CAPE COD CANAL & SANDWICH BEACHES

SHORE DAMAGE MITIGATION PROJECT

APPENDIX B

COASTAL ENGINEERING REPORT

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Cape Cod Canal and Sandwich Beaches
Section 111 Shore Damage Mitigation Study
Coastal Engineering Report

1. Introduction and Report Description

1.1. Report Description

This report has been written to be included as the Coastal Engineering Appendix to the feasibility report. Contained within the Coastal Engineering Appendix is a combination of both the US Army Corps of Engineers (Corps) Coastal Engineering effort and the Woods Hole Group (WHG) effort. To help provide a complete Coastal Appendix much of the information provided in the WHG report (Appendix C) has been summarized within this appendix. Throughout this appendix the applicable WHG report sections are referred to in order to assist the reader in acquiring more detailed information as needed. Generally, the Corps' detailed analysis and coastal engineering efforts focused on the regional and residual shoreline change analyses, assessment of federal responsibility, sea level change analysis, beach fill performance assessments for variations in nourishment quantities, and discussion of the selected plan. These sections are interwoven with sections describing work done by WHG. As noted, if the reader is interested in greater detail regarding the work performed by WHG they are referred to Appendix C.

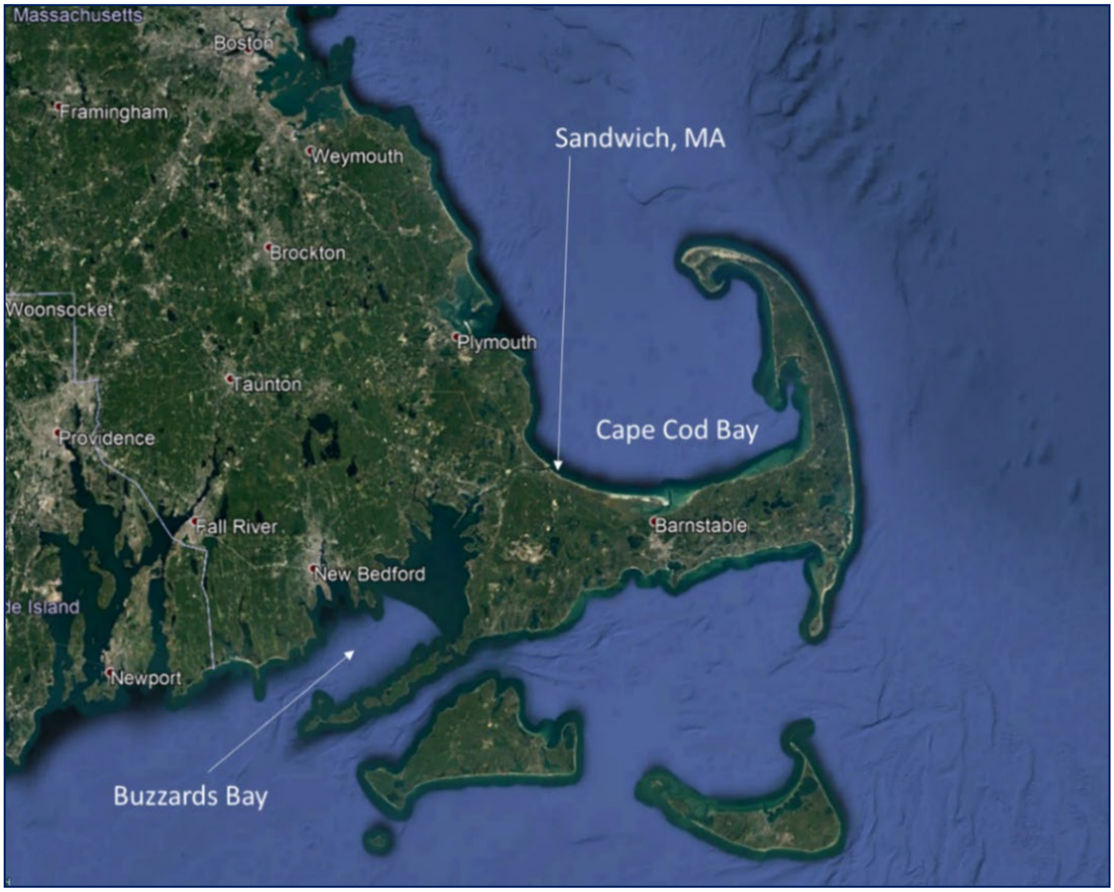
1.2. Project Description

Town Neck Beach is located directly adjacent and east of the east end of the Cape Cod Canal (Canal) and its jetties where it joins Cape Cod Bay (Figures B1-1 and B1-2) within the town of Sandwich, MA, approximately 50 miles southeast of Boston. The Canal was originally constructed by the Boston, Cape Cod, and New York Canal Company (Canal Company) under a charter issued by the Commonwealth of Massachusetts to provide a shorter, inshore, and more protected sea route connecting Buzzards Bay in the southwest with Cape Cod Bay in the northeast, avoiding the longer route around Cape Cod, the islands of Martha's Vineyard and Nantucket, Nantucket Shoals and Georges Bank. Construction began in 1909 and the Canal was opened to marine traffic in 1914. The Federal Government took control of the Canal during World War I, along with other national transportation infrastructure, and operated the Canal through the 1920s. The Canal was acquired by the Federal Government in 1928 under Section 2 of the River and Harbor Act, adopted 21 January 1927. In general, the project provides for a sea level canal 32 ft deep at Mean Low Water (MLW), 540 feet wide in a 7.7 mile land cut and having a total length of 17.5 miles. The project includes operating facilities, mooring basins for large and small vessels, a small-boat channel in Onset Bay, as well as two high level highway bridges and a railroad lift bridge.

A breakwater and jetty at the east end of the Canal were included in the purchase of the Canal properties. The 3,000 ft breakwater on the north (mainland) side of the Canal entrance was completed in 1913 by the Canal Company to arrest littoral drift, train the tidal current at the canal entrance to prevent shoaling, and reduce navigational difficulties. The design provided for a crest width of 25 ft at 18 ft above MLW, with side slopes of 1V:2H on the northwest side from the top to 12 ft below MLW, and 1V:1H below that depth and on the channel side. The structure was to be 3,000 feet in length, extending 2,600 feet from shore out to a depth of 32 feet. However, it appears to have actually been constructed to a total length of 2,700 ft, approximately to the 30 ft depth contour. The breakwater rehabilitation in 1962 placed larger stone on and around the existing structure at milder slopes. The breakwater head now has a crest width of 25 ft at 18 ft above MLW with side slopes of 1V:3H. The trunk section has a crest width of 20 ft at 18 ft above MLW with side slopes of 1V:2H. Repairs to the east end breakwater were also made in 1999-2000, focusing primarily on the breakwater head.

The armor stone jetty on the south (Cape) side of the Canal entrance was constructed in 1913 with a crest width of 20 ft at 10 ft above mean low water, with side slopes of 1V:1H. It was built to the 6 ft depth contour, a distance of about 600 ft. The jetty was constructed to prevent erosion of the shore and reduce shoaling of the channel. In 1975, repairs to the south jetty raised its top elevation to 13 ft above MLW.

Due to the interaction between the structures and the local coastal processes, it has been concluded that the beach east of the structures is being adversely affected. Based upon analysis to be discussed, it was determined that the jetty structures have significantly exacerbated erosion by cutting off the longshore transport of sand from the west. Under Section 111 – Mitigation of Damages Caused by Federal Navigation Projects (FNP), a feasibility study was undertaken to determine a solution to the erosion problem as it can be directly attributed to the Canal FNP. The immediate project area is the downdrift shoreline along Town Neck and Springhill Beaches within 10,800 ft of the Canal, but a larger area around the east entrance to the Canal is studied to define the sediment budget, or rates of sediment movement within the littoral system.


Massachusetts
Boston
Framingham
Weymouth
Sandwich, MA
Brockton
Woonsocket
Plymouth
Taunton
Providence
Cape Cod Bay
Fall River
Barnstable
New Bedford
de Island
Newport
Buzzards Bay


Scusset Beach
Cape Cod Bay
Cape Cod Canal
Town Neck Beach
Springhill Beach
Sandwich

Figure B1-2: Cape Cod Canal and Sandwich, MA beaches

Prior to the Canal's construction, the study area was comprised of a long, straight, uninterrupted sandy beach stretching south from Plymouth continuously to the opening at Old Harbor Inlet in Sandwich. Since the Canal's construction, significant erosion has occurred on the downdrift beaches in Sandwich. Efforts of varying size and nature have attempted to slow the rate of erosion. Throughout the 1950s the Town of Sandwich constructed shore protection structures in the form of stone groins along Town Neck and Springhill Beaches to maintain the position of the beach. While some of the groins on the western end of Town Neck Beach have had a stabilizing effect, the eastern end of Town Neck Beach and Springhill Beaches have continued to erode due to lack of sediment supply. Many of the groins along these beaches are in poor condition or are detached from the shoreline and have little effect on sediment transport. The lack of sediment supply also reduced the stability of the inlet beach system at Old Harbor Inlet which separates Town Neck Beach from Springhill Beach. The October 1991 "No-Name" storm, coupled with the lack of sediment supplied to the region, caused the inlet to breach out of its jetty structures. The inlet has since migrated southeast. With continued historical shoreline retreat, sea level rise, and the inlet breach, the current conditions expose a larger region of low-lying beach, marsh, dunes, roadways, and inland homes to a greater level of storm damage and flooding than in previous times in recent history.

Several artificial nourishment efforts have been undertaken by the Town of Sandwich and private homeowners. Nourishment has been permitted at Town Neck Beach through numerous small projects using upland sand sources and through three large-scale projects between 1990 and 2016 where dredged material from the Canal was pumped onto the beach and constructed as a beach and dune. Some individual homeowners have used sand-filled coir envelopes to reinforce the dunes as a last line of defense (Figure B1-3). Despite these efforts, continued erosion has, most recently, resulted in the loss of several homes (Figure B1-4).

1.2.1. <u>Geographic Setting and Coastal Environment</u>

Sandwich is located on the northern shoreline of Cape Cod facing Cape Cod Bay and is bisected by the Cape Cod Canal. Sediments are generally transported in a northwest to southeast direction from Plymouth through Sandwich. Astronomical tides in Cape Cod Bay are semi-diurnal, with the mean tidal range at Sandwich being approximately 9.6 ft (2.9 m) and the diurnal tidal range being 10.3 ft (3.1 m). Relevant tidal datums derived using NOAA's vDatum program for Town Neck Beach are provided in Table B1-1 relative to NAVD88. The coast is characterized by the Cape Cod Canal inlet and the Old Harbor Inlet with predominantly sandy beaches backed by dunes, bluffs, and salt marsh.

Table B1-1. Tidal Datums at Town Neck Beach

Tidal Datums at Town Neck Beach	
Datum	**Elevation (ft, NAVD88)**
Mean Higher High Water (MHHW)	4.69
Mean High Water (MHW)	4.22
NAVD88	0.0
Mean Sea Level (MSL)	-0.49
Mean Tide Level (MTL)	-0.56
Mean Low Water (MLW)	-5.34
Mean Lower Low Water (MLLW)	-5.63

Source: NOAA vDatum for latitude 41.768958, longitude -70.484506

1.2.2. Coastal Geology

Cape Cod and the Sandwich shoreline was formed by glacial deposition of sediments. In addition to their geological source, characteristics of sediments found in the Canal region are also the result of active coastal processes including winds, waves, tides and currents. Sediment has been supplied to Scusset Beach, and historically to Town Neck Beach, 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 supply of sediment to the beaches through longshore transport (Fitzgerald et al., 1994). The study area contains reworked sandy and gravelly glaciofluvial deposits and/or sandy and silty marine deposits. Complete details on the local geology can be found in Chapter 1.2 of Appendix C.

Grain size data from samples taken in 2016 at 24 stations on both Scusset and Town Neck Beaches indicate that the dominant sediment type is medium-to-coarse grained sand, with some gravel at Town Neck Beach. WHG also collected 11 sediment core samples from the nearshore at Scusset Beach to characterize the shore-parallel shoal and surrounding area. As with the beach grab samples, the offshore sediment is composed of poorly-graded sand, but is fine to medium grained. Having the same source material, the differences between the beach and intertidal samples (coarse-medium sand) and the offshore samples (medium-fine sand) are dominated by the physical processes that sort and transport the grain sizes (WHG, 2017).

1.3. Cause-and-Effect Relationship

An important criterion for defining a Section 111 project for USACE is determining whether and the extent to which damages to an adjacent area are directly attributable to an FNP. This analysis has shown that the Canal jetties prevent sand from Scusset Beach and points north from reaching the Town Neck and Springhill beach system. The long-term shoreline change analysis conducted for this study showed that shoreline recession attributable to the inlet extends approximately 10,800 ft to the east of the Canal. The percent of shoreline recession attributable to the Canal was determined to be 78 percent of the total erosion based on historical shoreline change, with the loss of 12,200 to 14,000 cy/year of material due to the Canal.

A sediment budget analysis also indicated the Canal and its structures have exacerbated the erosion at Town Neck and Springhill Beaches. The sediment budget indicated the volumetric loss rate attributable to the Canal to be 81,400 cy/year or 85 percent of the approximately 95,900 cy/year updrift alongshore transport. While this volumetric rate is about 6 times greater than the volume determined through the shoreline change approach, the percent attributable to the Canal FNP is comparable. Based on this information, increased erosion to the downdrift shoreline can be directly attributed to the Cape Cod Canal FNP.

2. Project Background/Coastal Engineering History

2.1. Coastal Engineering History

The USACE involvement with the Cape Cod Canal dates back to the early 1900's with the project acquisition in 1928. USACE improved navigability and deepened the Canal to its current depth. The entrance jetties have each undergone repair and improvement. Their design is discussed in Section 1.2. Today, the Canal is dredged approximately every seven years with an average of approximately 90,000 cubic yards of clean, beach compatible sand and gravel removed during each event.

The erosion at Town Neck and Springhill Beaches has been the subject of several past investigations and the construction of the jetties at the east end of the Canal has long been considered the primary reason for this coastal erosion (Giese, 1980). Working with the Town of Sandwich, WHG has conducted studies of the region, and designed and permitted projects for upwards of 20 years. Most recently, the Canal region and Sandwich beaches have been the subject of the following efforts:

- 2014: The Town of Sandwich Dune and Beach Reconstruction Project (EEA #15213), which seeks to place 388,000 cy of sand on Town Neck Beach and whose design is the selected alternative of this study, was reviewed and permitted by the Massachusetts Executive Office of Energy and Environmental Affairs. The investigation which led to the reconstruction project's design was conducted by the Town of Sandwich and WHG.

- 2015: USACE Section 204 Feasibility Study was conducted from 2014-2015. The study considered a cost-shared project to beneficially reuse dredged material from the Cape Cod Canal by directly placing approximately 120,000 cy of sand on Town Neck Beach. Due to unresolved policy and real estate concerns at the time that the Canal was dredged in 2016, this study was not completed and cost-sharing through the Section 204 authority was not approved.

- 2016: Although the USACE Section 204 study was unsuccessful, the Town of Sandwich was still able to construct the 120,000 cy beach fill with material dredged from the Canal without cost-share. The town, with help from the Commonwealth of Massachusetts was able to procure the funding needed to pay for the project in full and a Memorandum of Understanding between the Corps and the Town was signed, enabling the project to be constructed.

- 2018: Following the 2016 nourishment with material from the Canal, the Town of Sandwich, with WHG, studied, designed, and permitted a borrow site immediately offshore at Scusset Beach State Reservation. With the borrow area being located immediately updrift of the Canal's north jetty it can serve as a source of an additional 224,000 cy of material to build the previously permitted Town of Sandwich Dune and Beach Restoration Project (2014).

These studies provided a foundation for the Section 111 study. USACE selected WHG as the contractor to provide coastal engineering technical support given their history of work in the Canal region in Sandwich. Much of the shoreline change analyses and wave and sediment transport modeling conducted as part of this study built upon established, calibrated models that WHG had already developed through the efforts listed above.

3. Shoreline Mapping and Sediment Transport

3.1. Historical Shoreline Change Analysis

A computer-based mapping methodology, within a Geographic Information System (GIS) framework, was used to compile and analyze changes in the historical shoreline position between 1952 and 2018 at Town Neck and Springhill Beaches. The purpose of this task was to quantify changes in shoreline position using the most accurate data sources and compilation procedures available, and to characterize areas of erosion and accretion. For a more detailed discussion of historical shoreline change refer to Appendix C, Chapter 2.

3.1.1. Data Sources

For this project, five primary sources of data were used to evaluate changes in shoreline position during the period of 1952 to 2018. Shoreline data from Massachusetts Coastal Zone Management (Mass CZM) between 1952 and 2009 was obtained from the Massachusetts Shoreline Change Mapping and Analysis Project (USGS, 2013). More recent data from 2014 and 2018 was added to the analysis and obtained by digitizing the Mean High Water (MHW) line from georeferenced orthoimagery available from MassGIS and Google Earth.

3.1.2. Discussion of Shoreline Change

To evaluate trends in shoreline change at Sandwich, various graphical representations have been developed. Shoreline positions for each of the available dates between the period of 1952 and 2018 were developed and changes in shoreline position were evaluated along a series of 139 shore perpendicular transects spaced at 100 ft intervals from the Canal east spanning a study area of approximately 3.2 miles. At each shoreline change transect, distances of shoreline movement and annual rates of change were determined. Data from 1952 to 2018 was used to compute long-term rates of shoreline change while data from 2000 to 2018 was used to compute short-term rates of shoreline change using the linear regression method. Figures B3-1 and B3-2 show the long-term and short-term rates of change, respectively, at the shoreline change transects along Town Neck and Springhill Beaches. Negative values (yellow-orange-red) correspond to shoreline erosion, whereas positive values (green) correspond to shoreline accretion. Both the long- and short-term rates of change are plotted by transect in Figure B3-3. Transects 1 to 74 cover Town Neck Beach from the Canal to Old Harbor Inlet while transects 75 to 139 define Springhill Beach east of Old Harbor Inlet.

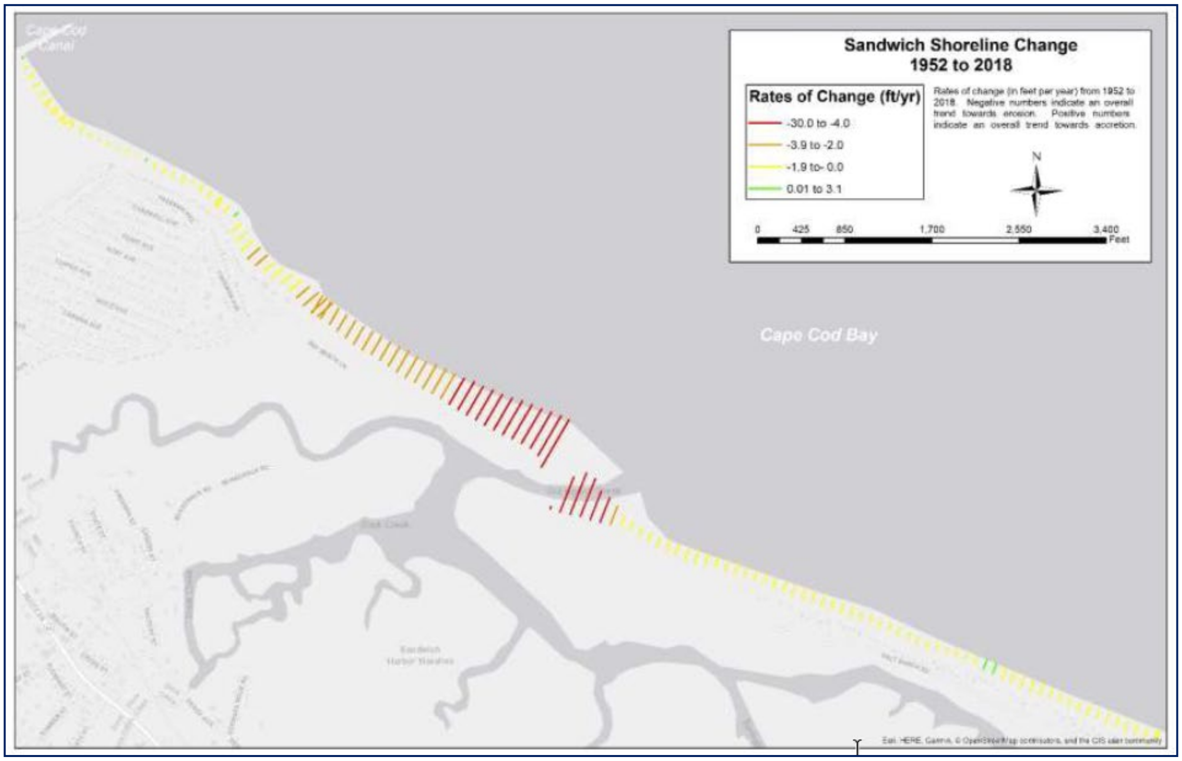


Figure B3-1: Long-term (1952-2018) shoreline change rates at Sandwich (WHG, 2020)

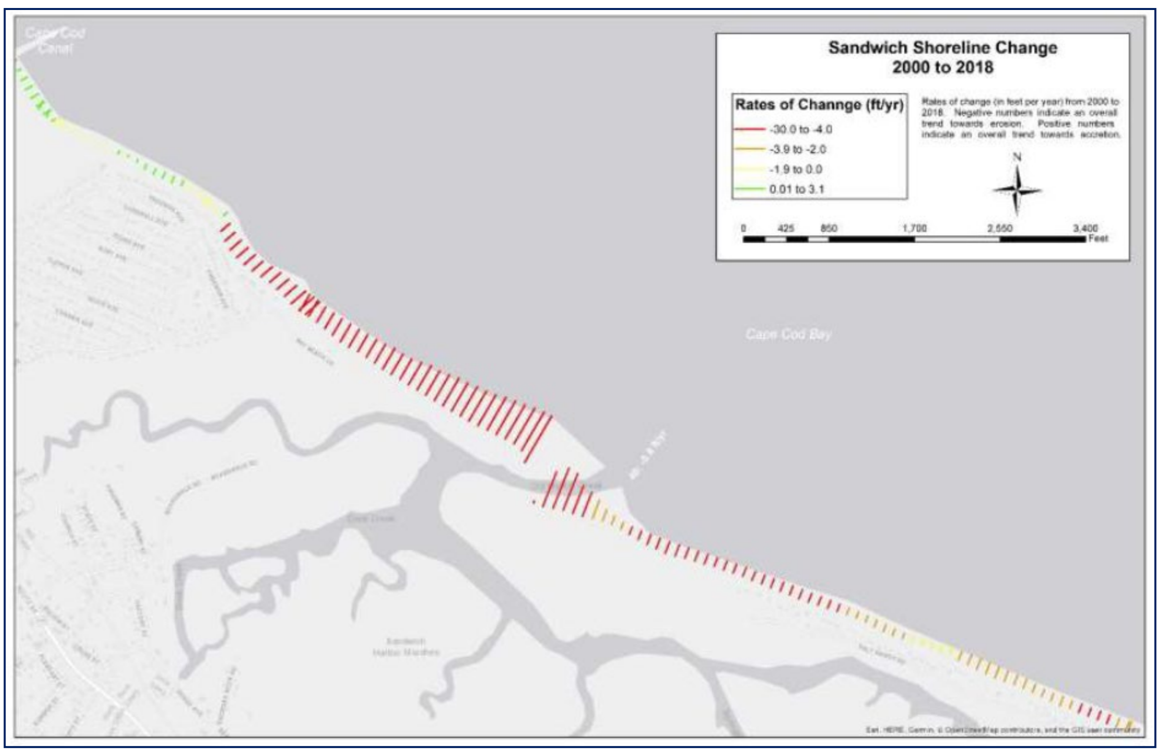


Figure B3-2: Short-term (2000-2018) shoreline change rates at Sandwich (WHG, 2020)

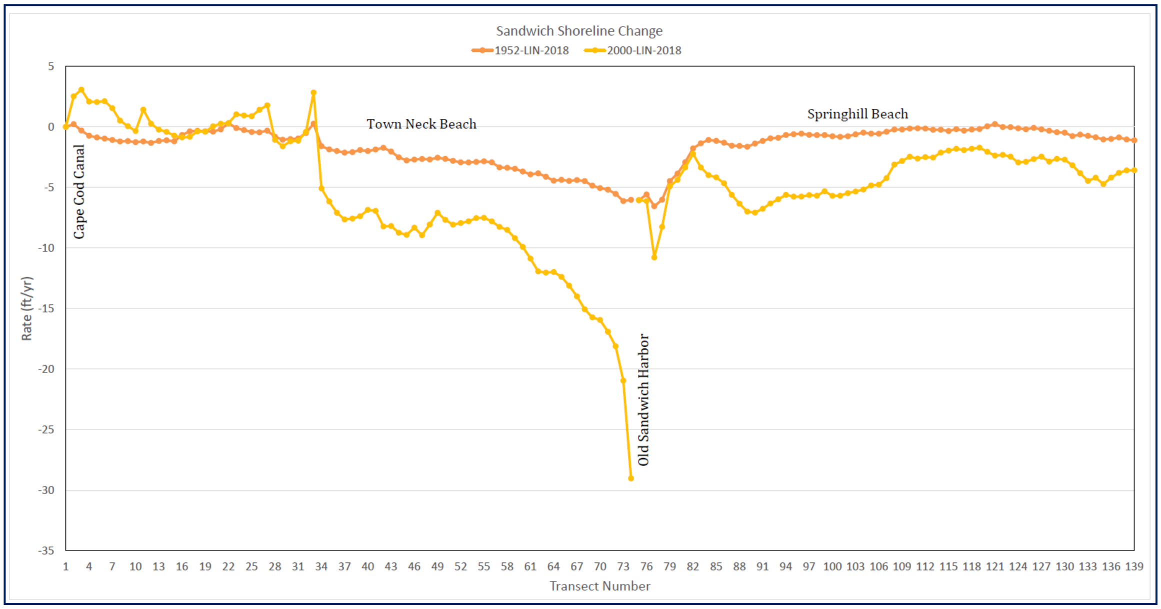


Figure B3-3: Long- and short-term rates of shoreline change at Sandwich by transect (WHG, 2020)

The shoreline change analysis indicates that Town Neck and Springhill Beaches have experienced erosive conditions over both the short and long term, with increased rates of erosion observed in the short term. The highest rates of erosion occur on both sides of Old Harbor Inlet, and along Town Neck Beach.

