Coastal Erosion, Sediment Transport, and Prioritization Management Strategy Assessment for Shoreline Protection

Scituate, Massachusetts

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1.0 INTRODUCTION

The Town of Scituate suffers extensive flood damage along many of its east-facing beaches, with total Federal Emergency Management Agency (FEMA) repetitive loss claims in excess of $61.8 million from 1978 to March 2015. Ongoing threats to public and private infrastructure continue to be a major concern for the Town, as both long-term coastal erosion and relative sea level rise in the coming decades will continue to exacerbate regional storm damage. With this understanding, the Town pursued a long-term planning effort to identify ongoing coastal erosion and the sediment transport pathways that govern this process, screen potential shore protection strategies to determine their applicability, assess both historical storm damage and needed shore improvement costs by shoreline reach, and prioritize shore protection and/or other management strategies based on potential costs and storm protection benefits. This town-wide optimization/prioritization shore protection planning effort received funding from the Executive Office of Energy & Environmental Affairs (EOEEA) Coastal Community Resilience Grant Program for Fiscal Year 2016. This report summarizes the findings of this joint effort between the Town of Scituate and the EOEEA.

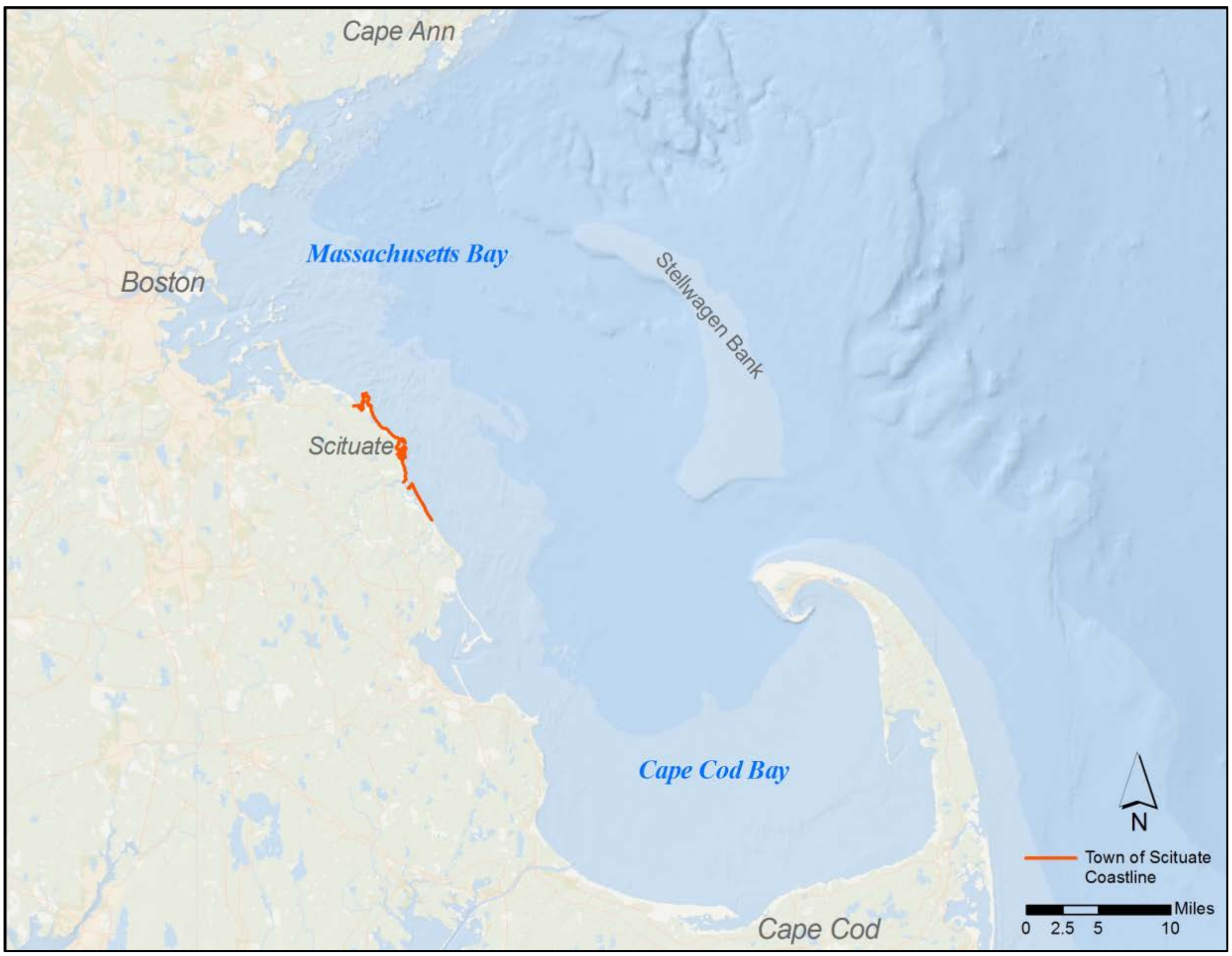


Figure 1.1 Map of Massachusetts showing the location and orientation of the Scituate coastline.

Based on recent work performed for the state-wide coastal structure inventory, even repairs to the existing seawall infrastructure fronting many of Scituate's shoreline will require approximately $70,000,000. These repairs will re-establish the "hardened" shoreline that prevents erosion of the upland; however, this approach does not address the longer-term concerns regarding ongoing shoreline migration and lowering of the beaches fronting the seawalls. Specifically, the increased water depth fronting the seawalls during coastal storms allows larger waves to impact the coastal infrastructure and this repair estimate does not include any enlargement of the shore protection structures that would be required to protect against rising sea levels. Therefore, repairs alone will not improve coastal resiliency along armored shorelines and other management and/or engineering strategies will need to be pursued to address the dual goals of coastal hazards management and climate adaptation.

Over the last several years, the Town has made great strides providing public outreach regarding coastal hazards and the effects of future sea level rise. In addition, work continues on upgrading existing seawalls (e.g. Oceanside Drive) and moving forward on other needed shore protection improvements (e.g. large-scale beach nourishment along North Scituate Beach). While these efforts continue, implementation has generally been performed in a reactionary manner, with storm damage repairs performed on an emergency basis. To provide for long-term coastal management, this effort represents initial steps by the Town to prepare a proactive planning approach to provide a broader town-wide perspective relative to shore protection needs and prioritization of projects.

To help build coastal resiliency into the long-term Town planning efforts, a multi-disciplined approach has been performed to address both the scientific/engineering and economic concerns. The approach can be divided into six (6) major tasks, which are described in more detail below:

- Analyze Coastal Change and Sediment Transport Processes
- Assess Historical Storm Damage Based on Storm Severity
- Develop Prioritization Criteria for Coastal Resiliency
- Determine Appropriate Shore Protection and/or Coastal Management Approaches
- Evaluate Shore Protection and/or Management Strategies by Shoreline Stretch
- Disseminate Findings at Two Public Working Sessions

The overall goal of the planning analysis is to produce a "roadmap" that the Town can utilize to proactively plan for projects that will improve the coastal resiliency of the community. By basing future shore protection decisions on a quantitative analysis of town-wide coastal processes, it is anticipated that more cost-effective and sustainable solutions can be developed as part of a long-term planning process.

1.1 Analysis of Coastal Change and Sediment Transport Processes

An evaluation of recent geological history of the Scituate coastline was developed utilizing standard reference materials (maps, aerial photographs, and regional geologic data), as well as knowledge regarding glacial geology and its role in shaping the Massachusetts Coast. The South Shore shoreline in the vicinity of Scituate consists of both glacial deposits and underlying bedrock outcrops, causing the undulating shape of the shoreline typified by pocket beaches punctuated with headlands. These headlands often consist of boulder/cobble lag deposits that are left behind as finer-grained clays/silts/sands are washed away. These more erosion-resistant deposits can act as natural armoring, thereby reducing or, in some cases, eliminating the landform as a sediment source to down-drift beaches. In addition, extensive armoring of the shoreline within Scituate also has eliminated many of the sediment sources. Understanding the geologic evolution of the beach system, as well as anthropogenic influences, allows determination of the limits of the regional “littoral system”. In this manner, determinations can be made regarding potential future sources of natural littoral sediments to the Scituate shoreline. This evaluation also will include how anthropogenic changes may have altered the natural sediment transport processes and the influence of sea level rise upon the long-term stability regional coastline.

In addition to an overall analysis of recent geologic history, an evaluation of local shoreline change (both recent and long-term) was performed for the Scituate shoreline. The analysis incorporates information from the North Scituate beach nourishment project, as well as other available information. Interpreted high water lines from recent aerial photography was utilized to evaluate recent changes in shoreline position.

A quantitative analysis of coastal processes was required to develop an understanding of alongshore sediment transport along the Scituate coast. However, numerical analysis techniques alone cannot provide the needed information for sediment transport that also is strongly dependent upon storm-induced changes (overwash) for portions of shoreline dominated by barrier beaches. An analysis of overtopping and its relative contribution to the sediment transport processes was performed, based on available data. Specifically, available LiDAR data was evaluated to assess short-term landward migration of dune ridges and the overall barrier beach system.

Prior to evaluating management options for addressing the shoreline recession and storm damage, a quantitative understanding of the coastal processes causing the local sediment deficit was required. Two numerical models were used to evaluate coastal processes: a wave refraction model and a longshore sediment transport/shoreline change model. The wave refraction modeling is required to estimate the driving forces governing longshore transport. Since the local bathymetry modifies the wave directions and heights, this model was used to determine how local changes in wave conditions modify sediment transport potential along the beach. The wave analysis was based upon both open-ocean and wind-driven waves that control local coastal processes. The study incorporated state-of-the-art wave refraction analysis techniques, utilizing the model SWAN (Simulating WAves Nearshore) developed by Delft University in the Netherlands, to transform the offshore waves to the shoreline for long-term sediment transport calculations.

Once wave heights and directions for various conditions had been determined, a sediment transport model was employed to estimate the annual longshore sediment transport rate. Sediment transport direction and rate are important parameters that characterize the stability of the nearshore system. In the longshore direction, a system in equilibrium will have a small net transport along the length of the shoreline due to balanced wave and current forces. The equilibrated shoreline may experience high wave energy conditions; however, there will be an overall balance in transported sediment volume in both longshore directions.

Utilizing a combination of the wave model information, existing historical shoreline change data, sediment grain size information, and seasonal beach form data, a predictive model of longshore sediment transport was calibrated to observed conditions. Once the shoreline change model had been calibrated, it was utilized to simulate the longevity and migration of potential beach nourishment projects. This aspect of the modeling effort was critical for assessing the viability of potential shore protection approaches.

Within the context of ongoing coastal evolution, the influence of relative sea level rise also was accounted for within the analysis. In this manner, quantitative information can inform the evaluation of engineering approaches for appropriate time horizons.

1.2 Assessment of Historical Storm Damage Based on Storm Severity

The Town has cataloged storm damage impacts for several decades; however, there has been a need to combine detailed storm damage data to the coastal infrastructure that protects this development. As the basis for prioritizing future actions to address coastal resiliency, it will be critical to relate ongoing (and future, to the extent possible) storm damage costs to the overall costs of infrastructure improvements needed to address these concerns. As the extensive flood damage that occurs along the Scituate coast primarily is driven by the influence of nor'easters, a more detailed understanding of impact related to storm severity was warranted. Specifically, the relationship between storm parameters and the severity of damage is critical for establishing expectations for shore protection strategies. With this understanding, four characteristic storms were selected to analyze the spatial distribution of the residential damage claims and the financial costs to the Town: the Blizzard of 1978, the 1991 No-Name Storm, Winter Storm Nemo (2013), and Winter Storm Juno (2015).

The Blizzard of 1978 was included as the storm of record, with a return period of greater than once every 100 years. While substantial infrastructure damage occurred during this storm event (Figure 1.2), the FEMA National Flood Insurance Program (NFIP) had not been officially implemented at the time of the event. Regardless, the severity of infrastructure damage associated with the highest storm surge experienced in Boston since the tide station went into operation in 1921 provides the benchmark for comparison of South Shore coastal storms in the past century. Although slightly less severe, the 1991 No-Name Storm was selected because it represented the most significant storm event in the past 30 years and represented the most severe storm event that occurred since FEMA NFIP had been implemented. In addition, the more recent Winter Storms Nemo and Juno also were reviewed to represent "typical" events that occur on a relatively frequent basis. These lower return period events were selected due to the detailed storm damage documentation available from the Town, including coastal infrastructure improvement needs associated with ongoing damage. Understanding the geographical distribution of storm damage for relatively frequent, as well as more severe infrequent, storm events allowed for a detailed economic assessment of damages relative to storm severity for different locations along the coastline.

1.3 Development of Prioritization Criteria for Coastal Resiliency

As described above, the coast of Scituate consists of both glacial deposits and underlying bedrock outcrops, causing the undulating shape of the shoreline typified by pocket beaches punctuated with headlands. In addition, extensive armoring of the shoreline has altered natural coastal processes. Understanding that the specific type of shoreline can be linked to its vulnerability to storm impacts, the coast was divided into characteristic sections that allowed for site-specific evaluation of appropriate prioritization criteria for addressing coastal resiliency concerns.

The overall goal was to create an objective set of technical criteria that could be utilized to create a rating system for the different sections of the Scituate coast. In addition to economics associated with damage susceptibility, a number of other factors were evaluated including landform elevation, need for providing emergency egress, breaching potential of the landform, and existing condition of any coastal engineering structures. Prioritization for infrastructure protection for a particular portion of shoreline depended upon potential damage to both private and public assets, as well as existing condition parameters. Development of prioritization criteria in this manner provides baseline information that the Town of Scituate can utilize to focus efforts on the most vulnerable areas.

Figure 1.2 Portions of the North Scituate Beach seawall destroyed during the Blizzard of 1978 (*image credit: The Boston Globe*).

1.4 Determination of Appropriate Shore Protection and/or Coastal Management Approaches

A number of potential shore protection options were evaluated to provide the basis for the site-specific assessment of alternative for each shoreline sections. The list of alternative shore protection strategies includes numerous “hard” (e.g. seawall and revetment) and “soft” (e.g. beach and dune nourishment) coastal engineering techniques, as well as potential innovative approaches (e.g. boulder dikes). In addition, the baseline alternative consists of maintaining the *status quo* of continuing to repair infrastructure as needed following storm damage and/or demonstrable failure.

Initially, each shore protection strategy was broadly reviewed relative to its applicability for the Scituate shoreline. Within this context, the shore protection options were evaluated relative for (a) the ability to provide the necessary level of shore protection, (b) the anticipated environmental impacts and associated ability to advance the option through the environmental regulatory process, and (c) the overall cost of the alternative including both initial construction and maintenance costs. Due to geological framework of the natural coastline, as well as anthropogenic changes that have occurred to provide shore protection, a wide variety of approaches exist for addressing coastal sustainability issues. The goal of providing an initial assessment of this broader range of shore protection approaches was to ensure that a broad range of approaches were carried forward into the site-specific assessment.

1.5 Evaluation of Shore Protection and/or Management Strategies by Shoreline Stretch

Several locations that have suffered historical flood damage within Scituate exist along barrier beach systems or other low-lying coastal features are fronted by seawalls and/or rubble mound revetments. For these shoreline stretches, it is critical to understand the major driving forces governing storm damage, as well as how future coastal erosion and sea level rise may exacerbate these conditions. For armored shoreline reaches within the Town, the influence of the existing armoring was evaluated relative to potential alternative shore protection measures that could enhance or replace existing infrastructure.

Once approaches were assessed relative to their applicability to shore protection, screening of these options was performed to determine the most appropriate approaches for each shoreline section. In general, discretionary criteria were utilized to assess the applicability of different options, considering aspects of each alternative including engineering, economics, and potential environmental impacts. Once the approach screening process was completed, a matrix of potential shore protection options was developed for each shoreline section based upon the assessment of vulnerability and "need" from the overall economic parameters. This scheme included both "hard" and "soft" shore protection measures, based on project need within each of the shoreline sections identified. In general, economic drivers were critical to this prioritization process; however, coastal resiliency was also addressed, as future shore protection expenditure planning required that a sustainable outcome will be achieved based upon a 50-year planning horizon. In some cases, the economics indicated that managed retreat is the most feasible alternative. The outcome of the prioritization assessment of shore protection management strategies based on both "need" and economic drivers is aimed at providing guidance for future Town planning efforts.

1.6 Public Working Sessions

A critical aspect of the overall prioritization plan for shore protection was to inform the public regarding both the process and the findings of this planning effort. Two public working sessions took place and results of the analysis at key stages were presented as stakeholder input was critical to the overall process of coastal planning efforts. For Scituate this is especially critical due to the level of potential local funding required to address the ongoing issues associated with coastal storm damage mitigation, as well as extensive involvement by homeowner groups affected by coastal storm damage.

After the development of the prioritization criteria, a public working session was conducted on April 28, 2016 to present the findings of the coastal processes analysis, the evaluation of historical storm damage, and the overall approach for developing prioritization criteria to utilize as the basis for coastal resiliency planning. A second public working session was held on June 16, 2016 to present the findings of the overall analysis. During this presentation, detailed evaluations of each shoreline section were presented, including information regarding the storm protection benefits, longevity, and costs for applicable measures for maintaining coastal development.

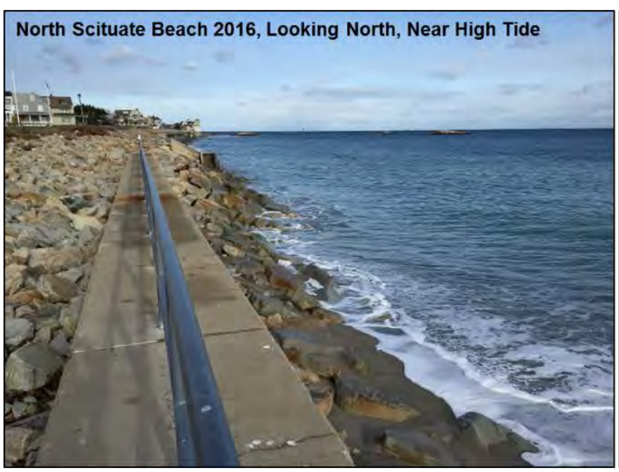
Presenting the study findings at these public working sessions represented the first step in the public process for town-wide coastal resiliency planning efforts. As the Town moves forward with potential recommendations, a more comprehensive project-specific public process will be performed as part of more detailed design analyses and the environmental permitting process.

2.0 HISTORICAL COASTAL CHANGE

The Town of Scituate enjoys scenic, economic, and recreational advantages associated with the ocean shores. However, inherent in being an oceanfront community, there exists a unique set of challenges that the Town faces, specifically impact from the ocean's sheer force. Direct effects of climate change, including sea level rise and increasing intense storm events, magnify these challenges. As a result of northeast facing of the Scituate shoreline and the exposure to open North Atlantic Ocean wave conditions, the coast of Scituate is a highly dynamic region, where natural forces continue to reshape the shoreline.

In addition, construction of shore protection measures over the past 90+ years has altered the sediment transport patterns along the shoreline. The glacially derived morphology of the Scituate coastal system creates an irregular shoreline with a series of barrier beaches, tidal inlets, more erosion-resistant coastal banks, and occasional non-erodible bedrock outcrops. Historically, natural coastal erosion processes eroded sediments from both the coastal banks and coastal beaches and formed a series of barrier beaches along the Town of Scituate shoreline.

Regionally, the Scituate shoreline consists of glacial till headlands, bedrock outcrops, and outwash deposits, as well as associated marine deposits in the form of barrier beaches. Glacial deposits historically provided the principal source of beach sediments, consisting of a broad range of sand, gravel, cobbles, and boulders, depending on the composition of the eroding glacial deposit. Many of these original sources of beach materials have been largely eliminated due to the construction of revetments and seawalls along the shoreline. Figure 2.1 and Figure 2.2 illustrate the change in beach conditions exacerbated by coastal armoring at North Scituate Beach and Third Cliff, respectively.



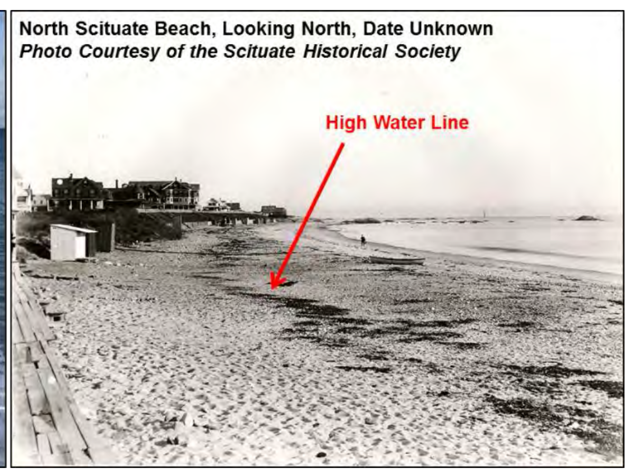


Figure 2.1 Photographs of North Scituate Beach from 2016 (left) at the time of high tide and likely in the early 1900s (right) indicating the location of the high water line. As shown, significant landward migration of the high water line has occurred over the past 100 years.

Figure 2.2 Photographs of Third Cliff from 1913 (top left) and 2016 (bottom), as well as a sketch from 1915 (top right). As shown, significant landward migration of the high water line has occurred over the past 100 years.

Use of shoreline and bathymetric change information allows quantification of coastal processes by providing a measure of nearshore accretion or erosion. For the Scituate shoreline, high quality shoreline data sets are available dating back to the mid-1800s. This 160+ year time period covers the transition of the shoreline from an unaltered natural beach system to the highly engineered shoreline that exists today. Due to the substantial natural and anthropogenic alterations to the shoreline over this time period, the more recent time period (1950s to present) is more appropriate for assessing the shoreline dynamics associated with present conditions. Based on the available shoreline data, the 1950/1952 shoreline represents the most modern shoreline prior to large-scale construction of revetments and seawalls along the Scituate coast. By utilizing the 1950/1952 shoreline as an “initial condition” and more recent (2000s and onwards) shorelines as the “current condition”, the shoreline change analysis can effectively determine the influence of the contemporary sediment transport on shoreline position.

2.1 Shoreline Change Analysis

Shoreline change is typically minimal along stretches where coastal engineering structures have been built. In many of these areas, notably at North Scituate Beach and along Oceanside Drive, the fronting beaches are submerged at high tide. The heavily armored Scituate shoreline leaves only several areas where the shoreline migration is not limited by seawalls and revetments: Mann Hill Beach, Peggotty Beach, and Humarock Beach. The shoreline change analysis focused on these three areas; however, shoreline change rates for the entire Scituate coastline from 1950/1952 to 2001 are presented in Appendix A. It should be noted that the change rates represent the horizontal shoreline migration only and do not include changes in the beach elevation (i.e. beach lowering) over time. Where the shoreline migration is limited by seawalls and revetments, the shoreline change rates may indicate that little or no horizontal change has occurred but the beach elevation may have lowered substantially over the same time period.

High water shorelines were obtained from 1950/1952 National Oceanic and Atmospheric Administration (NOAA) T-Sheets and by delineating the high water line from 2008 United States Geological Survey (USGS) aerial photographs. The high water shoreline position change rates were calculated by casting perpendicular transects to the later input shoreline at each analysis point (every 32.8 feet) along the line to the earlier shoreline. The result is a table of shoreline change magnitudes and rates for each transect where shoreline change denoted with a minus sign represents erosion.

All shoreline position data contain inherent errors and/or uncertainties associated with field and laboratory compilation procedures. The potential measurement and analysis uncertainty between the data sets is additive when shoreline positions are compared. Because the individual uncertainties are considered to represent standard deviations, a root-mean-square (RMS) method was used to estimate the combined potential uncertainties in the data sets. The positional uncertainty estimates for each shoreline were calculated using the information in Table 2.1. These calculations estimated the total RMS uncertainty to be ±0.5 feet/year from 1950/1952 to 2008. Transects with calculated shoreline change rates within the RMS uncertainty are shown in gray in the following figures.

The 2008 high water line approaches the five homes on Stanton Lane, as these structures now reside seaward of the dune crest, as shown in Figure 2.4. Figure 2.3 shows the shoreline change rate along Mann Hill Beach. The shoreline has generally been eroding at 0.5 to 1.0 feet/year at the north end of the beach. The cobble beach berm at the north end gets repaired with after major storms to control the overtopping of water into Mushquashcut Pond. These repairs have included importation of additional compatible material to augment material that has overwashed into the pond. At the south end of the beach, towards Egypt Beach, erosion has been occurring at 1.5 to 2.0 feet/year. North Scituate Beach and Egypt Beach/Oceanside Drive, located adjacent to Mann Hill Beach, show nearly no shoreline change over the 58 year period due to armoring of the shoreline and seawalls.

Shoreline change along Peggotty Beach is shown in Figure 2.5. Peggotty Beach has been eroding since the 1950's with erosion rates of up to 4 feet/year at the south end of the beach. This erosion rate represents the highest shoreline change rate along the developed portions of the Scituate coastline. In 2008, the high water line is shown to reach the seaward face of the first row of homes on Town Way Extension (Figure 2.6). Due to the generally low elevation of Peggotty Beach, much of the erosion is caused by storm surge and wave action overtopping the barrier beach system and pushing sediment into the marsh system landward of the beach. Peggotty Beach is situated between Second Cliff and Third Cliff; the cliffs have been armored and show no long-term erosion of the horizontal shoreline position.

Humarock Beach has generally experienced shoreline erosion from the 1950's to 2008, as shown in Figure 2.7. Long-term erosion is higher at the south end, where a landward shoreline migration rate of nearly 4 feet/year has been observed. Near the north end of the beach, the 2008 high water line is located approximately 50 feet seaward of the periodic public and private coastal engineering structures and this distance increases up to 100 feet at the south end of the beach.

Table 2.1 Estimates of potential error/uncertainty associated with shoreline position surveys.	
Traditional Engineering Field Surveys	
Position of rodded points Location of plane table Interpretation of high-water shoreline position at rodded points Error due to sketching between rodded points	±3 feet ±7 to 10 feet ±10 to 13 feet up to ±16 feet
Cartographic Errors (1950/1952)	Map Scale 1:10,000
Inaccurate location of control points on map relative to true field location Placement of shoreline on map Line width representing shoreline Digitizer error Operator error	Up to ±10 feet ±16 feet ±10 feet ±3 feet ±3 feet
Historical Aerial Surveys (1950/1952)	Map Scale 1:10,000
Delineating high-water shoreline position	±16 feet
Orthophotography (2008)	
Delineating high-water shoreline position Position of measured points	±10 feet ±10 feet
GPS Surveys	
Delineating high-water shoreline position Position of measured points	±3 to ±10 feet ±3 to ±10 feet

Figure 2.3 Historical shoreline change for Mann Hill Beach from 1950/1952 to 2008.

Figure 2.4 Dwellings along Stanton Lane illustrating proximity of the typical high tide line to the structures, as well as the position of the houses on the seaward side of the cobble dune crest (photo taken by Applied Coastal on May 10, 2016).

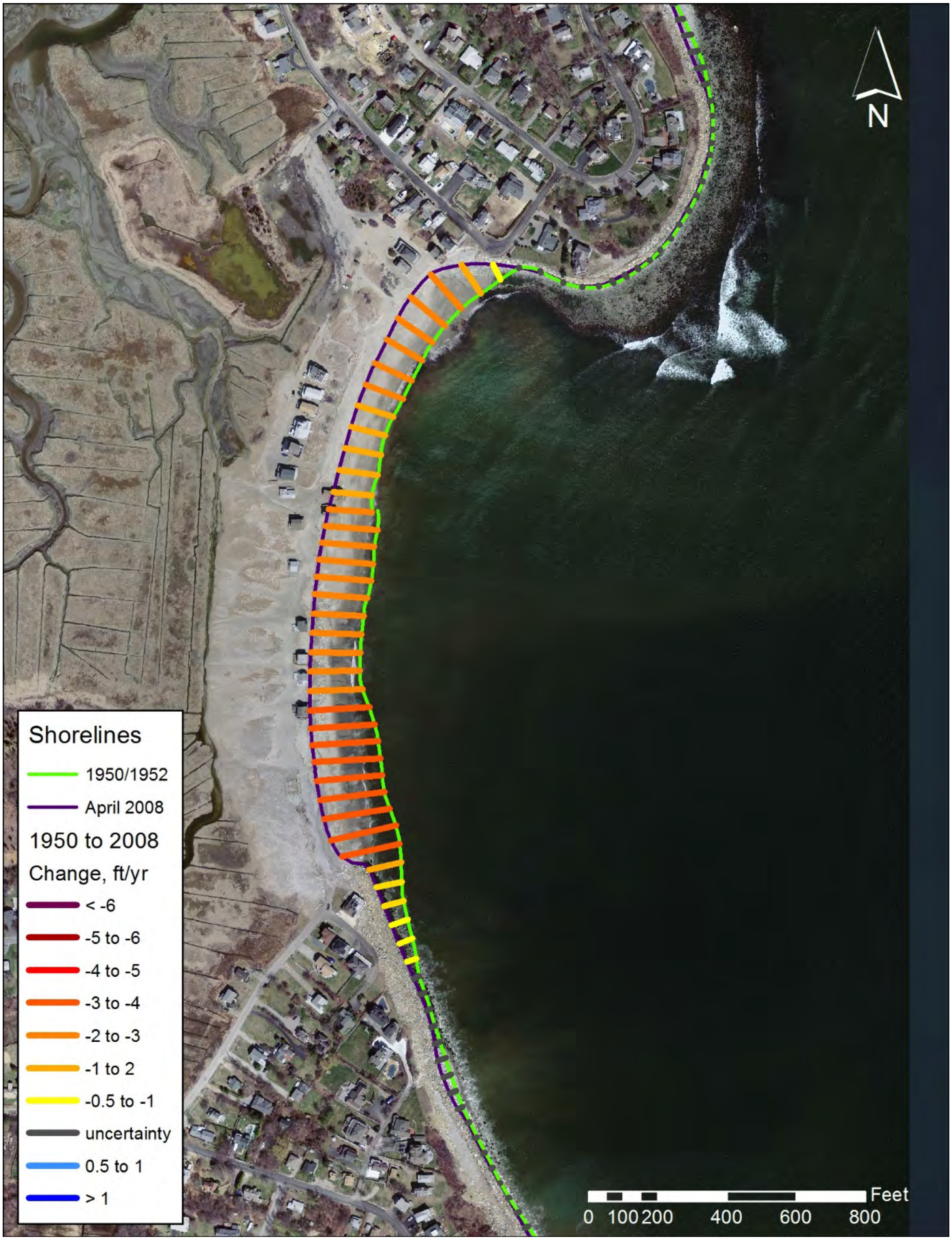


Figure 2.5 Historical shoreline change for Peggotty Beach from 1950/1952 to 2008.

