach. As the beach evolves to its summer profile, the build-up of the beach can create a temporary growth that may be misinterpreted as a success.
- Successful performance must be demonstrated with more than just before and after photographs. Long-term and large-scale measurement programs are required to validate the performance of the erosion control devices. This should include monitoring of not only the coastal site where the alternative technology is applied, but also of offshore and adjacent coastal regions to ensure negative impacts are not caused by the structure.
- The alternative technology must be able to withstand the forces of nature in open coastal environments. Engineering design and calculation should indicate that erosion control devices are able to withstand all the forces present during storms and the normal corrosion and fatigue associated with oscillatory wave action. In many cases, the erosion control devices are destroyed during storm events on the open coast.

In order to determine if an alternative technology is a reasonable approach for mitigating coastal erosion at a site, it must be carefully examined in order to ensure it is able to meet its promised function, minimize impact on the environment, survive for a predictable lifetime, and is cost effective. To further the development of innovative technologies, Pope (1997) raises a number of questions that should be considered when evaluating an alternative technology. Most of the technologies developed do not satisfactorily answer these questions. For example, some of the questions Pope (1997) poses include:

- Is the alternative technology heavy enough, especially considering the forces of storm waves?
- If the technology does fail, could the structural components become an environmental or public safety hazard?
- How will the technology perform and will it perform the way it is expected to perform?
- Will the technology be tolerant of erosion and scour effects?
- Will the technology be stable enough and anchored such that it doesn't fall apart?
- Does the technology perform as promised, and are there any adverse impacts to adjacent areas? Has this been document and shown using long-term data?
- What is the technology-effective life?

- How much will construction of the nontraditional or innovative system cost compared to more traditional methods?
- What are the design criteria?
- Is the material that is being constructed from survivable in a high-energy wave environment?
- What will it cost to remove the system (if necessary)?
- Has long-term monitoring of the performance of the alternative technology been conducted both at the site, as well as offshore and at adjacent beaches?

Nontraditional and innovative technologies need to be subject to the same design criteria and constraints as the more established traditional methods. Additionally, the alternative technology has the extra burden of overcoming previous shortcomings to prove that they effectively function. These alternative technologies are not recommended for mitigating the erosion at Sandwich beaches due to the lack of sediment supply. Adequate information is not available to support their use at a site of relatively high wave energy such as Town Neck Beach. Additionally, the scale and overall value of Town Neck Beach is not conducive to implementation of these alternative technologies.

### 5.11 REMOVAL OF THE CAPE COD CANAL JETTIES

Earlier alternatives evaluated modifications (lengthening or shortening) the jetties at the Cape Cod Canal. However, there have been suggestions that the solution to the sediment starvation of Sandwich beaches would be to remove the jetties at the CCC. The Cape Cod Canal serves up to 14,000 vessels a year, saving up to 150 miles on a route around the outer cape, which otherwise would include navigation around dangerous shoals and currents. The canal jetties serve to maintain the entrance to the canal.

While removing the jetties may appear to offer a potential alternative for restoring natural sediment transport from Scusset Beach to Town Neck Beach; the landscape has been drastically altered by human structures, and it is unlikely that removing the jetties would cause the canal to revert to its natural form immediately. Complete removal of both jetties has the potential to result in significant negative impacts on the entire system, destabilizing an area that has experienced the presence of the jetties for over 100 years. In the short-term, removing the jetties would immediately cause significant shoaling in the CCC; increasing dredge requirements, frequency, and expenses. At minimum, based on the sediment transport modeling, dredging requirements would likely be needed every year (100,000 cubic yards entering the canal). Shoals would likely make the canal navigation impossible for larger vessels. The loss of the jetties could also eventually result in destabilization of the geomorphology of the area surrounding the CCC. So, while the intent would be to return a sediment supply to the Town of Sandwich Beaches, this may actually result in serious repercussions for both the shorelines updrift (Scusset Beach) and downdrift (Town Neck Beach) of the CCC. A vast majority of the sediment would end up in the CCC itself and the destabilized entrance would likely migrate to the southeast, resulting in drastically increased erosion on Town Neck Beach as the channel shifts positions. Additionally, removing the jetties has the potential to cause significant impacts on Scusset Beach, as the northern jetty has been causing accretion on this beach. Without the jetties in place, erosion could be expected, which may outpace the intended sediment transport. The complete removal of both jetties may likely have unintended negative impacts on all systems within the Cape Cod Canal region, as well as significantly limiting navigation through the canal, and as such was not further evaluated as a potential alternative.