The Town Neck Beach shoreline from the Cape Cod Canal to the longer groin located near the intersection of Dillingham Avenue and Freeman Avenue (approximately Transect 31) has been relatively stable in both the short- and long-term, experiencing smaller erosion rates in the long-term and areas of accretion in the short-term. It is likely that this area, which lies in the shadow of the Canal jetties, experiences reduced wave energy afforded by the influence of the Canal jetties on local wave transformations. This is typical of an area immediately downdrift of a large coastal inlet with jetties, where the area immediately downdrift of the structures may experience reduced erosion rates and a reversal in sediment transport. This stretch of shoreline is sheltered by the Canal jetties from waves approaching from the north. This energy reduction, coupled with the stabilizing effects of the groins in the area and the slight reversal in sediment transport direction, has produced a more stable section of shoreline relative to areas further east.

Increasing rates of erosion are observed in both the short- and long-term moving east of the stabilizing groin in the vicinity of Transect 31 to Old Harbor Inlet at Transect 74. Although long-term erosion rates in this area range from -2 to -5 ft/year per year, much of this area has short-term erosion rates between -6 and -10 ft/year, with the 1,400 ft stretch of shoreline updrift of the inlet showing a dramatic increase in erosion up to -25 ft/year. The higher rates of erosion nearest the inlet are due to the inlet's migration outside its jetties, which is primarily the result of the lack of sediment supplied to the region which has reduced stability of the inlet system. During the October 1991 “No-Name” storm, the inlet breached out of its existing jetties. The inlet has continued to migrate east since then. Figure B3-4 shows Old Harbor Inlet prior to the October 1991 storm and its present condition. In addition to the inlet’s migration and separation from its jetties, the loss of beach is evidenced by the groins at Town Neck Beach which are now detached from the shoreline and contain no sediment.

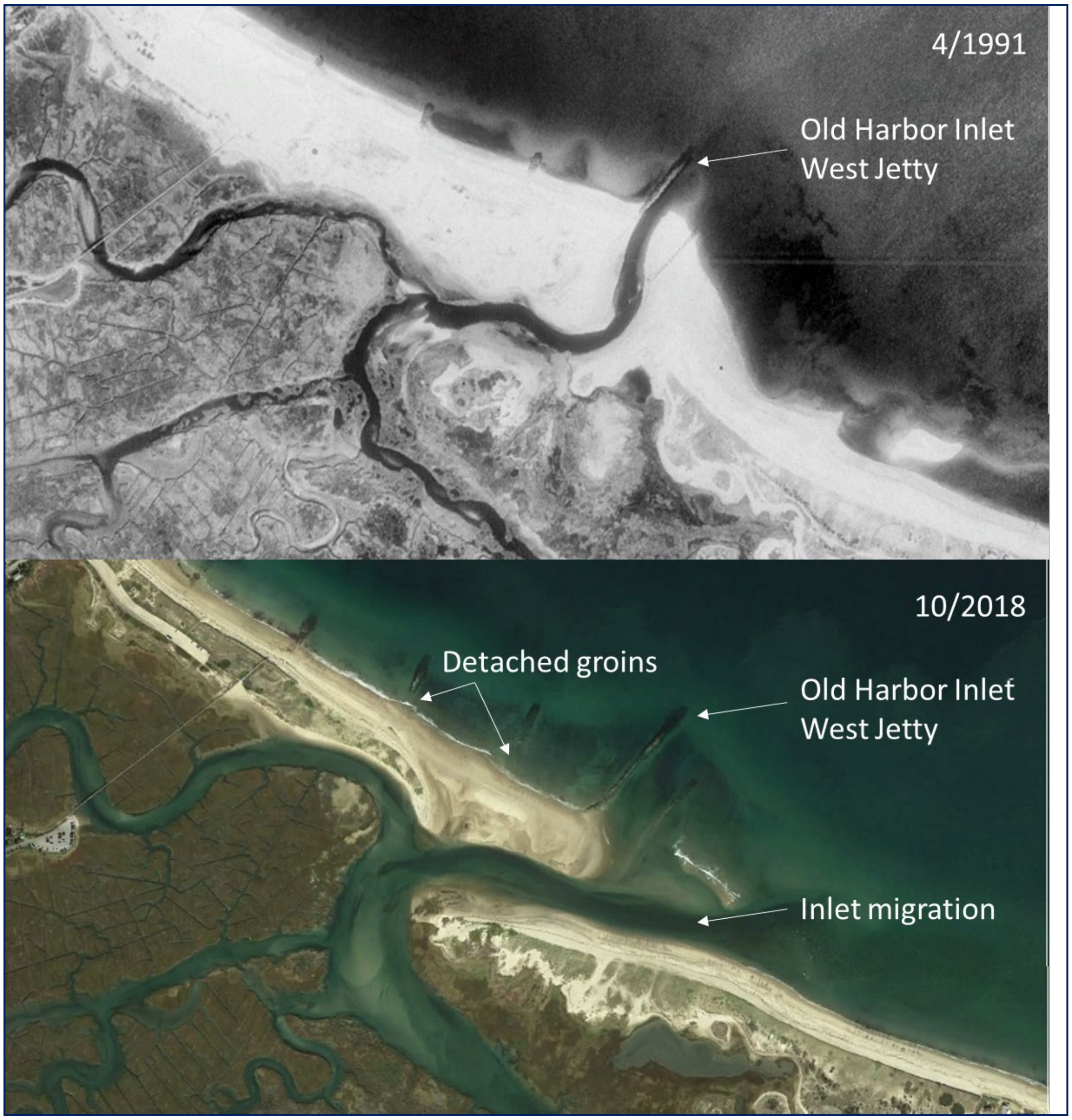


Figure B3-4: Aerial images showing shoreline changes in the vicinity of Old Harbor Inlet

Springhill Beach, east of Old Harbor Inlet, shows consistent but decreasing rates of erosion in both the short- and long-term. Long-term erosion rates at Springhill Beach are approximately -2 ft/year whereas short-term erosion rates are greater at approximately -5 ft/year. The erosional trend continues to approximately Transect 108, or 10,800 ft downdrift of the Cape Cod Canal, where the rates of erosion level off and the shoreline is increasingly stable. This distance of 10,800 ft was selected as a reasonable estimated extent of influence that the Canal has on downdrift erosion. In other words, the disruption in the natural sediment transport caused by the Canal and its structures was estimated to extend approximately 10,800 ft downdrift.

3.2. Regional Shoreline Change Discussion

To put the shoreline change at Sandwich in context with the region and confirm erosion within the study area was not due to coastal storm activity alone, shoreline change rates within the study area were compared with those in the region. This shoreline change analysis was based on the shoreline data assembled as part of the Massachusetts Shoreline Change Project 2018 Update (Himmelstoss et al, 2019).

Mass CZM launched the Shoreline Change Project in 1989 to identify erosion-prone areas of the coast. The project, which illustrates how the shoreline of Massachusetts has shifted since the mid-1800s, has been updated several times since its initial publication as new shorelines have been incorporated. The most recent update (2018) includes shorelines through 2014. Using data from historical and modern sources, shorelines depicting the local high water line have been generated with transects spaced at 50-m (approximately 164-ft) intervals along the shore. At each transect, net distances of shoreline movement, shoreline change rates, and uncertainty values are provided. Long-term rates of shoreline change were determined by fitting a least squares regression line to all shoreline positions from the earliest (mid-1800s) to the most recent (2014), spanning an approximately 150-year record. Short-term rates of shoreline change were calculated using the most recent shoreline positions from 1978 and 2014, a 36 year record. Long-term rates of shoreline change calculated with many shoreline positions can increase confidence in the data by reducing potential errors associated with the data source and fluctuating short-term changes.

For this study, the Massachusetts Shoreline Change Project shoreline change rate data was used to determine regional and local shoreline change rates within the littoral cell about the Cape Cod Canal from Stage Point in Plymouth to Sandy Neck in Barnstable (Figure B3-5). Ideally, shoreline and beach volume change should be evaluated for two distinct periods: one before project construction to determine the natural or background change, and the other after construction to quantify the response of the coast to the project. For the Cape Cod Canal, data available prior to the data of construction (1909-1914) had a high degree of error, inherent with the available early survey data. Therefore, regional shoreline change trends were determined and compared to the rate of shoreline change in the project area. This approach allowed comparison of shoreline change over contemporary time periods during which coastal processes are similar. For this study, the regional change was calculated between Stage Point in Plymouth, the western boundary, and Sandy Neck in Barnstable, the eastern boundary, about 23 miles of shoreline. It was found that for the period of 1860 to 2014, the regional shoreline change trend was recession at an average rate of 0.29 ft/year.

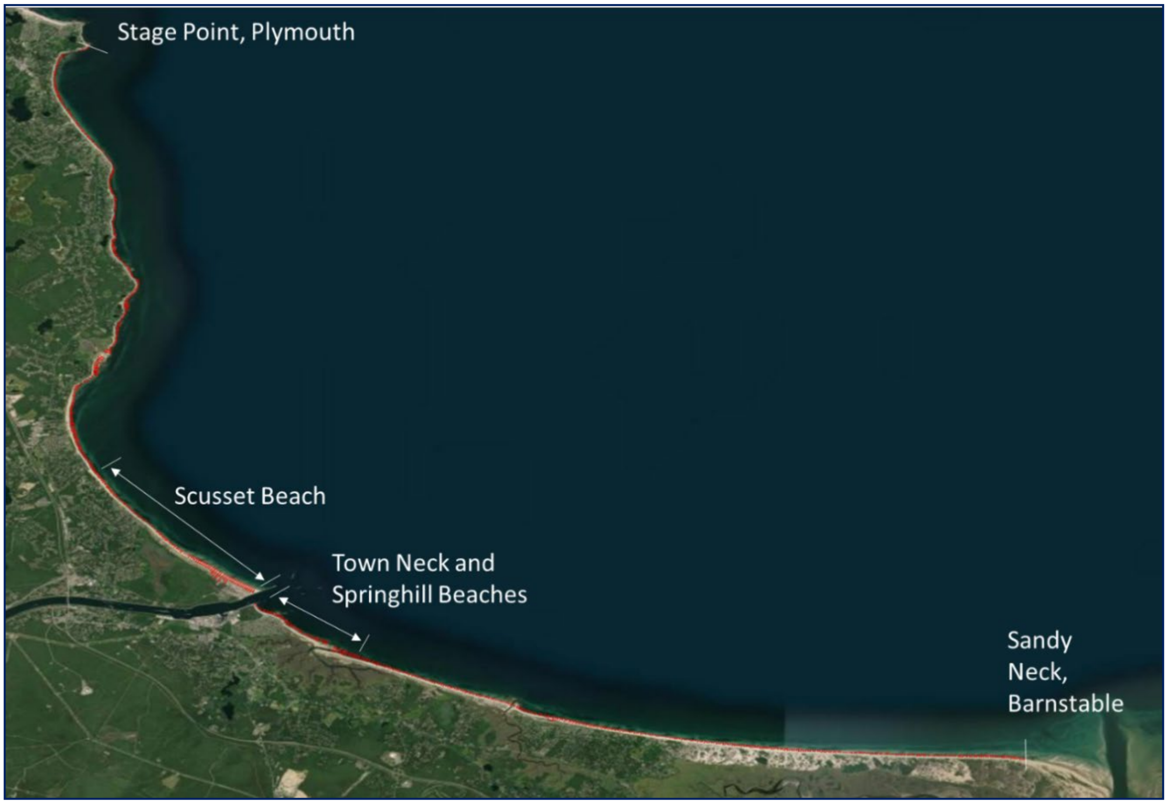


Figure B3-5: Regional shoreline change area from Stage Point, Plymouth to Sandy Neck, Barnstable

Within the region, rates of change along shorter segments of shoreline were evaluated to define more localized shoreline change rates. Similar to the regional trend of recession (-0.29 ft/year), long-term recession was observed both updrift and downdrift of the Canal. However, shoreline recession updrift of the Canal, from Stage Point to the Canal, was less than the regional average (-0.11 ft/year) while shoreline recession downdrift of the Canal, from the Canal to Sandy Neck, exceeded the regional average (-0.48 ft/year). The influence of the Canal and its structures is most pronounced along Scusset Beach, just updrift of the Canal, and downdrift of the Canal along Town Neck and Springhill Beaches. Along Scusset Beach, the long-term shoreline change trend is accretionary at 1.16 ft/year. On Town Neck and Springhill Beaches, the average shoreline change rate within 10,800 feet of the Canal is -1.33 ft/year. A summary of the short- and long-term shoreline change rates in the Canal region is provided in Table B3-1.

Table B3-1. Short- and long-term shoreline change rates in the Canal region

Area	Short Term Rate (ft/year)	Long Term Rate (ft/year)
Region	0.07	-0.29
Stage Point to Canal	0.07	-0.11

Canal to Sandy Neck	0.06	-0.48
Town Neck and Springhill Beaches -- 10,800 ft of Shoreline East of Canal (WHG defined area of impact)	-2.58	-1.33
Scusset Beach	2.25	1.16

As noted in Section 3.1.2 and Appendix C, Chapter 2, analysis of long-term shoreline change on the project scale showed that shoreline recession attributable to the Canal extends for approximately 10,800 feet to the east of the inlet. Within this area, the long-term shoreline change averaged -1.33 ft/year. A residual shoreline change rate, or the change attributable to the FNP, was then determined by removing the average regional recession rate from the shoreline change rate for the 10,800 ft of shoreline adjacent to the project. This procedure gave a residual recession rate of 1.04 ft/year. Thus, the amount of shoreline recession directly attributable to the FNP was determined to 1.04 ft/year (i.e. 78%) based on the shoreline change analysis. Overall, the shoreline change data indicate that the Canal and its structures have modified the evolution of the adjacent shorelines, resulting in shoreline advance updrift of the inlet and shoreline recession downdrift of the inlet.

3.3. Shoreline Change—Future Position

WHG estimated the future shoreline position at Town Neck and Springhill Beaches using the long-term shoreline change rates and future sea level projections. An overview of sea level change is provided before future shoreline positions are discussed.

3.3.1. Sea Level Change

Based on ER 1100-2-8162 and EP 1100-2-1, USACE studies must consider future rates of sea level change that are higher than the historical rates to account for the potential impacts of climate change. Due to the uncertainty associated with future sea level change the USACE policy is to look at three scenarios of sea level change and investigate the impact to project feasibility. These rates are the historic rate at the project site, an intermediate rate and a high rate of sea level change. The intermediate and high rates are modified from the National Research Council (NRC) curves I and III, respectively. All three local sea level change curves include the global (eustatic) sea level rise rate (approximately 1.7 mm/year according to IPCC 2007) as well as vertical land movement. USACE guidance allows for the consideration of additional curves.

In order to calculate these various rates for a project site, USACE developed an online calculator tool, the Sea-Level Change Curve Calculator (Version 2019.21) (http://corpsmapu.usace.army.mil/rccinfo/slc/slcc_calc.html). The tool uses the nearest NOAA tide station with an adequately long water level record to determine the historical trend. The tool then uses this historical trend along with a formulation provided in EP 1100-2-1 to determine the intermediate and high rates of change. The online calculator can also provide NOAA sea level change curves.

For the historic mean sea level trend, the Boston, MA NOAA station (NOAA 8443970), located 49 miles northwest of the east entrance to the Canal, was used. The Sea-Level Change Curve Calculator, as a default, uses the historic mean sea level rate published in 2006. However, the user may also select the regional rate (NOAA, 2013) or enter a user-specified rate. The 2006 mean sea level trend at Boston is 0.00863 feet/year or 0.863 ft/century. The regional trend is 0.00833 ft/year or 0.833 ft/century. The NOAA Sea Level Trends web page contains the historic mean sea level rate through 2019. At Boston, this mean sea level trend is 0.00938 ft/year (2.86 mm/year) or 0.938 ft/century based on regionally-corrected mean sea level data from the station's establishment through 2019, 98 years. This long-term linear trend was selected for use in the Sea-Level Change Calculator and is shown in Figure B3-6 (https://tidesandcurrents.noaa.gov/sltrends/sltrends_station.shtml?id=8443970). Also shown is the monthly mean sea level without the regular seasonal fluctuations due to coastal ocean temperatures, salinities, winds, atmospheric pressures, and ocean currents. The short-term sea level change rate varies due to yearly and decadal cycles.

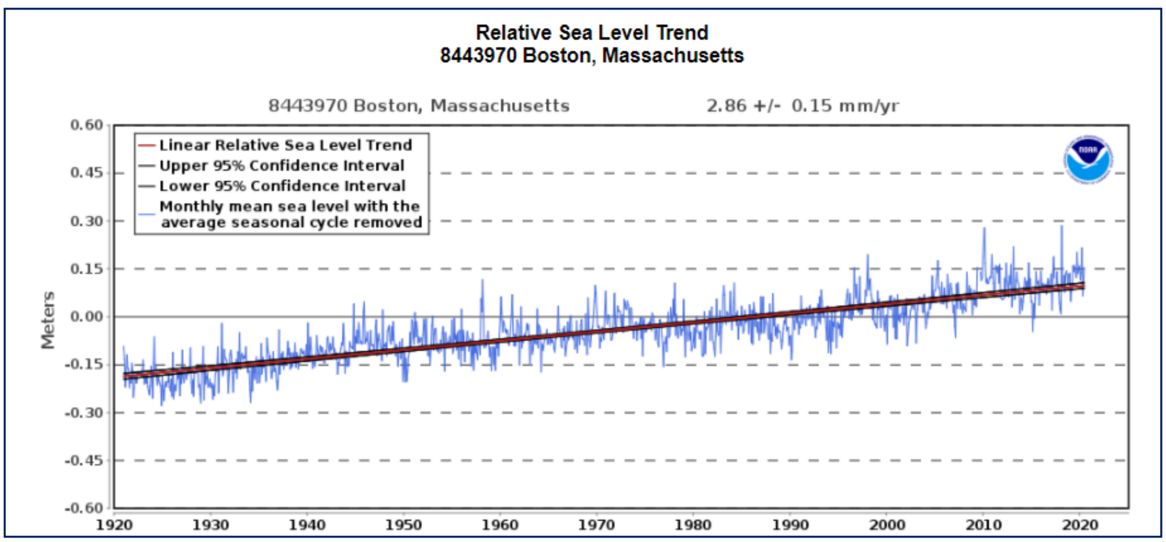


Figure B3-6: Relative sea level trend at Boston, MA (NOAA)

The sea level change rates required by USACE for scenario-based analysis for future conditions through 2120 are shown in Figure B3-7 and Table B3-2. The projected sea level changes after 50 and 100 years from 2020 are shown highlighted in green in Table B3-2. Sea level change values are in feet relative to Mean Sea Level starting from 1992, the midpoint of the present tidal epoch (1983-2001). As shown, the historic rate results in an increase of 0.73 ft through 2070 while the intermediate and high rates would cause increases of 1.27 ft and 2.99 ft respectively, within that same period. Looking out 100 years, a rise of 1.20 ft can be anticipated using the historic rate. The intermediate and high rates of sea level change estimate rises of 2.66 ft and 7.28 ft, respectively.