Figure 2.6 Dwellings along Town Way Extension, where the daily high tide line is under the buildings and the position of the houses is well seaward side of the low dune at Peggotty Beach (photo taken by Applied Coastal on May 10, 2016).

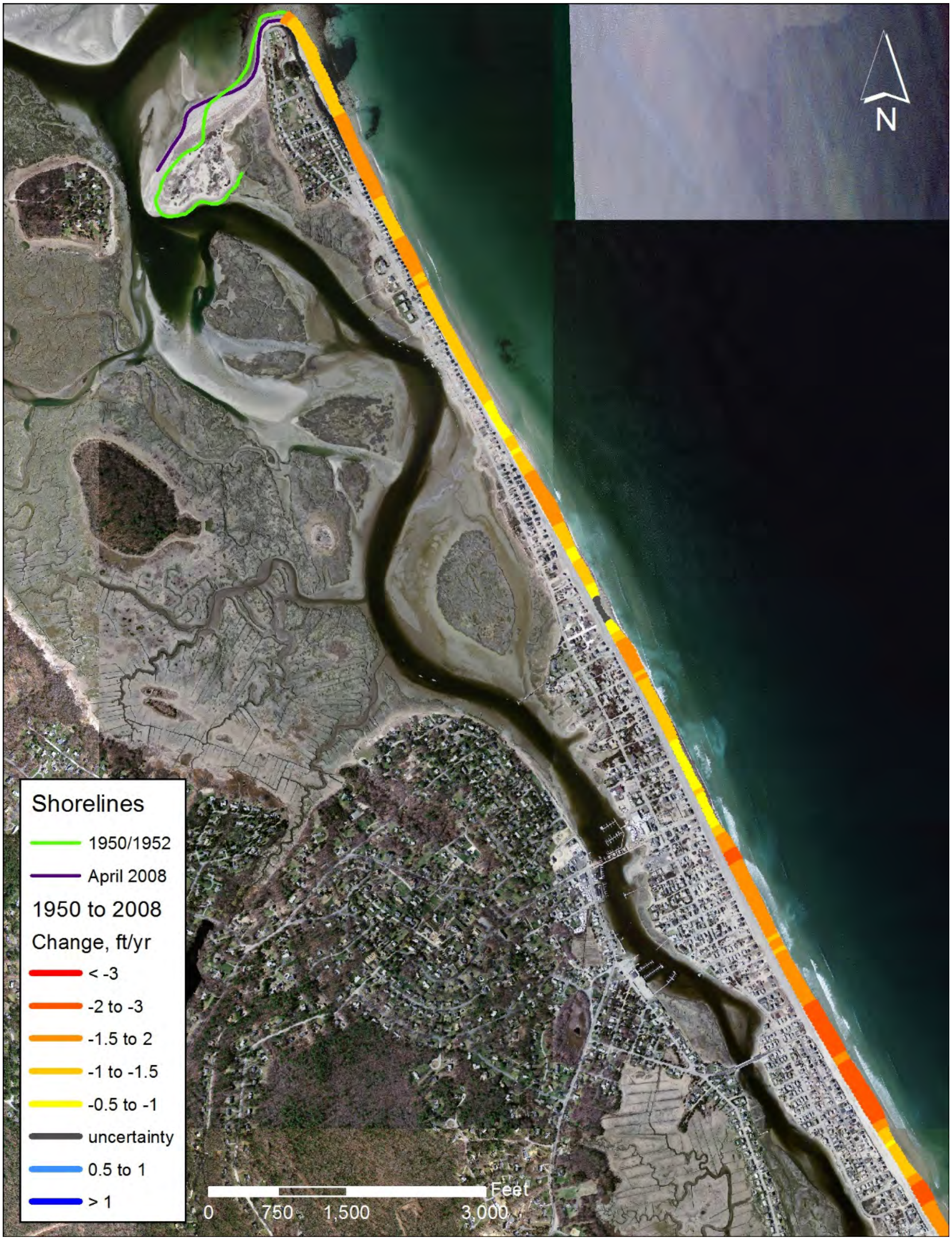


Figure 2.7 Historical shoreline change for Humarock Beach from 1950/1952 to 2008.

2.2 Topographic/Bathymetric Change Analysis

Light Detection and Ranging (LiDAR) survey data was evaluated to provide a more detailed assessment of barrier beach migration, as storm-driven overwash appears to be the dominant process controlling long-term performance of these beach areas. LiDAR data provided three dimensional surfaces of topographic, as well as limited nearshore bathymetric, information that could be evaluated within appropriate mapping software. Comparison of these topographic/bathymetric surfaces between years allowed for an analysis of sediment movement. Specifically for the barrier beach areas (i.e. Mann Hill/Egypt Beaches, Peggotty Beach, and Humarock Beach), the LiDAR comparisons allowed a more detailed assessment of recent time periods between 2000 and 2014. The series of LiDAR datasets available were utilized to the maximum extent possible to develop a clear understanding of the cross-shore sediment transport and barrier beach migration processes. Due to the anthropogenic manipulation of sediments along many of the developed barrier beach areas after storms, it was not always possible to track the natural barrier beach dynamics. However, to the maximum extent possible, the LiDAR data was utilized to assess the influence of cross-shore processes during significant storm events.

The LiDAR datasets were incorporated into ArcGIS. To simplify processing, elevation data points are interpolated onto a regular 2 foot by 2 foot raster grid to provide the final elevation surface. Raster models store the elevation data in uniform data units (cells); therefore, they are less computationally complex and allow for more rapid processing. Cross-shore profiles were extracted from 2000, 2010, 2011, and 2014 LiDAR datasets. The datasets from 2000 and 2010 included bathymetry (underwater topography) measurements while the other datasets are limited to dry land. Figure 2.8 shows the location of five representative transects obtained from Mann Hill Beach, Egypt Beach, Peggotty Beach, the north section of Humarock Beach, and the south section of Humarock Beach. The location of these transects were selected to represent characteristic areas of beach and dune width/elevation for these areas of the Scituate shoreline. The LiDAR datasets were also subsequently used in this study to analyze flooding extends, road and structure elevations, and dune volumes.

Figure 2.9 shows the profile along Transect 1 at Mann Hill Beach. The dune along this area has been reshaped, including the addition of rounded gravel/cobble fill, in recent years by the Town after moderate storms to maintain a consistent crest elevation of approximately 21 feet NAVD88 (North American Vertical Datum of 1988). While the reshaping maintains the height of the dune, the LiDAR profile indicates that the nearshore beach profile has lowered by approximately 3 to 4 feet between 2000 and 2010. This information indicates that in the long-term, only maintaining the gravel/cobble dune will not effectively maintain the entire beach profile. Long-term sediment starvation of this shoreline reach eventually will lead to more frequent maintenance of the barrier beach system, as nearshore sediments are no longer available to replace the lost sediments from the beach.

Without ongoing maintenance, the dune crest of Transect 1 would likely look similar to Transect 2, located at Egypt Beach (Figure 2.10). Since 2000, overwash of the cobble dune has reduced the crest elevation and moved the material landward into Sheep's Pond, creating overwash fans shown in Figure 2.11. Storm overwash into Sheep's Pond has led to concerns regarding infilling of the natural inland drainage path connecting this pond to Mushquashcut Pond. Lowering of the nearshore profile also was observed between the 2000 and 2010 surveys; however, the profile lowering offshore is only 1 to 2 feet. In addition to lowering of the beach/dune berm, landward migration of the dune crest has led to dwellings constructed along this shoreline stretch to become more exposed to storm wave action, as the homes along Stanton Lane are now located seaward of the dune crest (Figure 2.4).

Figure 2.8 Location of cross-shore profile change analysis transects.

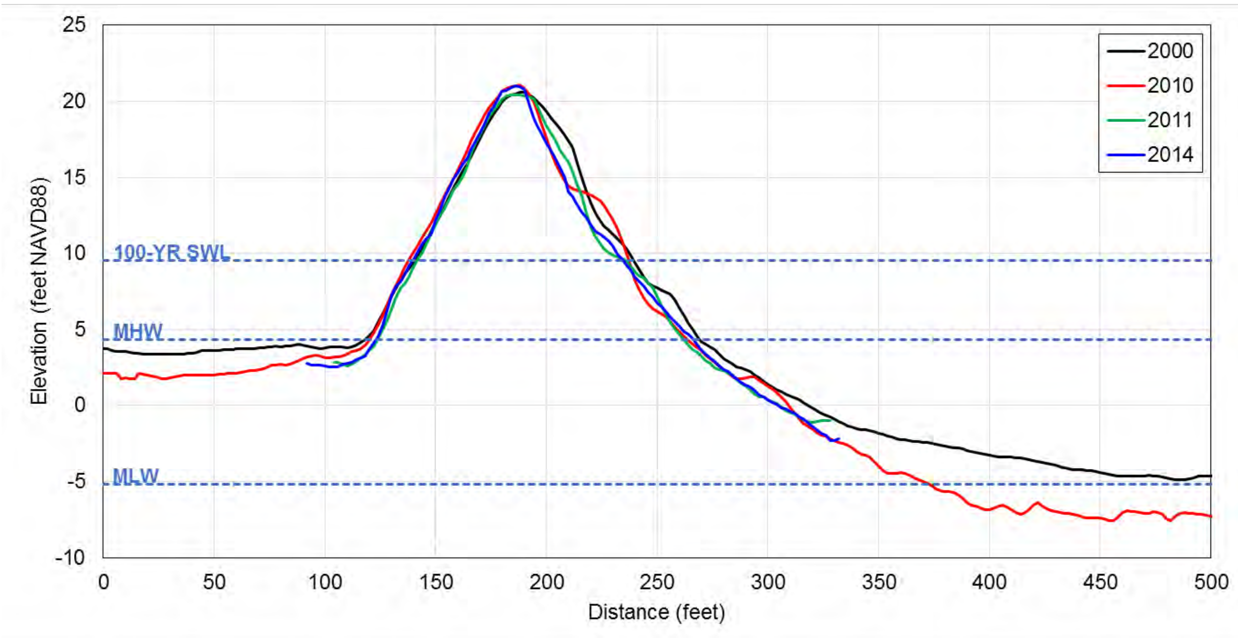


Figure 2.9 Profile change from 2000, 2010, 2011, and 2014 at Transect 1 (Mann Hill Beach). In addition to the LiDAR profiles, the dashed lines indicate the elevation of Mean Low Water (MLW), Mean High Water (MHW), and the 100-year Still Water Elevation (100-YR SWL).

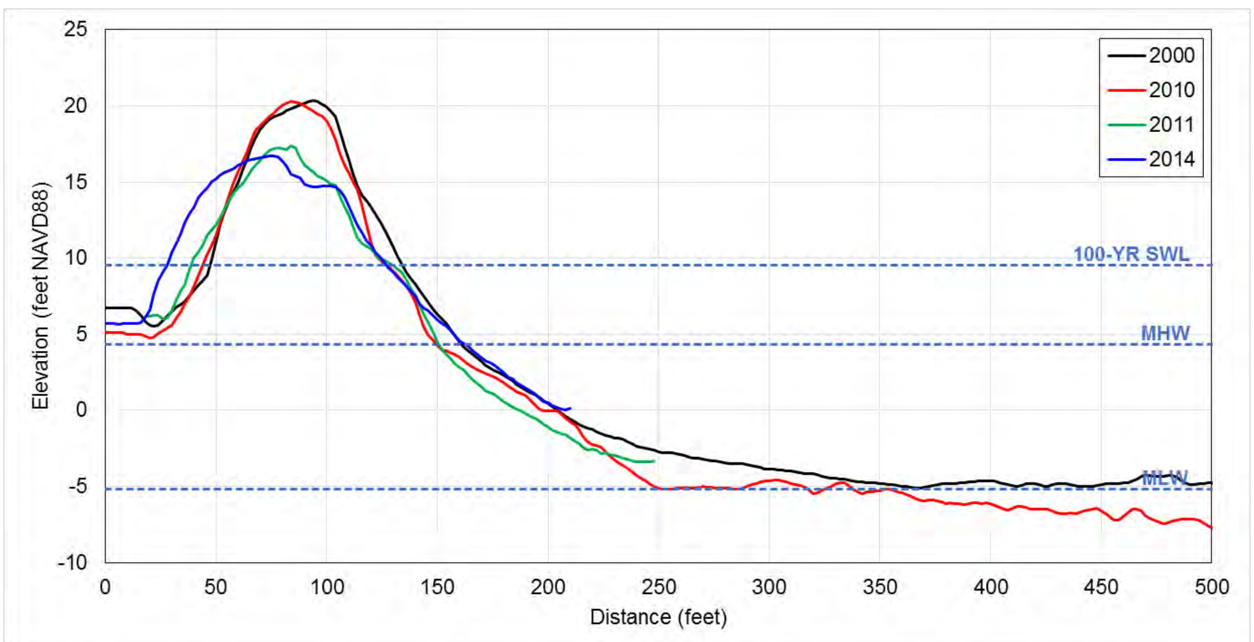


Figure 2.10 Profile change from 2000, 2010, 2011, and 2014 at Transect 2 (Egypt Beach). In addition to the LiDAR profiles, the dashed lines indicate the elevation of Mean Low Water (MLW), Mean High Water (MHW), and the 100-year Still Water Elevation (100-YR SWL).

Figure 2.11 Overwash of beach material from Egypt Beach into Sheep's Pond creates large overwash fans (photo taken by Applied Coastal on May 10, 2016).

At Peggotty Beach, frequent storm overwash across the beach has caused the shoreline to retreat and material to build up on the back of the beach across the eastern portion of the bordering salt marsh as shown in Figure 2.12. The Town Way Extension, once located at approximately 10 feet NAVD88, has since been filled in by more than 3 feet of material. Substantial lowering, up to 5 feet, of the fronting beach was observed from the 10-year period between 2000 and 2010. Overall, the elevation of the dune system along Peggotty Beach is substantially lower than other barrier beach systems in Scituate that are exposed to similar wave climate. This relatively low elevation allows for significant overtopping of the barrier even during periods of moderate storm activity. As shown in Figure 2.13, overtopping of the barrier beach extended to February 2, 2016, even though the nor'easter had pulled offshore the previous day. This observed overtopping was a result of higher than normal tides created by the nor'easters low pressure system and moderate wave action.

The landward migration of the barrier beach system indicated by the 14-year LiDAR measurements is illustrative of a rapidly shifting coastal dune system. At present, the volume of barrier beach sediments is not sufficient to withstand typical storm overtopping events. A review of historical aerial photographs and charts indicates that the dune overwash process has caused infilling of salt marsh channels along the landward fringe of the barrier beach system.

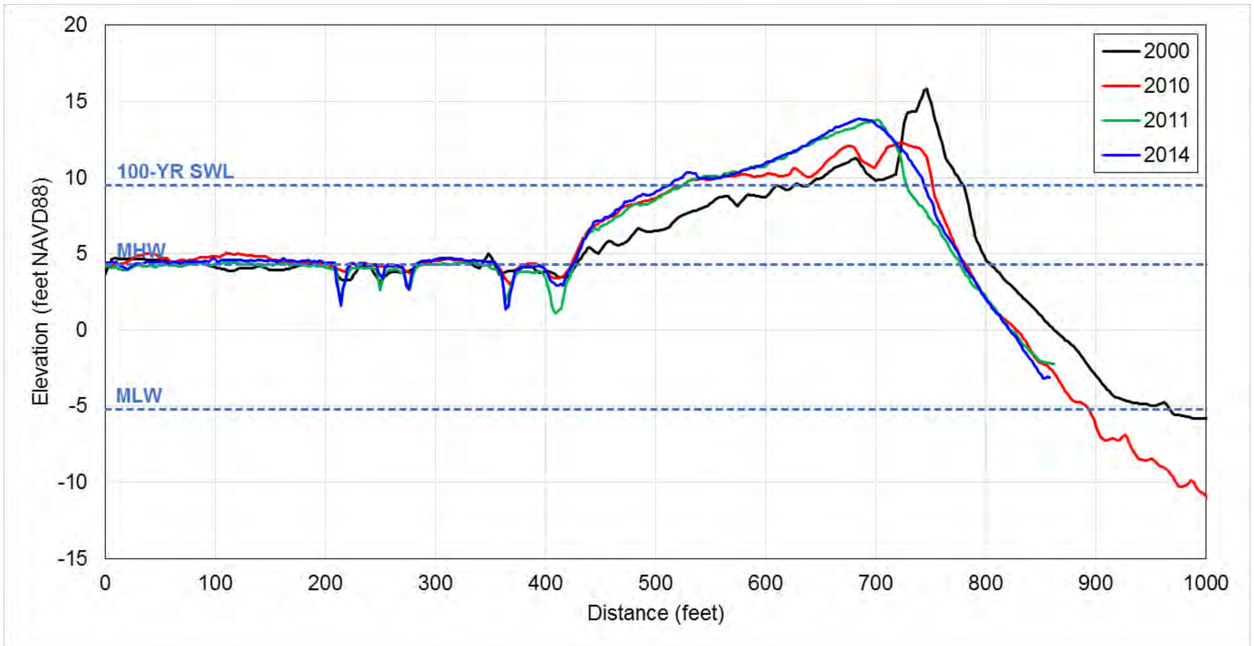


Figure 2.12 Profile change from 2000, 2010, 2011, and 2014 at Transect 3 (Peggotty Beach). In addition to the LiDAR profiles, the dashed lines indicate the elevation of Mean Low Water (MLW), Mean High Water (MHW), and the 100-year Still Water Elevation (100-YR SWL).

Figure 2.13 Overwashing of Peggotty Beach during the high tide after a storm on February 2, 2016 (photo by Peter Miles).

The location of Transects 4 and 5 are shown on Figure 2.8. Transect 4 (Figure 2.14), located in the north section of Humarock Beach at the base of Fourth Cliff, shows retreat of the beach face over the 14-year period. The elevation of the beach dune and roadway immediately landward of the primary dune are consistent over the years. However, it should be noted that both the dune elevation/form and the roadway are maintained following every significant nor'easter, where removal of overwashed beach and dune material off the roads post-storm by the Town and placement of the sediment back onto the beach by the homeowners is required (Figure 2.15).

Figure 2.16 shows that the dune elevation and berm width of Transect 5, located in the south section of Humarock Beach, varies over the period from 2000 to 2014. In general, the beach width along South Humarock is greater than North Humarock and sufficient beach material exists to maintain seasonal beach fluctuations. Therefore, the beach tends to be steeper and narrower during the winter months.

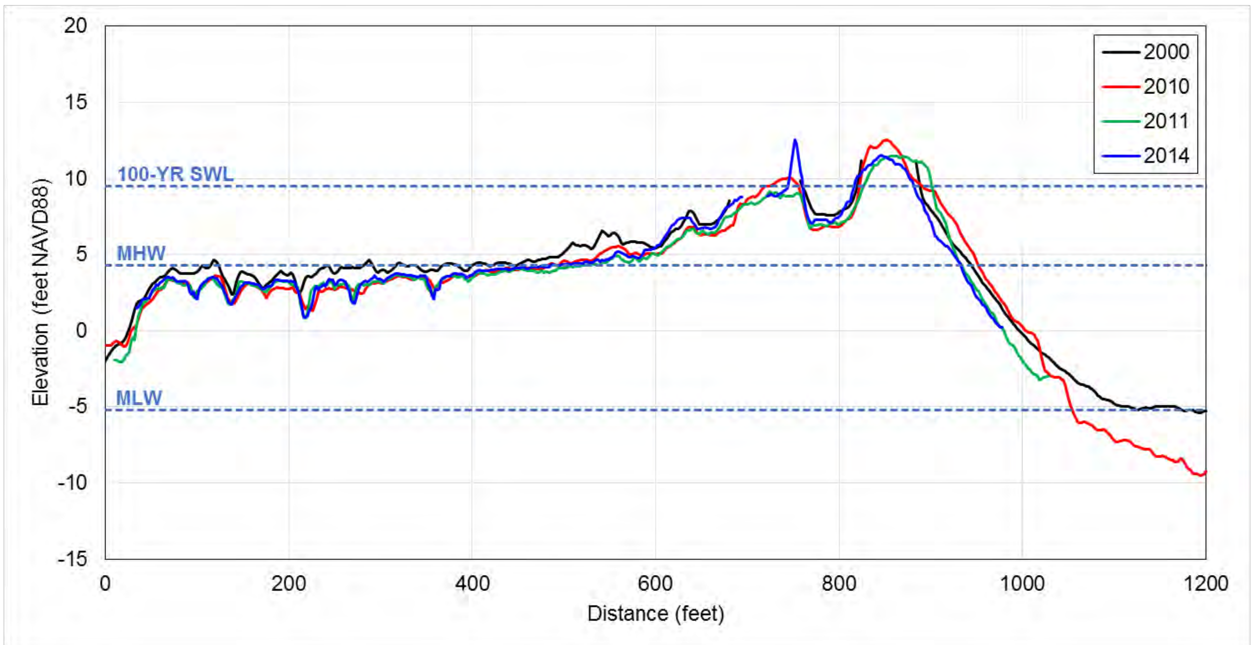


Figure 2.14 Profile change from 2000, 2010, 2011, and 2014 at Transect 4 (north section of Humarock Beach). In addition to the LiDAR profiles, the dashed lines indicate the elevation of Mean Low Water (MLW), Mean High Water (MHW), and the 100-year Still Water Elevation (100-YR SWL).

Figure 2.15 Clearing efforts along Central Avenue in the north section of Humarock Beach following a storm in 2013 (photo by Jason Burtner, *www.mycoast.org*).

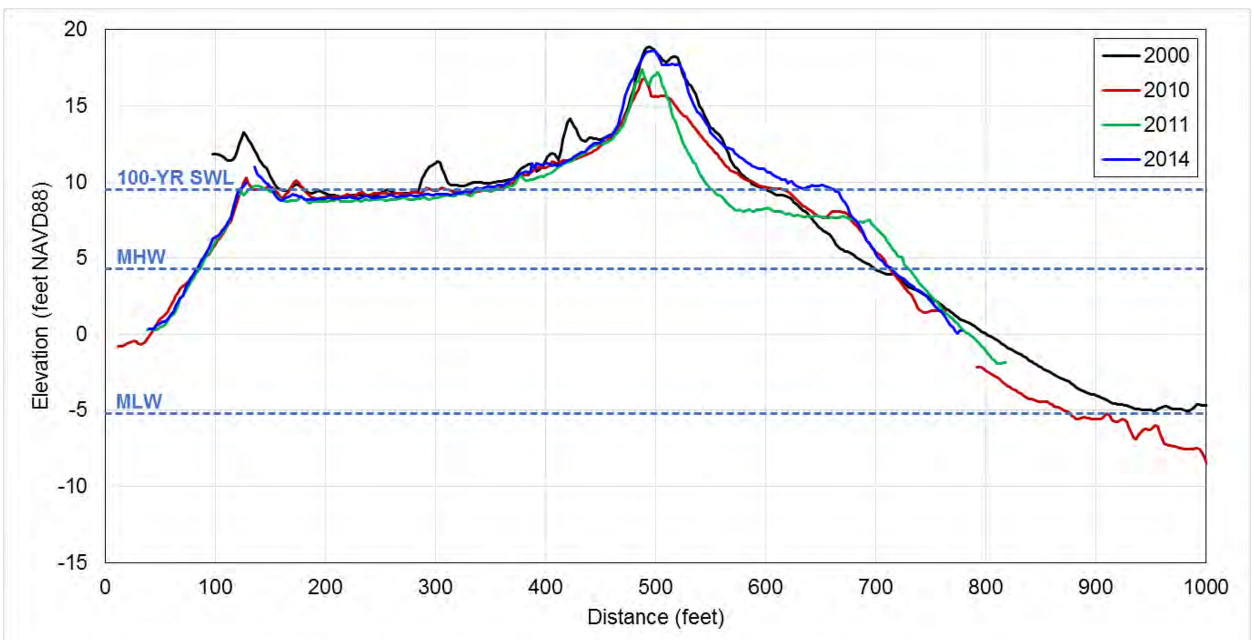


Figure 2.16 Profile change from 2000, 2010, 2011, and 2014 at Transect 5 (south section of Humarock Beach). In addition to the LiDAR profiles, the dashed lines indicate the elevation of Mean Low Water (MLW), Mean High Water (MHW), and the 100-year Still Water Elevation (100-YR SWL).

In addition to the evaluation of beach profiles, the general characteristics of beach and dune change can be viewed in three-dimensions by spatial analysis of the LiDAR data sets. As described above, anthropogenic manipulation of the dune system along northern Humarock Beach tends to bias the results of topographic change measurements derived from the LiDAR analysis. However, natural landward migration of the barrier beach and dune systems at both Mann Hill/Egypt Beaches and Peggotty Beach can be readily observed from the 14-year LiDAR data set (Figure 2.17 and Figure 2.18).

Along Mann Hill Beach (Figure 2.17), there has been a consistent lowering of the beach area seaward of the dune crest. In addition, it appears that the dune crest immediately south of Surfside also has lowered by as much as 5 to 6 feet. Since portions of the Mann Hill Beach dune system have been reconstructed by the Town periodically, the topographic change for the northern portion of Mann Hill Beach may not represent natural barrier beach/dune migration patterns. Further to the south, along the areas immediately north of Mann Hill Road and along Stanton Lane, the beach face elevation appears more stable; however, substantial loss of dune elevation (greater than 5 feet) has occurred over the 14-year period. As this dune area has not been nourished by the Town, the reduction in dune height appears to be a result of dune overwash into Musquashcut Pond, leading to substantial infilling of the area landward of original dune as this feature has migrated landward (e.g. Figure 2.10). In addition, the beach area along southern Mann Hill Beach may be maintaining its elevation as a result of being located downdrift of the reconstructed and renourished dune system along the northern portion of Mann Hill Beach. As shown in Figure 2.4, dwellings along Stanton Lane are now located seaward of the dune crest and more readily exposed to storm wave action. The northern undeveloped portion of Egypt Beach indicates similar trends to southern Mann Hill Beach, but both the dune erosion and subsequent infilling landward of the dune appear less dramatic.

As described previously, Peggotty Beach represents the most rapidly eroding developed area within Scituate. As shown in Figure 2.18, this rapid erosion of the beach face and primary dune is accompanied by an equally rapid infilling of the landward portion of the barrier beach and the eastern fringe of the salt marsh. Areas that had been historically salt marsh as recently as 10 years ago have now become barrier beach, as migration of the entire barrier beach system marches to the west. The most rapid accretion with the back beach and salt marsh areas appears to be along the southern two-thirds of the beach. Similar to the northern Mann Hill Beach area, the measured change along northern Peggotty Beach also likely illustrates anthropogenic effects, in this case clearing of the parking lot and the roadway landward of the dwellings. Landward migration of the fronting dune at the northern end of the beach has reduced storm damage protection to dwellings in this area.