### 5.12 SAND BYPASSING

Sand bypassing could be accomplished either through a periodic dredge program (e.g., dredging from the CCC or updrift areas) or via an established bypassing plant/system located on the updrift side of the CCC that would result in more frequent, but smaller, bypass results. In either case, the purpose of the bypassing mechanism would be to provide a more continuous supply of sediment to the downdrift Town of Sandwich beaches. The intent of this alternative would be to attempt to replicate the nature transport of sediment along the Town of Sandwich shorelines. This alternative is more of a long-term maintenance approach rather than an initial mitigation measure.

The periodic dredge program would be relatively easy to implement given the ongoing maintenance dredging that occurs at the CCC, as well as the soon-to-be permitted borrow location that is being established offshore of Scusset Beach by the Town of Sandwich (EEA #15213). These two sources may potentially provide a long-term source of sediment that could be periodically bypassed around the CCC. This sediment is also the material that would normally be transported to the Town of Sandwich beaches. Twelve (12) sediment cores were collected at the borrow location to characterize grain size off of Scusset Beach in May of 2016. Table 15 shows grain size data from composite samples of the six (6) sediment cores which fall within the borrow location.

Fixed bypassing plants are an alternative to dredging and bypassing sand. Fixed bypassing plants are used in similar situations to the Cape Cod Canal such as the Indian River Inlet in Delaware, or the Lake Worth Inlet in Palm Beach, FL. For example, a fixed plant sand bypassing system is utilized at the Indian River Inlet to mitigate beach erosion caused by a 500 ft wide inlet which is stabilized by two rubble mound jetties. The fixed bypassing system utilizes jet pumps to move sand from the up-drift beaches to the down-drift beaches. As significant sediment is impounded updrift of the northern Cape Cod Canal jetty, a sediment bypass system (such as a fixed bypass system) that transfers sediment from Scusset Beach to Town Neck Beach offers numerous benefits.

A sediment bypass system reduces the reliance on offshore borrow sites, which has the added benefit of minimizing environmental impacts to those offshore borrow sites. Additionally, a sediment bypass system would have the goal of mimicking the natural littoral system, and as such would reduce sediment impoundment updrift of the canal, while increasing sediment transport to Town Neck Beach. As less sediment would be impounded by the northern canal jetty, on Scusset Beach, there would be less sediment transport from Scusset Beach through, over, and around the jetty, and as such less sediment would be deposited within the canal. This may reduce the frequency of necessary canal dredging to maintain the canal channel.

**Table 15. Grain Size Results from Cores Collected from offshore Scusset Beach on May 20, 2016 from within the soon-to-be permitted borrow site footprint.**

| Core ID # | Latitude | Longitude | Core Length (ft) | Sample # (section length, feet) | Gravel % | Sand % | Silt & Clay % | $D_{50}$ (mm) | ASTM Classification |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 41.780773 | -70.491875 | 11.3 | 1A (0-6.04’) | 0.5 | 98.7 | 0.8 | 0.23 | Poorly Graded Sand |
| | | | | 1B (6.04-11.34’) | 1 | 98.2 | 0.8 | 0.24 | Poorly Graded Sand |
| 2 | 41.781548 | -70.494111 | 11.2 | 2A (0-6.6’) | 0 | 99.1 | 0.9 | 0.21 | Poorly Graded Sand |
| | | | | 2B (6.6-11.16’) | 4.3 | 93.5 | 2.2 | 0.23 | Poorly Graded Sand |
| 3 | 41.781896 | -70.495801 | 11.9 | 3A (0-6.18’) | 0 | 99 | 1 | 0.19 | Poorly Graded Sand |
| | | | | 3B (6.18-11.9) | 0 | 98.3 | 1.7 | 0.18 | Poorly Graded Sand |
| 9 | 41.780293 | -70.492386 | 7.9 | 9 (complete) | 10.3 | 88.1 | 1.6 | 0.20 | Poorly Graded Sand |
| 10 | 41.781175 | -70.491633 | 5.6 | 10 (complete) | 0 | 99.2 | 0.8 | 0.20 | Poorly Graded Sand |
| 11 | 41.781068 | -70.494411 | 1.9 | 11 (complete) | 0.3 | 98.6 | 1.1 | 0.22 | Poorly Graded Sand |