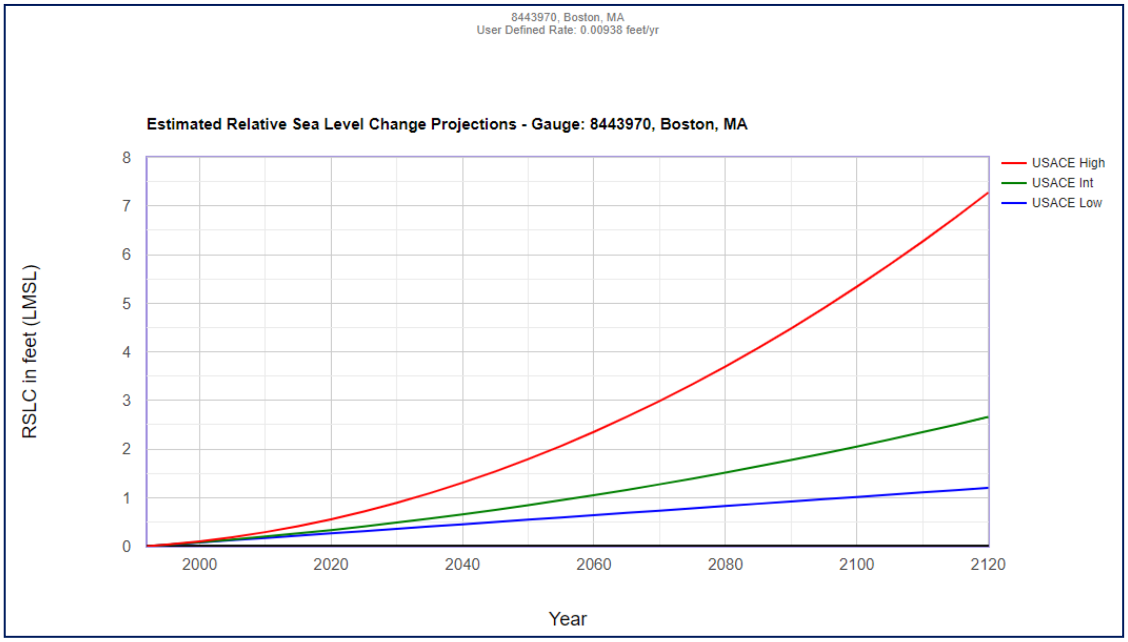


Figure B3-7: USACE sea level change curves for Boston, MA from 1992-2120

Table B3-2: USACE sea level change predictions for Boston, MA

8443970, Boston, MA							
User Defined Rate: 0.00938 feet/yr							
All values are expressed in feet relative to LMSL							
Year	**USACE Low**	**USACE Int**	**USACE High**	**Year**	**USACE Low**	**USACE Int**	**USACE High**
1992	0	0	0	2060	0.64	1.05	2.35
1995	0.03	0.03	0.03	2065	0.69	1.16	2.66
2000	0.08	0.08	0.1	2070	0.73	1.27	2.99
2005	0.12	0.14	0.19	2075	0.78	1.39	3.33
2010	0.17	0.2	0.29	2080	0.83	1.51	3.7
2015	0.22	0.26	0.41	2085	0.87	1.64	4.08
2020	0.26	0.33	0.55	2090	0.92	1.77	4.48
2025	0.31	0.41	0.71	2095	0.97	1.91	4.9
2030	0.36	0.49	0.89	2100	1.01	2.05	5.34
2035	0.4	0.57	1.09	2105	1.06	2.2	5.79
2040	0.45	0.66	1.31	2110	1.11	2.35	6.27
2045	0.5	0.75	1.54	2115	1.15	2.5	6.76
2050	0.54	0.84	1.79	2120	1.2	2.66	7.28
2055	0.59	0.94	2.06				

A comparison of the USACE sea level change projections to the tide gage record at Boston using the USACE Sea Level Tracker (https://climate.sec.usace.army.mil/slr_app/) (Figure B3-8) shows the 19-year Mean Sea Level moving average to presently be nearest to the USACE high curve.

The WHG analysis utilized a sea level change projection consistent with those being applied across the Commonwealth of Massachusetts and published by Mass CZM of 4.29 feet by 2070. This sea level rise assumes the Representative Concentration Pathway (RCP) 8.5 and was developed specifically for the Commonwealth of Massachusetts (DeConto and Kopp, 2017). While this sea level change projection exceeds the three USACE curve projections for 2070, it falls within the range of NOAA's 2017 predictions (Table B3-3). The projection of 4.29 feet is between the NOAA 2017 Intermediate-High (3.79 ft) and NOAA 2017 High (5.16 ft) estimates. Use of the projection of 4.29 ft was limited to the WHG analyses for approximating future shoreline position and estimation of future erosional losses at Town Neck and Springhill Beaches, as described in Section 3.2-3.4 and Appendix C, Chapter 2.2-2.3.

Given the sponsor's preference of aligning with the Mass CZM projections and the position of the Sea Level Tracker relative to the three USACE curves, plans were initially formulated using the USACE high curve. However, as will be discussed in Section 6 and Section 7, plan selection is not sensitive to sea level change scenario.

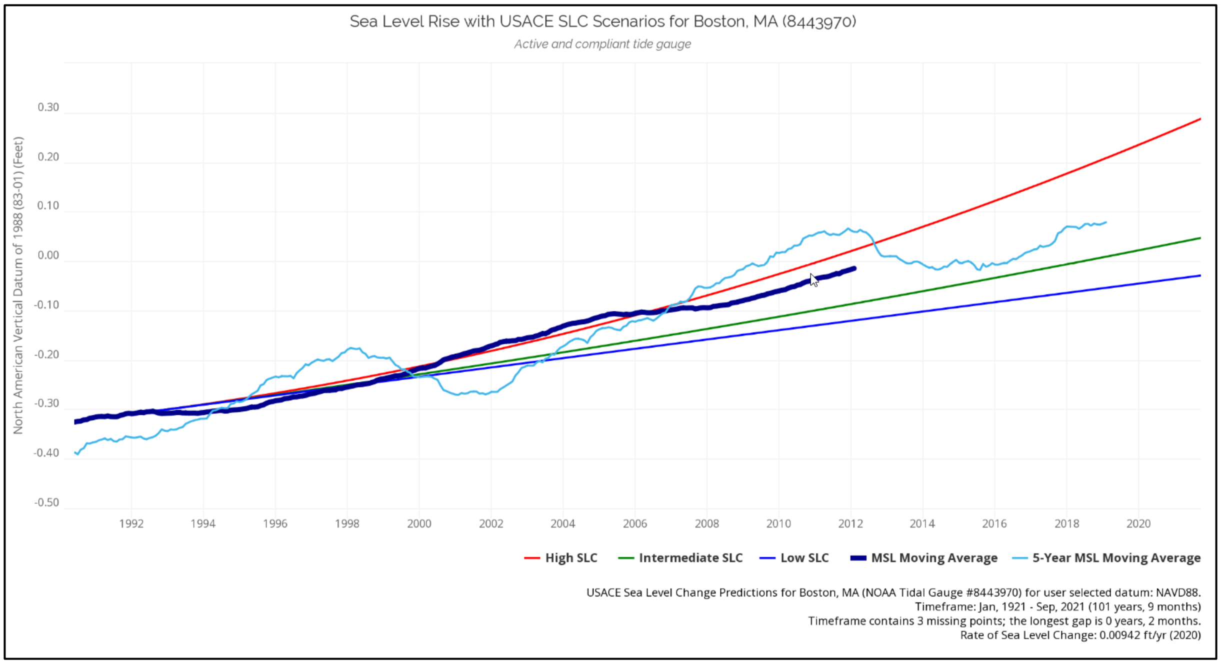


Figure B3-8: USACE Sea Level Tracker for Boston, MA from 1992-present

Table B3-3: NOAA 2017 sea level change projections for Boston, MA

Scenarios for BOSTON							
NOAA2017 VLM: 0.00259 feet/yr							
All values are expressed in feet relative to LMSL							
Year	**NOAA2017 VLM**	**NOAA2017 Low**	**NOAA2017 Int-Low**	**NOAA2017 Intermediate**	**NOAA2017 Int-High**	**NOAA2017 High**	**NOAA2017 Extreme**
2000	0.11	0.11	0.11	0.11	0.11	0.11	0.11
2010	0.14	0.27	0.31	0.37	0.47	0.57	0.6
2020	0.16	0.44	0.5	0.67	0.87	1	1.06
2030	0.19	0.57	0.67	0.96	1.26	1.55	1.69
2040	0.21	0.7	0.87	1.29	1.78	2.31	2.54
2050	0.24	0.87	1.06	1.72	2.37	3.13	3.56
2060	0.27	1.06	1.29	2.18	3.03	4.15	4.8
2070	0.29	1.23	1.49	2.64	3.79	5.16	6.18
2080	0.32	1.29	1.62	3.13	4.57	6.25	7.56
2090	0.34	1.42	1.78	3.65	5.43	7.59	9.33
2100	0.37	1.49	1.95	4.21	6.41	9.07	11.2

3.3.2. Future Shoreline Position—Methodology

WHG estimated the future shoreline position considering the long-term shoreline change rates at Town Neck and Springhill Beaches and a sea level change projection of 4.29 ft by 2070. First, a projected 2068 shoreline (50 years from 2018) was generated using the long-term rates of change at each shoreline change transect. In this step, the rate of erosion determined from the long-term shoreline change analysis was assumed to remain constant over the next 50 years. The present shoreline position and profiles were translated landward using these rates over 50 years. Next, the sea level rise projection was applied. The MHW shoreline in 2068 was estimated by adding the sea level rise of 4.29 ft to the present day MHW elevation to yield a projected MHW shoreline at elevation 8.4 ft NAVD88 (Figure B3-15). This methodology is further detailed in Appendix C, Chapter 2.2-2.3.

3.3.3. Future Shoreline Position—Results

The present and projected future shoreline positions along Town Neck and Springhill Beaches are shown from west to east in Figures B3-9 to B3-11. The present MHW shoreline is depicted in black while the 2068 shoreline is shown in red. Figure B3-12 shows areas projected to be inundated at MHW using the future MHW elevation of 8.4 ft NAVD88. The predicted MHW shoreline is again depicted in red. While shoreline loss is predicted throughout the project area, it is most severe along the east end of Town Neck Beach and in the vicinity of Old Harbor Inlet. In fact, almost a complete loss of the barrier beach at Town Neck is predicted. In addition to the direct loss of beach areas, the future condition would result in significant ecological impacts to the expansive saltmarsh system inland of Old Harbor Inlet as well as lead to increased flooding of the Route 6A/Downtown area during storm events.

Future shoreline retreat due to sea level change was also evaluated using the Bruun rule. The Bruun rule assumes an equilibrium beach profile is maintained as sea level rises. Therefore, for a given increase in water level, a response in the horizontal recession of the beach profile can be predicted. Shoreline retreat using Bruun rule was calculated for the three USACE sea level change scenarios and the Mass CZM sea level change projection on a representative beach profile for the Town Neck Beach area. Greater shoreline recession is predicted in response to greater increases in sea level. While results from the Bruun rule analysis were comparable to the future shoreline positions mapped in Figures B3-9 and B3-10 along Town Neck Beach, the Bruun rule predicted greater recession in the western end of Town Neck Beach where the beach is backed by development and less recession to the eastern end of Town Neck Beach where the barrier beach fronts the marsh. This is likely due to the simplified approach taken to use a representative beach profile in the Bruun rule analysis while the projected shoreline positions generated by WHG were based on localized erosion rates at multiple transects.

Figure B3-9: Existing (2018) and projected (2068) MHW shoreline positions, west end of Town Neck Beach (WHG, 2020)

Figure B3-10: Existing (2018) and projected (2068) MHW shoreline positions, east end of Town Neck Beach (WHG, 2020)

Figure B3-11: Existing (2018) and projected (2068) MHW shoreline positions, Springhill Beach (WHG, 2020)

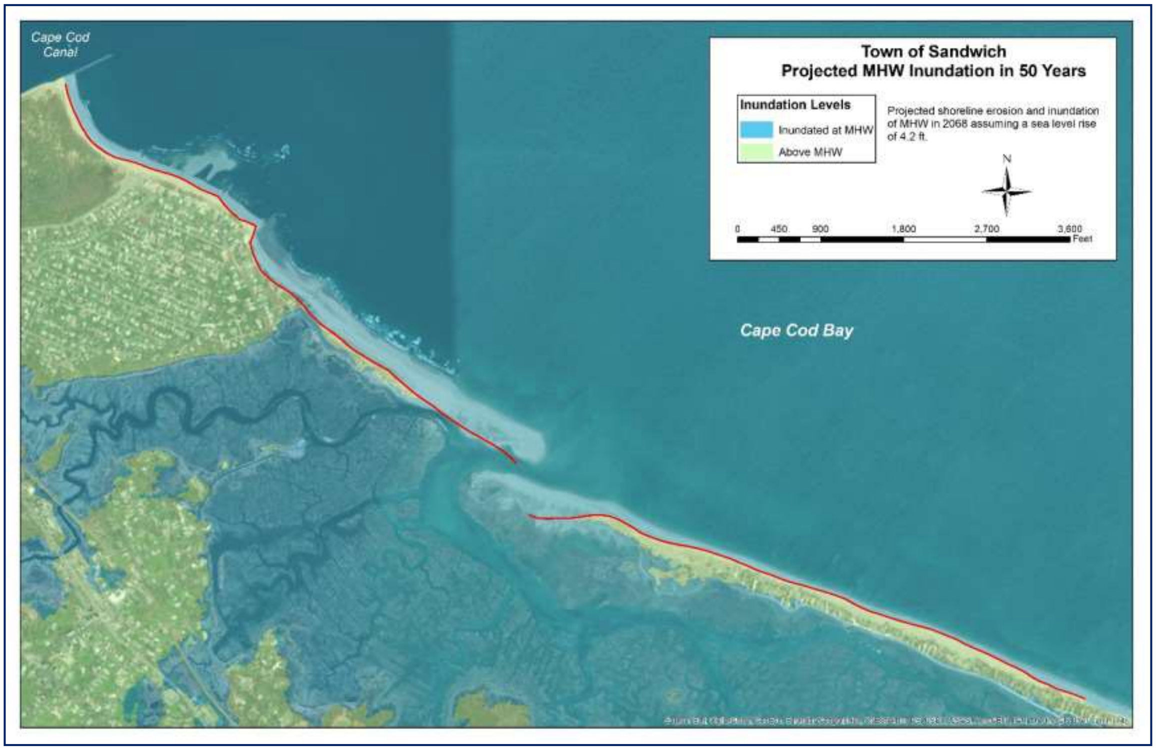


Figure B3-12: Projected (2068) areas of MHW inundation (WHG, 2020)

3.4. Volume Losses

Volume losses associated with shoreline change were predicted by WHG along Town Neck and Springhill Beaches for the previous 50 years (from approximately 1968 to 2018) and over the next 50 years (2018 to 2068). Additional detail can be found in Appendix C, Chapter 2.3. To determine volume losses, thirty shore perpendicular transects spaced at 500 ft intervals were used 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 beach. By comparing the present-day beach profiles to profiles representing the past and future shoreline conditions, WHG developed estimates of the volume of sediment lost in the past 50 years and anticipated to be lost over the next 50 years. The locations of the thirty transects used in this analysis are shown in Figure B3-13.

Figure B3-13: Volume loss transect locations (WHG, 2020)

The 2068 beach profiles were generated by translating the 2018 profiles landward, using the previously calculated long-term shoreline change rates. A similar but seaward translation of the 2018 shoreline was used to represent the position of the 1968 shoreline. A projection of the volume of sand lost over the past 50 years and expected to be lost over the next 50 years was estimated by determining the change in area between the present and past as well as the present and future shorelines, while also accounting for the distance along the shoreline. An example of the beach profile translation is shown in Figures B3-14 and B3-15.

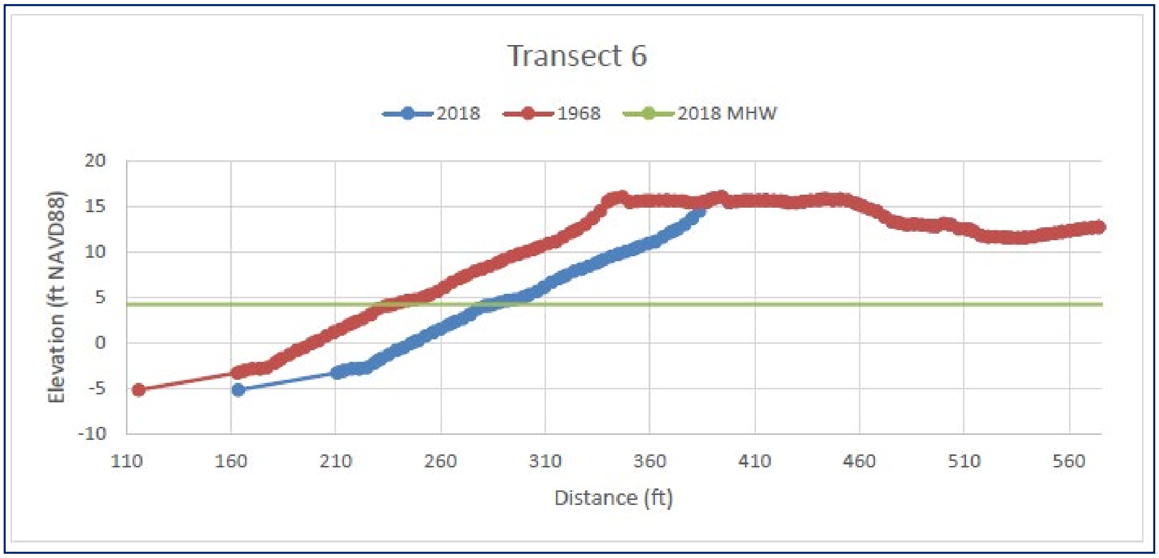


Figure B3-14: Example transect profile translation from 1968-2018(WHG, 2020)

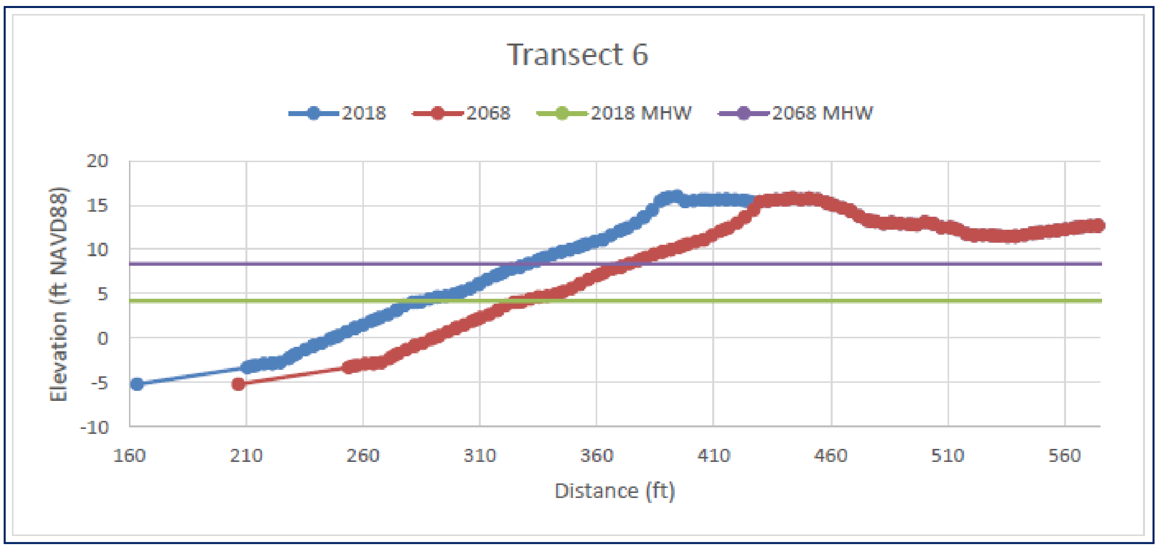


Figure B3-15: Example transect profile translation from 2018 to 2068(WHG, 2020)

This analysis determined the volumetric loss of shoreline over the past 50 years to be 782,000 cubic yards or 1.45 cubic yards per foot of shoreline per year. Over the next 50 years, the estimated volume loss of beach is predicted to be approximately 900,000 cubic yards or 1.66 cubic yard per foot of shoreline per year. It should be noted that because these profiles terminate offshore at -5 ft NAVD88 (0 ft MLW) and do not extend out to the depth of closure, these volumes may not fully capture the cross-shore volume loss.

The volume change below the measured beach profiles between -5 ft NAVD88 and the depth of closure, -23 ft NAVD88, was approximated separately using the average long-term erosion rate. This volumetric loss was estimated to be 0.725 cubic yards per foot of shoreline per year. Added to the volume changes computed above through shoreline translation above -5 ft

NAVD88, volumetric loss of shoreline over the past 50 years could be closer to 1,175,000 cubic yards or 2.175 cubic yards per foot per year. Over the next 50 years, the estimated volume of loss could be as much as 1,288,000 cubic yards or 2.385 cubic yards per foot per year, considering losses below MLW to the depth of closure. Considering the range of potential sea level change presented in Section 3.3.1, future volumetric losses will likely be between the value calculated using the past 50 years, assuming the low SLC curve represents a continuation of the past trend, and the volumetric loss calculated using a high rate of SLC.

3.5. Shoreline Movement Summary

Local and regional analyses of historical shoreline change were performed for the Canal region. The data used to compile the analyses were derived from the Massachusetts Shoreline Change Project, aerial photography, historical maps, and digital ortho-photographic quads.

WHG examined the short- and long-term rates of shoreline change at 139 shore-normal transects on Town Neck and Springhill Beaches in Sandwich.

The Town Neck Beach shoreline from the Cape Cod Canal to the longer groin located near the intersection of Dillingham Avenue and Freeman Avenue (approximately Transect 31) has been relatively stable in both the short- and long-term, experiencing smaller erosion rates in the long-term (-1 ft/year) and areas of accretion in the short-term.

Increasing rates of erosion were observed in both the short- and long-term moving east of the stabilizing groin in the vicinity of Transect 31 to Old Harbor Inlet at Transect 74. While long-term erosion rates in this area range from -2 to -5 ft/year per year, much of this area has short-term erosion rates between -6 and -10 ft/year, while the 1,400 ft stretch of shoreline updrift of the inlet shows a dramatic increase in erosion up to -25 ft/year.

Springhill Beach shows consistent but decreasing rates of erosion in both the short- and long-term. Long-term erosion rates are approximately -2 ft/year whereas short-term erosion rates are greater at approximately -5 ft/year. The erosional trend continues to approximately Transect 108, or 10,800 ft downdrift of the Cape Cod Canal, where the rates of erosion level off and the shoreline is increasingly stable. This distance of 10,800 ft was selected as a reasonable estimate of the extent of the influence of the Canal on downdrift erosion.

USACE investigated shoreline change rates along an approximately 23 mile segment of shoreline about the Canal from Stage Point in Plymouth to Sandy Neck in Barnstable to define a regional rate of shoreline change and put the erosion at Sandwich in context with a larger area.

The long-term regional shoreline change rate was -0.29 ft/year for the period from the 1860s to 2018. Within the same time frame, long-term shoreline change for the 10,800 ft shoreline segment east of the Canal was -1.33 ft/year. The difference between the regional and local shoreline change rates was more pronounced over the short-term. Within the region, shorelines accreted at an average of 0.07 ft/year while the shoreline at Sandwich eroded at a rate of 2.58 ft/year. Using the long-term shoreline change information, a residual shoreline change rate was determined by removing the average regional recession rate from the shoreline change rate for the 10,800 ft of shoreline adjacent to the project. This procedure gave a residual recession rate of 1.04 ft/year. Thus, the amount of shoreline recession attributable to the FNP was determined to be 78 percent based on the shoreline change analysis.

WHG also used shoreline translation to estimate volumetric losses for the past 50 years using historic shoreline change rates and for the next 50 years using the same rates of change and a conservative sea level change projection. This analysis determined the volumetric loss of shoreline over the past 50 years to be 782,000 cubic yards or 1.45 cubic yards per foot of shoreline per year. Over the next 50 years, the estimated volume loss of beach is predicted to be approximately 900,000 cubic yards or 1.66 cubic yard per foot of shoreline per year. Extending the volumetric losses calculated using shoreline translation offshore to the depth of closure increased the volumetric loss estimations to 1,175,000 cubic yards or 2.175 cubic yards per foot per year for the past 50 years and up to 1,288,000 cubic yards or 2.385 cubic yards per foot per year for the next 50 years. Given the uncertainty in future sea level change, these volumetric loss estimates can be seen as lower and upper bounds, approximating shoreline response under the low and high sea level change scenarios.