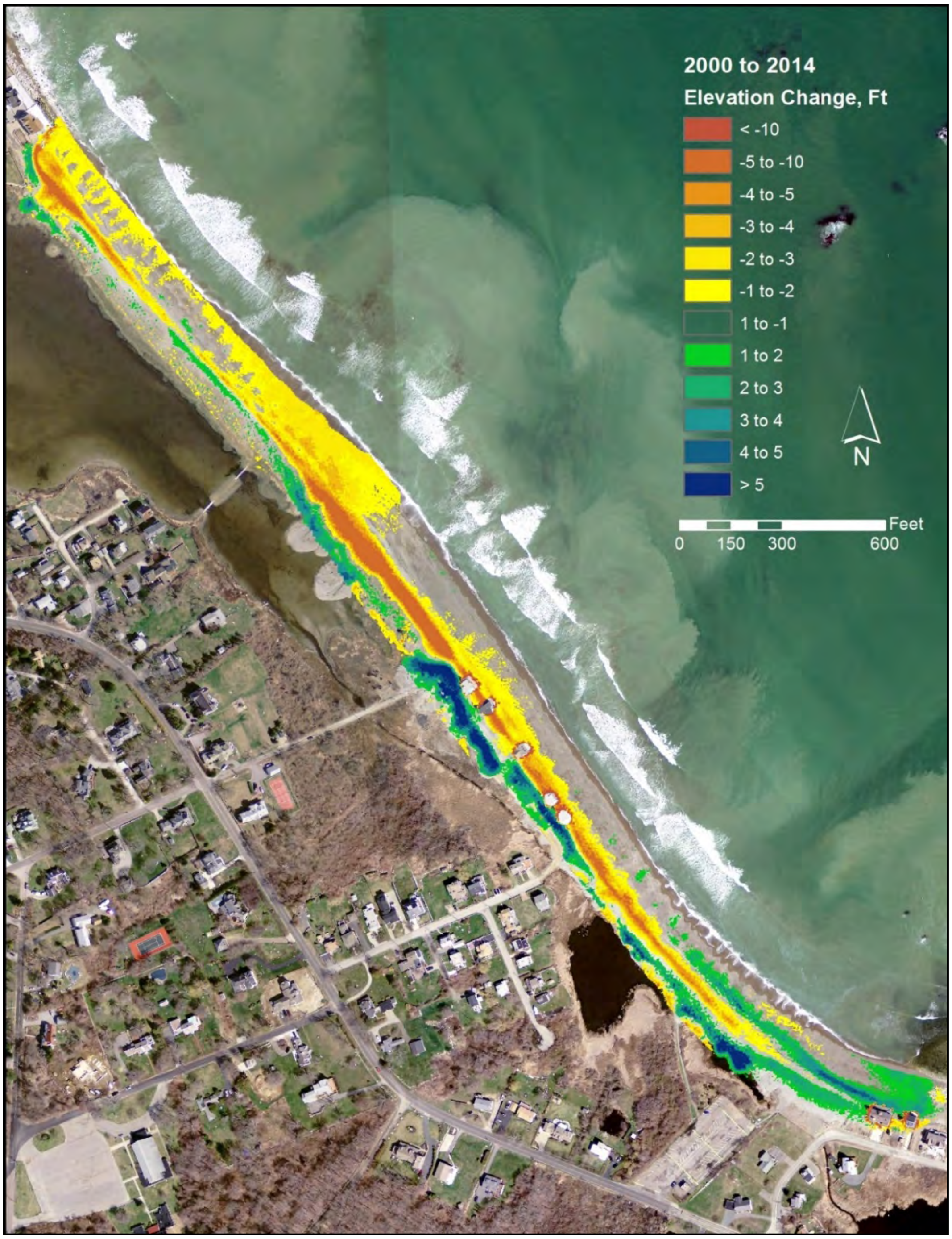


Figure 2.17 Topographic change from LiDAR data computed between 2000 and 2014 for Mann Hill and Egypt Beaches, where negative values indicate erosion and positive values indicate accretion.

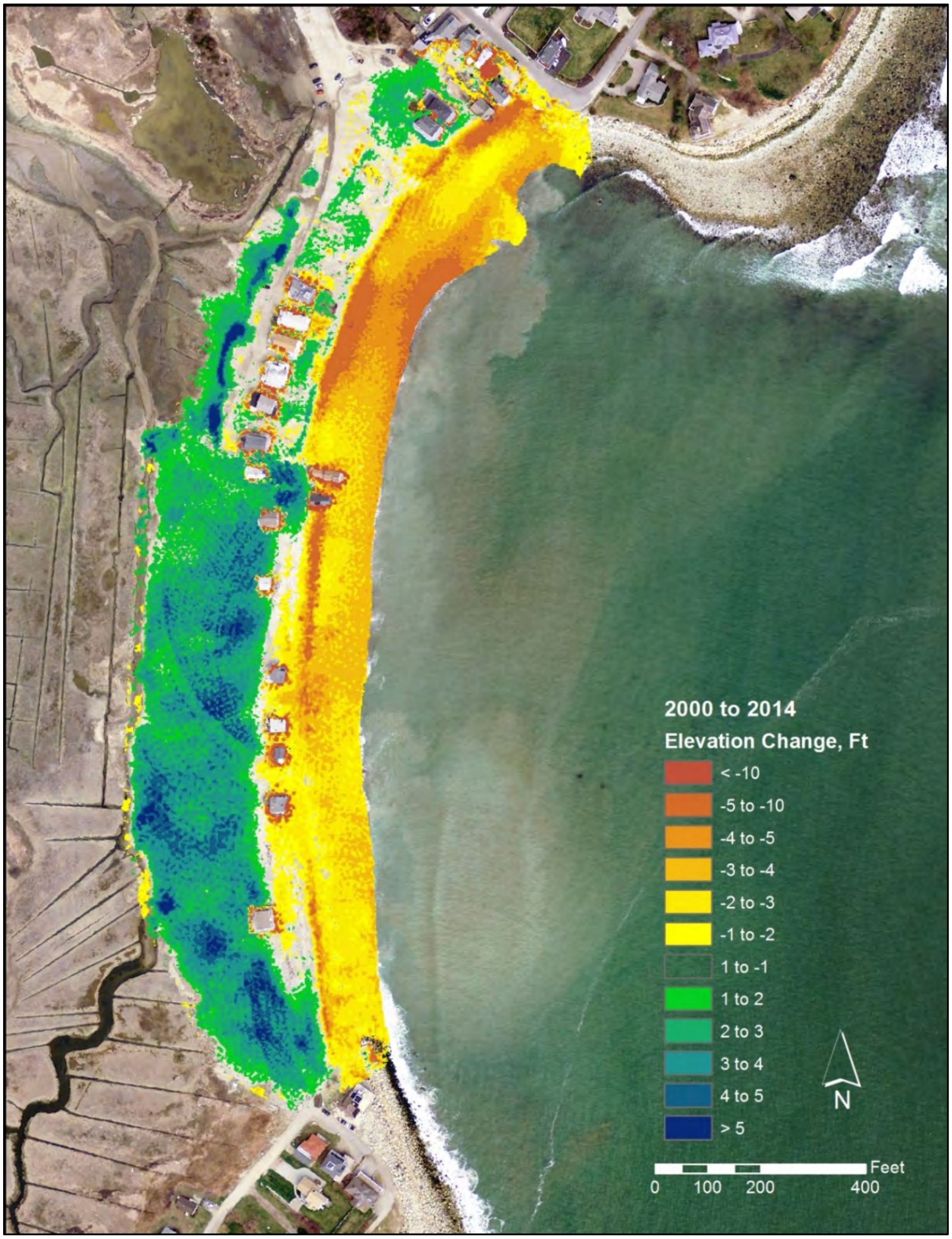


Figure 2.18 Topographic change from LiDAR data computed between 2000 and 2014 for Peggotty Beach, where negative values indicate erosion and positive values indicate accretion.

3.0 WAVE AND SEDIMENT TRANSPORT MODELING

As the main component for evaluating coastal processes, a shoreline modeling analysis was performed to assist in the development of shoreline management strategies for the Town of Scituate with a focus on areas of the coast where shore protection could be enhanced through beach and/or dune nourishment. To determine the local sediment transport pathways associated with the observed shoreline change, an in-depth scientific analysis was performed to quantitatively evaluate wave and longshore sediment transport processes that influence sand movement along the Town's shoreline.

Waves provide the driving forces governing erosion and the observed accretion/erosion along the Scituate shoreline. To predict areas of wave energy concentration and the direction of waves approaching the shoreline, a spectral wave refraction analysis was performed. This analysis computed the nearshore wave climate of the Scituate coastline based on offshore wave data. The wave modeling predicted the major effects of long-term average wave conditions on the beach areas and provided the basis for determining trends in sediment transport.

The sediment transport calculations along the Scituate shoreline depend upon a long-term wave data record. Ideally, this wave record would come from a data buoy stationed offshore of the site being modeled. In the absence of such a source of long-term data, there are few other options for retrieving wave data. For sites located on the open coast, simulated long-term wave records are available through the Wave Information Study (WIS) conducted by the U.S. Army Engineer Research and Development Center (ERDC). The WIS program has generated hindcast wave data for waves propagating from open Atlantic, through the use of computer simulations, for many sites along the U.S. coast.

In this study, a three-part wave analysis procedure was followed for the generation of wave input for the sediment transport analysis. First, a long-term wave data hindcast record was collected and processed. Second, the processed wave data were used as inputs into the two-dimensional wave transformation model SWAN. Third, output from this program was then used to generate the wave input record used in the sediment transport calculations.

Results from the spectral wave modeling formed the basis computed sediment transport rates along the modeled beach areas since wave-induced transport is a function of various parameters (e.g., wave breaking height, wave period, and wave direction). Longshore transport depends on long-term fluctuations in incident wave energy and the resulting longshore current; therefore, annual transport rates were calculated from the long-term average wave conditions. These sediment transport rates were incorporated into a shoreline change model (i.e. a "one-line" model) that could be utilized to predict future shoreline movement including the influence of beach nourishment and sand-trapping coastal engineering structures.

3.1 Wave and Wind Data

Wave conditions were generated using the data available from the WIS hindcast database from station 63053. The WIS data were used to develop offshore wave boundary conditions. The WIS station is located 11 miles northeast of Scituate Neck and has a record that spans the 33-year period between January 1980 and December 2012. Each hourly WIS time step includes parameters that describe the wave conditions (i.e., wave period, T_p; wave height, H_s; and direction, θ) and wind (direction and speed) at the station. The entire wave record from WIS hindcast is presented in Figure 3.1 as compass rose plots which show magnitude and percent occurrence as a function of direction.

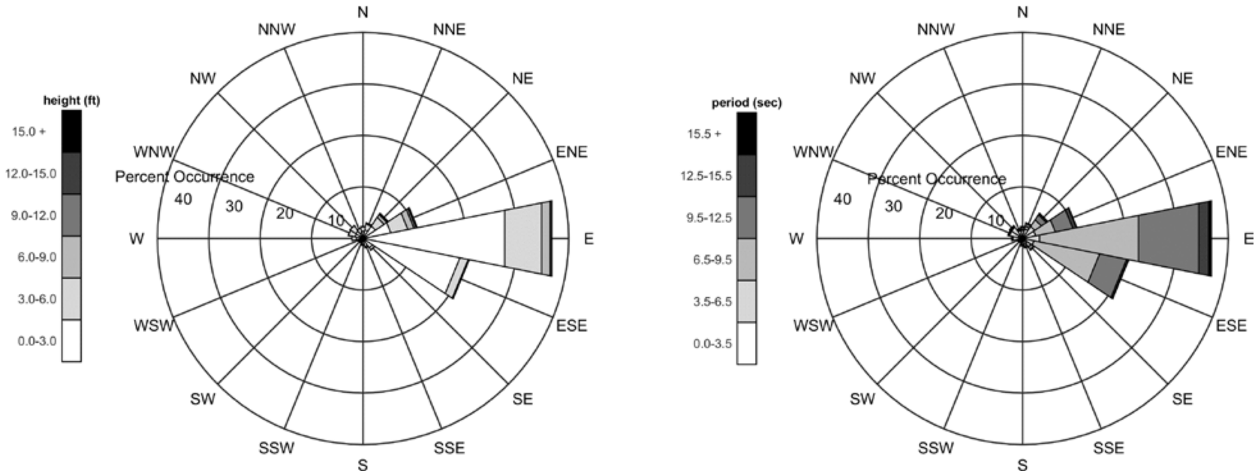


Figure 3.1 Wave height and period for hindcast data from WIS station 63053 (11 miles offshore Scituate Neck) for the 33-year period between January 1980 and December 2012. Direction indicates from where waves were traveling, relative to true north. Radial length of gray tone segments indicates percent occurrence for each range of wave heights and periods. Combined length of segments in each sector indicate percent occurrence of all waves from that direction.

For the wave data of the WIS hindcast, east is the predominant sector. Waves propagate from this direction 37.5% of the time. 75.0% of waves from this sector have a height less than 3 feet. Wave heights between 6 and 3 feet occur 19.6% of the time from the south sector. The second-most frequently occurring sector at this station is east-southeast, which occurs 21.1% of the time. From this sector, 91.2% of the waves have a height that is less than 3 feet, and 7.8% have a height between 6 and 3 feet.

To determine the offshore wave input conditions for the model, the wave parameters from each hourly wave record in the WIS were binned based on wave period and direction. The 180-degree compass sector from 330 degrees (north-northwest) to 150 degrees (south-southeast) was divided into 10 direction bins. Wave records were separated also into two bins based on their period (either less than or greater than 9.5 seconds). This method of sorting the wave data determines the average wave conditions that correspond to each binned wind case for input into the wave model.

Wave model input spectra were developed using a numerical routine that recreates a two dimensional spectrum for each individual hourly wave condition in the WIS record. The program computes the frequency and directional spread of a wave energy spectrum based on significant wave parameters (i.e., wave height, peak period, and peak direction) and wind speed (Goda, 1985). The result of this process is a wave energy spectrum that is based on parameters from the WIS record which distributes spectral energy based on wave peak frequency and wind speed.

3.2 SWAN Model Development

As waves propagate into shallower water near shore, the water depth will modify the height of the shoaling waves, and they will gradually change direction to conform to the bathymetry in that area. In order to determine how waves are transformed as they propagate toward the Scituate shoreline, the two-dimensional wave transformation program SWAN was used.

Developed at the Delft University of Technology of the Netherlands, SWAN Cycle III version 40.51AB is a steady state, spectral wave transformation model (Booij, *et al.*, 1999). Two-dimensional (frequency and direction vs. energy) spectra are used as input to the model. SWAN (an acronym for Simulating Waves Nearshore) is able to simulate wave refraction and shoaling induced by changes in bathymetry and by wave interactions with currents. The model includes a wave breaking model based on water depth and wave steepness. Model output includes significant wave height H_s, peak period T_p, and wave direction θ.

SWAN is a flexible and efficient program based on the wave action balance equation that can quickly solve wave conditions in a two-dimensional domain using the iterative Gauss-Seidel technique. For this study, the model was implemented using a steady state finite-difference scheme, on a regular Cartesian grid (computational cell dimensions in the *x* and *y* directions are equal), though other options are available (including a finite difference formulation using an unstructured mesh). A great advantage of the iterative technique employed in SWAN it that it can compute spectral wave components for the full 360-degree compass circle.

In addition to the spectral wave boundary conditions specified for each of the wave cases, bathymetry and several model parameters must be specified. The model parameters describe the extent and resolution of the computational mesh (separate from the bathymetry grid) including nested grids, the directional and frequency resolution of the wave spectrum, and wave physics (e.g., breaking, wave-wave interactions).

The SWAN model developed for the study shoreline used a coarse grid with 328 foot (100 meter) spacing for the region including the offshore region between the Scituate shoreline and the location of the WIS hindcast station (Figure 3.2) and a three fine mesh grids with 16.4 foot (5 meter) spacing that covers the study area in high resolution (Figure 3.3, Figure 3.4, and Figure 3.5). The National Ocean Service Geophysical Data System (GEODAS) database (NOS, 1998) was the main source of bathymetric data used to create the coarse grid. A 2010 LiDAR survey (JALBTCX, 2010) that covered the nearshore area of the Scituate shoreline was the primary source of bathymetry data used to develop the fine-scale nearshore wave grid. LiDAR is a system for collecting ultra-dense-coverage bathymetry and topography data using a laser system mounted to an airplane. This LiDAR survey measured bottom elevations as far as 3,700 feet offshore the study shoreline, and includes measurements deeper than -75 feet NAVD88. All bathymetry data were transformed to the NAVD88 datum.

The coarse grid was used to propagate the offshore wave conditions, developed from the analysis of the WIS hindcast record, to the nearshore. The fine mesh grids were nested within the domain of the coarse grid. This means that the input wave conditions along the open boundary of the fine grids are spatially varying output from the coarse grid (e.g., the 5 m grid open boundary waves are output from the 100 m grid). Therefore, the fine grid results are truly nested within the coarse grid simulations. This technique allows fine resolution of the model grid in areas where it is need, but also allows larger grid spacing where the fine resolution is not needed, and as a result, minimizes computational requirements, without sacrificing accuracy. Grid parameters of the coarse and fine grids are summarized in Figure 3.1.

The wave spectrum resolution specified for the model runs of all model meshes include the full 360-degree compass circle divided into 72, five-degree segments, with 40 discrete frequencies, between 0.06 and 1.00 Hz (corresponding to periods of between 16.7 and 1.0 seconds). The wave input parameters for the long-term conditions are summarized in Table 3.2.

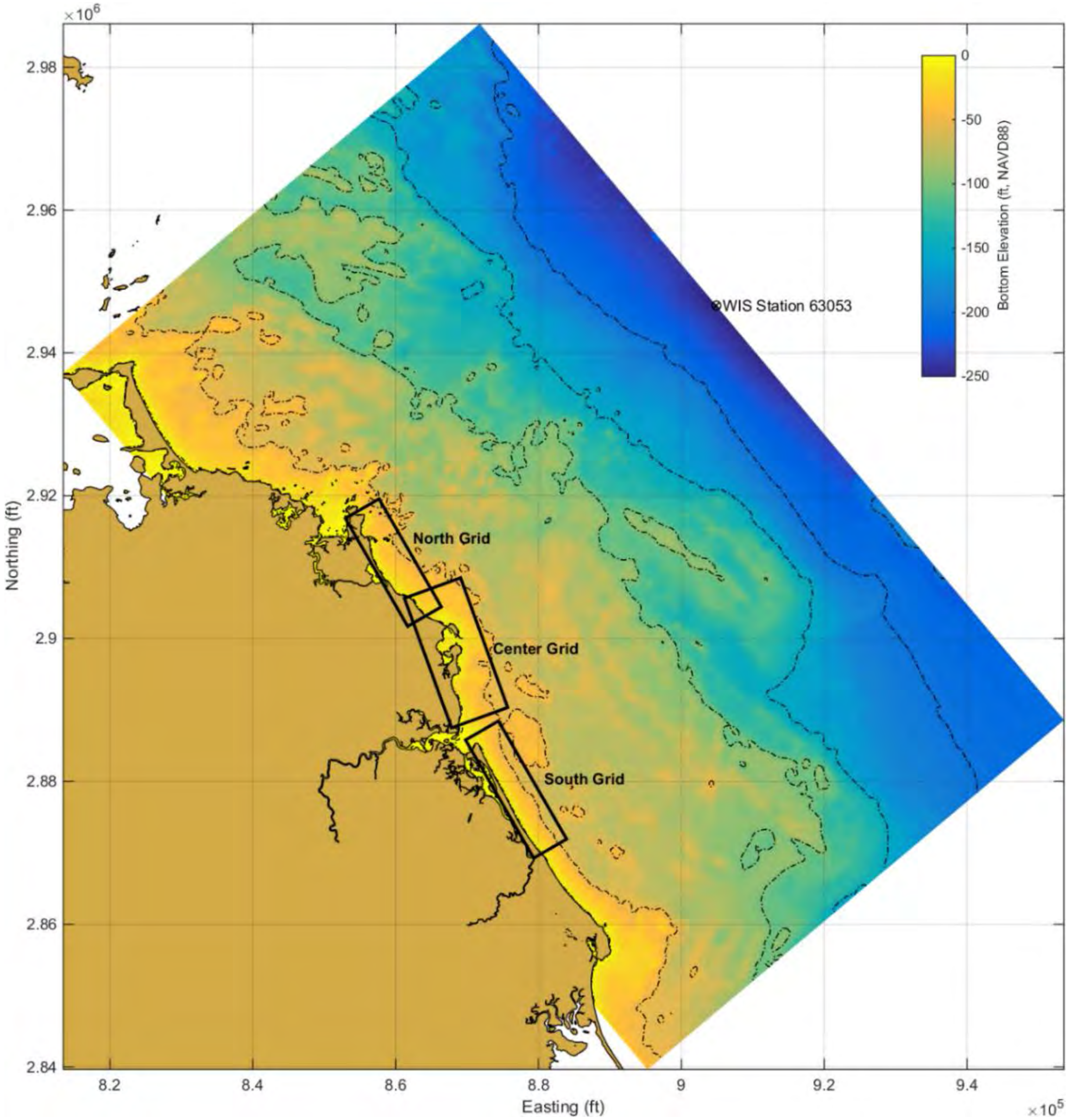


Figure 3.2 Map showing wave model grid limits and bathymetry of both the coarse model grid the offshore area between the WIS wave station and the Massachusetts coast. 50-foot bathymetric contour lines are also shown.

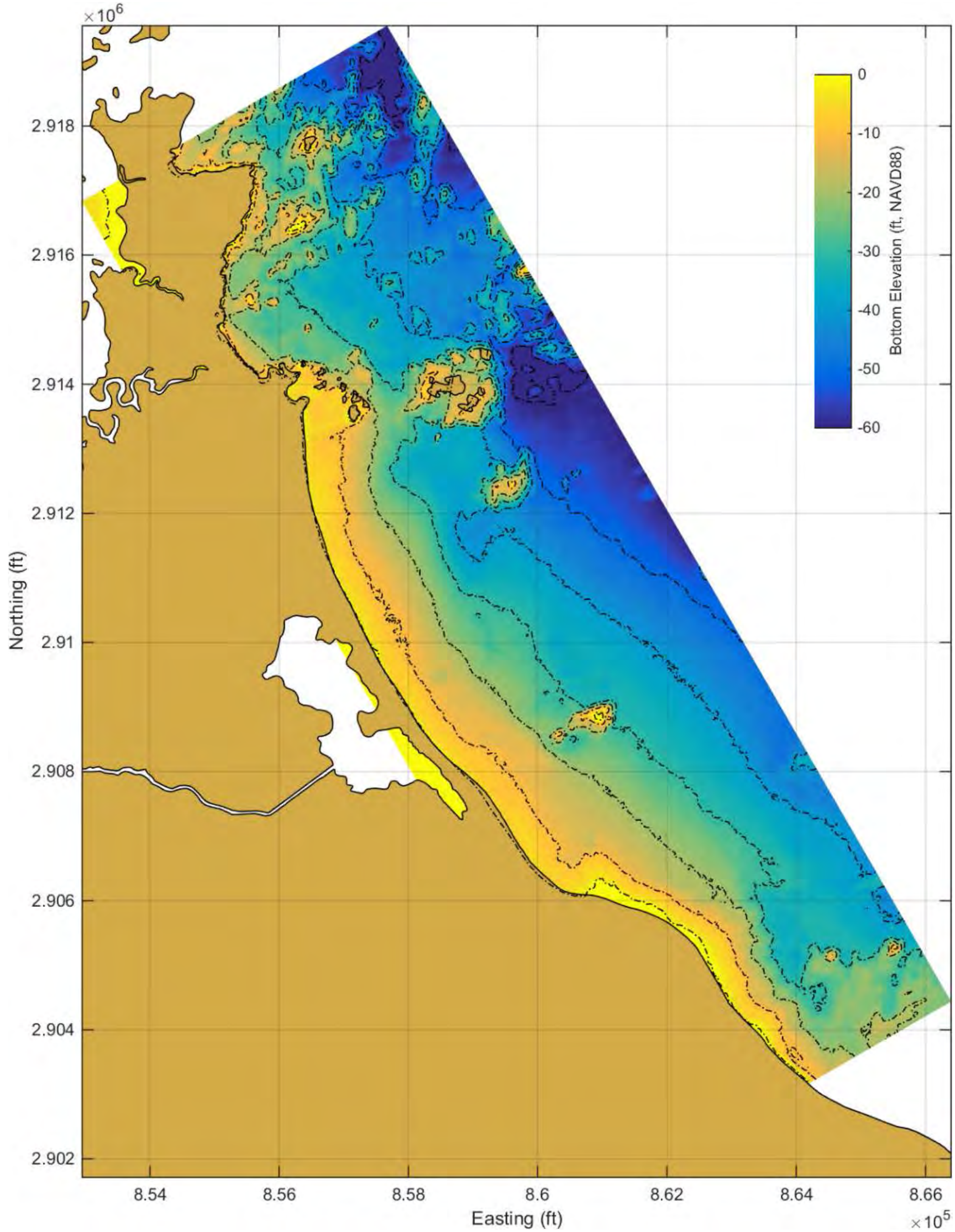


Figure 3.3 Map showing wave model grid limits and bathymetry of the fine model grid of the Scituate shoreline from Scituate Neck to Egypt Beach. 10-foot bathymetric contour lines are also indicated.

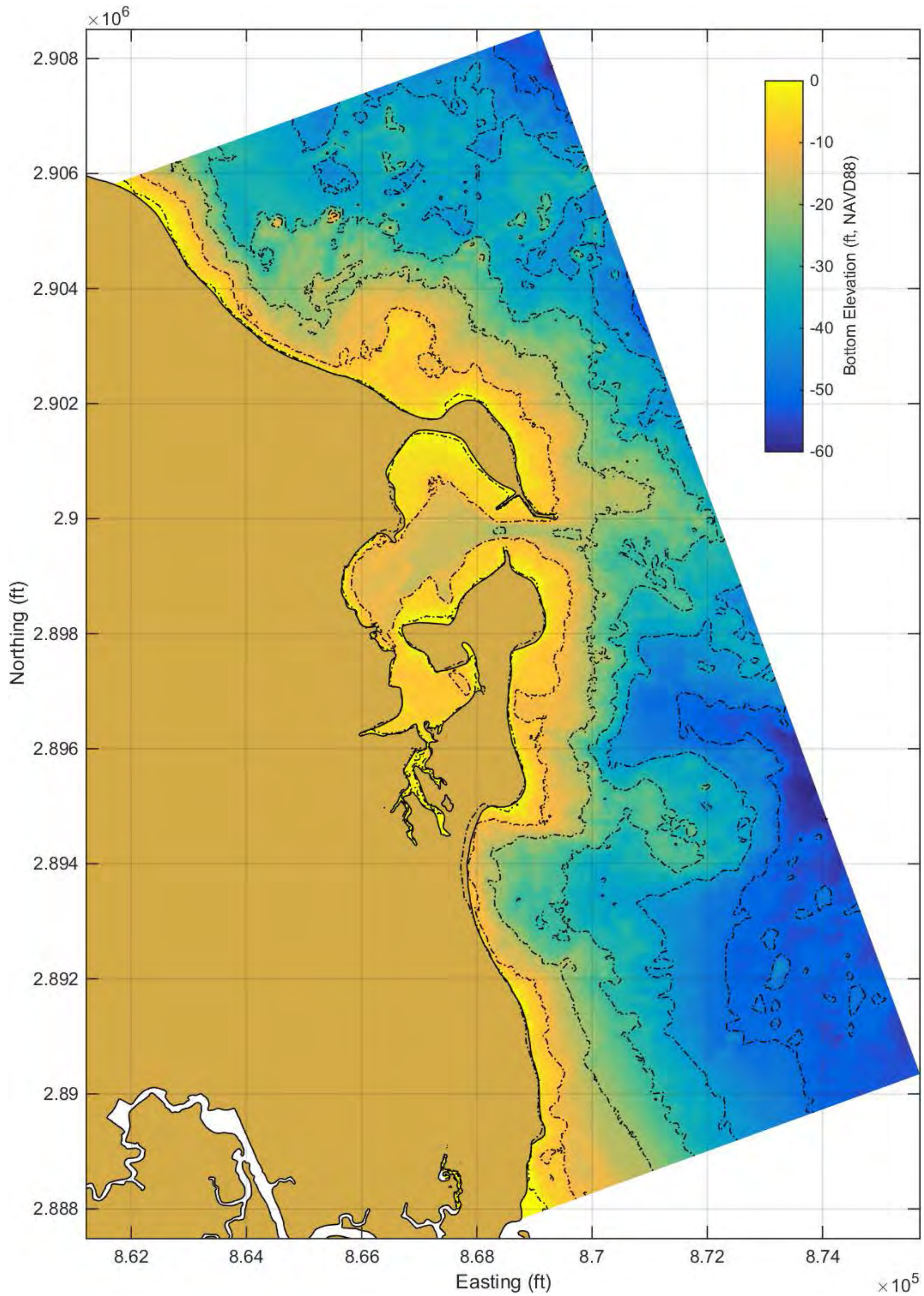


Figure 3.4 Map showing wave model grid limits and bathymetry of the fine model grid of the Scituate shoreline from Egypt Beach to Third Cliff. 10-foot bathymetric contour lines are also indicated.