However, the Cape Cod Canal also offers some significant challenges that would need to be overcome prior to implementation. The CCC location does not have an ideal location for a pipe crossing for transporting the sediment slurry across the Canal. The nearest infrastructure crossing (the Sagamore Bridge) is located 2.6 miles to the west of the shoreline, far too great a distance to pump material, while running a piped system underneath the Canal may results in complicated maintenance, navigation, and dredging interference issues. As such, the CCC is not the most ideal location for a permanent bypassing system.

To estimate the cost magnitude of a potential fixed bypass system, cost estimates for a similar fixed bypass system developed by the USACE (2004) for the Cape May Inlet in New Jersey are presented here:

- An initial construction cost of $6,345,000 for the fixed bypass plant
- O&M costs of $613,000 annually (bypassing from September to April, 5 days a week, 6 hours per day, bypassing 150,000 to 180,000 cy/year)
- Replacement of the pump system every 12-13 years at a fixed cost of $600,000
- Refurbishing/replacement of the system at year 25 for $6,345,000

Given the magnitude to the cost associated with this alternative, as well as the complicated nature of implementation at the CCC, a fixed bypass plant may be something that would need to be evaluated further. A periodic bypassing program based on recurring dredging of the CCC and potential updrift sources; however, offers numerous benefits and should be considered as a component of potential mitigative actions at the CCC.

## 6.0 SUMMARY AND KEY FINDINGS

- This report investigates the relationship between the Cape Cod Canal (CCC) and erosional problems experienced along the shorelines of the Town of Sandwich, Massachusetts, which lies directly downdrift to the southeast of the Cape Cod Canal. Due to the predominant northwest to southeast movement of sediment, the CCC has created an interruption in the natural transport of sediment to the Town of Sandwich beach system. As such, this report evaluates the influence the Cape Cod Canal and its associated structures may have on the adjacent shorelines, and then assesses potential alternatives to mitigate potential impacts attributed to the CCC Federal Navigation Project (FNP).

- The beaches in the Town of Sandwich, including Town Neck Beach and Springhill Beach have a history of erosion (as presented in Chapter 2). It has long been assumed that construction of jetties at the east end of the Cape Cod Canal in 1906 has been the primary reason for this coastal erosion (Giese, 1980). The two Canal jetties cause an interruption in the natural alongshore sediment transport from northwest to southeast. In order to combat the erosion, subsequent alterations to the Sandwich barrier beach system (e.g., development on the barrier beaches, construction of jetties, and the construction of groins) have further exacerbated the erosion on the Sandwich Beaches. Ultimately, the influence of the Cape Cod Canal has limited the sediment supply to the Town of Sandwich beaches such that the barrier beach system cannot maintain a healthy beach and dune complex.

- Over the long-term, shoreline change data show erosion along the entire 3.2 mile stretch of shoreline. The highest rates of erosion occur on both sides of the Old Sandwich Harbor inlet, and along the shoreline updrift (west) of the inlet. Lower rates of erosion occur along Springhill Beach and immediately downdrift of the Cape Cod Canal. Similar trends are seen over the short-term period between 2000 and 2018; however, the rates of erosion along Springhill Beach and updrift of Old Sandwich Harbor are higher, and an area of shoreline accretion is shown just downdrift of the Canal (in the shadow zone of the CCC jetties).

- The area immediately east of the Cape Cod Canal has experienced long-term erosion, but at a reduced rate compared to the shoreline further east. This is the area between the Cape Cod Canal to the longer groin located near the intersection of Dillingham Avenue and Freeman Avenue (approximately Transect 31) that has shown stability in both the long and short term. The Town of Sandwich has not nourished this section of the beach historically, nor is this area of the beach included in the permitted beach nourishment template (EEA #15213). Dune restoration only is proposed in this area, and only at the far eastern end of this section of the beach.