3.6. Sediment Transport – Without project

This section evaluates the regional sediment transport within Cape Cod Bay in the vicinity of the east entrance to the Cape Cod Canal for without-project conditions. For with-project conditions, sediment transport, will be discussed in Section 6.2 of this appendix. The wave modeling effort used to support this analysis will be discussed in Sections 4.2 and 6.1 of this appendix. For greater detail refer to Appendix C, Chapter 4.3.

3.6.1. Grain Size Analysis

Sediment samples at Town Neck and Scusset Beaches were collected by WHG in 2016 to physically characterize the sediments on the beaches surrounding the Canal. Samples were obtained as surface grabs during low tide on March 16-17, 2016. Twelve samples were collected at each beach along six shore-normal transects for a total of 24 samples. Each transect consisted of two samples: one sample collected on the beach above MHW and one collected on the intertidal beach. The locations of the samples taken are shown in Figure B3-15. Results from the Scusset Beach samples are summarized in Table B3-4. 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 B3-5. The results characterize Town Neck Beach 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 composite calculation. These results are consistent with a single glacial source of sediment supplying both Scusset and Town Neck beaches.

Figure B3-15: Grain size analysis sample locations (WHG, 2020)

Table B3-4: Table of Sediment Properties for Scusset Beach Samples

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 B3-5: Table of Sediment Properties for Town Neck Beach Samples

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

3.6.2. Methodology

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

The sediment transport model itself consists of a hydrodynamic component (to determine the wave-induced currents) and a sediment transport component (to quantify the amount of sediment moved by the wave-induced currents). The hydrodynamic component is based on a standard set of equations that are widely accepted and generally used, more specifically known as the steady-state depth-averaged mass and momentum equations for a fluid of constant density. These equations are standard in many surf zone applications (*e.g.,* Mei, 1983) and provide a state-of-the-art representation of the alongshore current. The sediment transport component is based on a recent peer-reviewed and published formulation by Haas & Hanes (2004), which has been shown to be consistent with recent complex formulae for wave-driven sediment transport and with the Coastal Engineering Research Center (CERC) formula for the total (laterally-integrated) alongshore sediment flux in the limit of a long straight beach subject to waves that are uniform alongshore. For a much more detailed explanation of the model the reader is referred to Section 4.3 of Appendix C.

3.6.3. Regional Sediment Transport

The regional wave modeling results (Section 4.2) were used as input into the non-linear sediment transport model. Wave results from each of the average annual directional spectra bin simulations were used to develop the complete summary of sediment movement for various wave conditions. Sediment transport results were also combined to define the average annual sediment transport regime in the vicinity of the east entrance to the Cape Cod Canal.

Sediment flux represents the potential rate of sediment moving along the coast. This rate can be used to quantify the annual sediment transport in reaches within Cape Cod Bay. Subsequently the flux divergence is calculated and indicates areas of erosion and/or deposition. A flux divergence represents erosion, while a flux convergence represents accretion. These calculations all assume that sediment is available for transport on the beach. If the shoreline is armored, doesn't have a sediment source readily available, or is interrupted by shore-normal structures such as groins or jetties, then the sediment transport rates may vary from those presented here. Therefore, these sediment transport rates are likely conservatively high as they assume an infinite supply of sediment, and do not account for morphologic changes to the shoreline.

Figure 3-16 presents the average yearly sediment flux determined using the process-based sediment transport model for the Cape Cod Canal region. The arrows on the figure indicate direction of sediment transport while the colors of arrows indicate magnitude. The sediment flux indicates there is a strong net alongshore sediment transport region from northwest to southeast, consistent with the prevalent northeast wave approach direction. Along Scusset Beach, north of the Canal, 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 is consistent with those presented by Berman (2011), Fitzgerald (1993), and Borrelli et al. (2016). Southeast of the Canal and ending in the vicinity of Knott Avenue, there is a small zone of transport reversal, located in the shadow of the Canal jetties, which limits wave energy from the northeast yet allows energy from the less predominant eastern directions. Net transport at this reversal ranges from 10,000 to 20,000 cy/year toward the northwest. Southeast of the reversal, net alongshore sediment transport patterns continue to be directed toward the southeast, where transport rates range from approximately 35,000 to 45,000 cy/year until reaching Old Harbor Inlet.

In addition to presenting the net overall transport results, Figure B3-16 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 ft/year). Negative values of shoreline change indicate erosion, while positive values indicate accretion. Areas of erosion and accretion generally match the expected patterns of alongshore transport based on the modeled results.

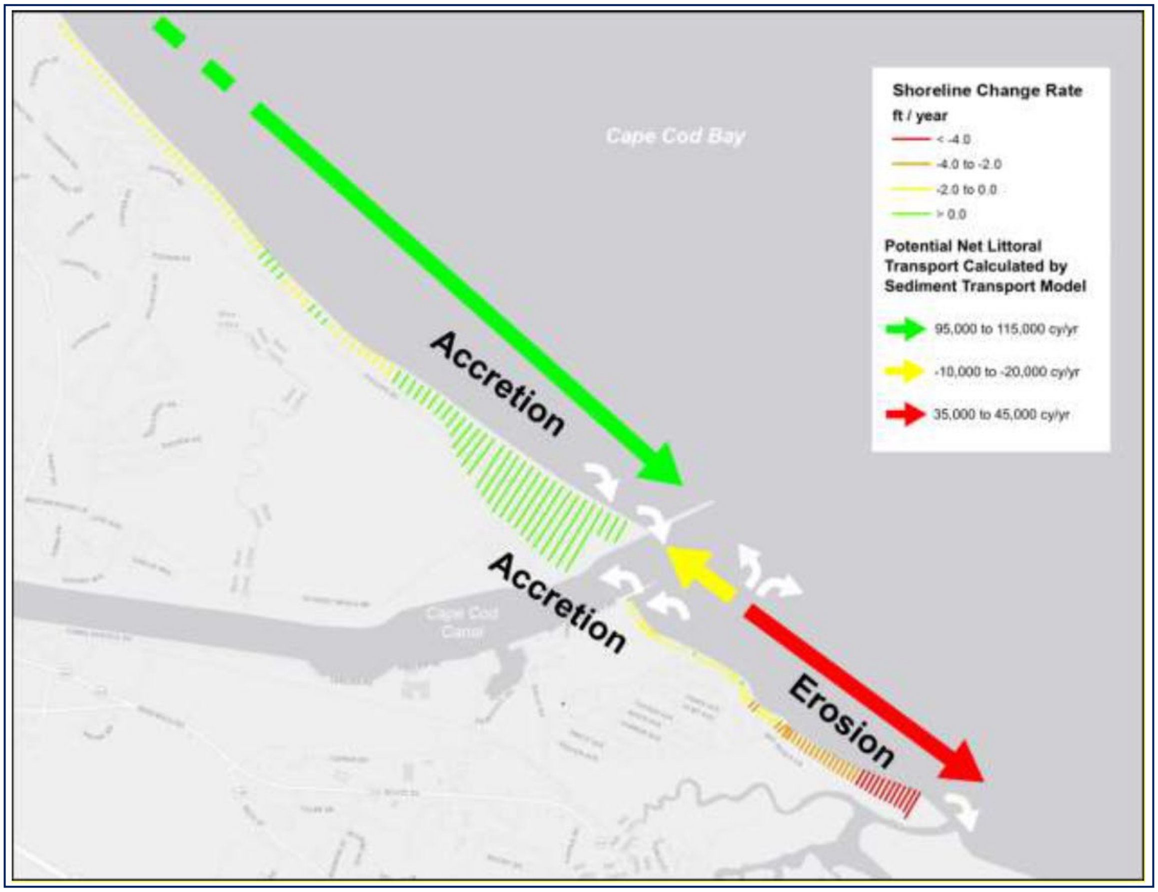


Figure B3-16: Annualized sediment flux and divergence for the Cape Cod Canal region (WHG, 2020)

3.7. Sediment Budget

A sediment budget at the east end of the Cape Cod Canal was developed by WHG to quantify sediment fluxes not captured in the shoreline change analysis and sediment transport model. 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. 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 Qsource and Qsink represent sources and sinks out of the budget cell, ΔV 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 B3-17 from USACE Coastal Engineering Technical Note (CETN IV-15) shows a conceptual box model version of the sediment budget equation with examples of the types of each parameter (Rosati and Kraus, 1999).

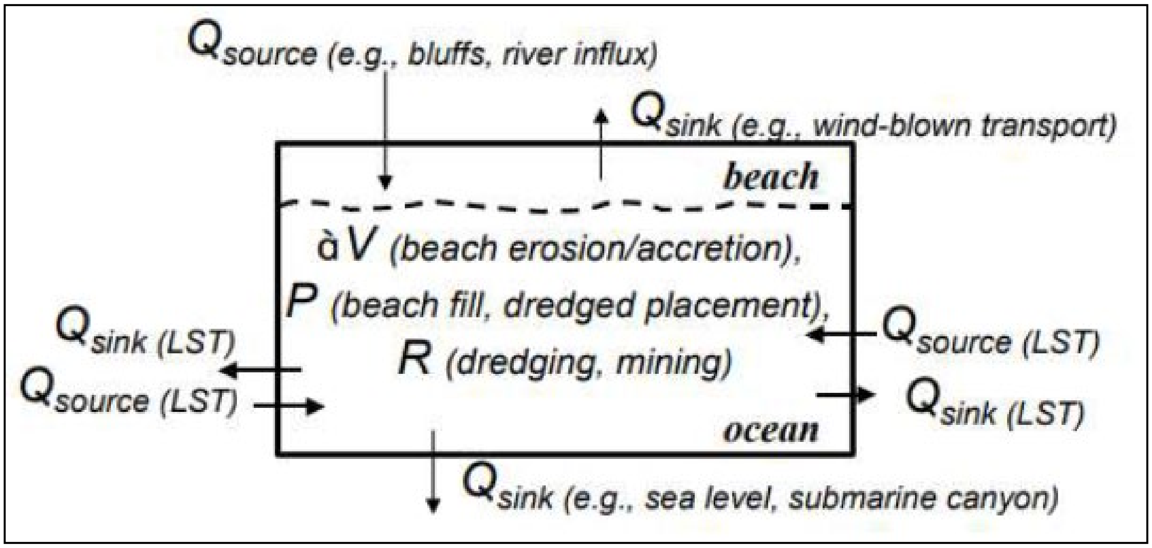


Figure B3-17: Conceptual box model of sediment budget (CETN IV-15)

Woods Hole Group established three sediment budget cells to represent the Cape Cod Canal sediment budget:

- Scusset Beach Cell – from the terminus of the cliffs north of Sagamore Beach to the Canal north jetty
- Cape Cod Canal Cell – the area in between and offshore of the Canal jetties
- Town Neck Beach Cell – from the Canal south jetty to the terminus of the Town Neck Beach spit

The components of the sediment budget, their data sources, and transport rates are summarized as follows:

Q_{LST}: Alongshore sediment transport rates into and out of each cell were estimated using the process-based, wave-induced sediment transport modeling described in Section 3.6 (Appendix C, Chapter 4).

Table B3-6: Alongshore transport rates at sediment budget cell boundaries

Transport Rate Location	Transport Rate (cy/year)
Q_{LST-SC}	95,000 to 96,800
Q_{LST-TN}	43,200 to 45,000

Q_{SLR}: Long-term offshore sediment losses due to sea level rise were estimated using Bruun rule, which relates sediment losses due to sea level rise with the local closure depth and the distance to the depth of closure.

Table B3-7: Sea level rise offshore sediment loss rates

Transport Rate Location	Transport Rate (cy/year)
QSLR-SC	5,700
QSLR-TN	3,200

P: While Town Neck Beach has been nourished a number of times, three significant nourishments have occurred between 1990 and 2016. The volumes of these nourishments shown in Table B3-8 were used to calculate an average annual placement rate of 11,200 cy/year.

Table B3-8: Town Neck Beach nourishments between 1990 and 2016

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

R: Canal dredge records from 1975 to 2016 were used to approximate the average annual volume of material that shoals in the eastern end of the Canal. The volumes shown in Table B3-9 were used to calculate an average annual volume of material shoaling in the channel of 28,100 cy/year.

Table B3-9: Canal dredge history from 1975 to 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

ΔV_{beach}: Volumetric changes on Town Neck and Scusset Beaches were calculated based on shoreline change data. Shoreline change rates were converted to volumetric rates of change (ΔV) by multiplying the rate of change in cross-shore position ($\Delta Y/\Delta t$) over the given alongshore length (ΔX), assuming that the shoreline translates parallel to itself over a given active depth (Da) defined as the height from the depth of closure to the berm height, such that:

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

This analysis resulted in a volumetric rate of change for the Scusset Beach cell, ΔV_{SC}, of 57,400 cy/year and a volumetric rate of change for the Town Neck Beach cell, ΔV_{TN}, of -38,500 cy/year.

ΔV_{canal}: The volumetric rate of change of material deposited offshore of the Canal was based on work done by Borelli et al. (2016) as part of a study evaluating sediment transport for the Sandwich and Barnstable coasts which noted that sediment moving around the Canal jetties was likely being deposited in deeper water offshore of the Canal entrance. Borelli et al. quantified the change in size of the offshore ebb shoal between 1933 and 2016 to arrive at a volumetric rate of change of 9,800 cy/year in the area offshore of the east end of the Canal.

Q_{canal}: Q_{canal1} and Q_{canal2} represent the rates of sediment transport from the surrounding beaches, Scusset and Town Neck, respectively, into the Canal. These values were determined by solving the system of equations developed for the sediment budget which describe the sediment movement in an out of the three sediment budget cells as follows:

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

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

Q_{canal1} was found to be in the range of 34,600 to 36,400 cy/year and Q_{canal2} was found be in the range of 1,500 to 3,300 cy/year.

Figure B3-18 shows a graphical representation of the sediment budget results as determined for this study by WHG.

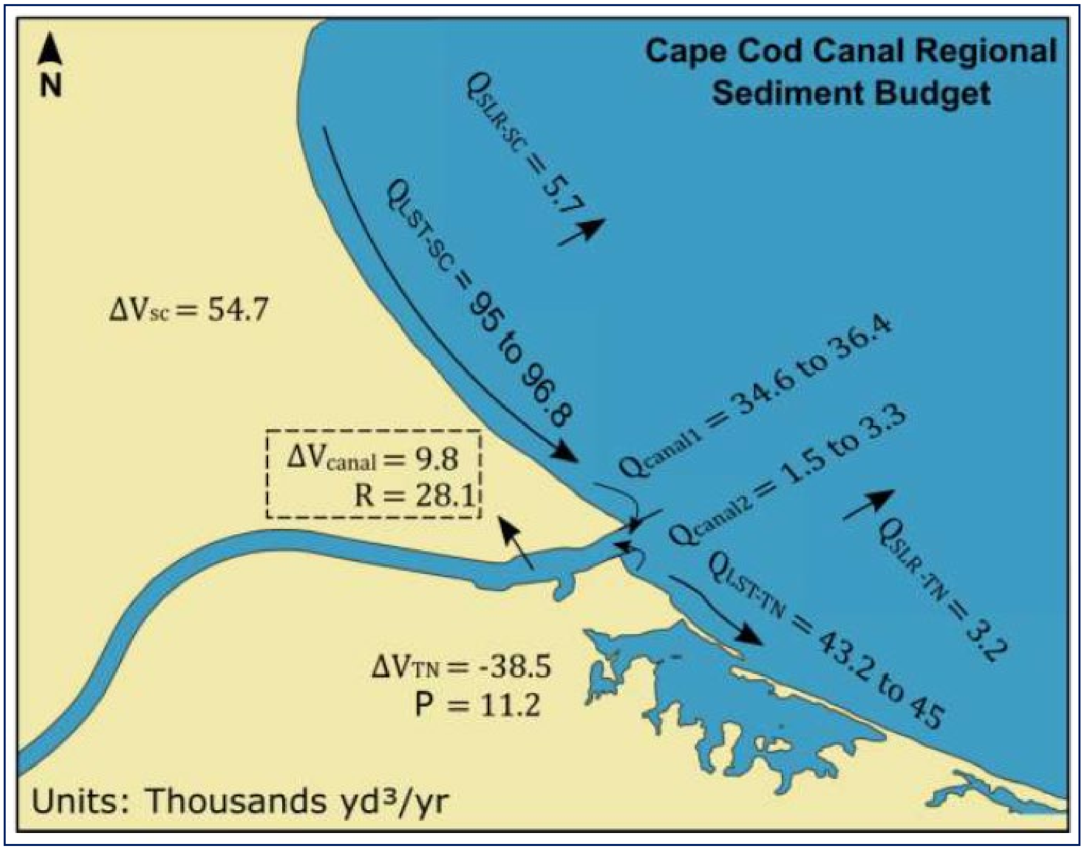


Figure B3-18. Sediment budget for the Canal region (WHG, 2020)

Based on the results of the sediment budget analysis, it is evident that the Canal and its navigation structures affect sediment transport processes in the vicinity of the Canal. The average annual longshore sediment transport rate updrift of the Canal was estimated at 95,000 to 96,800 cy/year. Of this, the impoundment rate updrift of the north jetty was estimated at 54,700 cy/year, the shoaling rate in the Canal from dredge records was 28,100 cy/year, and the volume lost offshore of the Canal was 9,800 cy/year. Material removed from the channel was placed on Town Neck Beach at a rate of 11,200 cy/year. Given this, the volumetric loss rate attributable to the Canal FNP was estimated to be 81,400 cy/year or 85 percent of the approximately 95,900 cy/year updrift alongshore transport.

3.8. Summary

As mentioned, the purpose of the Section 111 project is to determine if there are negative impacts from the Canal FNP on the downdrift shorelines in Sandwich, MA, to determine the level of impacts from the project, and develop mitigation alternatives for negative impacts. The impacts of the Canal and its navigation structures were assessed by explicit (measured) and implicit (potential transport) approaches. Shoreline change analysis provided an explicit evaluation of shoreline impacts, whereas the sediment budget analysis allowed for the evaluation of the potential loss of sediment to downdrift beaches.

Long-term shoreline change analysis showed that shoreline recession attributable to the inlet extends approximately 10,800 ft to the east of the Canal. A background shoreline recession rate was calculated at 0.29 ft/year from regional shoreline position data. Residual shoreline change rates, or the change attributable to the Canal inlet, were then determined by removing the background trend from the shoreline change rates. This subtraction resulted in a residual shoreline change rate of -1.04 ft/year. The percent of shoreline recession attributable to the Canal FNP was determined at 78 percent.

WHG estimated an annual volume loss per linear foot of beach of 1.45 cy/ft/year on Town Neck and Springhill Beaches from 1968 to 2018 and predicted an annual volume loss per linear foot of beach of 1.66 cy/ft/year for the next 50 years. Utilizing the percent of shoreline erosion attributable to the Canal FNP of 78 percent over the 10,800 ft length of shoreline experiencing erosion, the loss of 12,200 to 14,000 cy/year of material can be attributed to the Canal. Because the transect profiles used in this volume loss approximation did not extend offshore to the depth of closure, these volumes may not fully capture the cross-shore volume loss.

The shoreline change analysis focused on sediment losses at Town Neck and Springhill Beaches and does not account for volumes of sediment impounded updrift of the Canal at Scusset Beach, material that shoals in and is dredged from the Canal, and volumes deposited offshore of the Canal entrance. Because of these limitations in the explicit approach, an implicit assessment of volume lost to downdrift beaches based on the sediment budget was performed.

The sediment budget analysis indicated the volumetric loss rate attributable to the Canal FNP to be 81,400 cy/year or 85 percent of the approximately 95,900 cy/year updrift alongshore transport. While this volumetric rate is about 6 times greater than the volume determined through the shoreline change method, the percent attributable to the Canal FNP is comparable. The implicit sediment budget method also accounts for potential losses along all downdrift beaches and not just those in the impact erosion zone identified in the explicit shoreline change analysis method.

4. Wave Modeling

In order to arrive at an accurate estimation of the sediment transport in the region, wave model results can be used to generate the sediment transport flux. This would include waves coming from all directions and having various wave heights and periods. The combination of all the directional approach cases allows for an assessment of the average annual wave climate. Wave modeling was also used to assess changes in the wave climate in the evaluation of structural alternatives including modifications to the Canal jetties and the groin system on Town Neck Beach. For a complete discussion of the wave model approach and development, see Appendix C, Chapter 4.

4.1. Wave Data Analysis and Sources

The wave climate in the vicinity of the east entrance to the Cape Cod Canal was assessed by considering locally generated wind waves, regional swell waves, and high energy storm waves. The area surrounding the Canal 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 high frequency seas and longer period waves. Figure B4-1 illustrates the distribution of wave types and approaches influences the beaches surrounding the Canal. A sizeable portion of Cape Cod Bay is sheltered from the Atlantic Ocean by the outer Cape (indicated by the blue region in Figure B4-1) and waves from this direction are therefore produced by local winds. However, long period wave energy from the Atlantic Ocean (indicated by the yellow region), as well as sea conditions due to both regional and local winds, originates from the north. Additionally, a small approach angle (indicated by the red region) consists of both locally generated wind waves and swell waves from the northeast that enter Cape Cod Bay by wave refraction and diffraction about the tip of the Cape.

For this study, the USACE Wave Information Study (WIS) time series of wave and wind data were utilized to describe the wave climate. WIS Station 93, located offshore of Plymouth at the entrance to Cape Cod Bay provided 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 B4-1 (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.

While WIS Station 93 represents data from 1976 to 1995 and there is a more recent WIS data set available (1980-2014), data from WIS Station 93 was used for the wave transformation assessment for the following reasons:

1. The more recent data sets do not have the same spatial coverage as the previous WIS data set. WIS Station 93 is located much closer to the Sandwich shoreline and better represented ocean based swell conditions for Sandwich since it was located within Cape Cod Bay, thereby capturing the influence of the outer Cape. The more recent data is located well outside of Cape Cod Bay and would require a much more complicated wave transformation model to determine the transformations that propagate toward Sandwich.

2. The data from WIS Station 93 had been used extensively and successfully in prior wave modeling assessments conducted by WHG, including validation of sediment transport rates and involved local transformation to the Sandwich area.