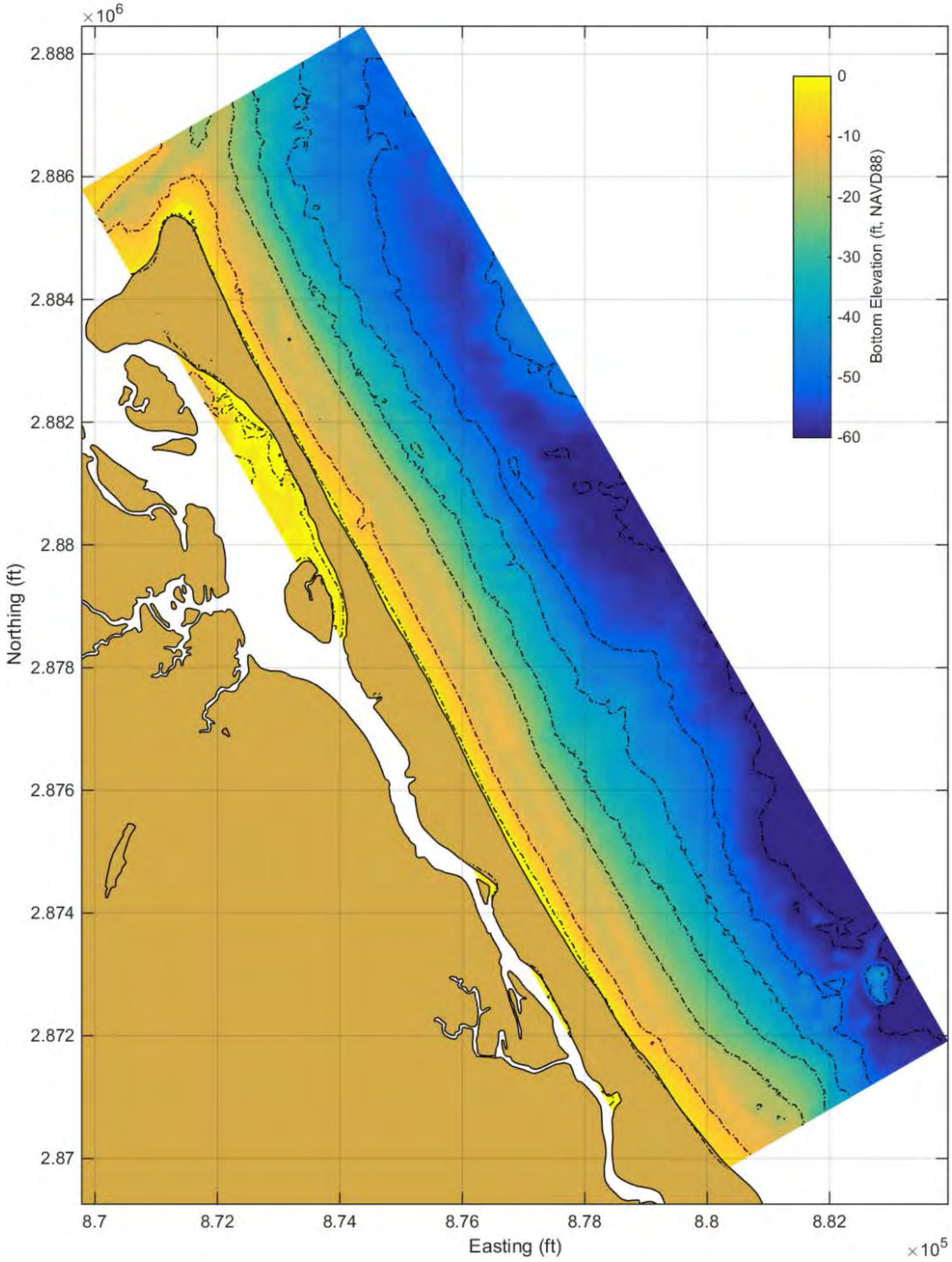


Figure 3.5 Map showing wave model grid limits and bathymetry of the fine model grid of the Scituate shoreline from Fourth Cliff to the southern town limits. 10-foot bathymetric contour lines are also indicated.

Table 3.1 SWAN grid parameters.

Grid	Coarse	Fine – North	Fine – Center	Fine – South
Cell Spacing	100 m (328 ft)	5 m (16.4 ft)	5 m (16.4 ft)	5 m (16.4 ft)
Computational Cells	90,016	353,248	600,780	377,136
Distance (alongshore)	24.1 miles	3.3 miles	3.7 miles	3.6 miles
Distance (cross-shore)	14.4 miles	1.0 miles	1.6 miles	1.0 miles
Orientation (compass)	140 degrees	150 degrees	160 degrees	150 degrees

Table 3.2 Wave model input parameters, listed by compass sector and wave period bin. Listed offshore wave parameters include peak direction θ_o, wave period T_o and wave height $H_{s,o}$. Angles are given in the meteorological convention.

Period Band	Case	Direction Sector	Percent Occurrence	θ_o (degrees)	T_o (seconds)	$H_{s,o}$ (feet)
Less than 9.5 seconds	1	339 to 357	1.44	349	4.2	3.3
	2	357 to 15	1.38	9	4.8	3.3
	3	15 to 33	1.63	24	5.6	3.5
	4	33 to 51	2.66	55	6.7	4.2
	5	51 to 69	3.91	64	8.3	4.8
	6	69 to 87	5.52	84	8.3	4.1
	7	87 to 105	16.60	99	8.3	2.6
	8	105 to 123	20.23	109	8.3	1.8
	9	123 to 141	2.31	129	8.3	2.8
	10	141 to 159	1.38	149	8.3	2.5
Greater than 9.5 seconds	11	339 to 357	0.29	349	11.1	3.0
	12	357 to 15	0.41	9	11.1	3.1
	13	15 to 33	0.48	24	11.1	3.3
	14	33 to 51	0.74	44	11.1	3.9
	15	51 to 69	1.58	64	11.1	5.2
	16	69 to 87	3.82	84	11.1	5.5
	17	87 to 105	11.21	94	11.1	3.8
	18	105 to 123	8.49	109	11.1	2.4
	19	123 to 141	0.94	129	11.1	2.3
	20	141 to 159	0.50	149	11.1	2.3

Examples of wave model output are presented in Figure 3.6 and Figure 3.7 for the coarse grid and center fine nearshore grid for wave case 4 (Table 3.2). In these plots the color contours indicate wave height and vectors are used to indicate the direction of wave propagation. Offshore waves with heights of 4.2 feet approach the Scituate shoreline from the northeast in the coarse grid. Waves are refracted as they approach the shoreline. In the results of the center fine nearshore grid, the waves are shown to diffract around the offshore feature at Sand Hills Beach, creating a relatively calm wave environment in the lee of the offshore shoal. Similarly, waves diffracting around the southern end of Second Cliff create a more quiescent wave environment at the north end of Peggotty Beach. Additional selected model results are presented in Appendix B.

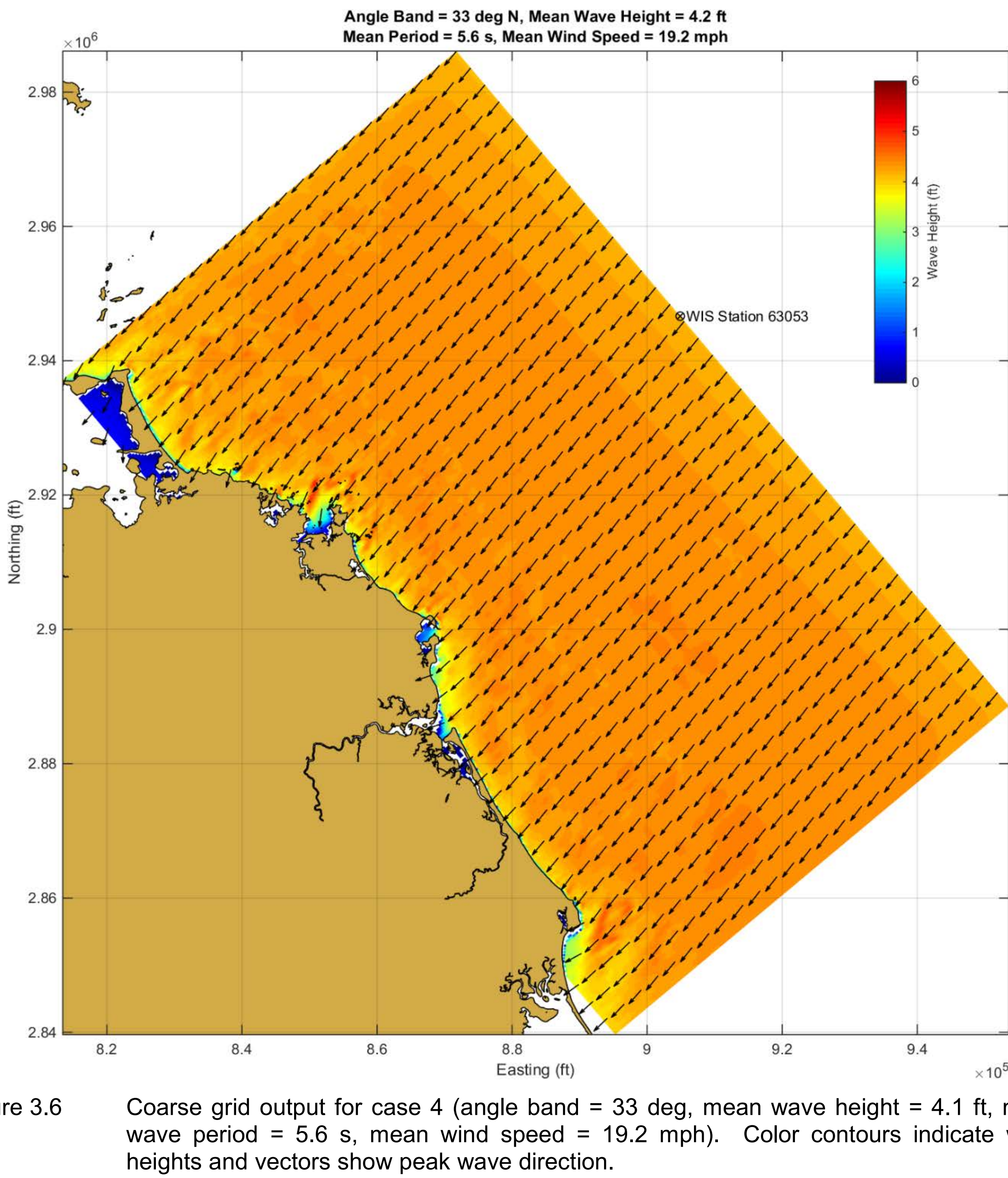


Figure 3.6 Coarse grid output for case 4 (angle band = 33 deg, mean wave height = 4.1 ft, mean wave period = 5.6 s, mean wind speed = 19.2 mph). Color contours indicate wave heights and vectors show peak wave direction.

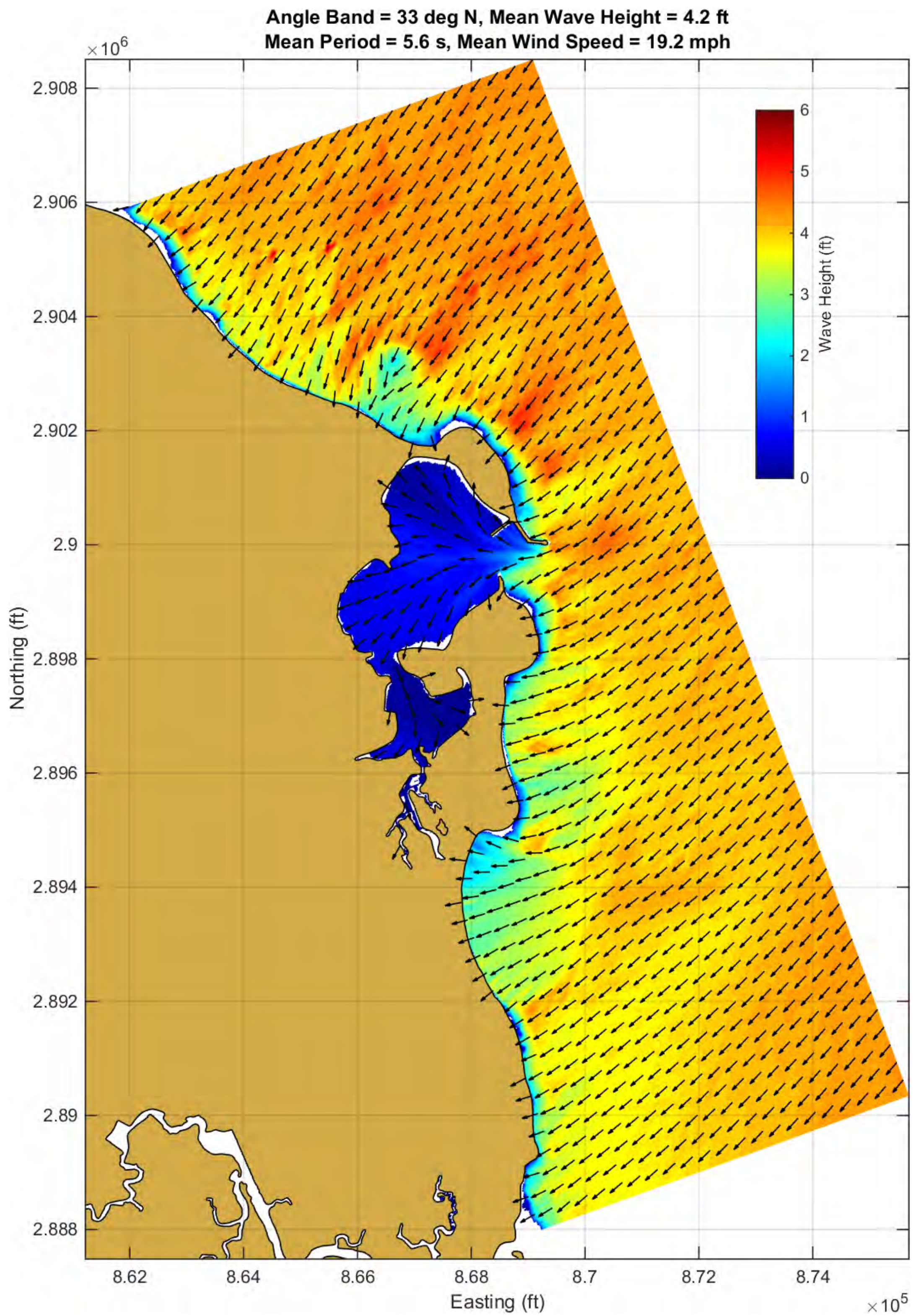


Figure 3.7 Fine grid output for case 4 (angle band = 33 deg, mean wave height = 4.1 ft, mean wave period = 5.6 s, mean wind speed = 19.2 mph). Color contours indicate wave heights and vectors show peak wave direction.

Extreme events (10-, 50-, and 100-year storms) were also modeled using SWAN to obtain design wave conditions for coastal engineering structures along the Scituate shoreline. For example, the 100-year storm conditions were used as the basis of the overtopping assessment. Figure 3.8 shows the significant wave heights obtained from WIS hindcast station 63053 extreme analysis. Peak wave period for each event storm was determined by relating wave periods to wave heights via combinations of greatest frequency, as shown in Figure 3.9. The 10%, 2% and 1% annual chance (10-, 50- and 100-year return period) still water elevation (SWL) was based on the Plymouth County Flood Insurance Study (FEMA, 2012). Wave input conditions for the extreme events are summarized in Table 3.3. In addition, evaluation of coastal engineering structures within the evaluation of approaches also included an anticipated sea level rise of 2 feet over the next 50 years, as described in the next section.

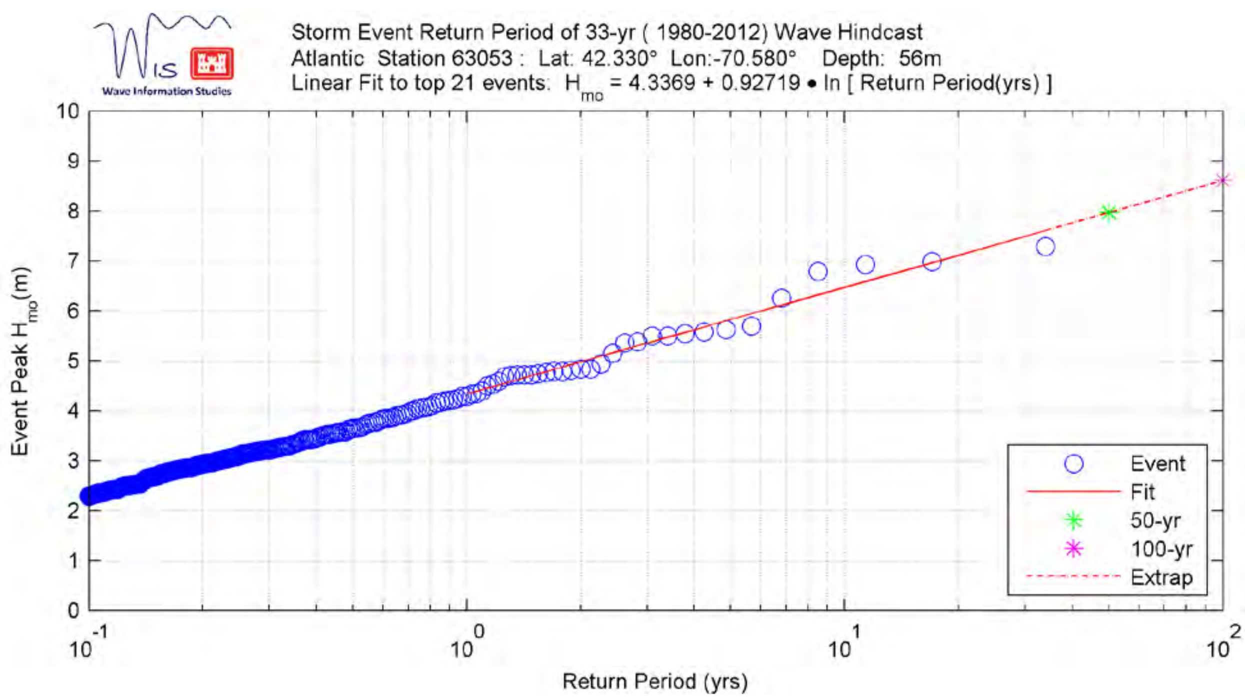


Figure 3.8 WIS hindcast station 63053 extreme analysis plot for significant wave height.

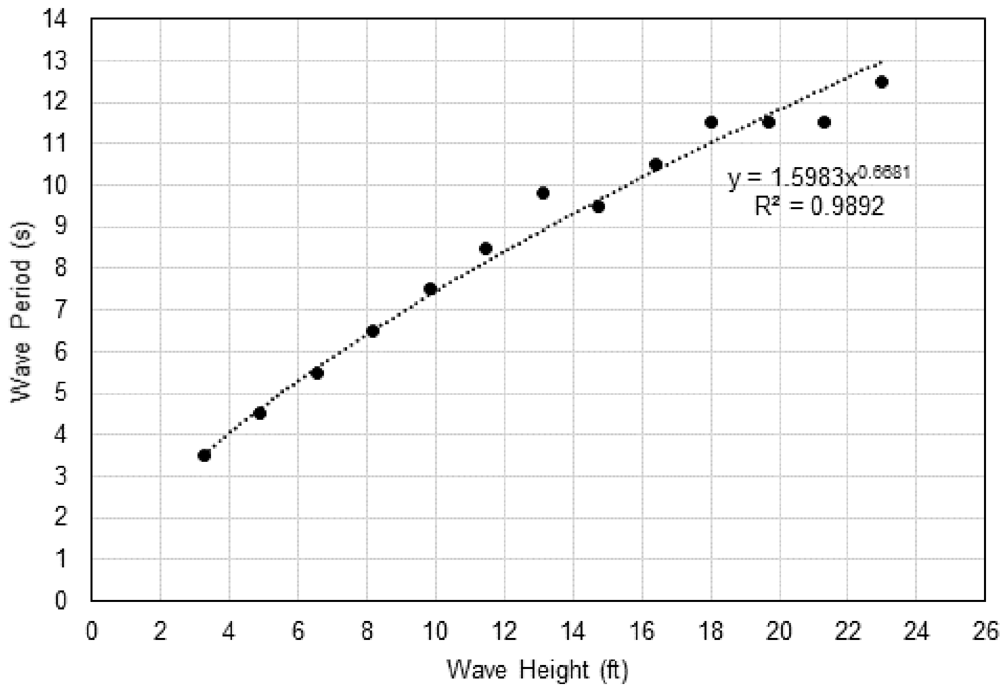


Figure 3.9 Wave period and wave height relationship for WIS hindcast station 63053.

Table 3.3 Wave characteristic and FEMA still water elevations (SWL) for 10-, 50- and 100-year return period events.

Return Period	Wind Speed (mph)	Significant Wave Height – Offshore (ft)	Significant Wave Height – Site (ft)	Peak Wave Period (s)	FEMA SWL (ft, NAVD88)
10-year	52.3	21.3	7.8	12.3	8.3
50-year	55.3	26.2	7.8	14.2	9.1
100-year	57.0	28.2	8.8	14.9	9.5

3.3 Sea Level Rise Considerations

Separate from the daily rise and fall of the tide, the average elevation of the ocean changes over time with respect to the land. This average position is called relative sea level and different geologic and atmospheric processes contribute to changes in relative sea level. Some of the causes include glacial ice melt, thermal expansion of the ocean as the global temperature increases, and the rising or sinking of the earth's crust itself. While the specific causes of relative sea level change are the topic of much scientific and political debate, historical evidence indicates that over the past 90+ years, the relative sea level in Boston, Massachusetts has been rising generally in a linear fashion (see Figure 3.10). Utilizing monthly mean sea level data, the long-term average relative sea level rise in Boston has been 2.79 mm per year or 0.92 feet per century.

The Massachusetts Office of Coastal Zone Management (MCZM) also published their own report in 2013 regarding future sea level rise projections along the Massachusetts coast based upon much of the information developed by NOAA (Parris, *et al*, 2012). These projections utilized estimates for the historical linear trend, an “intermediate low” scenario, an “intermediate high” scenario, and a “high” scenario as shown in Figure 3.11. For the evaluation of shore protection measures in this report, it is anticipated that a 50-year design life for new and/or reconstructed coastal engineering structures is appropriate. Utilizing the relatively conservative values associated with the “intermediate high” relative sea level rise projection for the region, the evaluation for future conditions assumed a 2-foot increase in relative sea level over the next 50 years.

It should be noted that simply increasing structure elevation by 2 feet might not address increased wave overtopping predictions over the next 50 years. Therefore, coastal engineering structure assessment also considered expansion of armor stone revetments fronting the structures to ensure appropriate designs under future sea level and storm wave conditions.

For non-structural coastal engineering measures (e.g. beach and/or dune nourishment), the design life generally is on the order of 5 to 15 years; therefore, designs could be readjusted as sea levels increased in the future. These design modifications would become part of the ongoing maintenance requirement for the project and there would be no need to incorporate sea level rise directly into the initial design.

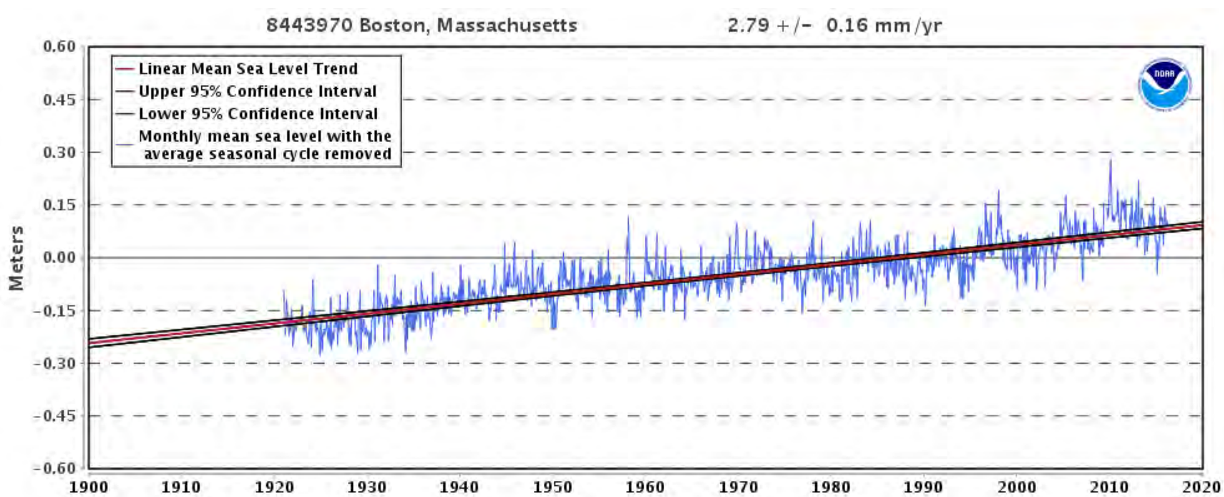


Figure 3.10 Long-term mean sea level data for NOAA Boston tide gauge station with linear trend and confidence interval.

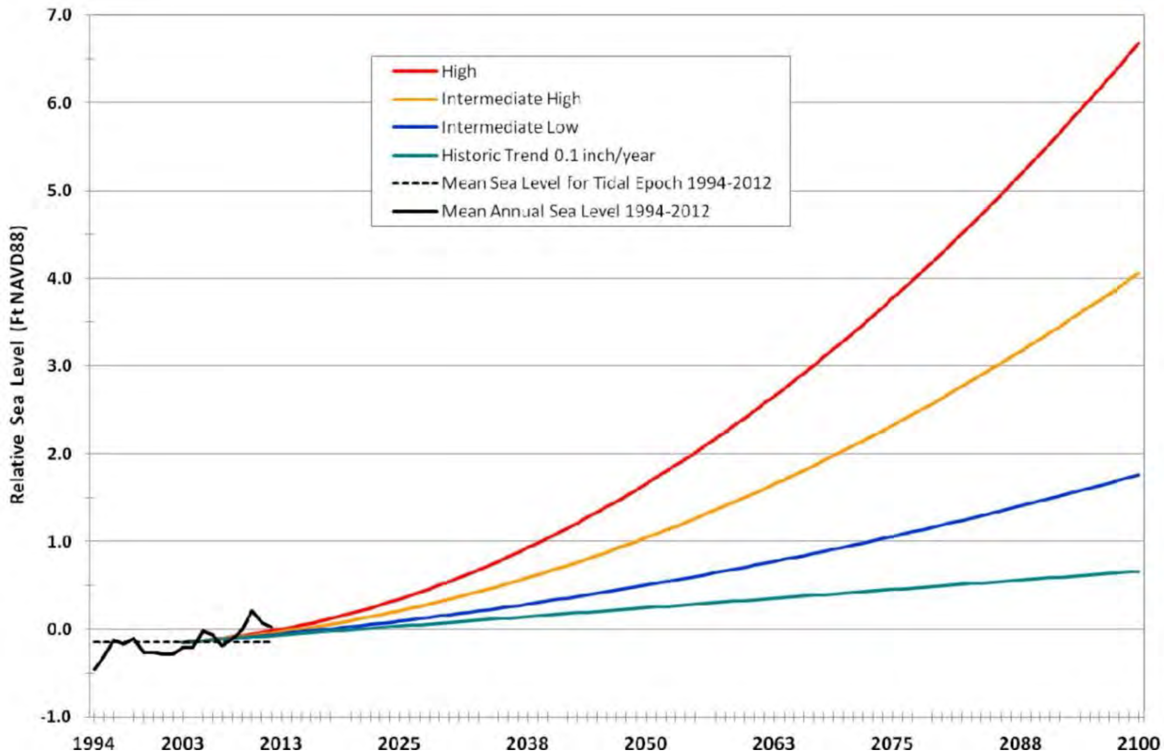


Figure 3.11 Relative sea level rise scenarios estimates (in feet NAVD88) for Boston, MA. Global scenarios from were adjusted to account for local vertical land movement with 2003 as the beginning year of analysis (*figure credit: MCZM, 2013*).

3.4 Sediment Transport and Shoreline Evolution Modeling

As integral part to the coastal processes that are at work to shape the shoreline of Scituate, an evaluation of sediment transport along the shoreline is necessary. Results from the spectral wave modeling formed the basis for the computed sediment transport rates along the modeled stretch of beach since wave-induced transport is a function of various parameters (e.g., wave breaking height, wave period, and wave direction). Longshore transport depends on long-term fluctuations in incident wave energy and the resulting longshore current; therefore, annual transport rates were calculated from the long-term average wave conditions described in the previous section.

Various types of models may be utilized for studying the transport of beach sediment and the consequent shoreline change resulting from waves. The technical sophistication of models ranges from simplified mathematical solutions of equations governing broad physical principles (*analytical* models) to highly complex computer models that simulate natural phenomena contributing to coastal erosion. The most complex computer models (three-dimensional models) require the most detailed input data. The model best suited for studying the Scituate shoreline falls in the middle of this technical range. While simplified analytical models ignore many of the important principles governing shoreline change along this beach, the most complex models attempt to simulate the inter-relation of complex physical phenomena not fully understood by scientists/engineers. Thus, a blend of advanced scientific principles with practical engineering assumptions are used in the development of a useful shoreline change model for Scituate.

Shoreline evolution modeling was performed using a “one-line” longshore transport computer code. So called “one-line” models simulate the evolution of a shoreline through time, at one specific contour level, e.g. the mean water level, based on the assumption that the nearshore bathymetry (to the depth of closure used to define the active extent of the beach profile) can be adequately represented by straight and parallel contours. The formulation of shoreline models is very well documented in the literature, e.g., Dean and Dalrymple (2001), Hansen and Kraus (1989).

3.4.1 Formulation of Sediment Transport and Shoreline Model

The sediment transport equation employed for the alongshore analyses is based on the work of the U.S. Army Corps of Engineers (CERC, 1984). In general, the longshore sediment transport rate is proportional to the longshore wave energy flux at the breaker line, which is dependent on wave height and direction. Since the transport equation was calibrated in sediment-rich environments, it typically over-predicts sediment transport rates. However, it provides a useful technique for comparing erosion/accretion trends along the shoreline of interest.

In the method described by the Army Corps, the volumetric longshore transport, Q, past a point on a shoreline is computed using the relationship:

$$Q = \frac{I}{(s-1)\rho g a'}$$

where I is the immersed weight longshore sediment transport rate, s is the specific gravity of the sediment, a' is the void ratio of the sediment, and ρ is the density of seawater.