- From the large groin (Transect 31) to Old Sandwich Harbor inlet the shoreline change rates show a trend of increasing erosion. Immediately east of Old Sandwich Harbor inlet, the long-term data reveal variability in rates of erosion, due in large part to inlet migration. Rates of erosion range between -2 to -5 feet/year.

- Further to the east, beyond the influence of the Old Harbor inlet, the data show a trend of decreasing erosion. This trend continues to approximately Transect 108, where the rates of erosion level off to a consistent value and the shoreline is stable. This distance is approximately 10,800 feet downdrift of the Cape Cod Canal and is a reasonable estimate of the influence distance of the CCC. In other words, the disruption in the natural sediment transport caused by the Canal and its structures appears to extend approximately 10,800 feet downdrift.

- Using the most recent state specific sea level rise conditions and the observed shoreline change rates, there is almost a complete loss of the barrier beach at Town Neck in 50 years (by 2068). This condition would also result in significant ecological impacts to the expansive salt marsh system, as well as lead to impacts directly on the center of the Town during storm events.

- Predicted volume loss over the next 50 years was estimated at approximately 900,000 cubic yards over the distance influenced by the Cape Cod Canal.

- Review of historical USC&GS T-sheets of the Cape Cod Canal region reveals that prior to the construction of the CCC (between the years of 1870 to 1909) the area of Town Neck Beach from the present-day northern jetty to Sandwich Harbor experienced accretion of approximately 1.1 ft per year. This value (accretion of approximately 1.1 ft/yr) represents the estimated conditions on Town Neck Beach prior to the construction of the Cape Cod Canal, and as such indicates that this beach was actually accreting prior to the construction of the Canal.

- The sediment budget indicated an insignificant amount of sediment is able to bypass the Cape Cod Canal.

- The Scusset Beach cell is gaining an average of 54,700 cubic yards per year, while the Town Neck Beach cell is losing an average of 38,500 cubic yards per year.

- Wave conditions in the vicinity of the Cape Cod Canal are comprised of both regional ocean swell waves and locally generated wind waves. The dominant wave approach direction is from the northeast.

- The sediment transport modeling indicates that there is a strong net alongshore transport in the CCC Region from northwest to the southeast, consistent with the prevalent northeast wave approach direction. Along Scusset Beach, north of the CCC, the average annual alongshore transport is directed to the southeast at an average rate of approximately 95,000 to 115,000 cy/year, ending at the western Cape Cod Canal jetty. Southeast of the CCC and ending approximately at Knott Avenue there is a small zone of transport reversal, located in the shadow of the Canal jetties, which limits the wave energy from the northeast, yet allow energy from the less predominant eastern directions. Net transport at this reversal ranges from approximately 10,000 to 20,000 cy/year toward the northwest. Southeast of the reversal, net alongshore sediment transport patterns continue to be directed towards the southeast, where transport rates range from approximately 35,000 to 45,000 cy/year until reaching Old Harbor Inlet.

- The existing beach and dune system would also be overtopped during all storm events, indicating the relative frailty of the existing barrier beach and dune system. Additionally, the 1-year and 10-year storm cases show retreat of the dune scarp, while the 50-year storm case shows complete failure of the dune.

- A wide variety of potential mitigative alternatives were evaluated, as developed in a joint meeting between the USACE and the Town of Sandwich.

- A proposed beach nourishment has already been designed and permitted as part of a previous project developed by the Town of Sandwich (EEA #15213). As part of that design development, a dune and beach restoration template was designed that offered a holistic approach by encompassing the entire Town Neck beach and dune system. Beach nourishment and dune creation in this alternative required approximately 388,000 cy of clean beach compatible sediment. The nourishment would primarily be used to stabilize, strengthen and rebuild weak and eroded beach and dune reaches throughout the Town Neck Beach system. This nourishment would also serve as a feeder system for eroding downdrift beaches (Springhill). The creation of additional beach and dune resources would expand critical habitat area, and serve the protectable interests of storm damage prevention and flood control.