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

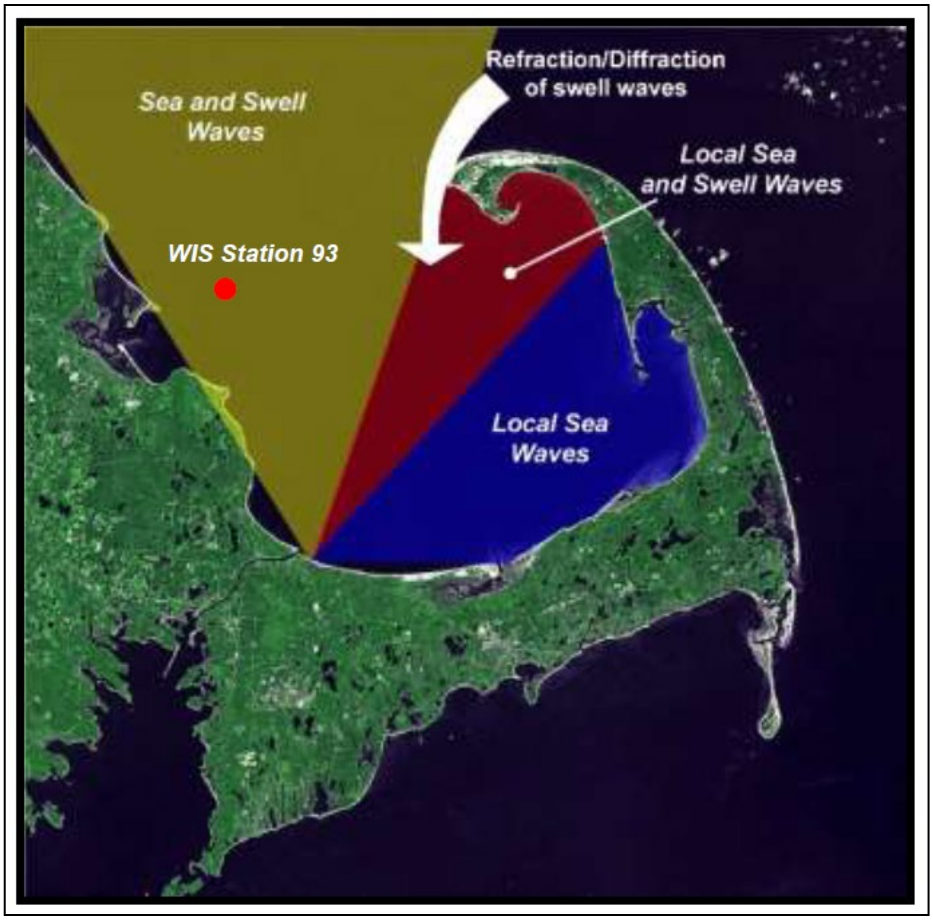


Figure B4-1. Wave climate by direction approaching the Canal region (WHG, 2020)

4.1.1. Locally Generated Waves

Local, historical wind information from WIS Station 93 was analyzed to determine the magnitude and direction of wind-generated waves in the area offshore of the Canal. Figure B4-2 shows the wind rose generated from the 20-year WIS time series. The most common direction of wind approach is from the west, with the strongest winds from the northwest.

However, 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 and influence nearshore sediment transport in the area. As such, locally generated wind waves were described by the data from between 25 degrees to 115 degrees (blue and red regions), while ocean generated waves were described by data from 295 degrees to 25 degrees (yellow region).

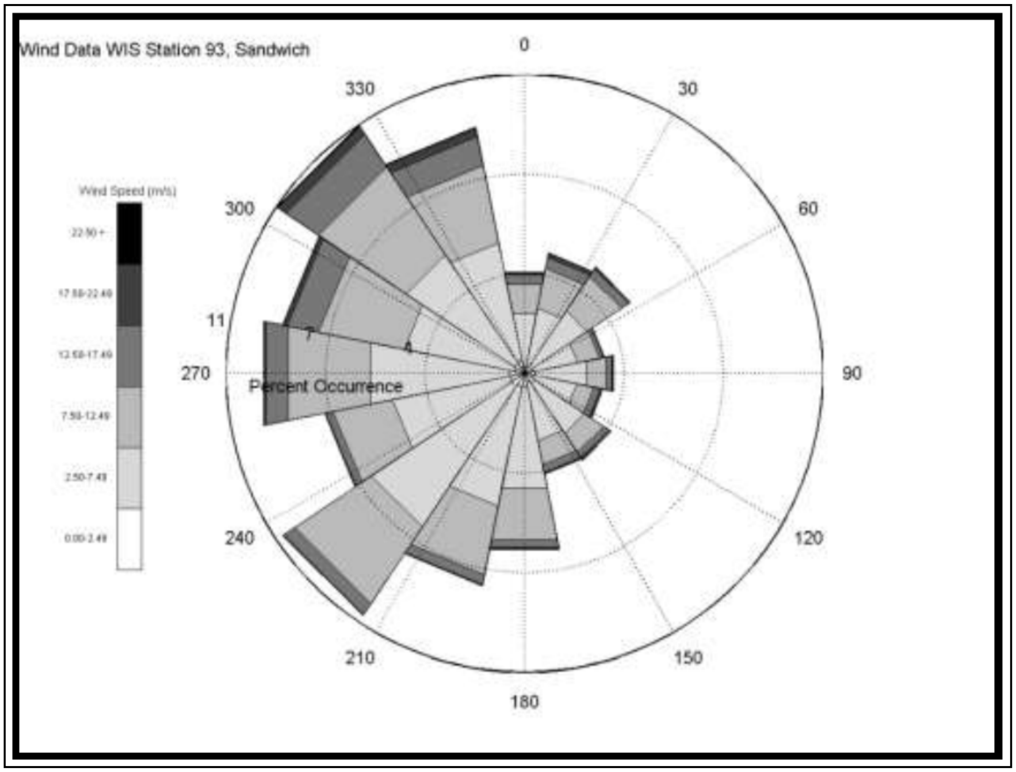


Figure B4-2: Wind rose generated from WIS Station 93 (January 1, 1976 – December 31, 1995)

Average winds speeds from each 22.5 degree approach direction were used to calculate wind-generated local wave conditions using the USACE Automated Coastal Engineering System (ACES) software. The resulting locally generated wave conditions predicted by WHG in ACES for each wind band are summarized in Table B4-1.

Table B4-1: Locally generated wave conditions from ACES predictions in Cape Cod Bay given average wind conditions

Wind Direction (Band), degrees	Average Wind Speed, mph	Significant Wave Height, ft	Peak Wave Period, sec	Wave Direction, degrees
36.25 (25-47.5)	15.7	2.37	3.5	43
58.75 (47.5-70)	15.5	2.35	3.5	61
81.25 (70-92.5)	14.8	2.15	3.3	76
103.75 (92.5-115)	15.5	1.67	2.8	82

4.1.2. Regional Swell Waves

Although a portion of Cape Cod Bay is sheltered from the Atlantic Ocean, the opening to the north provides an entry point for long period wave energy that has the potential to result in significantly longer period waves at the area surrounding the Cape Cod Canal. The energy associated with these waves was determined using the 20 year wave hindcast from WIS Station 93, located in a water depth of approximately 59 feet. Figure B4-3 shows a wave rose of the significant wave heights from the WIS point. Only waves that entered Cape Cod Bay through the northern opening (yellow and red regions on Figure B4-1) were considered significant for the processes affecting the Canal region. The WIS wave rose shows that both the largest waves and the most frequently occurring waves come from the north through north-northeast directions. For application to the nearshore wave modeling, described in Section 4.2, WHG transformed the wave conditions at the WIS station to a water depth of 26 feet offshore of the Canal using the wave transformation model WAVETRAN. Statistics of the transformed swell waves are presented in Table B4-2. The largest transformed swell waves generated in the Atlantic Ocean that affect the Canal region enter from the northeast bin (25-47.5 degrees).

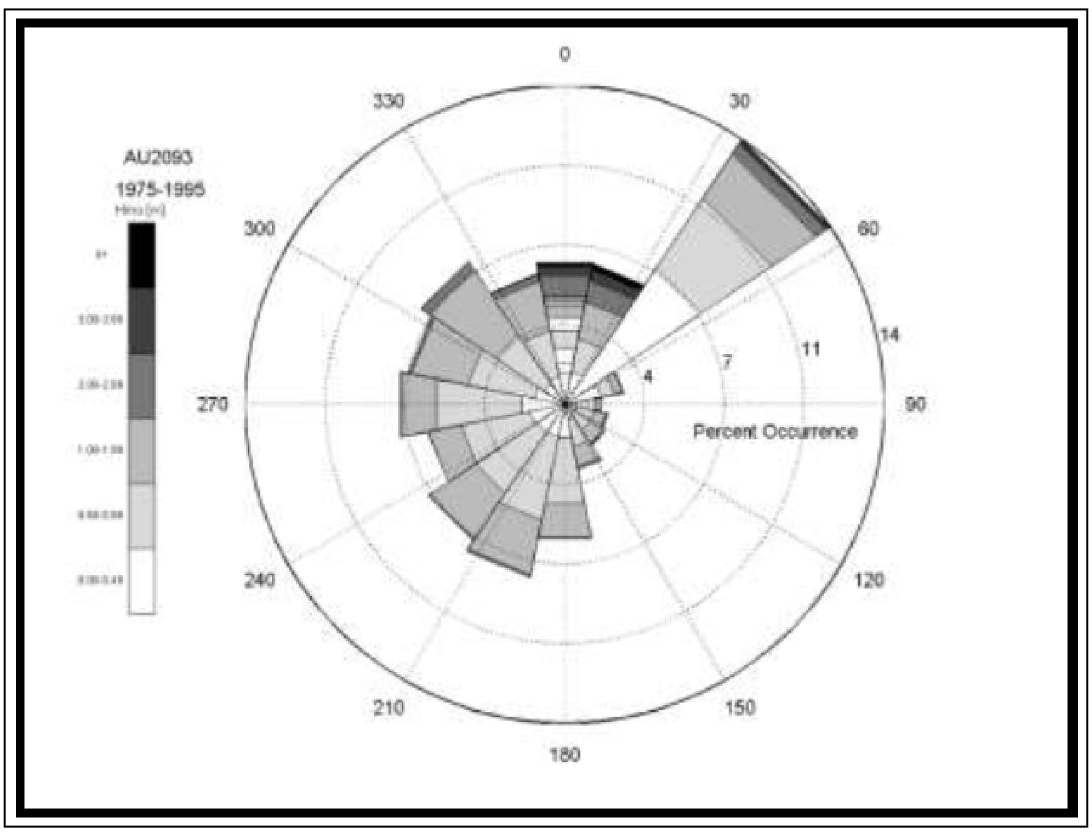


Figure B4-3: Wave rose generated from WIS Station 93 (January 1, 1976 – December 31, 1995)

Table C4-2: Average transformed swell wave conditions

Wave Direction Band, degrees	Significant Wave Height, ft	Peak Wave Period, sec	Wave Direction, degrees
295-317.5	2.40	4.0	306
317.5-340	2.82	4.2	329
340-2.5	3.08	4.9	351
2.5-25	3.61	5.4	10
25-47.5	2.23	11.0	25

4.1.3. Storm Waves

In addition to the average local wind-generated and regional swell wave conditions, a major component of the wave climate near the Cape Cod Canal consists of storm waves. Since high-energy events have a significant impact on many physical processes (and in most cases, dominate sediment transport), it is crucial to include storm simulations in wave modeling to assess the potential impact of a storm on the shoreline and the potential sediment transport within the Canal region. 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 hurricanes. To represent storm conditions extremal wave statistics were calculated using the transformed WIS wave conditions time series to estimate extreme storm wave conditions. Table B4-3 presents the results of the extremal analysis performed on the transformed wave information. These longer period waves are much larger than any potential wind generated wave in Cape Cod Bay.

Table B4-3: Extremal wave heights from WIS Station 93 transformed to nearshore location off Sandwich

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

4.2. Local Wave Modeling

A CMS-Wave model was used to evaluate nearshore wave transformation for the area surrounding the Cape Cod Canal. The CMS-Wave model estimated the refraction, diffraction, shoaling, and breaking of waves in the vicinity of the project area. CMS-Wave is a 2-dimensional half-plane, spectral wave transformation (phase-averaged) model. Developed by the USACE Coastal Inlets Research Program, CMS-Wave is designed for accurate and reliable representation of wave processes affecting coastal inlets. The model is regularly used and is widely accepted in coastal design studies. Many validation 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 ERDC/CHL Technical Report-08-13 (Lin et al., 2008).

4.2.1. Grid Generation

Figure B4-4 presents the modeling grid developed by WHG for the nearshore (local scale) wave modeling. The grid is comprised of rectangular elements with variable cell sizing. Grid resolution varied from 25 m (82.0 ft) at the grid boundaries to 10 m (32.8 ft) in the area of interest nearest the shoreline and Canal. The orientation of the grid was selected to closely parallel the shoreline. The offshore boundary of the grid was chosen at the location where the offshore data was transformed to, at a water depth deep enough that waves would not be significantly affected by ocean bottom friction. Figure B4-5 contains a zoomed in view of the grid around the Canal navigation structures, illustrating the density of the grid elements.

Multiple bathymetric and topographic data sets were utilized from existing available data sources to define the model grid. Bathymetric data previously collected by WHG in 2014 provided high resolution coverage in the nearshore areas surrounding Scusset and Town Neck Beaches. 2013 USACE topo-bathy LiDAR and NOAA's NOS H11695 hydrographic survey collected in 2007 were used to supplement these data and provide fully bathymetric coverage. The sole topographic data source used was the 2013-2014 USGS Post-Sandy LiDAR. All data were converted to the NAVD88 vertical datum and merged to create a seamless topo-bathy surface. Elevations were interpolated to the CMS-Wave grid.

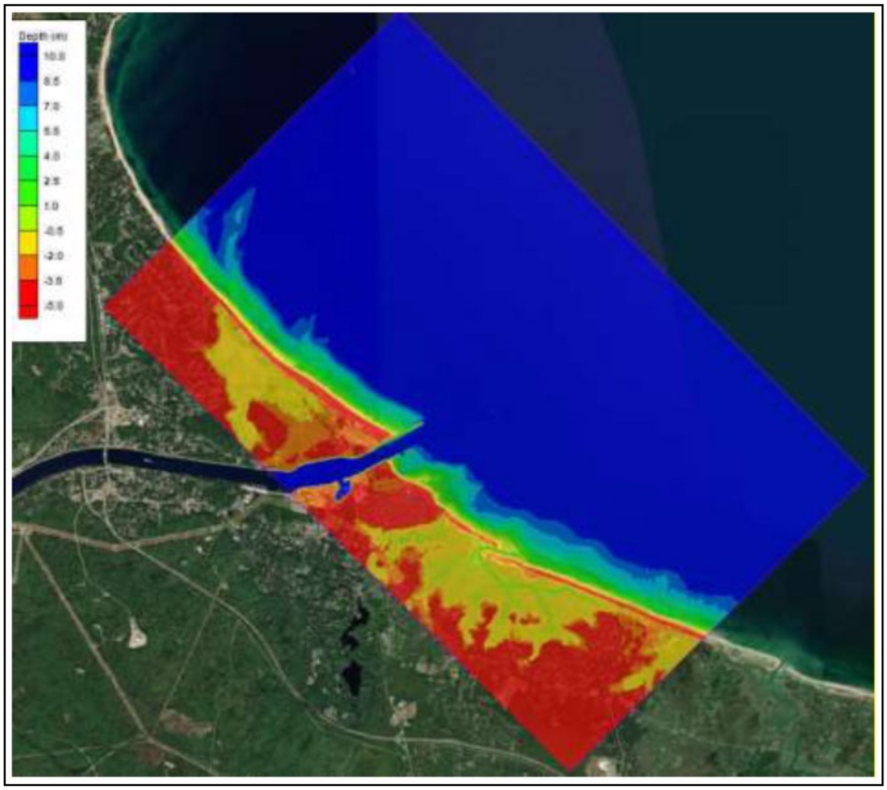


Figure B4-4: CMS-Wave model domain for existing conditions (meters-NAVD88) (WHG, 2020)

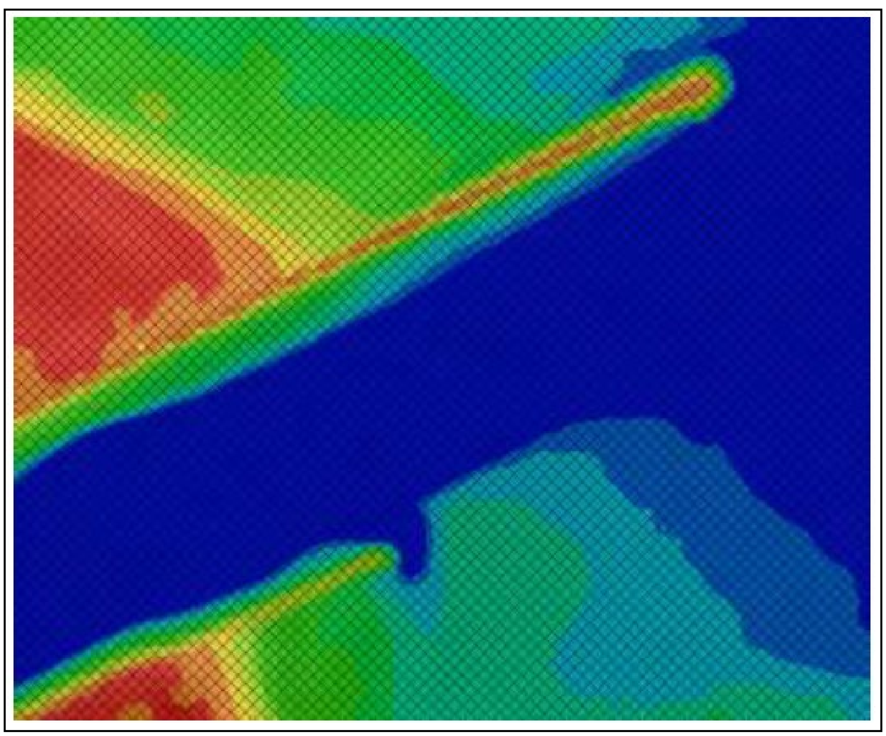


Figure B4-5: Detail of CMS-Wave model grid showing 10-m resolution at the Canal entrance (WHG, 2020)

4.2.2. Wave Input Spectra

CMS-Wave requires the input of a directional wave spectrum, which represents the distribution of wave energy in the frequency and directional 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 B4-4 presents the input conditions from the WIS station data utilized in creating the directional wave spectrum for the simulated hindcast. Data were segregated by direction of approach and the 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 B4-4: Input conditions and scenarios for the CMS-Wave numerical modeling

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

4.2.3. Existing Conditions Simulations

WHG performed existing conditions simulations using both the average annual directional spectra and high energy events. Results from each of the directional cases developed were assessed to assess the existing wave climate. The results from each directional case were combined with the percent occurrence from that direction to create a long term (20 year) evaluation of wave impacts at the shoreline. This long term evaluation was used to assess the existing sediment transport patterns around the Cape Cod Canal. In addition, results from the extreme event cases were utilized to assess the potential extreme waves experienced at the site, as well as sediment transport during extreme events.

4.2.4. Average Annual Directional Approaches

Figure 4-6 shows example sea surface results from the nearshore (local) CMS-Wave model for the north-northeastern approach spectrum (2.5-25 degree bin). The color map corresponds to the distribution of significant wave height (meters) within the modeling domain. Reds and yellows represent higher wave heights, while blues indicate smaller wave heights. Arrows indicate modeled wave directions as they propagate and approach the shoreline. Wave directions become increasingly shore-normal as they approach shore and interact with the bottom. As illustrated in Figure B4-6, wave shadowing occurs in the lee of the Canal navigation structures, particularly the north jetty.

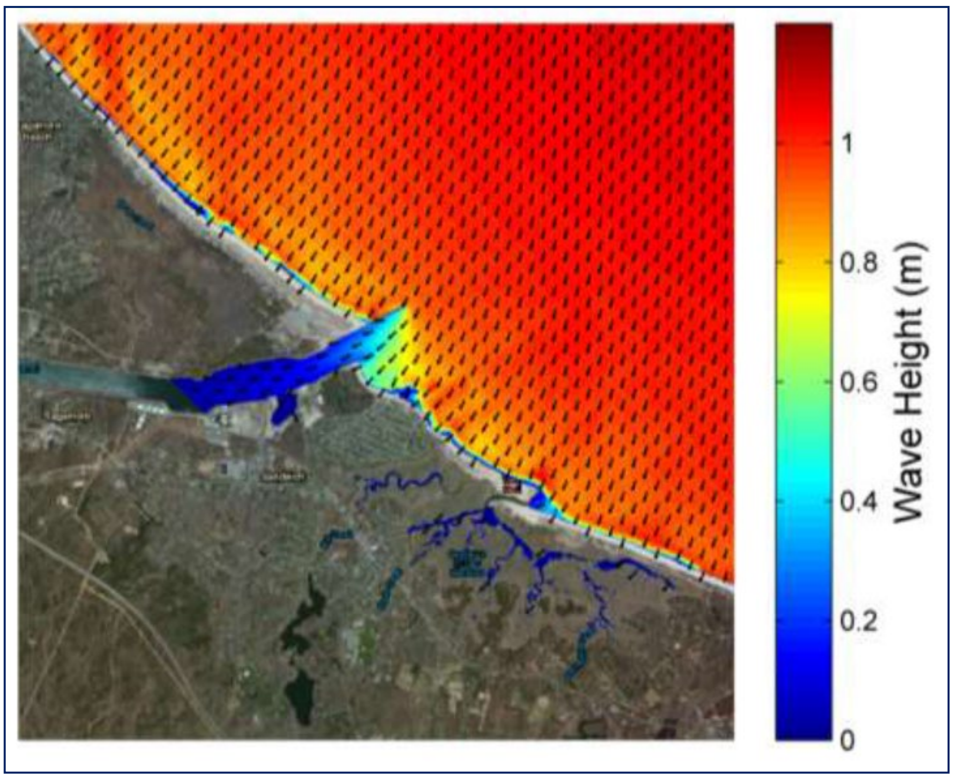


Figure B4-6: Example results of sea surface output from the nearshore (local) wave model (CMS-Wave). The simulation is for the north-northeastern approach spectrum (WHG, 2020)

Results from the nearshore (local) wave model were utilized to produce local sediment transport estimates and were used as part of the alternative analyses.

4.2.5. High Energy Event Simulations

In addition to the average annual approach directions, WHG also simulated high energy events to provide a more complete picture of the existing conditions impacting the Canal region. These simulations consisted of the 10- and 50-year return period storm events. Results indicate that wave heights during storm events are significantly higher than in the average annual directional cases. And, while these large wave events are short-lived, they can potentially have a significant impact on the mobilization of sediments on the beaches surrounding the Canal.

Figure B4-7 presents the sea surface results for the 10-year return period storm in the vicinity of the Canal. Again, reds and yellows indicate larger wave heights while blues represent smaller wave heights. The storm case, consisting of increased wave heights and water levels in the vicinity of the shoreline, presents a more well-structured wave field when compared to the average annual approach directions. The Canal navigation structures, particularly the north jetty, again provide a reduction in wave heights in their lee. The results from the nearshore storm simulations were used to quantify storm impacts on sediment transport and inform the alternatives analysis.