One method to compute the immersed weight longshore sediment transport, I, is based on the so-called “CERC formula” as,

$$I = KP_{\ell s}$$

where K is a dimensionless coefficient and P_{ls} is the longshore energy flux factor computed using the following relationship:

$$P_{\ell s} = \frac{\rho g^{3/2}}{16\sqrt{\gamma}} H_{sb}^{5/2} \sin 2\alpha_b$$

where H_{sb} is the significant wave height at breaking, γ is the coefficient for the inception of wave breaking ($\gamma = H_b/h_b$), and α_b is the breaking wave angle, as described by Bodge and Krause (1991). A value of K=0.4 is normally used, which is appropriate for significant wave heights (computed by SWAN), rather than the more familiar value K=0.77, which is used with RMS wave height.

The actual method used in this study to compute immersed weight longshore sediment transport was described by Kamphuis (1990). This method is basically a modification to the original CERC formula, and adds a dependency on the median grain diameter of the beach sediment, and also the surf similarity parameter, ξ_b, which is expressed as

$$\xi_b = \frac{m}{\left(H_b / L_0\right)^{0.5}}$$

where m is the bottom slope and L_o is the incident wave period. The complete expression of Kamphuis is written as:

$$I = K^{*} \rho g \left(\frac{g}{2\pi} \right)^{0.75} \xi_b T^{0.5} \left(m d_{50} \right)^{-0.25} H_s^{2.5} \sin^{0.6}\left(2\theta_b \right)$$

where the coefficient K^* = 0.0013 (Bodge and Krause, 1991). Therefore the instantaneous transport rate for sandy sediment, Q_s, is:

$$Q_s = \frac{K^{*}}{(s-1)a'} \left(\frac{g}{2\pi} \right)^{0.75} \xi_b T^{0.5} \left(m d_{50} \right)^{-0.25} H_s^{2.5} \sin^{0.6}\left(2\theta_b \right)$$

where Q_s has units of cubic meters per second.

Cobble transport was calculated using the method of Van Wellen, *et al*. (2000). The transport equation proposed by Van Wellen, *et al*., is specifically formulated for cobble (known as "shingle" in the United Kingdom) beaches, and includes sediment movement threshold term as well as transport in the swash zone, both of which are important for beaches of this type, as noted by the paper's authors. By this method, the instantaneous transport rate Q_c in cubic meters per second is computed as

$$Q_c = 1.34 \frac{(1+e)}{(\rho_s - \rho)} H_s^{2.49} T^{1.29} m^{0.88} d_{50}^{-0.62} \sin^{1.81}(2\theta_b)$$

where e is the void ratio and ρ_s is the density of the cobble. The value of transport potential derived using these methods represents the maximum possible at a particular location, given a rich sediment supply, and no structures (e.g., seawalls and groins) to modify the movement of sediment along the shoreline.

Using these expressions of sediment transport potential, a computer model was developed which simulates the conditions along actual shorelines, where coastal engineering structures impact actual sediment transport rates. The goal of the shoreline change modeling is first to predict measured shoreline change and long-shore sediment transport rates, and subsequently use the model to evaluate beach management approaches for the Scituate shoreline. For this application, shoreline modeling was performed using a 33 feet (10 meter) grid spacing, which is one-half the resolution (two times the spacing) that was used for the wave modeling. A 2001 shoreline, determined from the aerial photograph was used as the input shoreline. The model was calibrated using the seven year period between 2001 and 2008. This period was selected based on the availability of aerial photography and because it is sufficiently long to simulate long term trends in shoreline movement.

The model code incorporates the ability to simulate the effects of seawalls (and coastal dikes), groins and jetties on shoreline evolution. It also includes the ability to add sources and sinks of sediment along the shoreline. The model is formulated using a simple explicit upwind differencing scheme (e.g., Dean and Dalrymple, 2001), which computes change in shoreline position based on the computed gradient of sediment transport. The relationship between shoreline change and the gradient of sediment transport potential can be most simply expressed as:

$$\frac{\partial y}{\partial t} + \left(\frac{\partial Q}{\partial x} + q \right) \Big/ \left(D_B + D_c \right) = 0$$

where Q is sediment transport at a particular shoreline transect, x is alongshore width of a computational cell, y is the cross-shore position of the shoreline, t is time, q is a source tem, D_B

is the berm elevation of the beach, and D_C is the depth of closure. Values of sediment transport are computed at evenly spaced grid cells, with positions that correspond to alongshore grid cells of the wave transformation model grid. Groins and seawalls, which act to hinder sediment transport and prevent shoreline erosion, can be included anywhere within the model domain. The completed models were run and calibrated based on the comparison of the computed shoreline position and annualized change rates to measure shoreline data.

3.4.2 Model Calibration Example

The sediment transport and shoreline change model for Peggotty Beach is presented below. Shoreline change data representing past erosion and accretion patterns are essential to calibrate the model. During calibration, input coefficients are adjusted so that the modeled shoreline reasonably predicts the measured shoreline at the end of the calibration time period. Inputs into the sediment transport potential calculations include beach slope and sediment grain size.

Calibration results for the Peggotty Beach model are shown in Figure 3.12. The sediment grain size was estimated to be 0.7 mm (coarse sand). The comparison of modeled and measured 2008 shorelines shows generally good agreement. The shoreline is erosional along its entire length. The modeled beach loss rate is 6.6 feet per year while the measured loss rate is 7.0 feet per year.

The computed annual potential net sediment transport for existing conditions is presented in Figure 3.13 for the modeled segment of shoreline. The sediment transport vectors represent the net direction and magnitude of transport along the coast. The potential sediment transport rate assumes the sand supply is unlimited. Sediment transport at the north end of the beach is relatively lower due to the wave sheltering effects of Second Cliff.

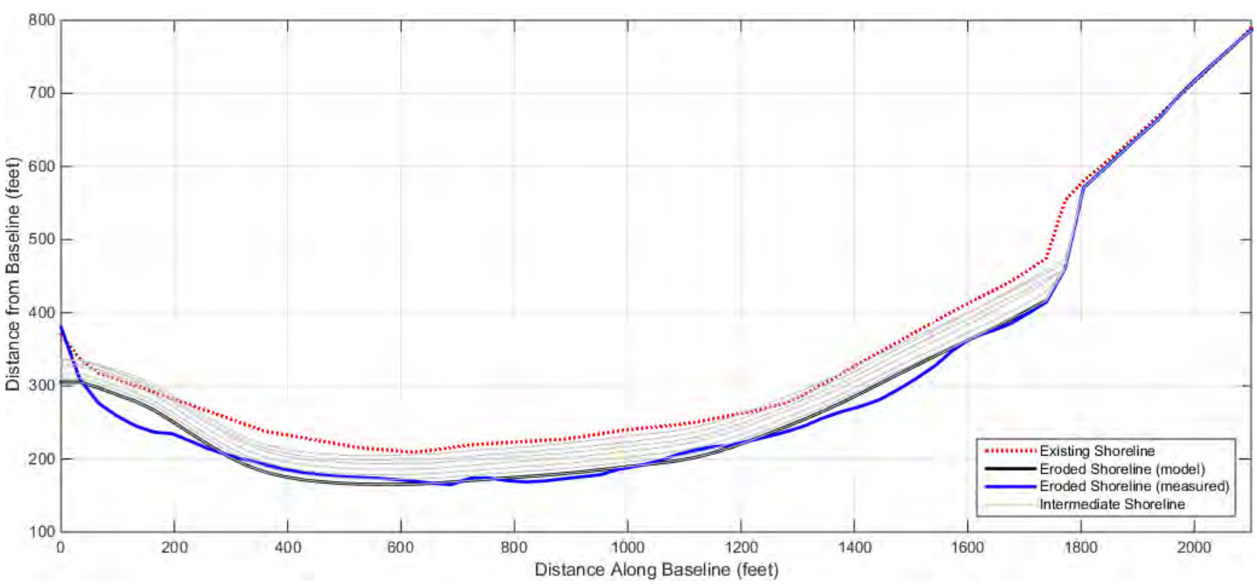


Figure 3.12 Comparison of measured and modeled shorelines along Peggotty Beach.

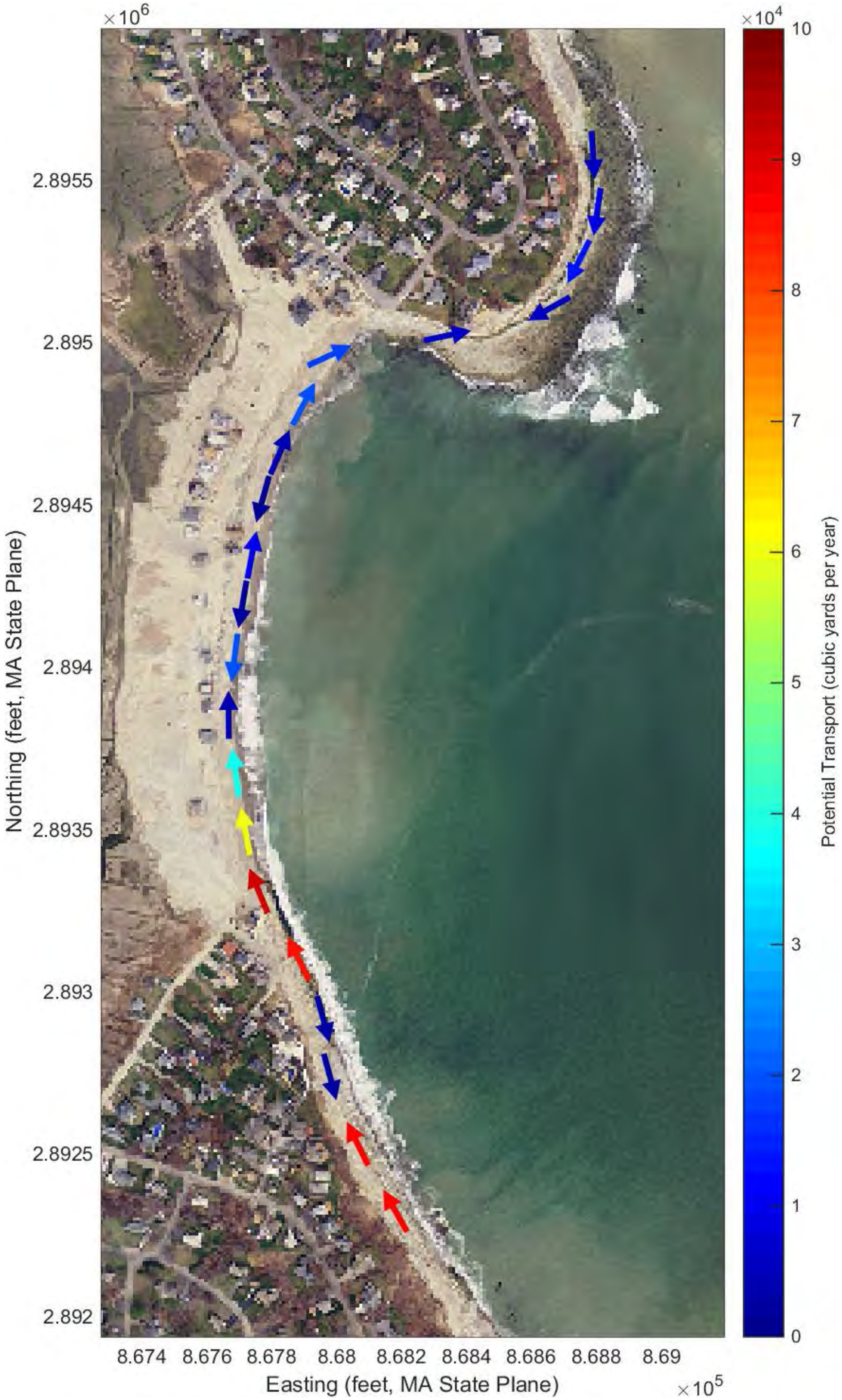


Figure 3.13 Modeled annual potential sediment transport at Peggotty Beach. The sediment transport vectors represent the net direction and magnitude of transport along the coast.

Similar model calibration procedures were followed for the remaining portions of the Scituate shoreline where beach sediments are available for transport (i.e. Minot and North Scituate Beaches, Oceanside, and Humarock Beach). The model-predicted net sediment transport rates Minot, North Scituate, and Surfside Beaches are shown in Figure 3.14. The sediment transport analysis for this region was calibrated based upon previous work associated with North Scituate Beach (Applied Coastal, 2015). In general, the net sediment transport direction along this shoreline is from north to south. The potential sediment transport rate assumes the sand supply is unlimited; however, it is observed that along the seawall the actual transport rate is nearly zero due to the lack of available beach material. A reversal in the net sediment transport direction can be observed near the northern limit of North Scituate Beach, as wave diffraction around the offshore bedrock outcrops (e.g. Well Rock and Bar Rock) causes redirection of the waves approaching the beach. Along Minot Beach, the sediment supply again is quite limited due to the existence of both the seawall and the offshore armor stone dike. Similar to North Scituate Beach, Minot Beach also is influenced by wave diffraction associated with the same offshore bedrock outcrops, but in this case, the waves are redirected to cause a net southerly drift. In contrast, sediment transport along the northern portion of Minot Beach is directed to the north. Due to the lack of shoreline change data available as a result of the large-scale coastal armoring of this shoreline, it was not possible to develop a calibrated sediment transport model for the Minot Beach section.

For Oceanside Drive, the long-term influence of the seawall on the beach system does not allow calibration of the longshore sediment transport model, as the long-term horizontal shoreline change for this stretch of shoreline is near zero. In general, the beach along this entire stretch is intertidal; therefore, the observed high water line is along the face of the seawall and it is not possible to discern either erosion or accretion utilizing standard techniques associated with shoreline change. The sediment transport rates illustrated in Figure 3.15 utilized similar parameters to other beaches along the Scituate shoreline and the net transport rates shown are appropriate for evaluating potential placement of nourishment along this area. However, a more detailed analysis incorporating nearshore bathymetric change through an evaluation of LiDAR datasets may be warranted to support beach nourishment design if the option is pursued in the future.

Overall, the net sediment transport direction along Oceanside Drive is from north to south, with the highest net transport rates in the vicinity of 10th Avenue and Turner Road. Alongshore sediment transport rates decrease markedly at the south end of the beach adjacent to Cedar Point. Toward the north end of Oceanside Drive, the shallow cobble and boulder platforms tend to re-direct waves, leading to variable net sediment transport directions.

Along Humarock Beach (Figure 3.16), the flat wide beach profile that exists along much of the intertidal and shallow sub-tidal barrier beach system leads to a highly variable alongshore sediment transport direction. At the north end of Humarock, the net sediment transport direction is shown to be south to north, where sediments eroded from the beach migrate towards Fourth Cliff and into the entrance of the North River. Along much of the remainder of the beach, the net sediment transport direction varies dramatically. However, a review of the gross transport model results (i.e. the total transport to both the north and south directions) indicates that the transport in both directions is nearly balanced, and the net transport at a specific location is generally a small value relative to the gross transport. Therefore, the highly variable direction of transport presented by the arrows in may be misleading, as the transport rates are very low along most of this shoreline. It should be noted, as a result of the this relatively balanced gross transport, the net longshore sediment transport rates along much of Humarock Beach are an order of magnitude lower than at Oceanside Drive or North Scituate Beach. This bi-directional sediment transport along Humarock Beach ensures a generally stable beach width along much of this shoreline, where the variable nearshore wave conditions can continuously resupply eroded areas. In contrast, on average about 80% of the longshore transport along Oceanside Drive is from north-to-south, creating a constant loss of sediment from this region.

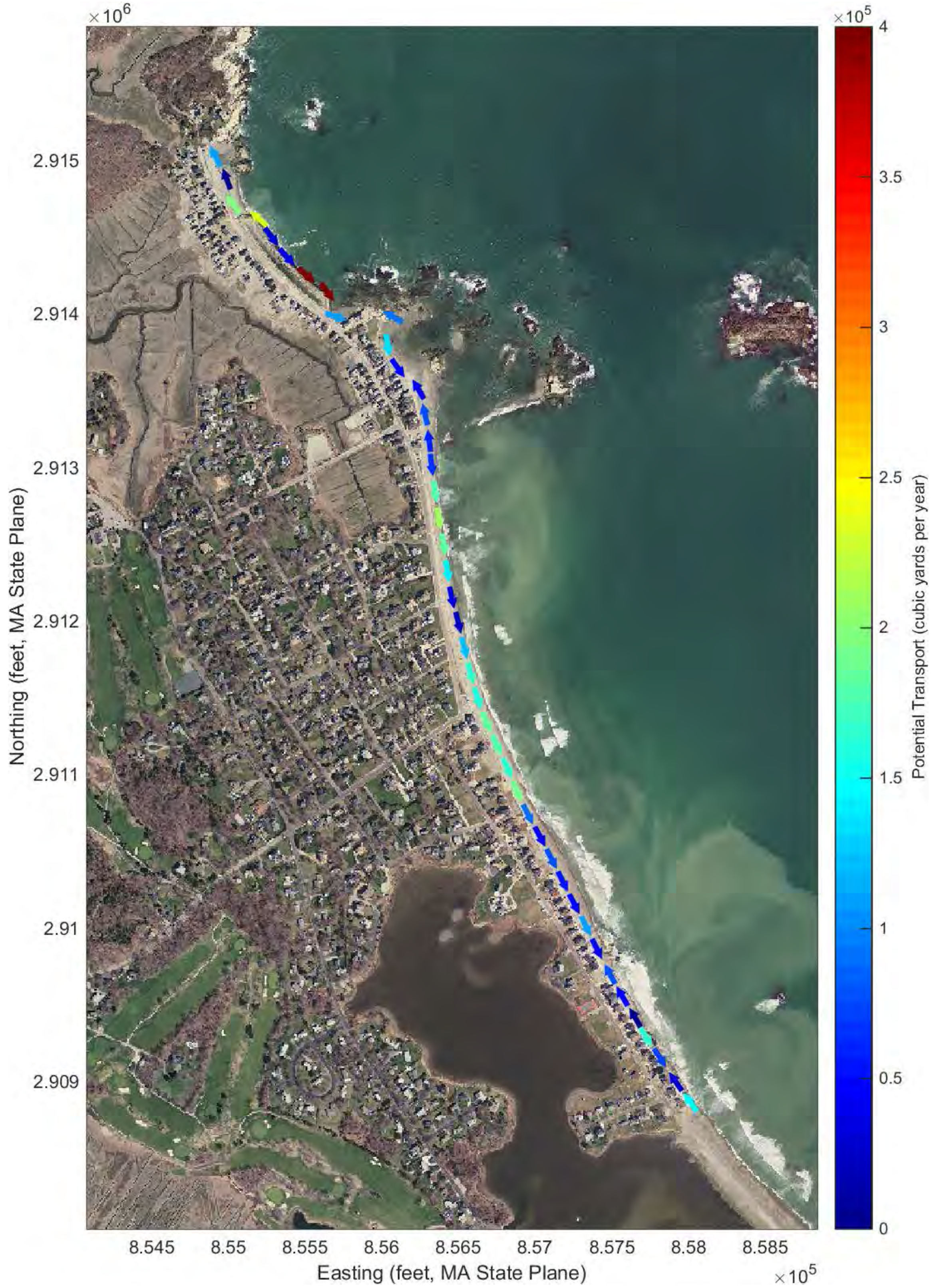


Figure 3.14 Modeled annual potential sediment transport along Minot, North Scituate, and Surfside Beaches. The sediment transport vectors represent the net direction and magnitude of transport along the coast.

Figure 3.15 Modeled annual potential sediment transport along Oceanside Drive. The sediment transport vectors represent the net direction and magnitude of transport along the coast.

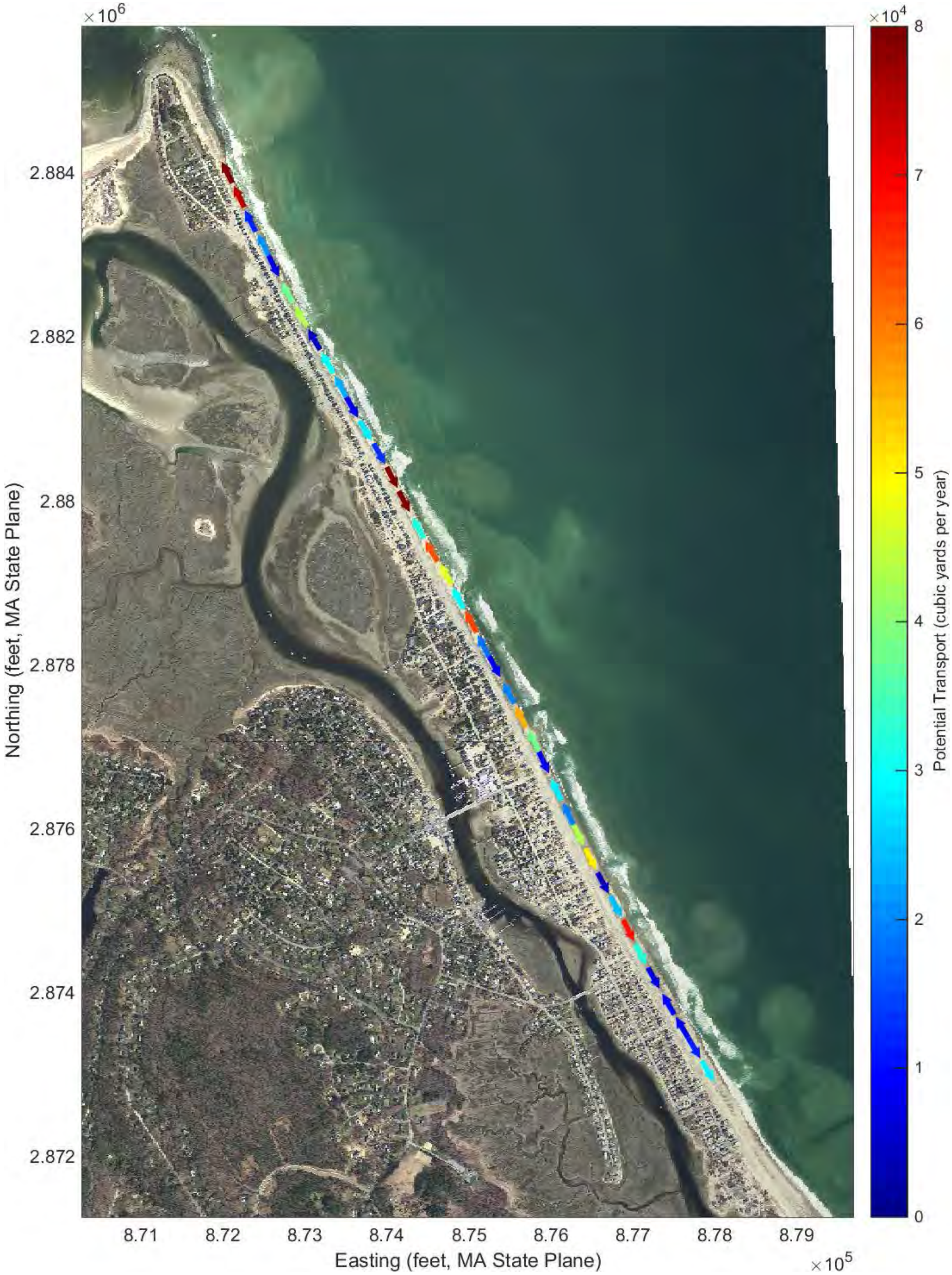


Figure 3.16 Modeled annual potential sediment transport along Humarock Beach. The sediment transport vectors represent the net direction and magnitude of transport along the coast.

4.0 HISTORICAL STORM DAMAGE

One of the primary issues facing the Town is the sustainability of its coastal development. The shoreline was extensively developed in the 1920's through the 1960's. High property values increased the pressure to develop the few remaining vacant lots on the shoreline. Today, many of the Town's highest priced homes are located there and these structures are becoming increasingly vulnerable. In addition to destruction of homes, storm conditions pose a significant danger to human life. During storm events, flood water and wind-swept debris may trap people preventing emergency personnel from providing assistance; these dangers make increased coastal development less and less sustainable.

As referenced in the *Massachusetts Climate Change Adaptation Report* (2011), Scituate's coastline is a classic example of a developed coastline that faces east or northeast and is vulnerable to nor'easters, which are common winter storms in Massachusetts. Existing foreshore protection stands landward of sediment starved beaches and is not capable of withstanding projected future conditions. Potential overwash, undermining, and collapse by higher sea levels and storm surge are serious concerns, particularly since at normal high tides there is no beach present in many areas to dissipate wave energy or to stabilize the structures.

The Town has cataloged storm damage impacts for several decades; however, there has been a need to combine detailed storm damage data to the coastal infrastructure that protects this development. As the basis for prioritizing future actions to address coastal resiliency, it was critical to relate ongoing (and future, to the extent possible) storm damage costs to the overall costs of infrastructure improvements needed to address these concerns. As the extensive flood damage that occurs along the Scituate coast primarily is driven by the influence of nor'easters, a more detailed understanding of impact related to storm severity was warranted. Specifically, the relationship between storm parameters and the severity of damage is critical for establishing expectations for shore protection strategies. With this understanding, four characteristic storms were selected to analyze the spatial distribution of the residential damage claims and the financial costs to the Town: the Blizzard of 1978, the 1991 No-Name Storm, Winter Storm Nemo (2013), and Winter Storm Juno (2015). Overall, these four storms provide the full range of severity associated with nor'easters that impact the Scituate shoreline, with return periods ranging from once every 158 years (the Blizzard of 1978) to once every 4 years (Winter Storm Nemo in 2013).

The Blizzard of 1978 was included as the storm of record, with a return period of greater than once every 100 years. While substantial infrastructure damage occurred during this storm event, FEMA NFIP had not been officially implemented at the time of the event. Regardless, the severity of infrastructure damage associated with the highest storm surge experienced in Boston since the tide station went into operation in 1921 provides the benchmark for comparison of South Shore coastal storms in the past century. Although slightly less severe, the 1991 No-Name Storm was selected because it represented the most significant storm event in the past 30 years and the most severe storm event that occurred since FEMA NFIP had been implemented. In addition, the more recent Winter Storms Nemo and Juno also were reviewed to represent "typical" events that occur on a relatively frequent basis. These lower return period events were selected due to the detailed storm damage documentation available from the Town, including coastal infrastructure improvement needs associated with ongoing damage. Understanding the geographical distribution of storm damage for relatively frequent, as well as more severe infrequent, storm events allowed for a detailed economic assessment of damages relative to storm severity for different locations along the coastline.

4.1 FEMA Repetitive Loss Claims by Storm (1978-2015)

FEMA defines a repetitive loss property as any insurable building for which two or more claims of more than $1,000 were paid by FEMA NFIP within any rolling ten-year period, since 1978. Repetitive loss property data was obtained from FEMA NFIP from 1978 to 2015; the information in the dataset included: the location/address of the properties, number of FEMA claims, the associated claim dates and claim amounts. It is acknowledged that the repetitive loss data does not include all claims to FEMA and does not take into account damages that property owners decided to not claim; however, the data gives an indication of the spatial distribution and the relative scale of damage costs. To maintain confidentiality, the exact location of the repetitive loss properties are obscured.

The dataset was sorted by claim date to determine time periods where large numbers of claims were filed. These dates generally coincided with significant storms and high surge events. Offshore wave records and measured water levels were obtained to characterize the storms. Table 4.1 summarizes the storm events that resulted in 5 or more FEMA repetitive loss claims in Scituate. All claim values have been updated to 2015 dollars.

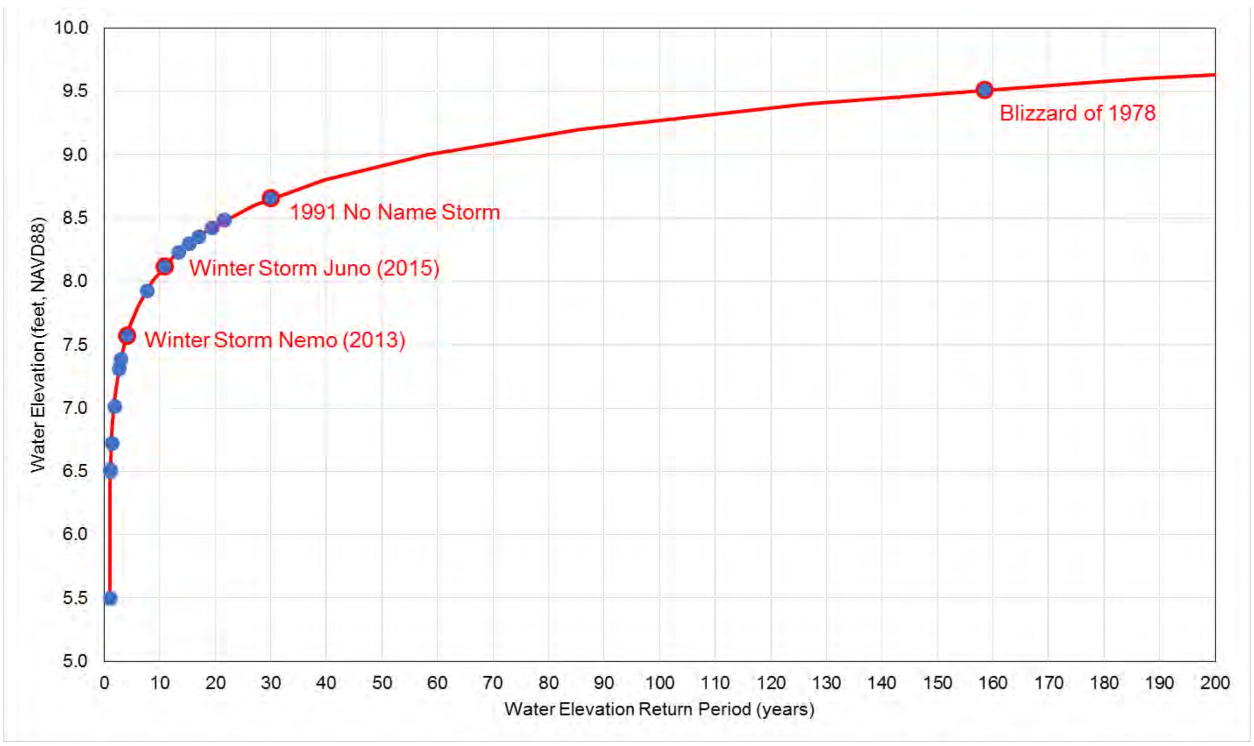


Figure 4.1 Water elevation return period of selected storm. Water elevations are measured in Boston at NOAA Station 8443970.