- The assessment of the beach nourishment alternative indicated that once the initial nourishment has decayed to width of approximately 30 feet, a 10-year storm event could cause significant overtopping of the dune system and potential upland damage. The critical width threshold was then applied to the performance curves to determine the estimated renourishment interval. At a critical width value of 30 feet the renourishment interval for the beach nourishment is 9 years (9 year service life).

- Adding coir envelopes to the dune were shown to provide some small incremental improvement to the critical width level, indicating approximately 8-10 feet less width is required. This would indicate that there is a potential improvement of approximately 2 years (to a total of 11) for the service life beach and dune restoration project prior to required maintenance of the beach system. Additionally, the actual biodegradable coir envelopes components likely need to be serviced every 5-7 years.

- Although a decrease in the length of the northern jetty may slightly increase littoral transport past the jetty, it may also produce negative effects that should be considered. Based on this analysis, the maximum gain in bypassing that could be expected under this north jetty modification alternative is approximately 160 cy/year. Additionally, any sediment bypassing the existing, or shortened northern jetty would most likely be deposited in the deep channel, or in the CCC itself, and not available for transport on the downdrift beaches. The required maintenance depths of the CCC limit the ability of sediment to bypass even without the influence of the jetties. As such, any sediment being mobilized due to the north jetty modification would likely just cause increased shoaling, and thus increase dredging costs.

- Lengthening the south jetty would not significantly alter the sediment dynamics on the south side of the CCC. It is unlikely that significantly more sediment would be captured by a longer jetty. Even if all the sediment that may be moving back into the CCC from the downdrift side is retained on Town Neck Beach, this is a minimal amount of sediment that would not justify the extension of the south jetty.

- After a number of model iterations, the optimized configuration for supplemental groins along Town Neck Beach involved removing two existing non-functional jetties at Old Sandwich Harbor, as well as deconstructing the five groins along the eastern portion of Town Neck Beach. Beneficially reusing this material, four (4) new notched groins were constructed on the eastern portion of Town Neck Beach. They would be approximately 250 in length (across shore) and include low profile notches along the shoreline in the intertidal zone.

- The influence of the notched groins shows some gains in beach nourishment performance. For example, the additional modified notched groins increased service life by approximately 4 years when 30% of the initial fill remains. Using the critical width criteria developed for the beach nourishment, the notched groins extend the renourishment interval by approximately 4.5 years. Therefore, instead of replenishments being required every 9 years (nourishment alone), replenishments would be required every 13.5 years.

- There is limited to no room to construct and maintain a seawall or revetment structure along Town Neck Beach. For a recreational beach setting like Sandwich Town Neck Beach, where maintaining a beach is important, a seawall or revetment would not be a preferred solution. Given the valuable bird habitat area that exists on the Town Neck and Springhill beaches, seawalls and revetments would not be permittable. Therefore, seawalls and revetments are not recommended for further assessment.

- At Town Neck Beach, the high construction costs, interference with recreational uses, and difficulty with likely permitting outweigh the anticipated benefits, making offshore breakwaters infeasible.

- While removing the jetties may appear to offer a potential alternative for restoring natural sediment transport from Scusset Beach to Town Neck Beach, removing the jetties would immediately cause significant shoaling in the CCC; increasing dredge requirements, frequency, and expenses. The loss of the jetties could also eventually result in destabilization of the geomorphology of the area surrounding the CCC. So, while the intent would be to return a sediment supply to the Town of Sandwich Beaches, this may actually result in serious repercussions for both the shorelines updrift (Scusset Beach) and downdrift (Town Neck Beach) of the CCC.

- A number of alternative technologies were evaluated as options to potential mitigate the erosion on Sandwich Town beaches. These alternative technologies are not recommended for mitigating the erosion at Sandwich beaches due to the lack of sediment supply. Adequate information is not available to support their use at a site of relatively high wave energy such as Town Neck Beach. Additionally, the scale and overall value of Town Neck Beach is not conducive to implementation of these alternative technologies.

- Given the magnitude to the cost associated with constructing a fixed bypassing plant/system, as well as the complicated nature of implementation at the CCC, a more detailed study would be required to determine the efficacy of such a system. A periodic bypassing program based on recurring dredging of the CCC and potential updrift sources; however, offers numerous benefits and should be considered as a component of potential mitigative actions at the CCC.

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