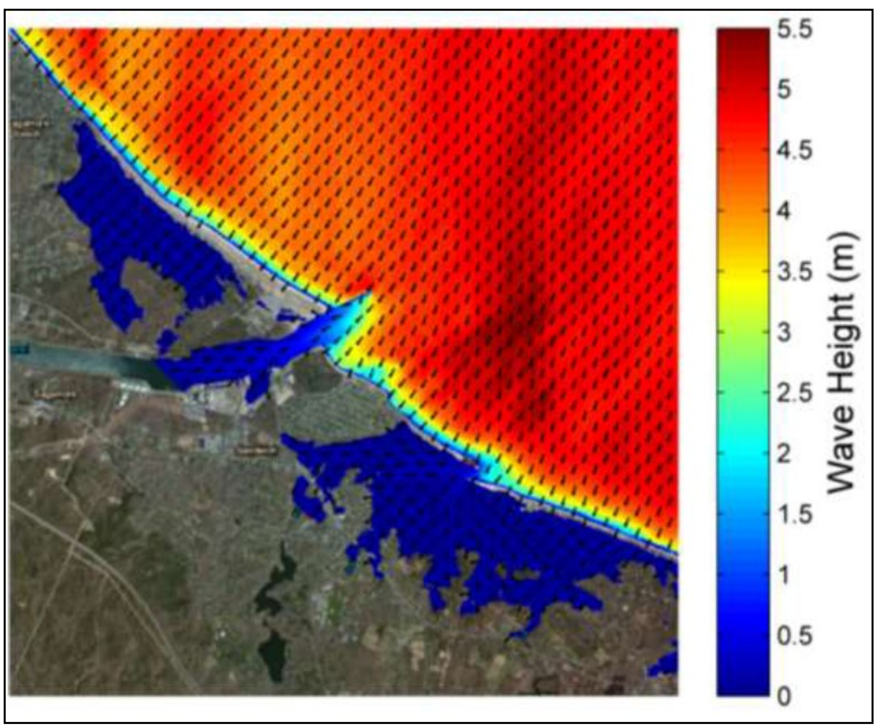


Figure B4-7: Sea surface results from the nearshore (local) wave model for the 10-year return period storm event (WHG, 2020)

4.2.6. Alternative Simulations

The ultimate goal of the overall modeling system was application towards the evaluation of the wide range of alternatives presented in Section 5. The alternatives were geared towards mitigation of the ongoing erosion occurring at Town Neck and Springhill Beaches. The resolution of the local nearshore model allows for the simulation of these alternatives with accurate dimensions and layouts. In order to simulate the alternatives, the existing conditions model grid was numerically modified to include the proposed layouts.

Alternative formulation is discussed in Section 5 and the evaluation of these alternatives, which includes the local wave modeling discussed in the previous sections, is presented in Section 6.1.

4.2.7. Local Wave Modeling Summary

The numerical wave model CMS-Wave was used to model the local, nearshore wave environment for the Canal region. The nearshore (local) wave model was simulated using the average annual wave climate as well as higher energy storm events and captured the local physical processes, (e.g., wave reflection, wave-induced currents, wave dispersion, nearshore wave refraction and diffraction, etc.), and subsequently the engineering alternatives.

Evaluation of the sea surface results for the existing conditions revealed: (1) significant wave shadowing occurs in the lee of the Canal navigation structures, particularly the north jetty, (2) higher energy storm events generate significantly higher wave heights in Cape Cod Bay in the Canal region which contribute to considerable sediment transport, and (3) wave directions become increasingly shore-normal as they approach shore and interact with the bottom, with a more well-structured wave field observed in the storm simulations when compared to the average annual approach directions.

5. Alternative Formulation & Design

The process of designing and evaluating the numerous alternatives considered during this investigation evolved throughout the study. Overall, a variety of factors were considered when designing and evaluating the various alternatives (e.g., cost, constructability, feasibility, performance, environmental impacts, etc.), with the overall objective focused on identifying the optimal solution. However, the focus of this section is primarily on the coastal engineering design of the alternatives related to providing protection and maintaining Town Neck and Springhill Beaches. The alternatives were designed to mitigate the ongoing erosion occurring at Town Neck Beach and not for storm damage reduction. It is understood that there will be storm damage reduction benefits, but they were not considered in the analysis. The assessment and performance of the alternatives is discussed in Section 6.

5.1. Development of Alternatives

The Cape Cod Canal Section 111 Project alternatives were developed jointly between the USACE New England District, WHG, and the Town of Sandwich. During this iterative process, many solutions were discussed and considered, and an initial series of alternatives was selected for the analysis procedure. Careful consideration was given to all factors associated with each alternative. For example, potential impacts on the neighboring shoreline, engineering feasibility, likelihood of success, etc. were all considered in the final selection process. The alternatives that were viewed as the most highly effective were jointly selected for further analysis. The initial array of alternatives included six primary alternatives, with several sub-alternatives considering sources and quantities of material.

5.2. Alternatives Considered

Table B5-1 presents a list of the alternatives considered. Beach nourishment was considered as a standalone alternative (Alternative 1) as well as the base measure for two additional alternatives (Alternatives 2 and 3). These alternatives considered additional project elements in order to help create a more sustainable beach. Two alternatives (Alternatives 4 and 5) focused on alterations to the Canal navigation structures with the goal of increasing sediment bypassing at the north jetty and sediment retention at the south jetty. The final alternative considered was a permanent sand bypassing system which would pump sand from the updrift fillet at Scusset Beach to the downdrift Town Neck Beach shoreline, mimicking more natural sediment transport processes that would be in place without the Canal's interruption of the shoreline.

Table B5-1: Alternatives considered for analysis

Alternative Number	Description
1	Beach nourishment alone
2	Beach nourishment with dune core stabilization
3	Beach nourishment with groin modifications
4	North jetty shortening
5	South jetty lengthening
6	Permanent sand bypassing system

5.3. Alternative Descriptions

5.3.1. No Action Alternative

The no action alternative implies there would be no change to the present conditions in the vicinity of the Cape Cod Canal. This is an unacceptable alternative, as the existing shoreline would continue to be eroded, a sustainable beach and/or any protective action would not be undertaken, and the landward homes and structures would face potential damage/loss. Further, loss of the barrier beach at Old Harbor Inlet could have devastating consequences on the marsh ecosystem and would increase potential for flooding the Town of Sandwich inland along Route 6A. This alternative does not address the required mitigation purview of the Section 111 Authority.

5.3.2. Alternative 1: Beach Nourishment Alone

This alternative consists of a beach nourishment in the area fronting Town Neck Beach as permitted by the Town. The nourishment design consists of approximately 388,000 cubic yards of material extending approximately 5,000 ft (1,525 m), beginning 1,000 ft southeast of the Canal in the west and extending to within 600 ft of Old Harbor Inlet in the east. The beach nourishment includes dune and berm sections which vary in height and width to avoid rocky intertidal and complex hard bottom resources as well as meet habitat requirements for endangered and threatened shorebirds. The dune crest will be at an elevation of 15 to 21 ft NAVD88 with a width ranging from 50 to 150 ft. 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 with a 1V:20H slope extending seaward from the berm to existing grade. Dunes would have a slope of 1V:10H to 1V:15H. At the western end of the project area, dunes would have a slope of 1V:5H, and the beach would slope seaward from the toe of the dune at a slope of 1V:10H. At both ends of the project, the sand would be graded to transition between the project and existing beach and dune grades. This alternative would 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. This beach nourishment is also considered as a component of Alternatives 2 and 3.

Two variations of this beach nourishment design, with smaller placement volumes, were also considered for this study. In all cases, the dune section of the design, which calls for 138,000 cy of material, and the fill length of 5,000 ft were retained. In order to reduce the nourishment volume, the berm width was reduced. Alternate beach nourishments with sand volumes of 324,000 cy and 224,000 cy were considered. The 324,000 cy beach fill will have a berm width of 74.4 ft and the 224,000 cy nourishment will have a berm width of 34.4 ft.

5.3.3. Alternative 2: Beach Nourishment with Dune Core Stabilization

This alternative would consist of the engineered dune and berm beach fill described in Alternative 1 as well as biodegradable sand filled coir envelopes used to fortify the dune. The coir envelopes would act as a last line of defense and provide additional storm damage protection. With the coir envelopes in place, the dune may not erode as quickly once the dune erodes back to the location of the envelopes. However, when erosion does occur, these envelopes would become exposed, impeding shorebirds from accessing foraging habitat and be subject to accelerated degradation due to UV light and saltwater exposure. The use of coir envelopes in beach fills has been employed by individual homeowners at Town Neck Beach (Figure B1-3) and elsewhere in Massachusetts.

5.3.4. Alternative 3: Beach Nourishment with Groin Modifications

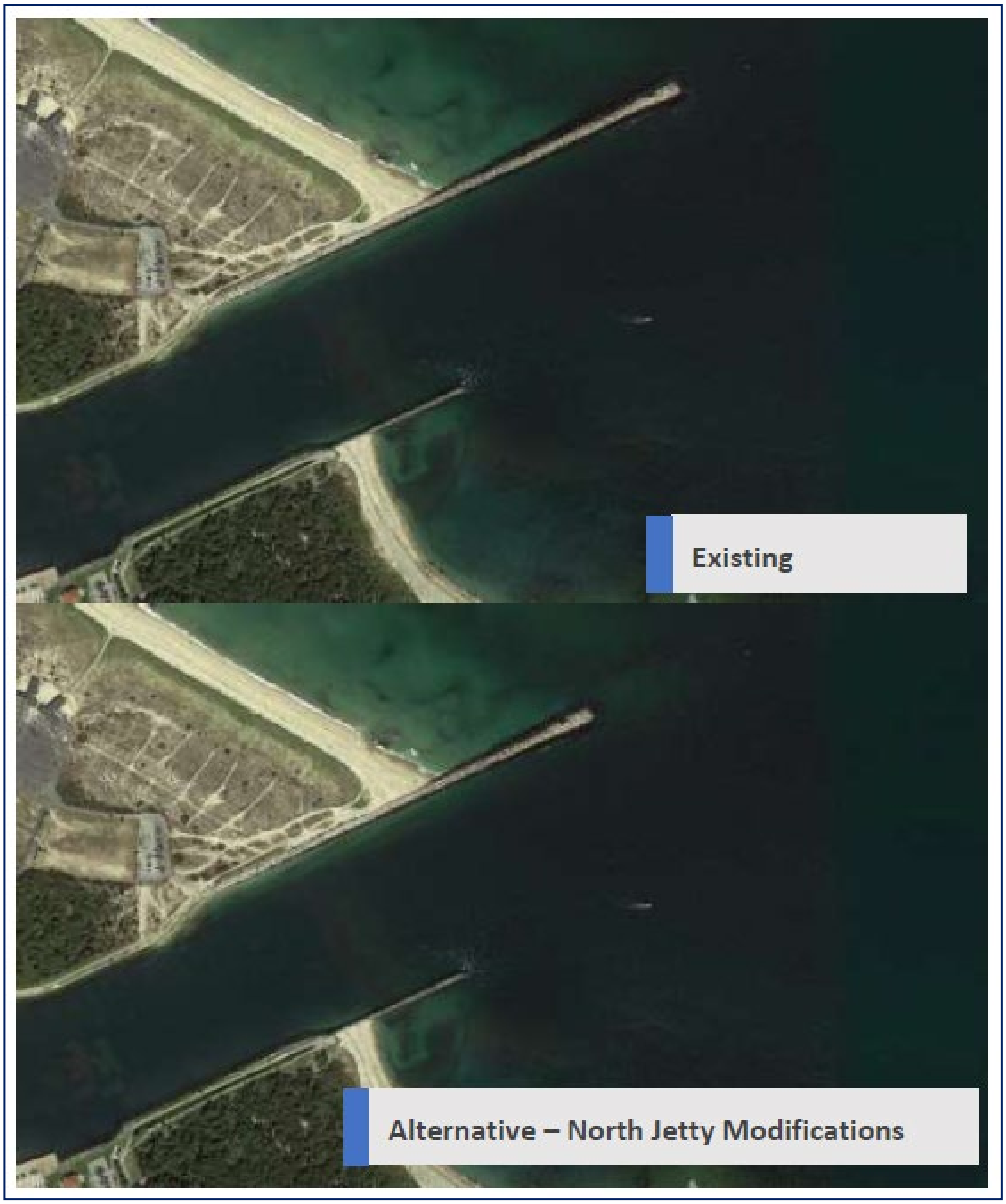
This alternative would consist of the engineered dune and berm beach fill described in Alternative 1 as well as modifications to the groin field at Town Neck Beach to improve its ability retain beach fill and extend the longevity of the nourishment project. Town Neck Beach currently contains 9 shore-perpendicular 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 alongshore sediment transport, while others have a significant impact on processes along the beach. WHG inspected the integrity, condition, and functionality of the existing groins and jetties at Old harbor Inlet and, after a number of iterations, recommended an optimized design for the groin field. This optimized design involves the removal of the two non-functional jetties at Old Harbor Inlet and five groins along the eastern portion of Town Neck Beach. Reusing the material from these structures, this alternative proposes the construction of four 250 linear foot notched groins along the eastern portion of Town Neck Beach (Figure B5-1). These groins would help retain the newly placed beach fill while the notches would allow for the natural migration of some sediment to Springhill Beach to the east. See Section 5.7 of Appendix C for additional discussion of the considerations made in formulating the groin modification alternative.

The variations in beach fill volume described for Alternative 1 were also considered for Alternative 3. Again, the full dune section of the design and 5,000 ft length of the nourishment were included in all variations, with the only the berm width changing with the placement volume. The 388,000 cy volume was assumed to have a berm width of 100 ft while the 324,000 cy and 224,000 cy nourishments will have berm widths of 74.4 ft and 34.4 ft, respectively.

Figure B5-1: Plan view of Alternative 3 – Beach nourishment with groin modifications (WHG, 2020)

5.3.5. Alternative 4: North Jetty Shortening

This alternative consists of shortening the north jetty by a length of 550 feet to decrease the amount of material impounded within the updrift fillet at Scusset Beach and increase the amount of sand which bypasses the Canal and reaches the downdrift shorelines at Sandwich (Figure B5-2).



5.3.6. <u>Alternative 5: South Jetty Lengthening</u>
This alternative consists of lengthening the south jetty by a length of 900 feet in order to prevent material on Town Neck Beach from migrating into the Canal where a localized reversal in sediment transport toward the Canal has been observed (Figure B5-3). This alternative would improve sand retention on the section of Town Neck Beach immediately downdrift of the Canal.

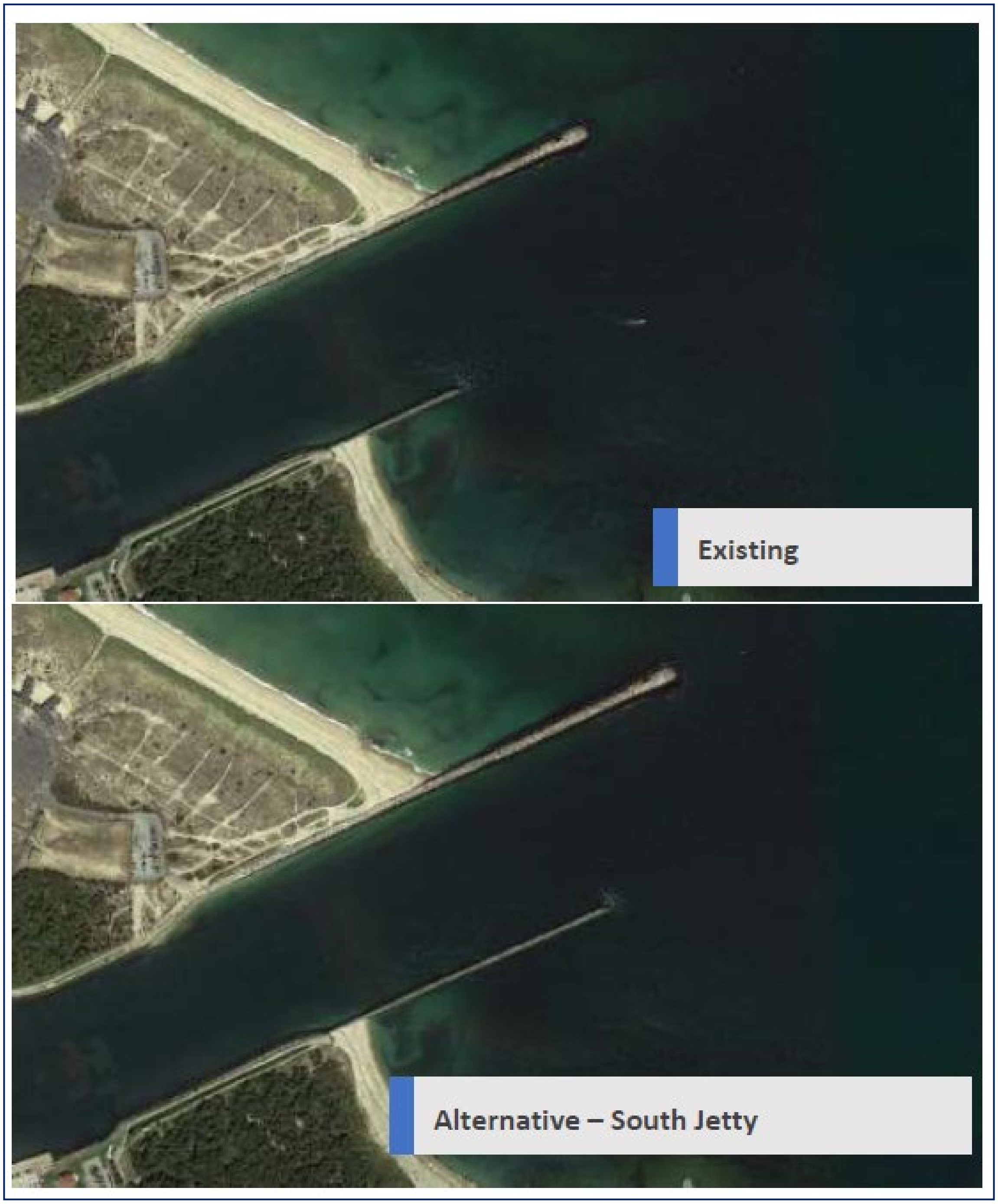


Figure B5-3: Plan view of Alternative 5 – South jetty lengthening (WHG, 2020)

5.3.7. Alternative 6: Permanent Sand Bypassing System

This alternative would construct a permanent bypassing system to pump accumulated sand on the updrift side of the Canal at Scusset Beach to the downdrift shoreline at Town Neck Beach. The bypassing system would remove material from the nearshore area at Scusset Beach which has already been permitted as a borrow site by the Town of Sandwich. The bypassing system would supply more frequent, but smaller quantities of sand to Town Neck Beach than beach fills and would replicate the natural transport of sediment within the region. As it would remove material from the updrift fillet adjacent to the north jetty, there would be less sediment transport from Scusset Beach around and through the jetty, and likely less sediment deposition in the Canal, potentially lowering the Canal maintenance needs. While the sand bypassing system alternative is described here, its performance was not evaluated further as it was screened out due to long term costs associated with its operation and maintenance.

6. Alternatives Evaluation

Alternatives were evaluated based on their ability to nourish and create a more stable shoreline at Town Neck and Springhill Beaches, while also considering potential impacts on neighboring shorelines, engineering feasibility, and likelihood of success. Different methods were employed to evaluate each alternative's ability to meet this objective.

6.1. Wave Height and Direction Evaluation

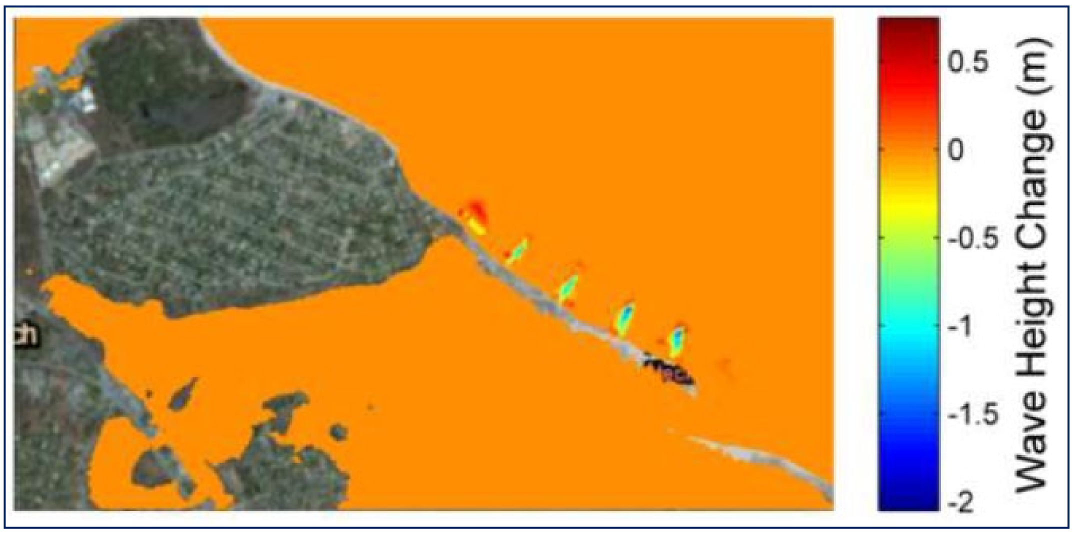
WHG modified the existing conditions CMS-Wave model grid for alternatives with structural modifications that would alter the local wave climate. Modified model grids were generated for each Alternative 3, 4, and 5. The modified model grids were then simulated for the same set of wave conditions run on the existing conditions grid (i.e., average annual approach directional bins and high energy events). These simulation results were used to evaluate the overall performance of each alternative and, in concert with the existing conditions simulations for each scenario, to generate differences in wave height and direction within the vicinity of the Cape Cod Canal. Alternatives 1, 2, and 6 were not expected to alter the local wave climate as their plans do not contain features that would affect incident wave energy. Therefore, no changes in wave climate from the existing conditions simulations were expected for Alternatives 1, 2 and 6.

Differences in wave heights (between existing conditions and alternative cases) were computed at each grid cell within the model domain. Difference plots were created (subtracting alternative wave heights from existing) that indicate regions of increased and/or decreased wave heights and assessed to determine the overall impact of the alternative on the wave heights in the region. Similar plots were generated illustrating changes in wave direction between the existing conditions simulations and the alternative cases.

An important aspect of any potential alternative and/or solution is the potential negative impacts that may be associated with the alternative. This may include, increased wave energy in other shoreline regions, increased wave energy in the navigational channel, or alternatives that result in significant maintenance concerns.