Table 4.1 Storm event characteristics and FEMA repetitive loss claims (1978-2015).

Start of Storm	End of Storm	FEMA Repetitive Loss Claims	Total Claim Amount	Storm Duration (hours)	Maximum Surge (feet)	Maximum Wave Height (feet)	Maximum Water Elevation (feet, NAVD88)	Water Elevation Return Period (years)
2/6/1978	2/8/1978	-	-	47	4.4	-	9.5	158
1/24/1979	1/27/1979	52	$630,429	29	3.8	-	8.4	19.3
3/29/1984	3/31/1984	21	$159,685	37	4.5	-	6.5	1.1
1/2/1987	1/3/1987	162	$2,715,502	24	2.4	15.4	8.5	21.6
10/28/1991	11/2/1991	446	$34,505,878	99	4.9	29.9	8.7	30.0
12/10/1992	12/14/1992	385	$9,494,777	92	3.2	24.0	8.5	21.6
3/31/1997	4/2/1997	10	$218,653	33	3.2	24.2	6.5	1.1
3/5/2001	3/9/2001	105	$1,603,277	52	3.2	24.0	7.3	2.7
1/1/2003	1/5/2003	61	$1,076,543	71	2.4	20.3	7.9	7.6
12/5/2003	12/9/2003	34	$512,202	39	3.5	26.4	6.7	1.3
1/22/2005	1/24/2005	20	$325,617	37	2.2	27.8	5.5	1.0
5/22/2005	5/30/2005	25	$298,145	57	2.4	17.4	8.1	10.7
4/15/2007	4/19/2007	32	$1,050,634	61	3.4	25.9	8.3	15.2
2/23/2010	2/28/2010	6	$168,587	77	4.0	25.3	7.0	1.8
12/16/2010	12/28/2010	147	$4,169,119	150	3.9	25.3	8.2	13.3
2/7/2013	2/11/2013	145	$4,672,018	30	4.2	25.7	7.6	4.1
3/4/2013	3/11/2013	21	$454,648	61	3.2	20.7	7.4	3.0
1/2/2014	1/4/2014	13	$386,059	31	3.1	19.3	8.4	16.9
1/26/2015	1/29/2015	47	$1,154,415	30	4.8	27.2	8.1	10.9

4.2 Selected Storms

Four storms were selected to analyze the spatial distribution of the residential damage claims relative to storm intensity, and the financial costs to the Town. As described above, the storms chosen were: the Blizzard of 1978, the 1991 No-Name Storm, Winter Storm Nemo (2013), and Winter Storm Juno (2015). The Blizzard of 1978 was included as the storm of record; however, it was understood *a priori* that detailed FEMA claims information was not available for this storm due to the implementation timeframe for the FEMA NFIP program in the late 1970s. Of similar magnitude, the Portland Gale in 1898 breached the barrier beach between Third and Fourth Cliffs to form New Inlet, eventually leading to the closure of the inlet to the South River near Rexhame Beach in Marshfield.

The 1991 No-Name Storm was selected because it represents the most significant recorded storm since the FEMA NFIP program went into effect and of the large number of recorded claims. More recently, Winter Storm Nemo and Juno, although of lower overall magnitude than the substantial 1978 and 1991 storms, were well documented within the Town records. Overall, the range of the four selected storms provides the general range of anticipated damages to Town and private coastal infrastructure based on storm intensity. As shown in Figure 4.2, along with information from Table 4.1, the return period of storms can be linked directly to FEMA repetitive loss claims and the four storms evaluated represent each of these three categories. In addition, the FEMA damage records can be utilized to discern which geographic areas are impacted by the more frequent storms versus those areas that only sustain major damage during the most significant storm events. This data can help guide the prioritization process, as it highlights the most vulnerable areas.

For consistency, wave and water level data for the following plots were obtained from the National Data Buoy Center Station 44013 (Figure 4.3) and from NOAA Station 8443970 in Boston Harbor. Typically, storm severity is based upon storm surge elevations alone, where both storm duration and offshore wave height generally are not considered important parameters. As shown in Figure 4.4, offshore wave height appears to be independent of storm intensity (i.e. return period), indicating that substantial storm waves are generated offshore regardless of storm severity. However, for depth-limited conditions closer to shore, large waves can propagate unattenuated under higher water surface conditions; therefore, higher storm surge conditions allow larger waves to impact the coastline. Unlike hurricanes in New England, extra-tropical storms or nor’easters often linger for more than 24 hours. While no direct numerical link existed, it is apparent from the data provided in Table 4.1 that all major storms had durations in excess of two tidal cycles and could exhibit elevated water levels for as much as 6 days.

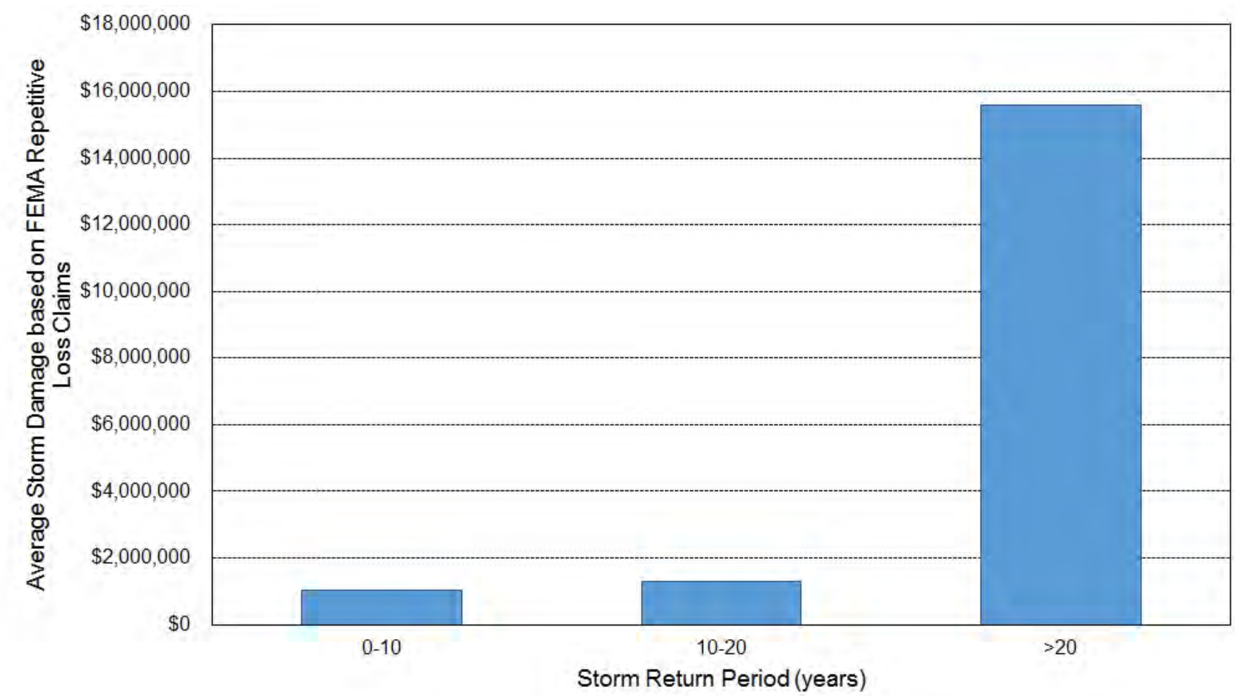


Figure 4.2 Average monetary value of FEMA repetitive loss claims versus return period of storm event since 1978.

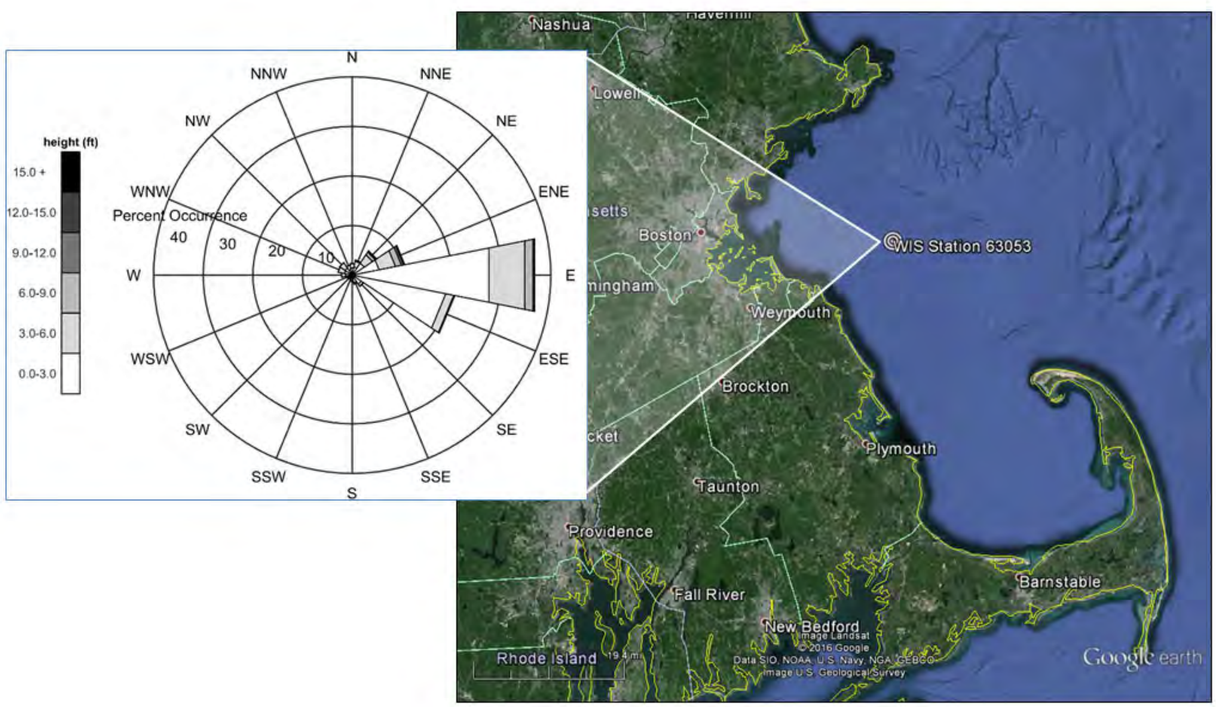


Figure 4.3 Location of National Data Buoy Center Station 44013 offshore with the associated long-term wave rose that indicates waves propagating towards the Massachusetts coast from the east and east-southeast approximately 59% of the time.

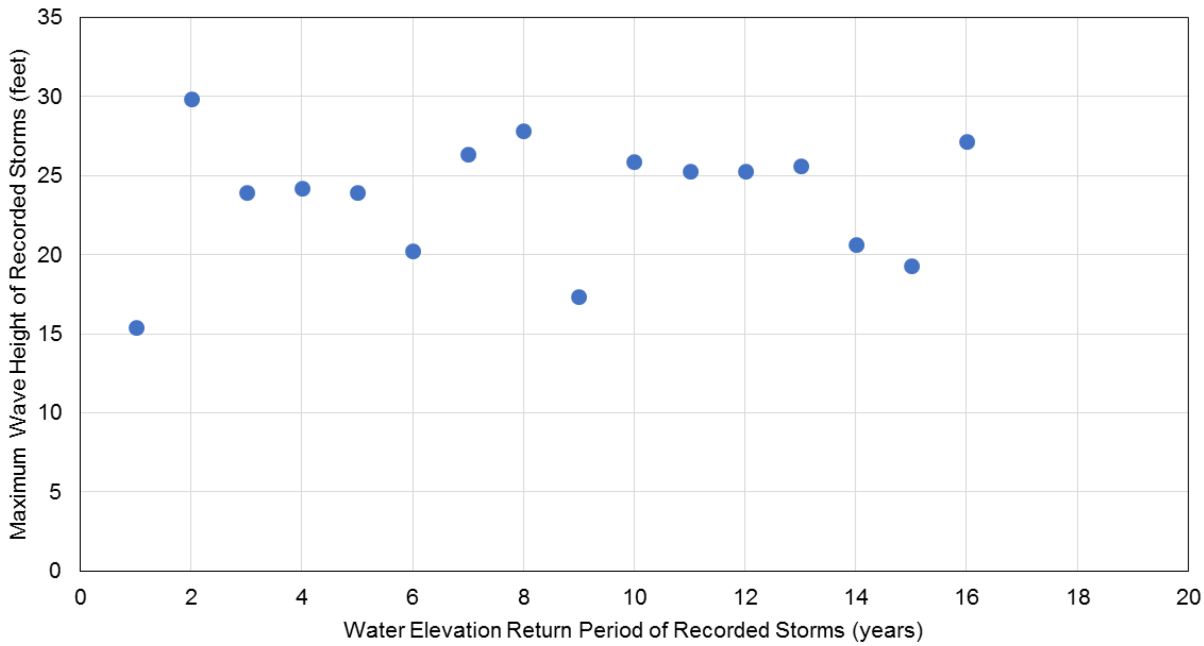


Figure 4.4 Recorded maximum wave height at National Data Buoy Center Station 44013 versus maximum storm surge elevations recorded for a range of storm return periods based upon tide level observations at NOAA Station 8443970.

4.2.1 Blizzard of 1978

A time series plot of the water elevation during the Blizzard of 1978 is shown in Figure 4.5. The black and red lines represent the predicted and observed water elevation, respectively. The difference in elevation between the lines represents the storm surge. The yellow line indicates the storm duration, where significant deviation existed between the predicted and observed tide elevations. The storm lasted approximately 47 hours with peak surges aligning with two of the high tides. Due to the relatively high tide range in the Boston area, coincident high storm surge and high tides generally are rare. With a return period of 158 years, the blizzard is the storm of record; however, detailed residential property damage claims to FEMA were not available.

Based on available information, over 300 people in Scituate were evacuated and 189 homes destroyed with over 400 homes sustaining major damage (MCZM, 1993). Figure 4.6 shows the extensive storm damage along Turner Road (looking north). The Town submitted a FEMA claim for damages to public infrastructure and incurred costs as a result of the storm (costs in 2015 dollars):

- $22,000,000 in seawall/revetments
- $650,000 in road damage
- $1,500,000 in debris clearing
- $700,000 in damage to public utilities
- $130,000 in damage to public buildings
- $1,700,000 for protective measures (police, fire, public shelters, etc.)
- Total: $26,700,000

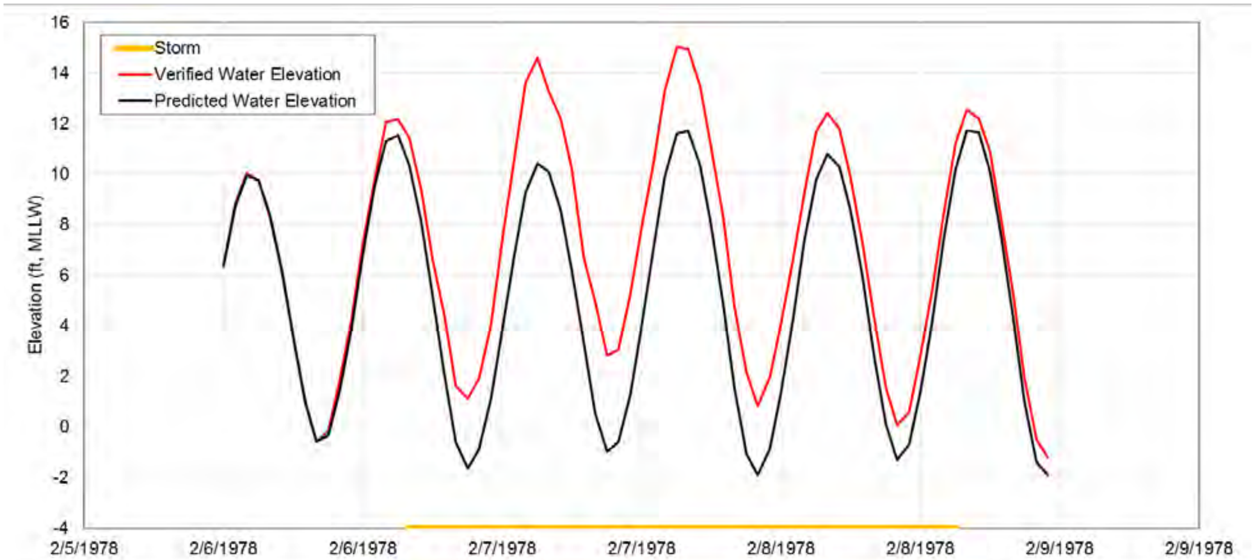


Figure 4.5 Time series plot of the Blizzard of 1978. The black and red lines represent the predicted and observed water elevation, respectively. The yellow line indicates the storm duration.

Figure 4.6 Turner Road damage from the Blizzard of 1978 (*image credit: www.blizzardof78.org*).

4.2.2 1991 No-Name Storm

The 1991 No-Name Storm, also known at the 1991 Perfect Storm, lasted approximately 99 hours with a peak surge of almost 5 feet. Offshore waves reached a peak height of nearly 30 feet around the same time as the high tide and peak surge. A time series plot of the storm including predicted and observed water levels, wave height, and duration is presented in Figure 4.7. The 1991 No-Name storm had a return period of 30 years.

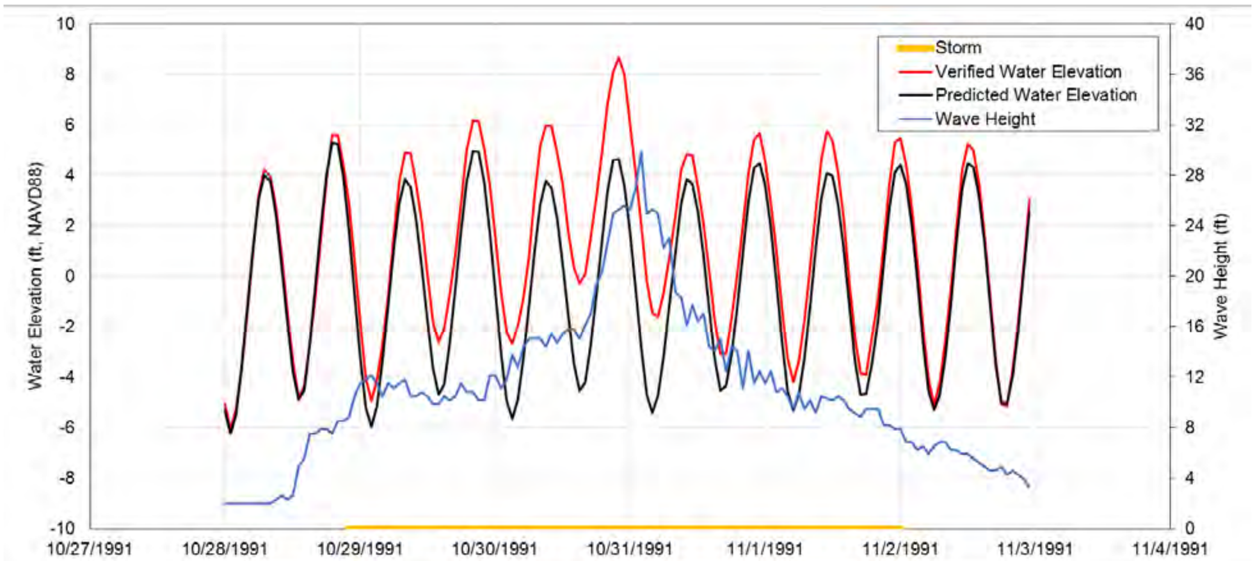


Figure 4.7 Time series plot of the 1991 No-Name Storm. The black and red lines represent the predicted and observed water elevation, respectively. The blue line represents the offshore wave height and the yellow line indicates the storm duration.

Spatial distribution of the FEMA repetitive loss claims incurred by the storm are mapped in Figure 4.8. Widespread damage was recorded along the entire Scituate shoreline, as shown in Figure 4.9, with the exception of the Cliffs. In total, 446 FEMA repetitive loss claims were filed totaling over $34 million. The costs to Town were summarized in a FEMA claim:

- $2,400,000 in seawall/revetments
- $130,000 in road damage
- $140,000 in debris clearing
- $100,000 in damage to public utilities
- $90,000 in damage to public buildings
- $90,000 for protective measures
- Total: $3,000,000

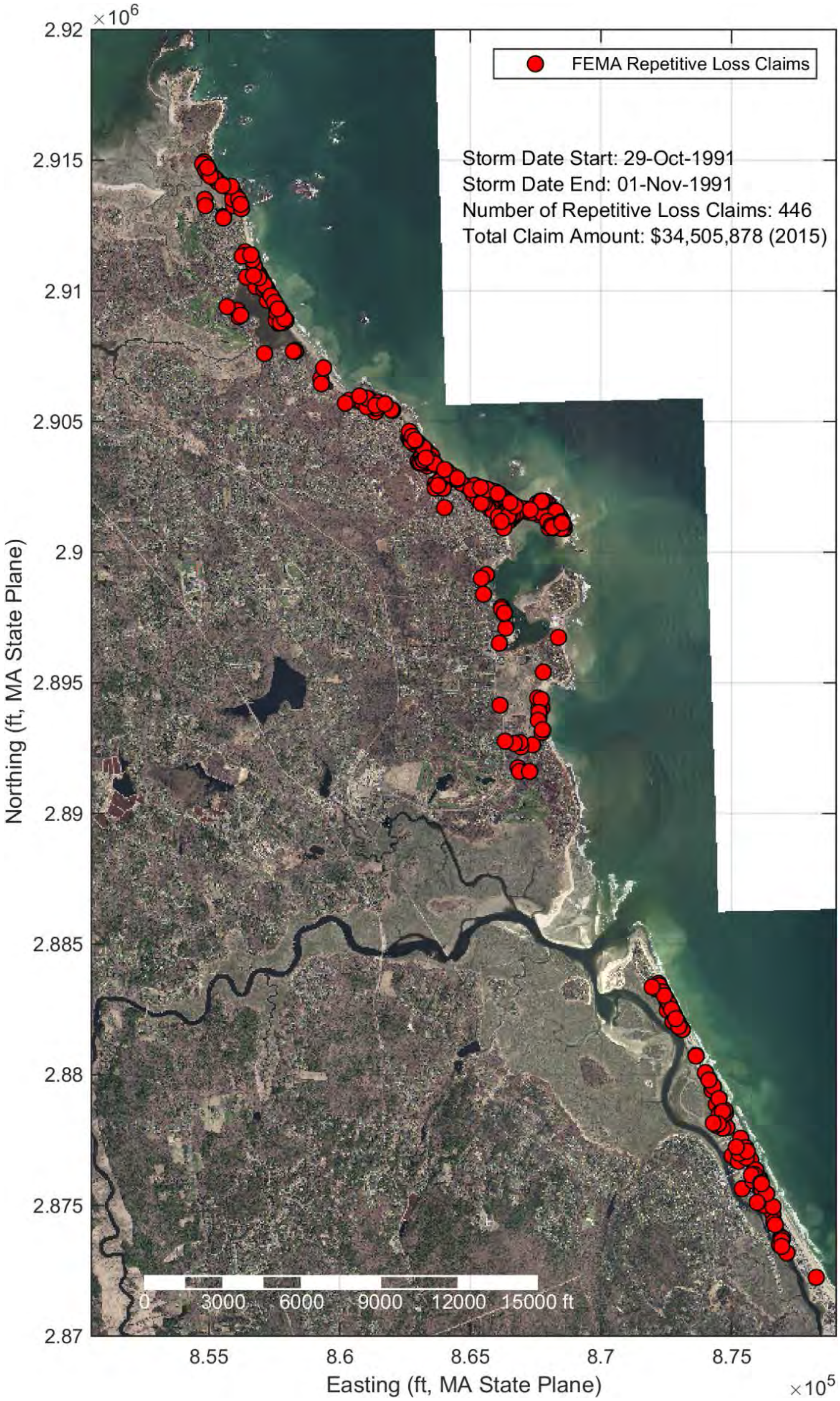


Figure 4.8 Spatial distribution of FEMA repetitive loss claims from the 1991 No-Name Storm.

Figure 4.9 Destroyed home from the 1991 No-Name Storm (photo from the Scituate Historical Society).

4.2.3 Recent Storms – Winter Storm Nemo (2013) and Winter Storm Juno (2015)

Winter storms Nemo and Juno, with 4- and 10-year return period, respectively, were not as severe as the storms described above, however these smaller nor'easters aid in indicating where susceptible damage areas are located in Scituate. The maximum surge and offshore wave height for both storms occurred during low tide and the storms were relatively short, about 30 hours in duration. Time series plot of the storms are presented in Figure 4.10 and Figure 4.11. A photo of Oceanside Drive, rendered impassible due to flooding during Winter Storm Nemo, is shown in Figure 4.12.

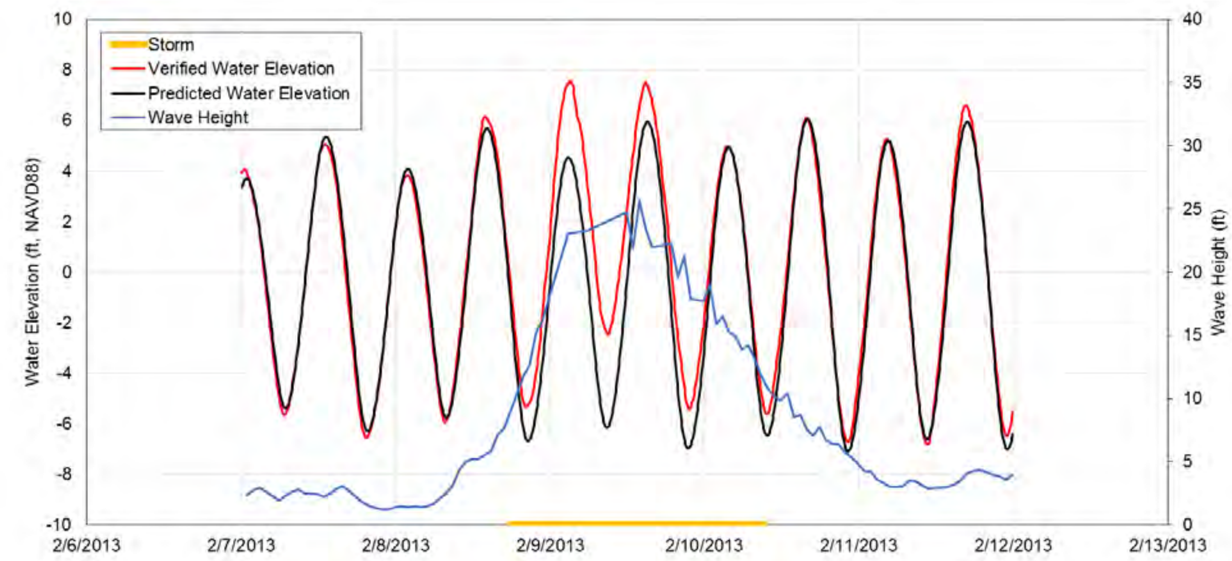


Figure 4.10 Time series plot of Winter Storm Nemo. The black and red lines represent the predicted and observed water elevation, respectively. The blue line represents the offshore wave height and the yellow line indicates the storm duration.

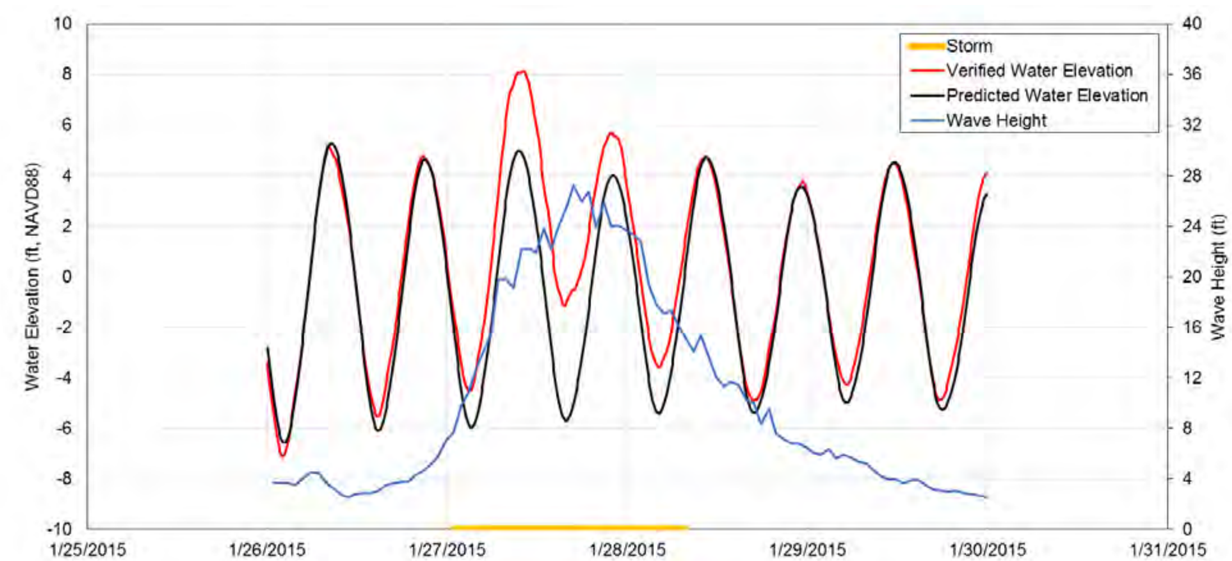


Figure 4.11 Time series plot of Winter Storm Juno. The black and red lines represent the predicted and observed water elevation, respectively. The blue line represents the offshore wave height and the yellow line indicates the storm duration.

Figure 4.12 Impassible roads along Oceanside Drive during Winter Storm Nemo (photo by Jason Burtner, *www.mycoast.org*).

FEMA repetitive loss claims during Nemo and Juno are shown in Figure 4.13 and Figure 4.14, respectively. Compared to the extent of damages of the 1991 No-Name Storm, the damages for the smaller storms are concentrated in several areas: Oceanside Drive and Turner Road, Cedar Point, and the north part of Humarock. The Town estimates that a total of $11.3 million in damages were sustained by the publically-owned coastal engineering structures as a result of the two winter storms. During storms of similar magnitude, the road clearing costs incurred by the Town are approximately:

- $12,000 for Surfside Road
- $10,000 for Peggotty Beach
- $30,000 for Central Avenue (Humarock)

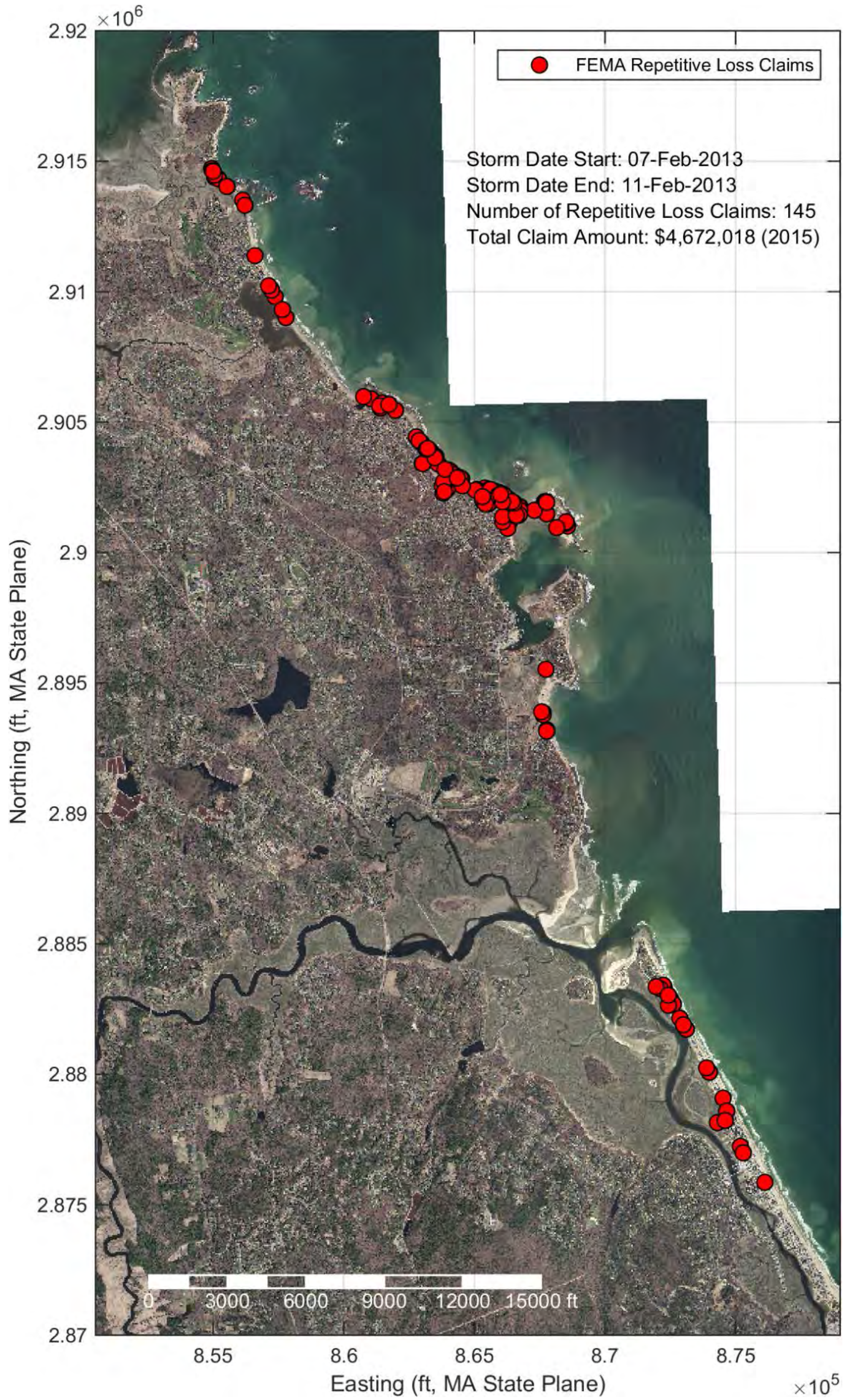


Figure 4.13 Spatial distribution of FEMA repetitive loss claims from Winter Storm Nemo.

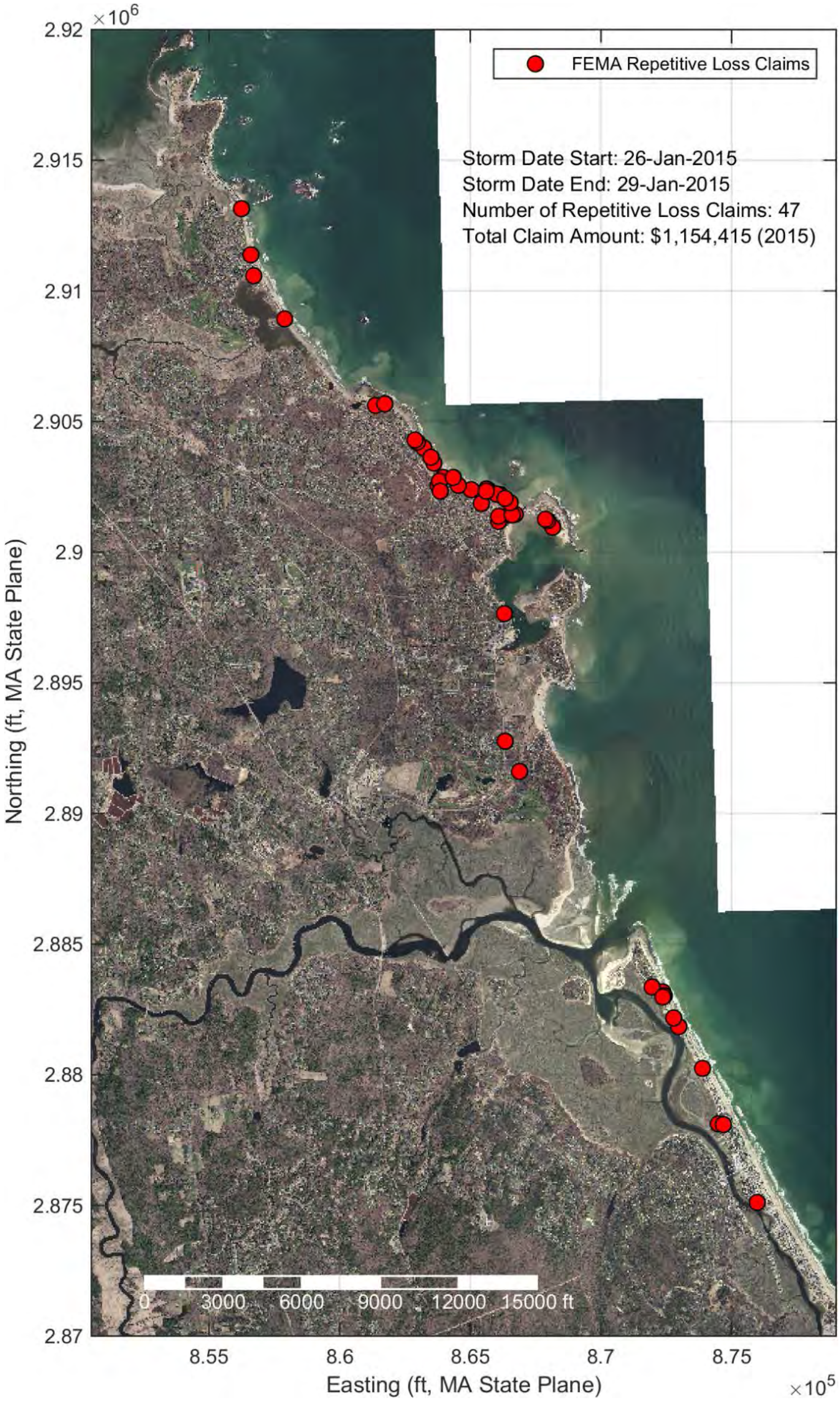


Figure 4.14 Spatial distribution of FEMA repetitive loss claims from Winter Storm Juno.

5.0 PRIORITIZATION CRITERIA

Development of prioritization criteria for evaluating vulnerability of both private and public infrastructure is a critical initial step for developing a meaningful assessment of management strategies for shore protection. The overall goal was to create an objective set of technical criteria that could be utilized to create a rating system for the different sections of the Scituate coast. In addition to economics associated with damage susceptibility, a number of other factors were evaluated including landform elevation, need for providing emergency egress, breaching potential of the landform, and existing condition of any coastal engineering structures. Prioritization for infrastructure protection for a particular portion of shoreline depended upon potential damage to both private and public assets, as well as existing condition parameters. Development of prioritization criteria in this manner provides baseline information that the Town of Scituate can utilize to focus efforts on the most vulnerable areas.

The coast of Scituate consists of glacial deposits, outwash areas formed from the erosion of these deposits, and underlying bedrock outcrops, causing the undulating shape of the shoreline typified by pocket beaches punctuated with headlands. In addition, extensive armoring of the shoreline has altered natural coastal processes. The extent of development on beaches, dunes, and upland areas can also influence how the landform responds to storm impacts. Understanding that the specific type of shoreline can be linked to its vulnerability to storm impacts, the coast was divided into characteristic sections that allowed for site-specific evaluation of appropriate prioritization criteria for addressing coastal resiliency concerns. Prioritization criteria could be evaluated for each of these shoreline sections, which then could be summed together to create an overall prioritization ranking. In this manner, a comparative analysis between different sections of the Scituate shoreline could be provided to inform the Town decision-making process.

It should be noted that the prioritization criteria were developed to help differentiate the different shoreline sections from each other. Therefore, the analysis did not include potential criterion that would be identical or nearly identical for all sections of the Scituate coast. For example, there was no criterion for vulnerability to impacts from large storm waves, as the entire coastal area evaluated is subjected to storm waves generated in the North Atlantic Ocean. Instead, prioritization criteria focused on the varying natural and anthropogenic features along the shoreline that increase the vulnerability to storm impacts.

5.1 Study Areas

Based upon both the natural and anthropogenic features along the Scituate shoreline, the coast was divided into 15 study areas to reflect the shoreline type, residential density, and presence of coastal engineering structures. The extents of each study are summarized in Table 5.1 and Figure 5.1. Within each study area, the overall features of the coastline are similar as illustrated through the following examples:

- *North Scituate Beach* – The entire length of shoreline consists of a vertical seawall fronted by a low elevation armor stone revetment that is generally appears to be dumped stone. Landward of the seawall, a combination paved and/or rip-rap splash apron exists to dissipate overtopping waves from impacting Glades Road that runs parallel to the seawall crest. A narrow intertidal sand/gravel/cobble (mixed sediment) beach exists along the length of seawall; however there is no beach at high tide.
- *Egypt Beach* – This section of shoreline transitions from the cobble berm and mixed sediment beach at the north end to a cobble and boulder platform in the intertidal area along the southern portion, backed by a mixed sediment beach. This platform

significantly attenuates incoming wave energy during normal tidal conditions. Although some armor stone revetments exist along this beach area, the contiguous beach system exists.

- *Second Cliff* – This section of shoreline consists of a relatively high armor stone revetment that has eliminated this glacial feature as a supply of sediment to adjacent beaches. An intertidal boulder and cobble platform exists along much of the Second Cliff shoreline that provides some limited wave attenuation during normal tide conditions.
- *Humarock North* – This barrier beach area is characterized by relatively low volume cobble dunes that are not sufficient to withstand moderate storm events. Even during modest nor'easters, the overwashed dune material completely blocks Central Avenue requiring excavation of several feet of material to re-open the roadway. The elevation of Central Avenue along this stretch is substantially below the 100-year still water elevation. The cobble dune is fronted by a mixed sediment beach.

Table 5.1 Study area limits along the Scituate shoreline.

Study Area	North Limit	South Limit	Length (feet)
Minot Beach	163 Glades Road	100 Glades Road	2,037
North Scituate Beach	96 Glades Road	4-6 Gannett Road	2,653
Surfside Road	1 Gannett Road	91 Surfside road	2,978
Mann Hill Beach	South Property Line of 91 Surfside Road	4 Stanton Lane	2,663
Egypt Beach	South Property Line of 4 Stanton Lane	30 Standish Ave	3,686
Oceanside Drive	146 Oceanside Drive	183 Turner Road	5,662
Cedar Point	11 Lighthouse Road	Scituate Lighthouse	2,828
First Cliff	184 Edward Foster Road	152 Edward Foster Road	1,762
Edward Foster Road	138 Edward Foster Road	114 Edward Foster Road	1,079
Second Cliff	108 Edward Foster Road	52 Peggotty Beach Road	2,295
Peggotty Beach	4 Peggotty Beach	6 Town Way Extension	1,932
Third Cliff	1 Dickens Road	53 Collier Road	4,853
Fourth Cliff	Fourth Cliff Military Reservation	16 Cliff Road South	1,735
Humarock North	10 Cliff Road South	130 Central Avenue	4,746
Humarock South	128 Central Avenue	9 Old Mouth Road	8,282

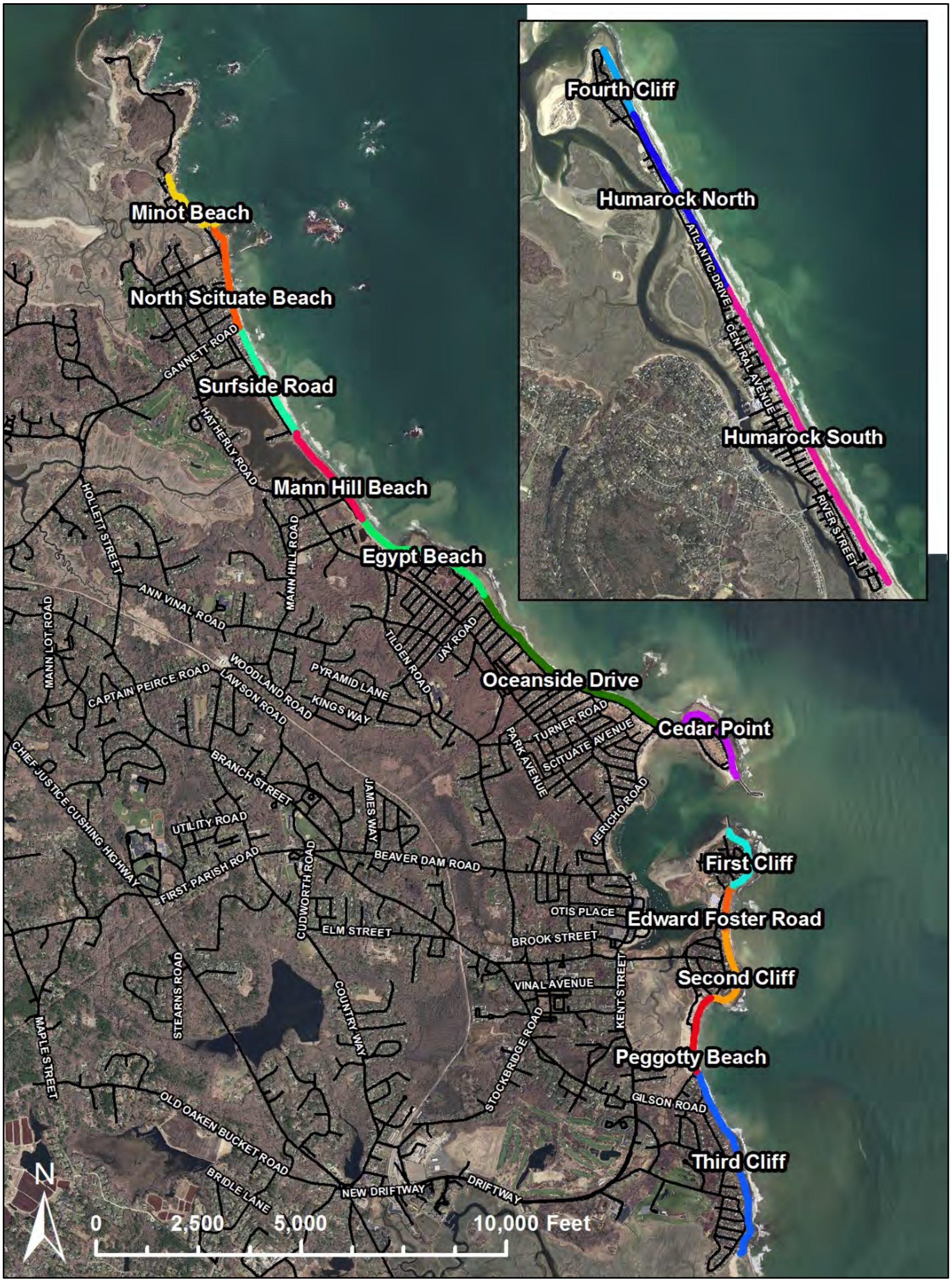


Figure 5.1 Study areas utilized to assess prioritization criteria along the Scituate shoreline.

5.2 Damage Susceptibility of Private Properties

While the Flood Insurance Rate Maps (FIRMs) developed by FEMA are often considered the best available information regarding the influence of both storm surge and wave runup/overtopping in the coastal zone, substantial technical concerns remain about the accuracy of these maps, specifically along the New England coast. This has led to a series of both appeals and Letters of Map Revision (LOMRs) to FEMA that have gradually improved the accuracy of maps produced over the past 5 years. For the case of Scituate, an appeal was filed and accepted by FEMA to address some general concerns about mapping accuracy. However, notable inaccuracies remain, and a review of the FIRMs for Scituate highlights these concerns. As an example, it is often informative to review the mapped flood zones with FEMA repetitive loss data to determine whether coastal areas experiencing frequent structural damage to properties align with the mapped highest flood hazard areas. Similar to many areas along coastal New England, there does not appear to be a consistent correlation between the mapped flood zones and the FEMA repetitive loss records; therefore, the mapped flood zones appear lacking. For this reason, it became apparent that the FEMA repetitive loss database provided a more accurate assessment of vulnerability to coastal storm damage.

The damage susceptibility of private properties in Scituate was evaluated by dividing the FEMA repetitive loss data based on the study areas each claim property was located within. To assess overall vulnerability, the evaluation quantified information related to both the number and cost of FEMA claims by shoreline study area. After reviewing information within the repetitive loss database, three measures were selected to evaluate damage susceptibility: the number of claims per 1,000 feet of study length, the value of the claims per 1,000 feet of study length, and the average number of claims per repetitive loss property.

The number of FEMA repetitive loss claims along 1,000 feet of each study length was calculated to determine the density of damage incidents. Oceanside Drive had the greatest number of claims at 157 claims per 1,000 feet. The second highest number of claims was at Cedar Point at 69 claims per 1,000 feet. The density of claims in each section is summarized visually in Figure 5.2.

The value of FEMA repetitive loss claims per 1,000 feet was calculated normalizing the total value of FEMA claims (converted to 2015 dollars) by the study area length. Based on this criteria, Oceanside Drive and Surfside Road show the largest claim values at $5.8 million and $3.7 million, respectively. Figure 5.3 summarizes the value of the claims per 1,000 feet for each study area.

The average claims per property is calculated by diving the total number of claims by the number of repetitive loss properties in each study area. Because the data is based on repetitive loss properties, each property has a minimum of two claims. Oceanside Drive and Peggotty Beach rated the highest using this criteria with 4.8 and 4.0 claims per property. The average claims per property are shown in Figure 5.4.

A priority rating scheme was developed for each of the above measures of damage susceptibility, as presented in Table 5.2. The rating scheme for each measured ranged from 0 to 5 with 5 being of highest priority. With no repetitive loss claims reported during the study period, the Cliffs received the lowest priority rating (0) for all the measures while Oceanside Drive received the highest maximum priority rating of 15. Summary of priority rating for each study area with respect to damage susceptibility is summarized in Table 5.3.

Table 5.2 Priority rating scheme for FEMA repetitive loss claims per 1,000 feet of study shoreline from 1978 to 2015.

Priority Rating (0-5)	Claims per 1,000 feet	Claim Value per 1,000 feet	Average Claims per Repetitive Loss Property
0	0	0	0
1	<10	<$500,000	<2.5
2	10-20	$500,000-$1.0 million	2.5-3.0
3	20-50	$1.0 million -$3.0 million	3.0-3.5
4	50-100	$3.0 million -$5.0 million	3.5-4.5
5	>100	>$5.0 million	>4.5

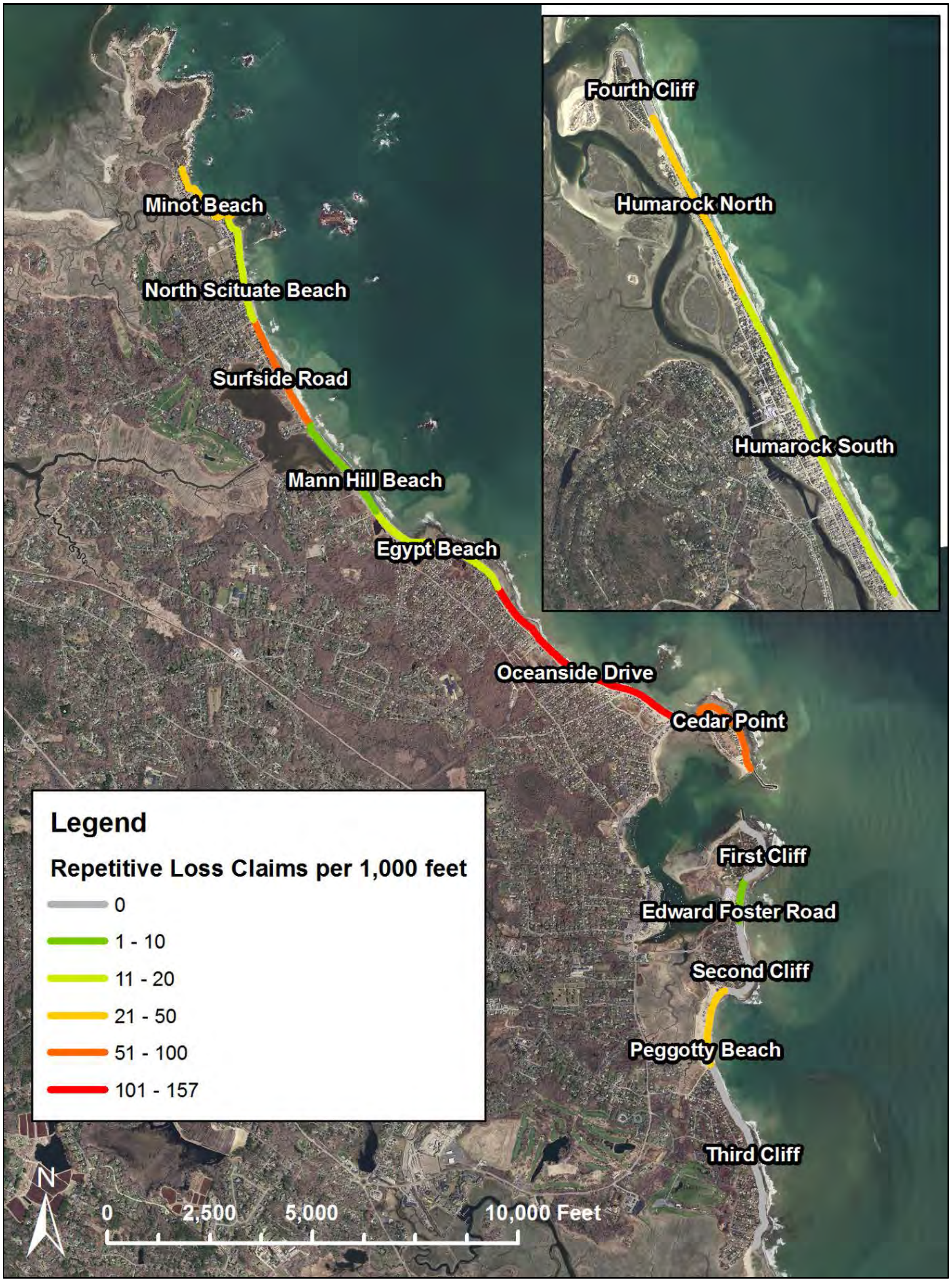


Figure 5.2 FEMA repetitive loss claims per 1,000 feet of study shoreline from 1978 to 2015.

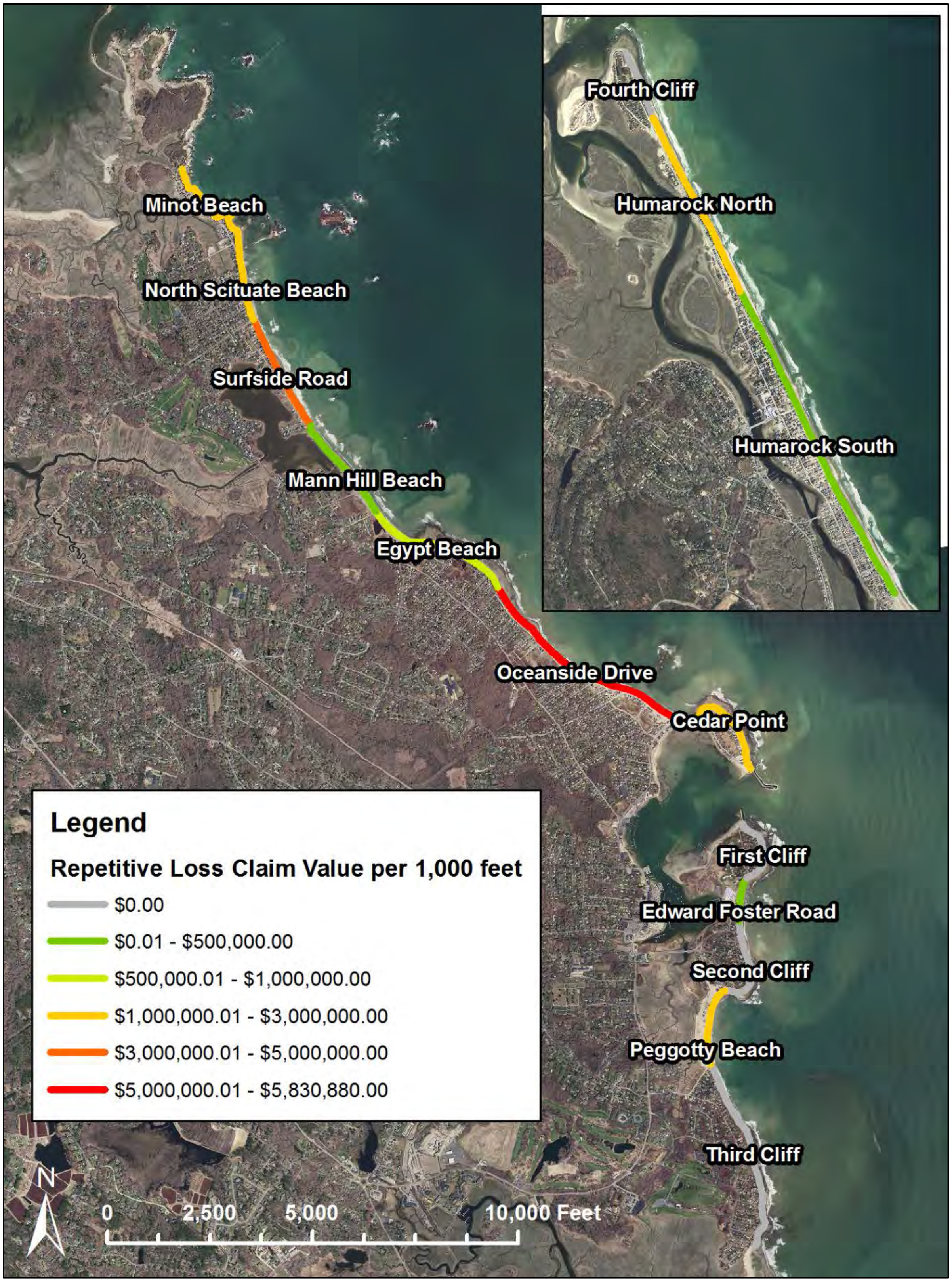


Figure 5.3 Value of FEMA repetitive loss claims per 1,000 feet of study shoreline from 1978 to 2015.