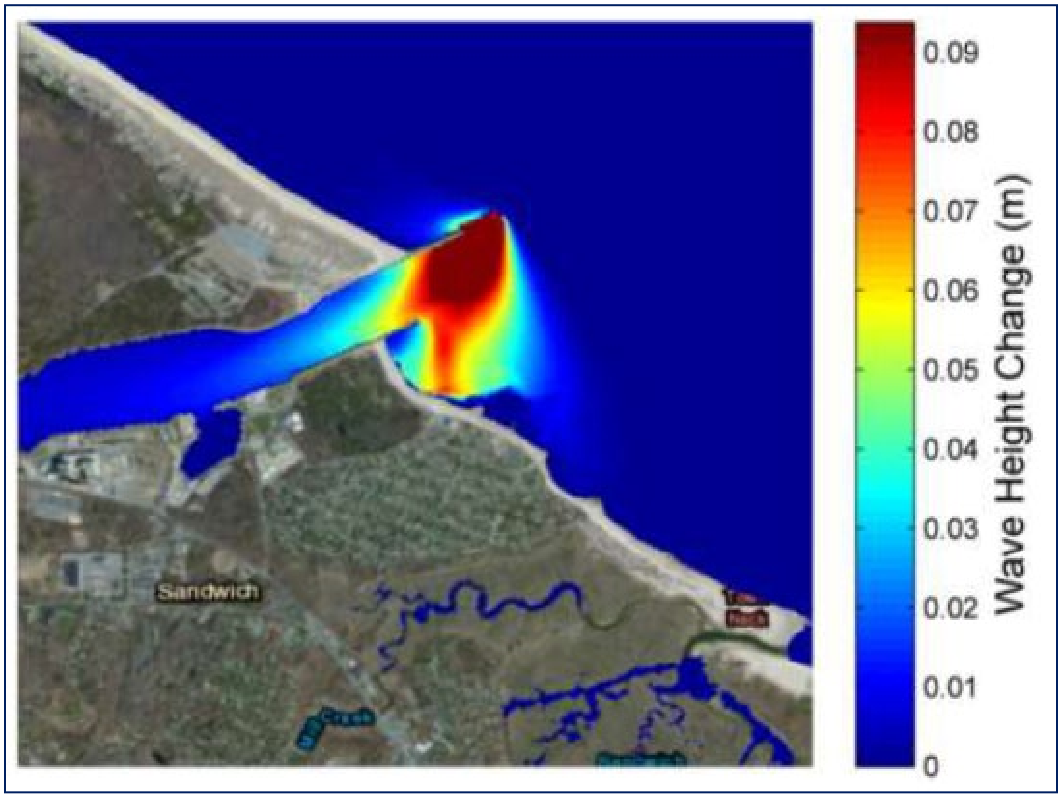
6.1.1. Alternative 3: Beach Nourishment with Groin Modifications

Groin modifications associated with this alternative were designed to be implemented with the beach fill described in Alternative 1. Local wave 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 areas where groins have been removed and not replaced. Increases in wave height are also observed at the heads of each groin where wave focusing occurs. Figure B6-1 shows an example of the comparison of the results for a simulation of a 50-year storm event with waves from a northeast direction. The difference in wave heights are shown with reds indicating increases in wave height with the groin modifications, and greens and blues indicating decreases in wave height associated with the groin modifications.



6.1.2. Alternative 4: North Jetty Shortening

Figure B6-2 presents an example of the wave height change plot for Alternative 4 (north jetty shortening) for average annual wave conditions from the northeast direction. In this figure, reds and yellows indicate areas where wave height increases are greatest. Light blues and greens indicate areas where wave heights are predicted to increase, but to a lesser degree. Wave model results show that wave energy will increase at the entrance to the Canal if the north jetty is shortened for waves approaching from the north and northeasterly directions, as well as due to storms. Increased wave energy is shown within the Canal as well as along the western portion of Town Neck Beach, indicating that it may pose an increased hazard to navigation and increase erosion along the Town Neck Beach shoreline, both adverse effects. No reductions in wave heights were observed with this alternative.



Changes in wave direction were also plotted between the existing conditions and alternative simulations. Figure B6-3 shows the direction change plot for Alternative 4 for average annual wave conditions from the northeast direction. Greater changes in wave direction are indicated by reds and yellows, while lesser changes in wave direction are indicated by greens and blues. The directions of wave propagation for the existing and alternative simulations are depicted by white and black arrows, respectively. By shortening the north jetty, waves are increasingly directed at the western portion of Town Neck Beach.

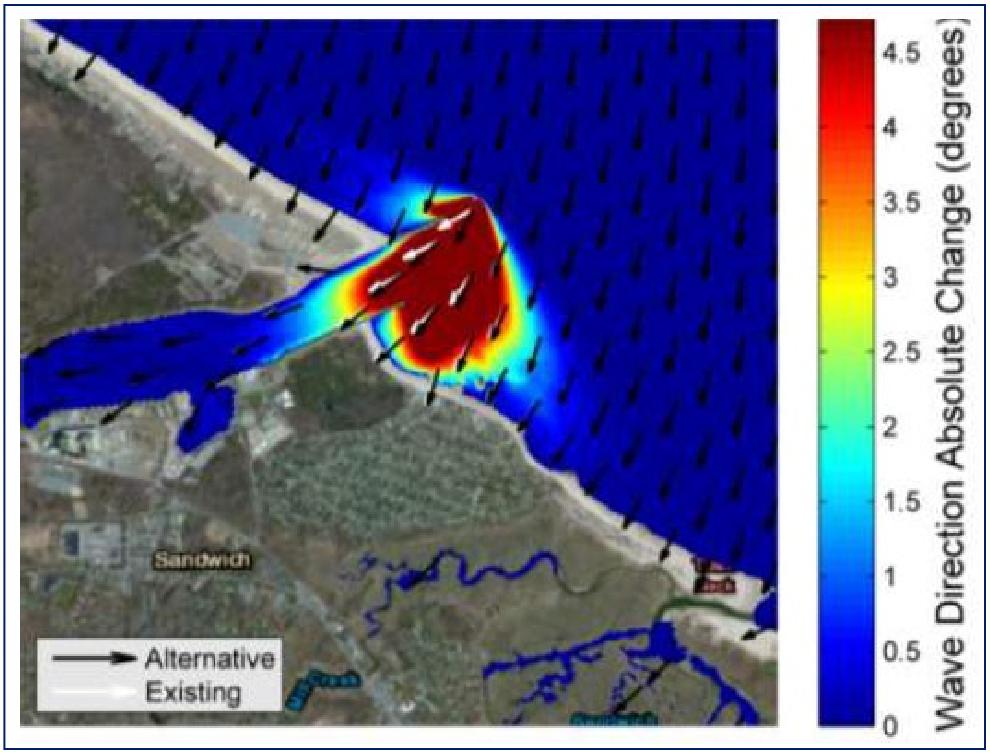


Figure B6-3: Wave direction changes for Alternative 4 for average annual wave conditions from the northeast (WHG, 2020)

6.1.3. <u>Alternative 5: South Jetty Lengthening</u>

Figure B6-4 presents an example of the wave height change plot for Alternative 5 (south jetty lengthening) for average annual wave conditions from the northeast direction. In this figure, blues and greens show areas where wave heights are greatest in the existing conditions simulation. Thus, lengthening the south jetty results in decreases in wave heights on the western section of Town Neck Beach when waves are approaching from the northeast and in storm events. This reduction in wave energy will reduce sediment transport along this segment of the shoreline. There are no increases in wave height shown for this alternative.

Figure B6-5 shows the wave direction change plot for Alternative 5 for average annual wave conditions from the northeast direction. Greater changes in wave direction are indicated by reds and yellows, while lesser changes in wave direction are indicated by greens and blues. The directions of wave propagation for the existing and alternative simulations are depicted by white and black arrows, respectively. By shortening the lengthening the south jetty, changes in wave direction are observed closest to the south jetty and along the western portion of Town Neck Beach.

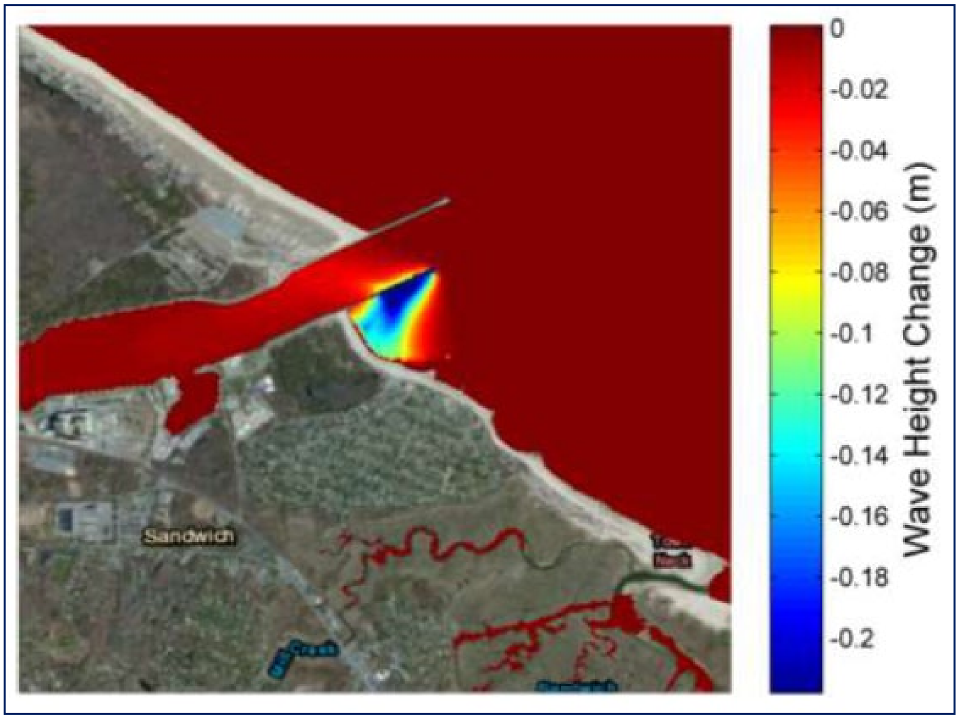


Figure B6-4: Wave direction changes for Alternative 5 for average annual wave conditions from the northeast (WHG, 2020)

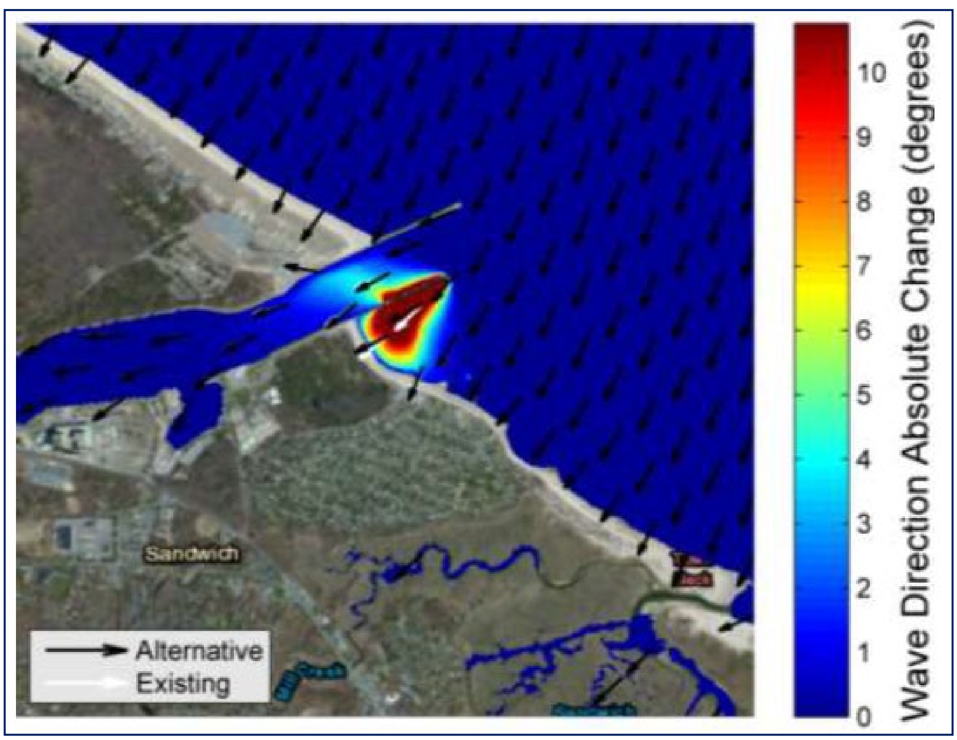


Figure B6-5: Wave direction changes for Alternative 5 for average annual wave conditions from the northeast (WHG, 2020)

6.2. Sediment Transport – Alternative Analysis

Where changes in wave climate were observed in Alternatives 3, 4, and 5, changes in sediment transport were also evaluated.

The methodologies to determine sediment transport were discussed in Section 3.6.3. In that section the existing conditions were discussed. In the following sub sections of Section 6.2, the alternatives are evaluated to determine their effects on sediment transport within the Canal region and their relative performance in terms of their ability to maintain a stable shoreline downdrift of the Canal along Town Neck and Springhill Beaches. Results from the nearshore (local) wave model were used to drive the local sediment transport analysis. Changes in potential sediment transport were evaluated for each modeled alternative (Alternatives 3, 4, and 5) and are reported herein.

6.2.1. Alternative 3: Beach Nourishment with Groin Modifications

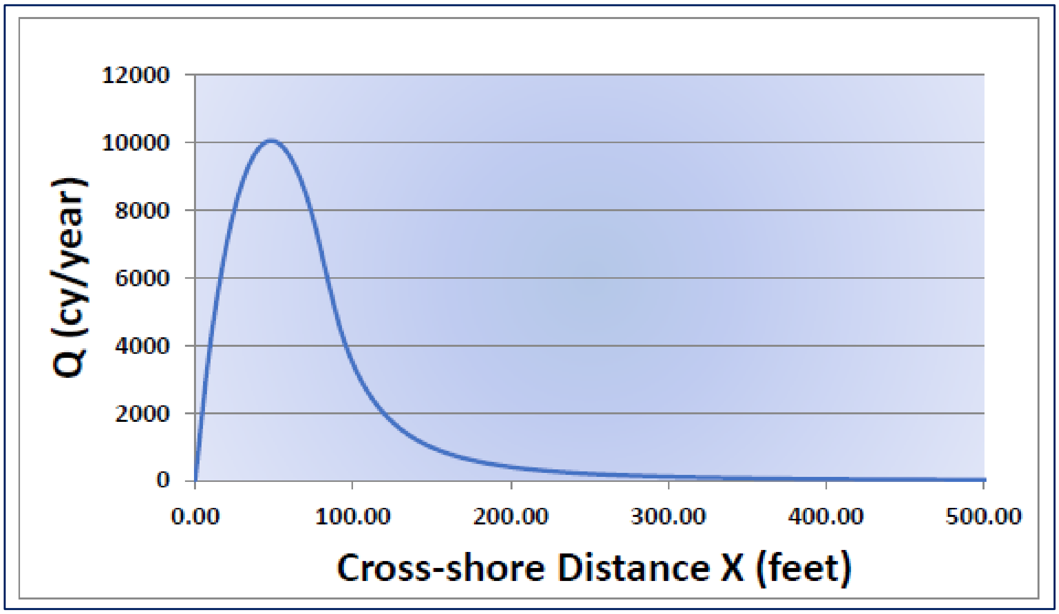
Results from the wave modeling of the groin modification alternative showed both localized increases and decreases in wave energy. However, it is expected that construction of an engineered system of groins, in combination with dune and beach nourishment, would provide the most stable shoreline at Town Neck Beach. As discussed further in Section 6.3, the groin system prevents lateral spreading of the beach fill outside the initial placement area, keeping more material in place longer. By incorporating notches near the shoreward end of each groin, some bypassing of sediment to the downdrift beaches to the east is maintained.

6.2.2. Alternative 4: North Jetty Shortening

WHG estimated the potential increase in littoral transport expected by shortening the north jetty by performing an analysis of the cross-shore distribution of alongshore transport using relationships developed by Longuet-Higgins (1970, 1970a). Based on the cross-shore distribution, the effect of a shore-perpendicular structure on reducing or increasing the alongshore sediment transport can be estimated. Refer to Appendix C, Chapter 5.5 for additional details.

The alongshore current on Scusset Beach, calculated using the process-based sediment transport model described in Section 3.6.3, 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 beach along that stretch of shoreline where it is impounded behind the north jetty.

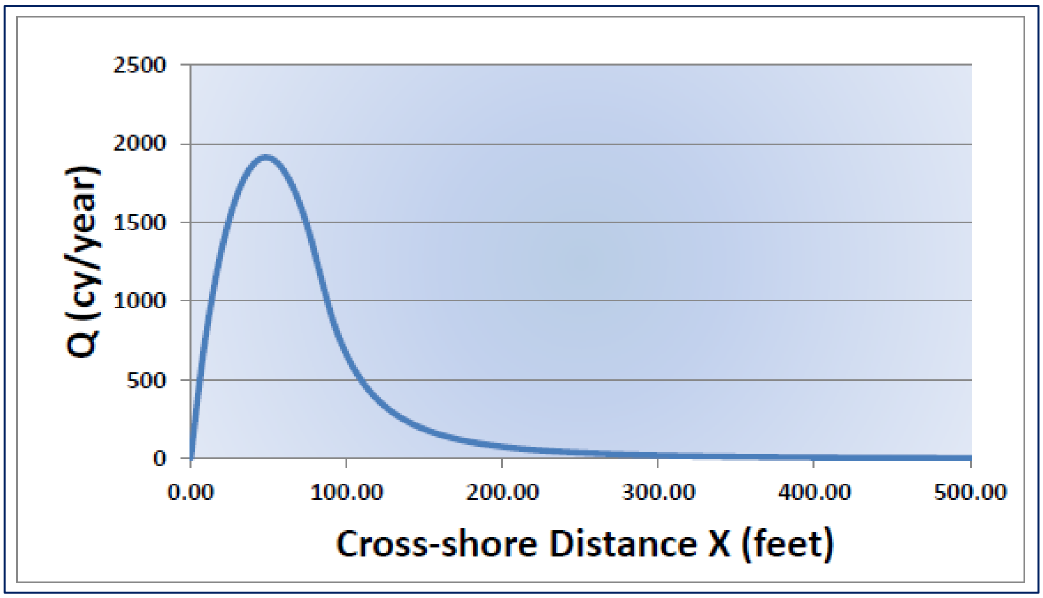
Using the cross-shore distribution developed for Scusset Beach (Figure B6-6), only 74 cubic yards/year is theoretically able to bypass the seaward end of the existing jetty. Decreasing the length of the north jetty by 550 feet adds 160 cubic yards/year to the volume of material that is transported around the seaward end of the jetty (234 cy/year total). This small sediment flux about the head of the jetty is not unexpected, as a majority of the alongshore flux occurs within the nearshore portion of the surf zone rather than at the depths of the end of the jetty (even with a reduced length).



This analysis shows shortening the north jetty does little to increase sediment transport about the seaward end of the jetty. In fact, it suggests that the majority of the sediment being transported past the north jetty under existing conditions (approximately 34,600 to 36,400 cy/year according to the sediment budget) is not going around the jetty, but is instead being transported through the jetty or being transported by wind and wave overtopping processes over the landward portion of the jetty. Both these processes would not be expected to change significantly by shortening the jetty.

6.2.3. <u>Alternative 5: South Jetty Lengthening</u>
To quantify the potential volume that would be impounded behind the lengthened south jetty, the cross-shore distribution of the alongshore transport was evaluated using relationships proposed by Longuet-Higgins (1970, 1970a). Refer to Appendix C, Chapter 5.6 for additional details.

The alongshore current caused by the reversal at the western end of Town Neck Beach nearest the Canal was calculated using the process-based sediment transport model which indicated potential transport rates of 10,000 to 20,000 cy/year. As calculated in the sediment budget, approximately 1,500 cy/year to 3,300 cy/year is transported by the reversal into the Canal. The Longuet-Higgins quadratic alongshore distribution for the sediment flux to the south jetty calculated using a conservative volume of 20,000 cy/year is shown in Figure B6-7. Using this distribution, approximately 19,920 cy/year is captured by the existing jetty length with 80 cy/year bypassing the seaward end of the south jetty. This calculation suggests 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 south jetty would not significantly alter the sediment dynamics on the south side of the Canal.



6.3. Sediment Transport – Beach Nourishment

The performance of alternatives involving beach nourishment is discussed within this section. Although Alternatives 1, 2, and 3 each feature beach nourishment, only Alternatives 1 and 3 are discussed herein. Alternative 2 was screened out as it had similar beach nourishment performance to Alternative 1, but had added costs associated with the need to frequently replace the biodegradable coir envelopes.

6.3.1. <u>Beach Nourishment Spreading</u>

Beach nourishment performance was evaluated using a standard approach which combines the conservation of sediment equation with the linearized transport equation. This formulation, called the Pelnard-Considére (1956) equation, is used in providing theoretical results to establish design and performance standards for nourishments (Equations B6-1 and B6-2). The detailed analysis can be found in Appendix C

Section 12.5.5. This analysis used the methods and formulations provided for in the CEM Section V-4-1 g. page V-4-46.

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

Equation B6-1

where:

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

The alongshore diffusivity is presented by Pelnard-Considére (1956) and has been provided as Equation B6-2.

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

Equation B6-2

where:

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

The Pelnard-Considére equation assumes the nourishment will spread symmetrically about the centerline of the project as material is transported to both sides of the nourishment. The longevity 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 as material spreads, but material is not necessarily lost from the system, as it spreads to regions outside the initial fill template.

Table B6-1 presents the adjusted length of the nourishment for each alternative based on the influence of the structures (both existing and proposed). These adjusted lengths do not represent an actual physical extension of the nourishment; however, the adjustment is used to represent the influence of structures on the rate of dispersion in the sediment transport model.

For Alternative 1, beach nourishment alone, it was assumed that the existing groins at Town Neck Beach, which are in various conditions, have little impact on sediment transport. Therefore, no adjustment was made to its nourishment length for the purposes of evaluating fill longevity. For the nourishment with groin modifications, an adjustment in the fill length was made to account for the slowed dispersion of the spreading. In cases where the nourishment is placed directly next to a shore perpendicular structure, the fill length is doubled as material is only allowed to spread in one direction. For this case, the groin system will slow the spreading of the nourishment, but it will still occur in both directions. Therefore, the adjusted nourishment length was based on inspection of the wave energy changes, and subsequent radiation stress-based transport, shown in Section 6.1.1. An increase in length of approximately 40 percent was used to represent the slower spreading of fill in Alternative 3.

Table B6-1: Adjusted nourishment lengths

Alternative	Adjusted Nourishment Length (ft)
1 – Beach Nourishment Alone	5,000
3 – Beach Nourishment with Groin Modifications	7,000

6.3.2. Background Erosion Rate Inclusion

As discussed in the CEM, the beach fill longevity formulation, discussed in the previous sections, only accounts for sediment being removed from the beach fill due to end losses, or material be transported from the edges of the beach fill. The loss rate does not account for sediment being removed due to the background erosion that previously existed before the fill was placed. The addition of background erosion is discussed in detail in the CEM in section V-4-1 g.3.c on page V-4-52.

A long-term background erosion rate of -1.1 ft/year was included in this analysis. The long-term shoreline change rate was considered to be a more representative background erosion rate than the short-term shoreline change rate for several reasons. First, the short-term rate is influenced by contemporary nourishments, with increased rates of shoreline change in those years which mask the true background erosion rate. Second, the short-term period from 2000-2018 was considered to be a relatively short timeframe to be used to generate a background erosion rate, prone to significant variation due to the limited number of shorelines used to derive the short-term rate. Therefore, the long-term change rate was used to evaluate performance of beach fill alternatives. Performance of the selected alternative using both the long-term and short-term change rates is reported in Section 7 to communicate project risk and illustrate sensitivity of project performance to increased rates of background erosion.

6.3.3. Beach Fill Performance

Alternatives were compared to one another based on their ability to maintain a beach at Town Neck Beach. Beach fill performance was evaluated using two metrics—remaining fill volume and minimum beach width. Once either of these thresholds is reached, it was assumed that renourishment would be needed.

6.3.4. Beach Volume

The percentage of beach nourishment remaining within the initial placement template is a common approach to measure beach performance and longevity. Figure B6-8 presents the performance of Alternative 1 (solid lines) and Alternative 3 (dashed lines) with nourishment volumes of 388,000 cy (red), 324,000 cy (blue), and 224,000 cy (orange), in terms of amount of material remaining, as a function of time. This includes background erosion corresponding to -1.1 ft/year. 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 (as indicated in the historical shoreline change analysis). The percent of initial material remaining is presented along the vertical axis, while the time elapsed from the initial placement (in years) is presented along the horizontal axis. The 30 percent volume remaining line is highlighted in green. This threshold is commonly used to estimate beach fill longevity and predict when a renourishment should be initiated. Use of this cutoff ensures that there will be some "cushion" to account for modeling errors and uncertainty associated with the beach fill longevity information. This information is also presented in Table B6-2.

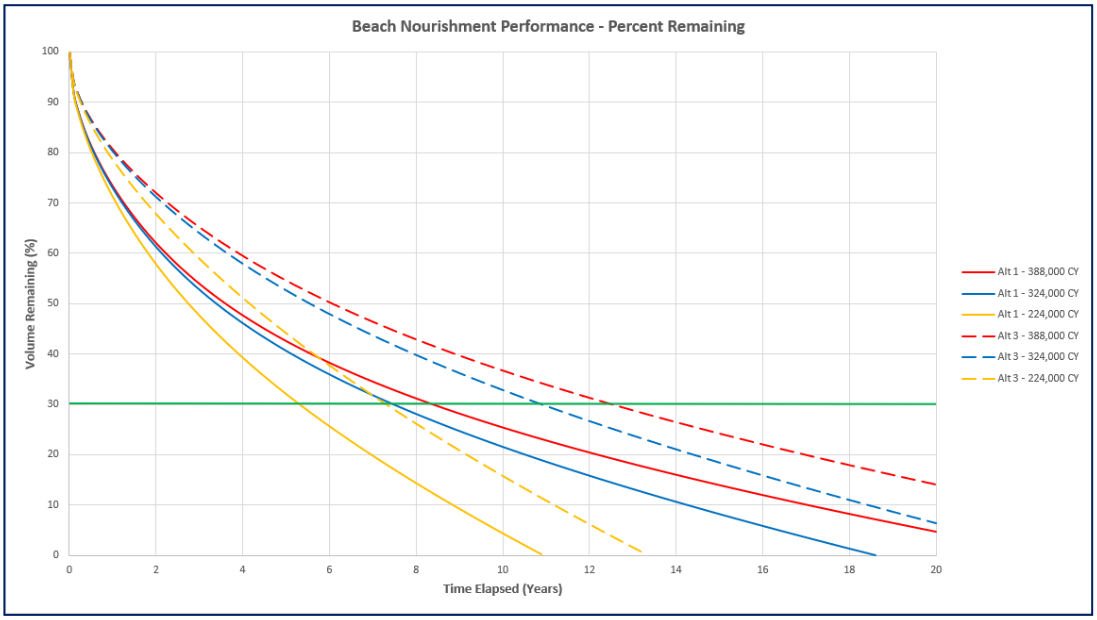


Table B6-2. Beach fill longevity based on percent fill remaining

Alternative 1				Alternative 3			
Fill Volume	388,000 cy	324,000 cy	224,000 cy	Fill Volume	388,000 cy	324,000 cy	224,000 cy
Time (Years)	% Remaining			Time (Years)	% Remaining		
0	100	100	100	0	100	100	100
1	73.3	72.9	71.2	1	80.6	80.2	78.5
2	62.0	61.2	57.8	2	71.9	71.2	67.7
3	53.9	52.8	47.6	3	65.1	64.0	58.8
4	47.6	46.1	39.2	4	59.5	57.9	51.1
5	42.5	40.6	32.1	5	54.6	52.7	44.1
6	38.2	36.0	25.6	6	50.2	48.0	37.7
7	34.5	31.8	19.8	7	46.4	43.7	31.7
8	31.2	28.1	14.4	8	42.9	39.8	26.1
9	28.1	24.7	9.3	9	39.7	36.3	20.8
10	25.4	21.6	4.4	10	36.7	32.9	15.7
11	22.8	18.7		11	33.9	29.7	10.8
12	20.4	15.9		12	31.3	26.7	6.1
13	18.1	13.2		13	28.8	23.9	1.5
14	16.0	10.7		14	26.4	21.1	
15	13.9	8.3		15	24.2	18.5	
16	12.0	5.9		16	22.0	16.0	
17	10.1	3.6		17	19.9	13.5	
18	8.2	1.4		18	17.9	11.1	
19	6.4			19	16.0	8.8	
20	4.7			20	14.1	6.5	

6.3.5. <u>Minimum Berm Width</u>

The longevity of each alternative was also evaluated considering the need to maintain a minimum berm width. This minimum beach width ensures that the beach nourishment project has capacity to absorb a storm event without a reduction in storm damage benefits and damages occurring. 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. It assumes that once the beach width reaches the critical width, a maintenance nourishment would be required.

To assess critical width, WHG modeled the beach cross-shore evolution along a representative 1-dimensional transect in response to storm conditions in XBeach. The assessment indicated that once the initial nourishment width has decayed to approximately 30 ft, a 10-year event could cause significant overtopping of the dune system and potential upland damage. For this reason, a critical width of 30 feet was selected to evaluate beach fill longevity. The WHG XBeach model setup, scenarios, and results are detailed in Appendix C, Chapter 4.

Figure B6-9 presents the performance of Alternative 1 (solid lines) and Alternative 3 (dashed lines) with nourishment volumes of 388,000 cy (red), 324,000 cy (blue), and 224,000 cy (orange), in terms of remaining berm width, as a function of time. This includes background erosion corresponding to -1.1 ft/year. The berm width is presented along the vertical axis, while the time elapsed from the initial placement (in years) is presented along the horizontal axis. The 30 ft critical width line is highlighted in green. This information is also presented in Table B6-3.

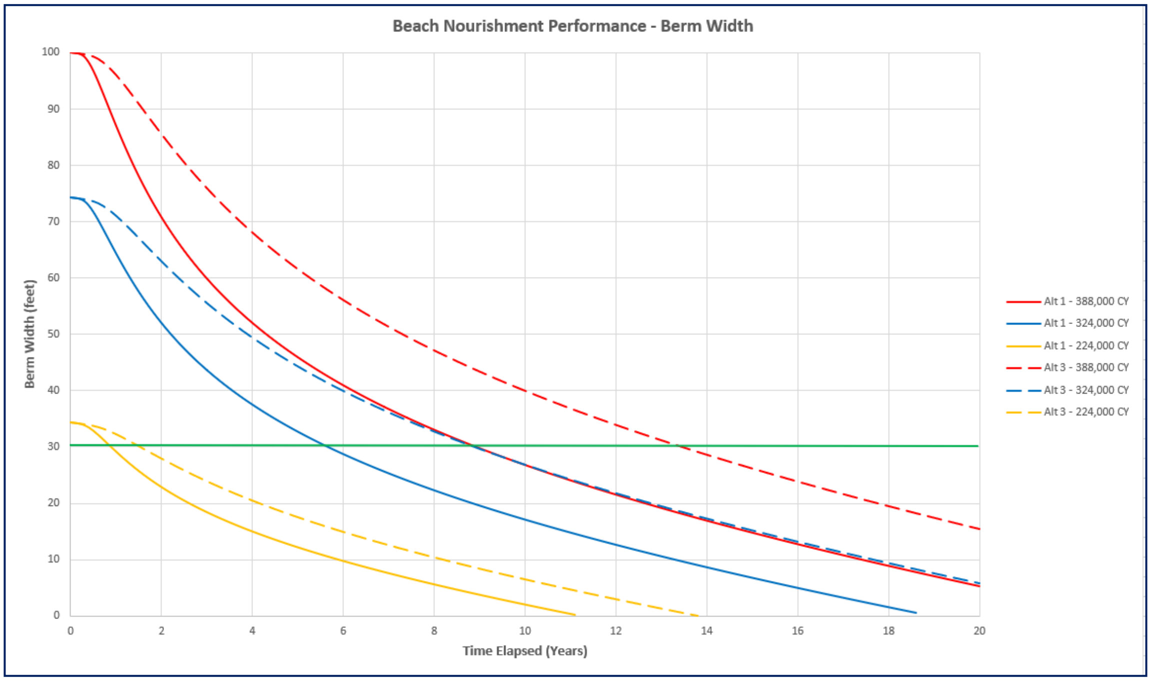


Table B6-3: Beach fill longevity based on berm width

Alternative 1				Alternative 3			
Fill Volume	388,000 cy	324,000 cy	224,000 cy	Fill Volume	388,000 cy	324,000 cy	224,000 cy
Time (Years)	Berm Width (feet)			Time (Years)	Berm Width (feet)		
0	100	74	34	0	100	74	34.4
1	86.9	64.4	29.2	1	96.0	71.1	32.3
2	70.7	52.0	22.9	2	85.5	63.0	28.0
3	59.8	43.7	18.4	3	75.9	55.6	23.9
4	51.9	37.5	15.0	4	68.0	49.5	20.5
5	45.9	32.7	12.2	5	61.5	44.4	17.6
6	40.9	28.7	9.7	6	56.0	40.0	14.9
7	36.7	25.3	7.6	7	51.3	36.2	12.6
8	33.0	22.3	5.6	8	47.1	32.8	10.4
9	29.7	19.6	3.7	9	43.3	29.7	8.4
10	26.7	17.1	2.0	10	39.9	26.9	6.5
11	24.0	14.8		11	36.8	24.3	4.7
12	21.5	12.6		12	33.9	21.8	3.0
13	19.1	10.5		13	31.1	19.5	1.3
14	16.9	8.6		14	28.6	17.3	
15	14.7	6.7		15	26.1	15.2	
16	12.7	4.9		16	23.8	13.2	
17	10.7	3.2		17	21.6	11.3	
18	8.8	1.5		18	19.4	9.4	
19	7.0			19	17.4	7.6	
20	5.2			20	15.4	5.8	

6.3.6. Beach Fill Longevity

Beach fill performance was evaluated by predicting the change in volume remaining within the initial placement template and the berm width with time. The expected beach longevities and the renourishment volumes needed to restore the beach to its initial fill volume are presented in Table B6-4.

Table B6-4: Beach fill volumes and longevity summary

Alternative	Initial Volume (cy)	Fill Longevity/ Renourishment Rate (years)	Renourishment Volume (cy)
Alternative 1 – Beach Nourishment Alone	388,000	9	279,000
	324,000	6	207,500
	224,000	1	65,000
Alternative 3 – Beach Nourishment with Groin Modifications	388,000	13.5	281,000
	324,000	9	206,500
	224,000	2	72,500

6.4. Alternatives Evaluation Summary

Alternatives were evaluated based on their ability to nourish and create a more stable shoreline at Town Neck and Springhill Beaches. The CMS-Wave model was utilized to evaluate changes in wave height and direction for alternatives which affected local wave transformations. Results from these alterative model runs informed the sediment transport model. Beach nourishment alternatives were evaluated based on their expected longevities and renourishment needs. Key findings from each alternative were:

- Alternative 1 – Beach nourishment would directly place up 388,000 cy of material on Town Neck Beach that would have a renourishment interval of 9 years. Smaller placement volumes of 324,000 cy and 224,000 cy would require more frequent renourishment, every 6 years and annually, respectively.

- Alternative 2 – Beach nourishment with dune coir envelopes was expected to perform similarly to Alternative 1, but have greater maintenance requirements associated with needing to frequently replace the biodegradable coir envelopes.

- Alternative 3 – Beach nourishment with groin modifications would increase the beach longevity of Alternative 1 by controlling the lateral spreading of the beach fill. The renourishment interval for the 388,000 cy placement would be 13.5 years. Smaller nourishment volumes of 324,000 cy and 224,000 cy would require more frequent renourishment, every 9 and 2 years, respectively.

- Alternative 4 – North jetty shortening has the potential to increase sediment transport around the north jetty by 160 cy/year. However, reducing the length of the jetty is expected to increase wave energy at the Canal entrance and along the western portion of Town Neck Beach.

- Alternative 5 – South jetty lengthening is expected to have little benefit of capturing additional sediment from being lost to the Canal as the existing conditions sediment transport estimates only 80 cy/year of material has the potential to presently make is around the jetty.

- Alternative 6 – Permanent sand bypassing system was not evaluated as part of the coastal analysis as it was screened out due to high operations and maintenance costs.

7. Selected Alternative

Through an iterative planning process it was determined that Alternative 1 – Beach Nourishment at Town Neck Beach would most effectively mitigate shoreline damages directly attributable to the Canal FNP.

While Alternative 3 – Beach nourishment with groin modifications was identified as being most cost-effective over the 50-year period of analysis, the project cost is limited under the Section 111 authority. With the initial construction expected to reach the Section 111 limit, there would not be a mechanism in place to ensure future periodic nourishments. Without a commitment to periodic nourishment, construction of a groin field could cause adverse impacts to the downdrift shorelines already impacted by the interruption of longshore transport caused by the Canal. Therefore, the intent of the recommended plan was to provide a readily implementable project which can place as much material as possible on Town Neck Beach in the near term with a longer term plan to study the area more comprehensively under another authority.

7.1. Description

Beach nourishment and dune creation in this alternative would require approximately 388,000 cy of clean, beach compatible material. While variations in beach nourishment volume were investigated as part of the alternatives evaluation, smaller placement volumes would require more frequent and costly renourishments.

The nourishment design extends approximately 5,000 ft (1,525 m), beginning 1,000 ft southeast of the Canal in the west and extending to within 600 ft of Old Harbor Inlet in the east, and includes dune and berm sections which vary in height and width to avoid rocky intertidal and complex hard bottom resources as well as meet habitat requirements for endangered and threatened shorebirds. The dune crest will be at an elevation of 15 to 21 ft NAVD88 with a width ranging from 50 to 150 ft. For the eastern barrier beach portion of the project, the beach berm is to be increased in width by at least 100 ft at an elevation of 6 ft

NAVD88 with a 1V:20H slope extending seaward from the berm to existing grade. Dunes would have a slope of 1V:10H to 1V:15H. At the western end of the project area, dunes would have a slope of 1V:5H, and the beach would slope seaward from the toe of the dune at a slope of 1V:10H. At both ends of the project, the sand would be graded to transition between the project and existing beach and dune grades. The beach nourishment would 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.

Using the long-term rate of background erosion of -1.1 ft/year, renourishment of the beach fill is anticipated to be needed after 9 years when 30 percent of the initial placement volume and a 30 ft wide berm remain. While the long-term rate of background erosion is considered more representative and reliable than the short-term rate, it is noted that increased rates of erosion have been observed. Should an increased rate of background erosion, consistent with the short-term shoreline change from 2000-2018, of -5 ft/year occur, the triggers for renourishment would be reached after 5 years.

Plan selection was not sensitive to sea level change. Because Alternatives 4, 5, and 6 did not meet study objectives, costs were developed primarily for the nourishment alternatives in Alternatives 1, 2, and 3 which are all expected to be equally vulnerable to sea level change. Renourishment rates and costs were developed based on the historic rate of sea level change and background erosion rate. Therefore, the rates of renourishment under higher sea level change scenarios such as the USACE intermediate and high curves would be more frequent and costly. However, because it was determined that initial construction cost will likely reach the Section 111 limit, only a single placement of beach fill is expected. Over the beach fill's longevity of 9 years before a nourishment would be recommended, sea level change across the three USACE curves ranges from 0.1-0.34 feet (1.2-1.4 inches). This amount of sea level change is considered negligible and will have little affect on the project design and performance.

7.2. Sand Source

The selected alternative recommends beach nourishment using 388,000 cy of material from the nearshore area off Scusset Beach. The Town of Sandwich recently obtained permits for a nearshore borrow area at Scusset Beach that allows for the removal of approximately 224,000 cy. Because this project would require more than 224,000 cy of material, consideration was given to other sources of sediment including upland areas, maintenance dredging of the Canal, and expanding the permitted borrow site at Scusset Beach.

The permitted borrow area at Scusset Beach was designed with the primary goal of identifying a borrow source that could provide 278,000 cy of clean, beach compatible sand while minimizing impacts to environmental resources, sediment transport, and the shorelines at and adjacent to the Scusset Beach Reservation. The full 388,000 cy volume of the beach nourishment was not sought as Town Neck Beach received 110,000 cy of material from the Canal's maintenance dredging in 2016. However, the application for the permitted borrow area at Scusset Beach (WHG, 2017) did include the evaluation of a larger volume (approximately 350,000 cy).

WHG modeled changes in wave climate and sediment transport between existing and proposed conditions under average annual wave conditions and in response to the 10- and 50-year storm events. Results from the wave and sediment transport models were comparable between both the permitted 224,500 cy and 350,000 cy borrow site alternatives. For both alternatives, any increases in wave height were localized to the area just off the Scusset Beach and did not impact neighboring areas. As Scusset Beach has grown considerably due to the impoundment of sand at the Canal's north jetty and since there is a healthy beach and dune system, the potential increase in wave energy was not expected to have an adverse impact on erosion. In addition, sediment infilling rates were estimated to predict how long it would take each borrow area to fill under average annual and storm conditions. The infilling rates for both alternatives were nearly identical (105 and 102 cy/day). Using these infilling rates, the permitted area was predicted to fill in 5.9 years while the larger area was estimated to fill in 9.4 years.

Based on the evaluation completed by WHG for the 2017 borrow site permit application, there are not anticipated to be adverse impacts from sourcing sand from a larger area at Scusset Beach for the construction of the beach nourishment project.

It is recommended that dredged material from Canal maintenance be considered for future renourishment and maintenance of the beach project. However, for initial construction of the beach fill project, it was considered unlikely to be able to sync the project construction with a Canal dredging, as they occur approximately once every seven years.

8. Conclusions and Recommendations

Town Neck and Springhill Beaches, located adjacent to the Cape Cod Canal in Sandwich, MA, have undergone significant shoreline change in the last century since the Canal's construction, including significant erosion over the past several decades. The purpose of this study was to evaluate potential alternatives that may be viable solutions to the ongoing erosion, most immediately, at Town Neck Beach, but also at Springhill Beach farther east. The study focused on evaluating the physical processes (concentrating on the wave environment) occurring within the vicinity of Canal's east entrance, and specifically the Town

Neck Beach and Springhill Beach area, to assess potential alternatives that may be used to mitigate the erosion along the shoreline.

There were two main components of the study, the evaluation of historic change and development of an understanding of existing conditions and physical processes, and the evaluation of alternatives intended to mitigate for the erosion directly attributable to the Canal. WHG was contracted to conduct coastal engineering analyses to complete this investigation. The historic environment was studied through analysis of shoreline change, development of a sediment budget, and review of existing studies to develop an initial understanding of the ongoing coastal processes that shape the shoreline in the Canal region. The numerical modeling component of the study consisted of accurately simulating the existing conditions within the vicinity of the Cape Cod Canal, and subsequently utilizing the models to simulate various alternatives for shoreline protection. The numerical modeling portion of the study ultimately evaluated the performance of each of the alternatives and their ability to sustain a beach Town Neck and Springhill Beaches.

8.1. Historical shoreline change

The shoreline adjacent to the Canal along Town Neck and Springhill Beaches has experienced significant erosion. The average long-term shoreline change rate within 10,800 ft of the Canal is -1.33 ft/year. Erosion has been more pronounced in the same area in the short term, with recession of the shoreline occurring at -2.58 ft/year. By comparing this shoreline change to the region, it was determined that the Canal FNP was responsible for 78 percent of the erosion at Springhill and Town Neck Beaches.

WHG also used shoreline translation to estimate volumetric losses for the past 50 years using historic shoreline change rates and for the next 50 years using the same rates of change and a conservative sea level change projection. This analysis determined the volumetric loss of shoreline over the past 50 years to be 782,000 cubic yards or 1.45 cubic yards per foot of shoreline per year. Over the next 50 years, the estimated volume loss of beach is predicted to be approximately 900,000 cubic yards or 1.66 cubic yard per foot of shoreline per year.

Extending the volumetric losses calculated using shoreline translation offshore to the depth of closure increased the volumetric loss estimations to 1,175,000 cubic yards or 2.175 cubic yards per foot per year for the past 50 years and up to 1,288,000 cubic yards or 2.385 cubic yards per foot per year for the next 50 years. Given the uncertainty in future sea level change, these volumetric loss estimates can seen as lower and upper bounds, approximating shoreline response under the low and high sea level change scenarios.

8.2. Sediment Budget

WHG developed a sediment budget at the east end of the Cape Cod Canal to quantify sediment fluxes not captured in the shoreline change analysis and sediment transport model. Based on the results of the sediment budget analysis, it is evident that the Canal and its navigation structures affect sediment transport processes in the vicinity of the Canal. The average annual longshore sediment transport rate updrift of the Canal was estimated at 95,000 to 96,800 cy/year. Of this, the impoundment rate updrift of the north jetty was estimated at 54,700 cy/year, the shoaling rate in the Canal from dredge records was 28,100 cy/year, and the volume lost offshore of the Canal was 9,800 cy/year. Material removed from the channel was placed on Town Neck Beach at a rate of 11,200 cy/year. Given this, the volumetric loss rate attributable to the Canal FNP was estimated to be 81,400 cy/year or 85 percent of the approximately 95,900 cy/year updrift alongshore transport.

8.3. Local Scale Wave Modeling

WHG simulated average annual wave conditions and storm events in a nearshore (local) wave model. The wave model was used to understand the existing wave climate and nearshore wave transformations and to evaluate changes in wave climate caused by the alternatives considered. Results from the wave model were also used to generate the sediment transport flux. Evaluation of the sea surface results for the existing conditions revealed: (1) significant wave shadowing occurs in the lee of the Canal navigation structures, particularly the north jetty, (2) higher energy storm events generate significantly higher wave heights in Cape Cod Bay in the Canal region which contribute to considerable sediment transport, and (3) wave directions become increasingly shore-normal as they approach shore and interact with the bottom, with a more well-structured wave field observed in the storm simulations when compared to the average annual approach directions.

8.4. Sediment Transport Assessment

The sediment flux in the vicinity of the Canal indicates there is a strong net alongshore transport from northwest to southeast, consistent with the prevalent northeast wave approach direction. However, the magnitude of the transport varies throughout the domain. Along Scusset Beach, updrift of the Canal, the average annual transport rate is approximately 95,000 to 115,000 cy/year to the southeast. Southeast of the Canal, there is a small zone of transport reversal in which the transport rate is approximately 10,000 to 20,000 cy/year to the northwest. Southeast of the reversal, net alongshore transport patterns are again directed toward the southeast at approximately 35,000 to 45,000 cy/year.

8.5. Alternative Screening

A variety of alternatives were considered for addressing the erosion at Sandwich. Alternatives were determined jointly between WHG, the Corps, and the Town of Sandwich.

The nearshore (local) wave model was used as a screening tool through evaluation of results, wave height changes, and assessment of potential impacts. Potential adverse impacts to neighboring beaches and navigation were also evaluated. Six alternatives were evaluation in terms of sediment transport changes and beach performance. Beach nourishment alone was selected as the preferred alternative to mitigate for erosional damages directly attributable to the Canal FNP.

8.6. Beach Fill Design

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

Beach nourishment and dune creation in this alternative will require approximately 388,000 cy of clean, beach compatible material. While variations in beach nourishment volume were investigated as part of the alternatives evaluation, smaller placement volumes would require more frequent and costly renourishments. The beach design provides the minimum fill requirements needed to be in place to withstand a 10 year storm, with a nourishment interval of 9 years.

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