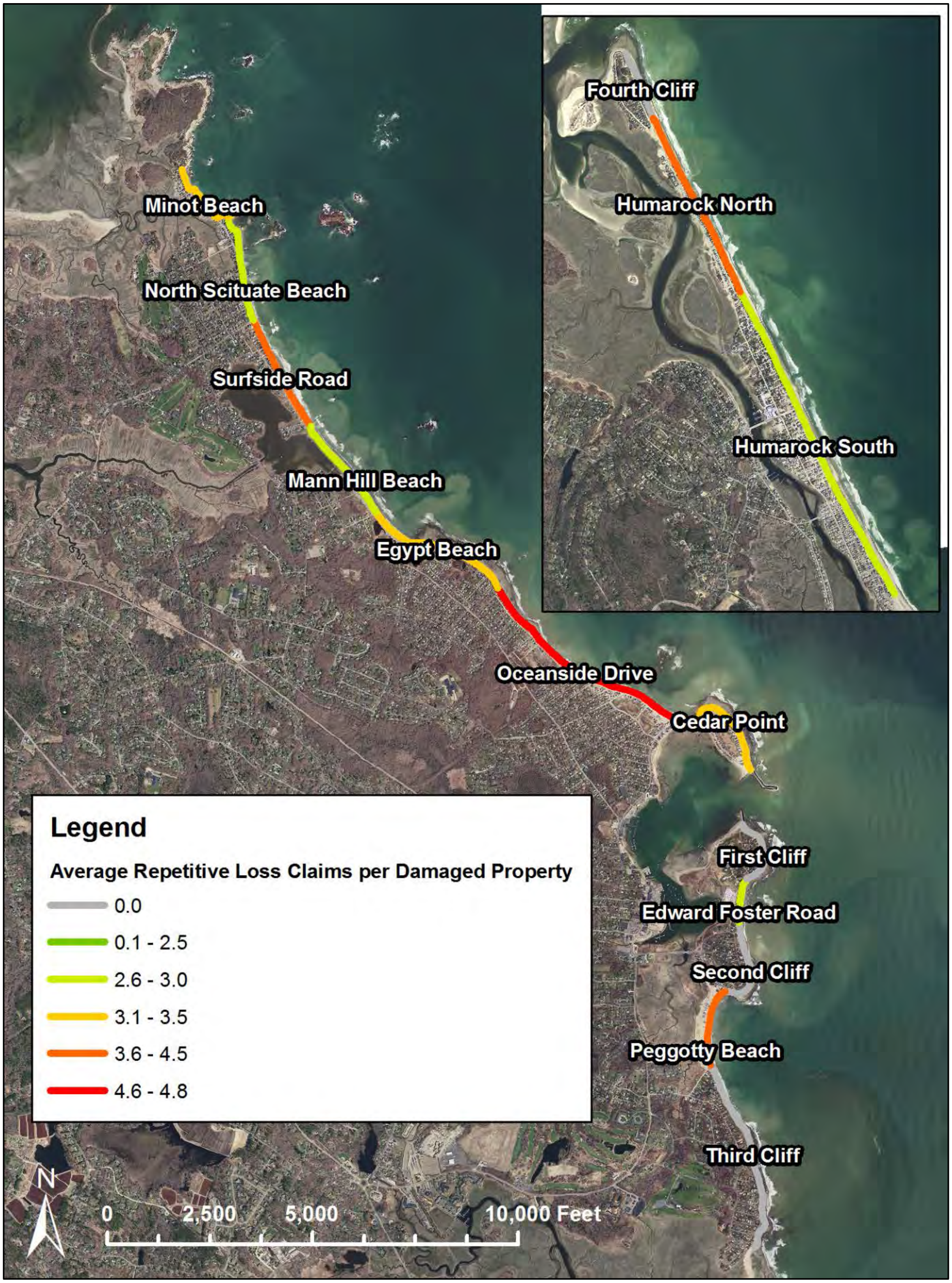


Figure 5.4 Average number of FEMA repetitive loss claims per damaged property for each study area from 1978 to 2015.

Table 5.3 Priority rating of study areas with respect to damage susceptibility of private properties.

Study Area	**Priority Rating**			
	Repetitive Loss Claims per 1,000 feet **0 – Low** **5 – High**	**Repetitive Loss Claim Value per 1,000 feet** **0 – Low** **5 – High**	**Average Repetitive Loss Claims per Damaged Property** **0 – Low** **5 – High**	**Total** **0 – Low** **15 – High**
Minot Beach	3	3	3	9
North Scituate Beach	2	3	2	7
Surfside Road	4	4	4	12
Mann Hill Beach	1	1	2	4
Egypt Beach	2	2	4	8
Oceanside Drive	5	5	5	15
Cedar Point	4	3	3	10
First Cliff	0	0	0	0
Edward Foster Road	1	1	2	4
Second Cliff	0	0	0	0
Peggotty Beach	3	3	4	10
Third Cliff	0	0	0	0
Fourth Cliff	0	0	0	0
Humarock North	3	3	4	10
Humarock South	2	1	2	5

5.3 Landform Elevation

Along the Scituate shoreline, the difference between the 10-year and 100-year still water elevation (SWL, the surface of the water if all wave and wind action were to cease) is less than 1.5 feet. Low elevation areas are more prone to flooding and merit a higher priority rating. For example, Figure 5.5 shows the comparitive flooding susceptibility of Cedar Point versus First Cliff during 10-year and 100-year storms. The majority of Cedar Point is shown to be below the 10-year still water elevation while First Cliff is above the elevation fo the 100-year storm. Additional figures for other areas of the Town are presented in Appendix C.

Figure 5.5 Mapping of landform elevation at Cedar Point and First Cliff. Dark and light blue shading indicates that the area is flooded during 10- and 100-year storms by still water (excluding waves), respectively.

The priority rating scheme for landform elevation ranges is presented in Table 5.4. Study areas where the primary elevation is above the 100-year still water elevation are given the lowest rating while areas that are inundated by the 10-year still water elevation or lower have the highest priority. The priority rating for each study area with respect to landform elevation is presented in Table 5.5.

Table 5.4 Priority rating scheme for the primary landform elevation of the study areas.

Primary Landform Elevation	**Priority Rating (1-5)**
Above 100-year SWL	1
Above 10-year SWL, but Below the 100-year SWL	3
Below 10-year SWL	5

Table 5.5 Priority rating of study areas with respect to primary landform elevation.

Study Area	**Priority Rating 1 – Low 5 – High**
Minot Beach	1
North Scituate Beach	3
Surfside Road	5
Mann Hill Beach	3
Egypt Beach	3
Oceanside Drive	5
Cedar Point	5
First Cliff	1
Edward Foster Road	3
Second Cliff	1
Peggotty Beach	5
Third Cliff	1
Fourth Cliff	1
Humarock North	5
Humarock South	5

5.4 Damage Susceptibility of Public Utilities

Along with public and private development, public utilities exist along the shoreline to service this development. The susceptibility of these public utilities to storm damage can increase the vulnerability of development along the open coastal areas of Scituate. For this study, the damage susceptibility of public utilities was determined by evaluating three measures: wastewater, pump stations, and electric lines. Natural gas lines are present through a majority of the Town and water lines are present for all properties. These two forms of underground utilities were not included in the prioritization assessment, due to their broad presence throughout the study area. However, it should be noted that gas and water lines are susceptible to storm damage and have been exposed by road scour from wave overtopping in the past (see an example in Figure 5.6).

Figure 5.6 Exposed gas lines along Glades Road behind Minot Beach. Photo from February 11, 2013 storm report by Jason Burtner (*mycoast.org*).

The majority of the Town uses individual septic systems to manage wastewater. The town wastewater system map is shown in Appendix D. In a 1993 MCZM report, 22 septic systems were reportedly located on Peggotty Beach and temporary closures of the adjacent public beach due to high coliform counts, presumably from septic system pollution, has occurred in the past. During the 1991 No-Name Storm, damage to septic systems caused public health concerns throughout the Town, as both overwash and beach erosion exposed these systems to the environment. Where septic systems exist behind a coastal engineering structure, the damage susceptibility is considered to be low, as the system is less likely to be scoured and exposed. In addition, in comparison to a sewer system where a break in the line could have widespread environmental impacts, damage to an individual septic system is localized. Study areas where septic tanks and sewer systems are present without a fronting coastal engineering structure are the most susceptible to damage, as they can are more likely to be exposed and damaged by wave overwash and/or coastal erosion. Figure 5.7 shows an exposed sewer manhole at Egypt Beach that is subjected to wave attack during storms or higher than normal tides.

As part of the municipal sewer system, 9 pump stations exist in Scituate, 5 of which are located in close proximity to the open coastline, as shown in Figure 5.8. The three pump stations located on First, Second, and Third Cliff and one station located behind Musquashcut Pond have a smaller service area than the Egypt Beach and Sand Hills pump stations. Service areas for the pump stations are shown in the wastewater map presented in Appendix D.

All areas in the Town have above-ground electric utility lines except for the Glades Road, south of Bailey's Causeway. Due to the nature of nor'easters occurring in the colder months (generally November through April), the combined influence of frozen precipitation and strong winds can make these utilities susceptible to significant storm damage. As an example ice-covered utility lines and leaning utility poles during a 2014 storm event are shown in Figure 5.9. Downed lines are dangerous, can obstruct roads, result in power outages, and can be costly to repair depending on the severity of damage. Therefore, a higher priority rating is given to above-ground electric utility lines.

Table 5.6 presents the priority rating scheme with respect to damage susceptibility of public utilities and Table 5.7 summarizes the priority rating given to each study area. Egypt Beach, with sewer lines and no fronting coastal engineering structure, a crucial pump station, and above-ground utility lines, receives the highest priority rating.

Figure 5.7 An exposed sewer manhole at Egypt Beach (photo taken by Applied Coastal on May 12, 2016).

Figure 5.8 Pump stations and wastewater treatment plant in Scituate.

Figure 5.9 Frozen utility lines and leaning poles on Lighthouse Road. Photo from January 3, 2014 storm report by Jason Burtner (*mycoast.org*).

Table 5.6 Priority rating scheme for wastewater system along study area.

Priority Rating (0-5)	**Type and Location of Wastewater system**	**Existence of Pump Station**	**Type of Utility Line**
0	-	No	-
1	Septic system with fronting coastal engineering structure	-	Buried
2	-	-	-
3	Sewer system with fronting coastal engineering structure	Yes	Above ground
4	-	-	-
5	Septic system with no fronting coastal engineering structure Sewer system with no fronting coastal engineering structure	Yes, with a significant service area	-

Table 5.7 Priority rating of study areas with respect to damage susceptibility of public utilities.

Study Area	**Priority Rating**			
	Wastewater **0 – Low** **5 – High**	**Pump Stations** **0 – Low** **5 – High**	**Electrical** **1 – Low** **3 – High**	**Total** **1 – Low** **13 – High**
Minot Beach	3	0	3	6
North Scituate Beach	0	0	1	1
Surfside Road	3	0	3	6
Mann Hill Beach	5	3	3	11
Egypt Beach	5	5	3	13
Oceanside Drive	3	5	3	11
Cedar Point	5	0	3	8
First Cliff	0	3	3	6
Edward Foster Road	0	0	3	3
Second Cliff	0	3	3	6
Peggotty Beach	5	0	3	8
Third Cliff	0	3	3	6
Fourth Cliff	0	0	3	3
Humarock North	5	0	3	8
Humarock South	5	0	3	8

5.5 Emergency Egress

Emergency egress is critical to public safety along the Scituate shoreline to ensure evacuation is possible and to provide access for emergency vehicles, if required. Access to emergency egress was determined by evaluating the likelihood of encountering obstruction in order to travel safely to or from a particular area during storm conditions. Highest priority is given when access must go through a high repetitive loss area (i.e. repetitive loss damage properties exist on both sides of the road). Slightly lower priority is given where access is required to pass through a historically flooded area based on the Town's experience. For example, while First Cliff and Second Cliff are not prone to flooding due to their high landform elevation, travel in and out to the Cliffs require accessing Edward Foster Road, which is known to flooding during particularly high tides. Lastly, areas that are generally unobstructed are given a low priority rating. Mann Hill Beach was the only study area to be given a priority of 0 with respect to emergency egress access, where it is possible to access the area via shore perpendicular roadways. The priority rating scheme for emergency egress access is presented in Table 5.8. Table 5.9 summarizes the priority rating given to each study area.

Table 5.8 Priority rating scheme for emergency egress along study areas.

Emergency Egress	**Priority Rating (0-5)**
Generally unobstructed access	0
Access is through a historically flooded area	3
Access is through a high damage repetitive loss area	5

Table 5.9 Priority rating of study areas with respect to emergency egress.

Study Area	Priority Rating 0 – Low 5 - High
Minot Beach	3
North Scituate Beach	3
Surfside Road	5
Mann Hill Beach	0
Egypt Beach	3
Oceanside Drive	5
Cedar Point	5
First Cliff	3
Edward Foster Road	3
Second Cliff	3
Peggotty Beach	5
Third Cliff	3
Fourth Cliff	5
Humarock North	5
Humarock South	5

5.6 Breaching Potential

One unique aspect of barrier beaches is their potential to breach during a severe storm event. A breach is where the barrier beach is broken down by waves, allowing the seawater to extend inland. In most cases, storm-induced breaches heal naturally, as the inlet formed typically is not the most hydraulically efficient inlet to the back barrier estuary. Notable regional exceptions include the recent inlets to Pleasant Bay through the Nauset Barrier Beach in Chatham (as a result of storms in 1987 and 2007) and the formation of New Inlet between Third and Fourth Cliffs in Scituate (as a result of the Portland Gale in 1898). Due to the extensive development along the Scituate shoreline since the Portland Gale, there are concerns that even a temporary breach of a barrier beach could create significant public safety concerns, as well as prohibiting access to year-round residences "cut-off" by the breach. However, the main concern remains the formation of a permanent breach or a breach that would be difficult to close once a daily tidal regime becomes established, similar to a recent beach in Hatteras Island, NC (Figure 5.10).

Figure 5.10 Oblique aerial photography looking north along Hatteras Island, NC on September 21, 2003 (left, after hurricane Isabel), May 6, 2008 (middle) and December 4, 2009 (right), roughly two weeks after the storm. The yellow arrows point to the same location in each photograph. The inlet formed during Isabel was filled, and a protective dune was constructed in front of the new road (*image credit: USGS*).

Once a tidal inlet is well-established, the cost of closure increases dramatically, as either bridging an unstructured inlet or filling the inlet with dredged material is challenging in this dynamic environment. Generally, along developed shorelines, it is more cost-effective to proactively reduce the likelihood of breach formation, rather than attempting to fill the breach once it becomes established.

Potential for breaching within Scituate was identified at two locations: Lighthouse Road at the “neck” of Cedar Point and at the north end of Humarock. Potential for breaching at Lighthouse Road is considered to be low as the area is reinforced by a seawall and any breach would likely be short-lived and relatively straight-forward to fill. In addition, a breach at this location likely would not create a more efficient flow pathway into Scituate Harbor, so strong tidal currents likely would not be established.

At the north end of Humarock, formation of a breach along the barrier beach system is more likely as regular overwash has lowered and narrowed the barrier beach over time. Figure 5.11 summarizes the dune volume along Humarock North above the 100-year still water elevation. As a guideline developed by FEMA, it is anticipated that dunes with a cross-sectional area of less than 540 square feet above the 100-year still water elevation cannot withstand a 100-year storm event. The section between D Street and Atlantic Drive is of particular concern as barrier beach is extremely narrow, as narrow as 200 feet, and the dune volumes are only about 10% of the dune volume needed to withstand a major storm event. Table 5.10 presents the priority rating scheme and Table 5.11 summarizes the priority rating given to each study area with respect to breaching potential.

Figure 5.11 Dune volume along transects in Humarock North. Note the particularly vulnerable areas between D Street and Atlantic Drive.

Table 5.10 Priority rating scheme for breach susceptibility along study areas.

Potential for Breaching	Priority Rating (0-5)
No potential	0
Potential, reinforced with coastal engineering structure	3
Potential, no reinforcement	5

Table 5.11 Priority rating for study areas with respect to breach susceptibility.

Study Area	Priority Rating 0 – Low 5 - High
Minot Beach	0
North Scituate Beach	0
Surfside Road	0
Mann Hill Beach	0
Egypt Beach	0
Oceanside Drive	0
Cedar Point	3
First Cliff	0
Edward Foster Road	0
Second Cliff	0
Peggotty Beach	0
Third Cliff	0
Fourth Cliff	0
Humarock North	5
Humarock South	0

5.7 Coastal Engineering Structure Condition

Much of the extensive shorefront development along the Scituate shoreline is protected by coastal engineering structures. These structures have been developed over the past 100+ years, with the most extensive development occurring over the past 60 to 70 years. Due to the age of these structures, as well as the continued erosion of the beach at many locations, the structures may not be adequate to provide the level of protection needed. As the ability of the coastal engineering structure to perform its intended role is critical to the sustainability of infrastructure along the shoreline, the condition of the existing structure was deemed critical to the overall prioritization assessment.

Based on the 2007 (as well as the updated information from 2013) South Shore Coastal Infrastructure Inventory and Assessment Demonstration Project, a visual assessment of the condition of each coastal engineering structure was performed. The condition assessment was based on a five-level rating system:

- A Rating: Structures not requiring any maintenance, repair or rehabilitation cost and would not be expected to experience damage if subject to a major coastal storm event.
- B Rating: Structure requiring limited or no repair and would be expected to experience only minor damage if subject to a major coastal storm event. The value of these maintenance costs is assumed to be 10% of the construction cost.
- C Rating: Structures requiring moderate to significant level of repair or reconstruction and would be expected to experience significant damage if subject to a major coastal storm event. The structure is presumed to be effective under a major storm event. The value of the repair costs is assumed to be 50% of the construction cost.
- D Rating: Structures requiring significant level of rehabilitation or total reconstruction and would be expected to experience significant damage or possibly fail if subject to a major coastal storm event. The value of the repair costs is assumed to be 100% of the construction cost.
- F Rating: Structures requiring complete reconstruction and would expect to provide little or no protection from a major coastal storm event. The value of the repair costs is assumed to be 100% of the construction cost plus a coast to removal/disposal of the original structure.

The condition of the coastal engineering structures was updated in 2013 by CLE Engineering. Of the 55 coastal engineering structures in Scituate (coastal beaches and structures along Scituate Harbor were not considered), only one structure has a condition rating of A, 17 were rated B, 18 were rated C, 18 were rated D, and one structure was rated F.

An overall structure condition rating was developed for each study section by weighting structure length by the expected damage cost and dividing by the total structure length. No rating was given for study areas where no public coastal engineering structure is present. The priority rating scheme developed for the average coastal engineering structure condition is presented in Table 5.12; results for each study area are summarized in Table 5.13.

Table 5.12 Priority rating scheme for the average coastal engineering structure condition along the study areas (for areas with structures only).

Average Coastal Engineering Structure Condition	Priority Rating (0-5)
A – Excellent	0
B – Good	1
C – Fair	3
D – Poor	5

Table 5.13 Priority rating for study areas with respect to average coastal engineering structure condition (for areas with structures only).

Study Area	Priority Rating 0 – Low 5 – High
Minot Beach	1
North Scituate Beach	3
Surfside Road	3
Mann Hill Beach	Not Applicable
Egypt Beach	Not Applicable
Oceanside Drive	3
Cedar Point	3
First Cliff	5
Edward Foster Road	1
Second Cliff	3
Peggotty Beach	Not Applicable
Third Cliff	5
Fourth Cliff	5
Humarock North	Not Applicable
Humarock South	5

5.8 Prioritization Matrix

Based upon the analysis of the different prioritization criteria, it was possible to generate an overall rating for each section of shoreline. This rating scheme attempts to provide an objective process to assist the Town with focusing planning efforts to address shore protection along the Scituate shoreline. Table 5.14 summarizes the results of prioritization analysis and ranks the study areas from high to low priority. Ranking was based on the "priority rating value", which is calculated by dividing the sum of the prioritization scores by the maximum potential score. If the study area contained a public coastal engineering structure according to the South Shore Coastal Infrastructure Inventory and Assessment Demonstration Project, the ranking is based on the sum of criteria 1 through 6 (maximum of 48). For those areas without public coastal engineering structures are based on the sum of criteria 1 through 5 (maximum of 43).

It should be noted that the prioritization ranking utilized different weighting of the criteria based upon importance relative to the overall storm damage concerns. Specifically, damage susceptibility of private properties had scores that ranged from 0 to 15 and damage susceptibility to public utilities had scores ranging from 1 to 13. The remaining categories received maximum scores of 5. Based on this approach, observed and potential susceptibility of direct damage to private infrastructure and public utilities were deemed most critical for prioritizing shore protection needs.

Table 5.14 Ranked prioritization matrix (high to low) for the study areas.

Study Area	1 Damage Susceptibility: Private Properties	2 Landform Elevation	3 Damage Susceptibility: Public Utilities	4 Emergency Egress	5 Breaching Potential	6 Structure Condition	Total	Priority Rating Value
Oceanside Drive	15	5	11	5	0	3	39	0.813
Humarock North	10	5	8	5	5	-	33	0.767
Cedar Point	10	5	8	5	3	3	34	0.708
Peggotty Beach	10	5	8	5	0	-	28	0.651
Surfside Road	12	5	6	5	0	3	31	0.646
Egypt Beach	8	3	13	3	0	-	27	0.628
Humarock South	5	3	8	5	0	5	26	0.542
Minot Beach	9	1	6	3	0	1	20	0.417
Mann Hill Beach	4	3	11	0	0	-	18	0.419
North Scituate Beach	7	3	1	3	0	3	17	0.354
First Cliff	0	1	6	3	0	5	15	0.313
Third Cliff	0	1	6	3	0	5	15	0.313
Edward Foster Road	4	3	3	3	0	1	14	0.292
Fourth Cliff	0	1	3	5	0	5	14	0.292
Second Cliff	0	1	6	3	0	3	13	0.271

6.0 ENGINEERED SHORE PROTECTION APPROACHES

A number of potential shore protection options were evaluated to provide the basis for the site-specific assessment of alternative for each shoreline sections. The list of alternative shore protection strategies includes numerous “hard” (e.g. seawall) and “soft” (e.g. beach and dune nourishment) coastal engineering techniques, as well as potential innovative approaches (e.g. boulder dikes). In addition, the baseline alternative consists of maintaining the *status quo* of continuing to repair infrastructure as needed following storm damage and/or demonstrable failure.

Initially, each shore protection strategy was broadly reviewed relative to its applicability for the Scituate shoreline. Within this context, the shore protection options were evaluated relative for (a) the ability to provide the necessary level of shore protection, (b) the anticipated environmental impacts and associated ability to advance the option through the environmental regulatory process, and (c) the overall cost of the alternative including both initial construction and maintenance costs. Due to geological framework of the natural coastline, as well as anthropogenic changes that have occurred to provide shore protection, a wide variety of approaches exist for addressing coastal sustainability issues. The goal of providing an initial assessment of this broader range of shore protection approaches was to ensure that a broad range of approaches were carried forward into the site-specific assessment. While the assessment ensured inclusion of this broad range of approaches, it should be noted that the initial evaluation of various technologies also allowed for elimination of shore protection techniques that were deemed to have “fatal flaws” either due to excessive environmental impacts and/or being cost-prohibitive.

6.1 Maintain Status Quo

To maintain the status quo, the Town would continue repairing and maintaining the shore protection structures in a reactive manner. The cost of ongoing maintenance for failing structures has minimal benefits and does not prevent the continued overtopping and storm damage to homes and public infrastructure. By not pro-actively addressing the shore protection needs of the Town, there remains and increasing threat to public safety, health, and welfare. Sea level rise and the chronic beach erosion along the shoreline may result in loss of tax revenue corresponding to lowering of property values and the loss of recreational resources. This alternative would place the residential properties and public infrastructure at increasing risk as the existing shore protection continues to degrade and the beaches continue to erode.

Maintaining the status quo indicates that implementation of repairs and/or minor improvements will generally be performed in a reactionary manner, with storm damage repairs performed on an emergency basis. Therefore, the cost of this limited maintenance can be substantial, especially if emergency repairs are required along any critical shoreline location (e.g. breaching of Central Avenue in Humarock or collapse of the seawall along Oceanside Drive). Recent efforts by the Town to repair and slightly improve seawalls along Oceanside Drive have been costly, due primarily to the proximity of dwellings to the seawall crest.

To provide a baseline for assessment, a 50-year time horizon for maintaining the status quo was developed based upon anticipated FEMA claims and repairs/maintenance to existing shore protection structures. The overall results of this analysis for the Scituate shoreline are shown in Table 6.1. These total cost is shown at the end of the 50-year period assuming a 3% inflation rate from the present.

Table 6.1 Estimated cost of Maintain Status Quo approach over 50 years.

Study Area	Projected FEMA Repetitive Loss Claims over 50 Years	Anticipated Costs for Maintaining Status Quo over 50 Years (see notes)	Total	Notes
Minot Beach	$12,967,492	$18,164,035	$31,131,527	Seawall maintenance
North Scituate	$9,865,854	$22,035,059	$31,900,914	Seawall maintenance
Surfside Road	$41,494,106	$32,457,047	$73,951,153	Seawall maintenance
Mann Hill	$1,982,300	$2,263,200**	$4,245,500	Value of homes
Egypt Beach	$7,529,427	$0	$7,529,427	
Oceanside Drive	$127,389,808	$119,406,200	$246,796,008	Seawall maintenance
Cedar Point	$20,918,650	$15,484,096	$36,402,746	Seawall maintenance
First Cliff	$0	$10,273,102	$10,273,102	Revetment maintenance
Edward Foster	$62,553	$11,613,072	$11,675,625	Seawall maintenance
Second Cliff	$0	$13,250,813	$13,250,813	Revetment maintenance
Peggotty Beach	$8,182,214	$8,674,500**	$16,856,714	Value of homes
Third Cliff	$0	$28,586,023	$28,586,023	Revetment maintenance
Fourth Cliff	$0	$4,317,681	$4,317,681	Revetment maintenance
Humarock North	$27,263,219	$43,305,176	$70,568,395	Road maintenance and road breach
Humarock South	$8,867,370	$24,119,457	$32,986,827	Seawall maintenance
Total	**$266,522,993**	**$353,949,460**	**$620,472,453**	

** 2016 assessed value of homes located in study area

It should be noted that this monetary value represents the cost of each future event based upon the 3% annual inflation; therefore, the numerical totals can be misleading, as all expenses represent future costs at the time they are incurred. Seawalls and revetments along the shoreline are assumed to require 5% maintenance annually over the life of the structure, with one major reconstruction events required over the 50-year cycle, each requiring 50% of the initial construction cost of the structure. It is anticipated that over the 50-year cycle, Humarock North will require a breach repair estimated at $30,000,000, annual clearing of Central Avenue, and reconstruction of Central Avenue twice of the planning cycle (each event estimated at 50% of an initial roadway construction. The estimated cost for the breach repair was based upon rapid placement of a temporary pile-supported bridge to maintain access, as well as filling of the breach with beach/dune compatible material from an upland source.

By only maintaining the status quo, it is anticipated that all dwellings along both Mann Hill Beach and Peggotty Beach are lost over the next 50 years. Therefore, this cost is shown as assessed value of each property. FEMA losses are anticipated to continue at the average rate observed over the past 25 years (1991-2016).

Overall, this analysis is conservative over the 50-year planning horizon, as it does not account for (a) sea level rise, (b) lowering of the beach fronting coastal engineering structures, and (c) ongoing erosion of the beach systems (except for Mann Hill and Peggotty Beaches). This analysis provides a basis for comparison of the shoreline improvements that could potentially address coastal sustainability along the Scituate shoreline, provided in Section 7 of this report. It should be noted that maintaining the status quo provides no improvement to the overall resilience of the Scituate coast. See Table 6.2 for a list of the pros and cons associated with Status Quo approach.

Table 6.2 Pros, cons, and challenges of the Maintain Status Quo approach.

Pros	**Cons**
• None	• Threat of endangerment of public safety, health, and welfare • Continued overtopping and storm damage to homes and public infrastructure • Further decay and potential failure of existing revetment and seawall structures • Increased future costs to repair or rehabilitate the structure • Loss in tax revenue (property values) • Continued loss of recreational resource • Ongoing maintenance costs for failing infrastructure with minimal benefits
Challenges • Public buy-in • Ability for the Town to provide adequate emergency response to storms	

6.2 Seawalls and Revetments

Seawalls and revetments are currently the main form of shore protection in Scituate; nearly 50% of the Scituate shoreline is armored with these “hard” engineering structures. Seawalls and large-scale armor stone revetments can provide increased wave dissipation, reduced wave overtopping, and increased storm protection. The storm protection is considered relatively short-term because seawalls and revetments (to a lesser extent) cause accelerated lowering of the fronting beach over time. This lowering of the beach is caused by increased wave reflection of the vertical or steeply sloping face of the structure relative to a natural beach. A lower beach elevation results in waves breaking closer to the shoreline with increased overtopping potential. Unlike groins and breakwaters, which may protect adjacent updrift beaches or improve the longevity of a beach fill, seawalls and revetments only protect the land directly behind them. Figure 6.1 shows that if there is no shore armoring in place, the eroding beach will move landward to maintain the width of the beach. With a seawall in place, the fronting beach becomes narrower with continued erosion, as the beach profile cannot migrate landward due to the presence of the seawall.

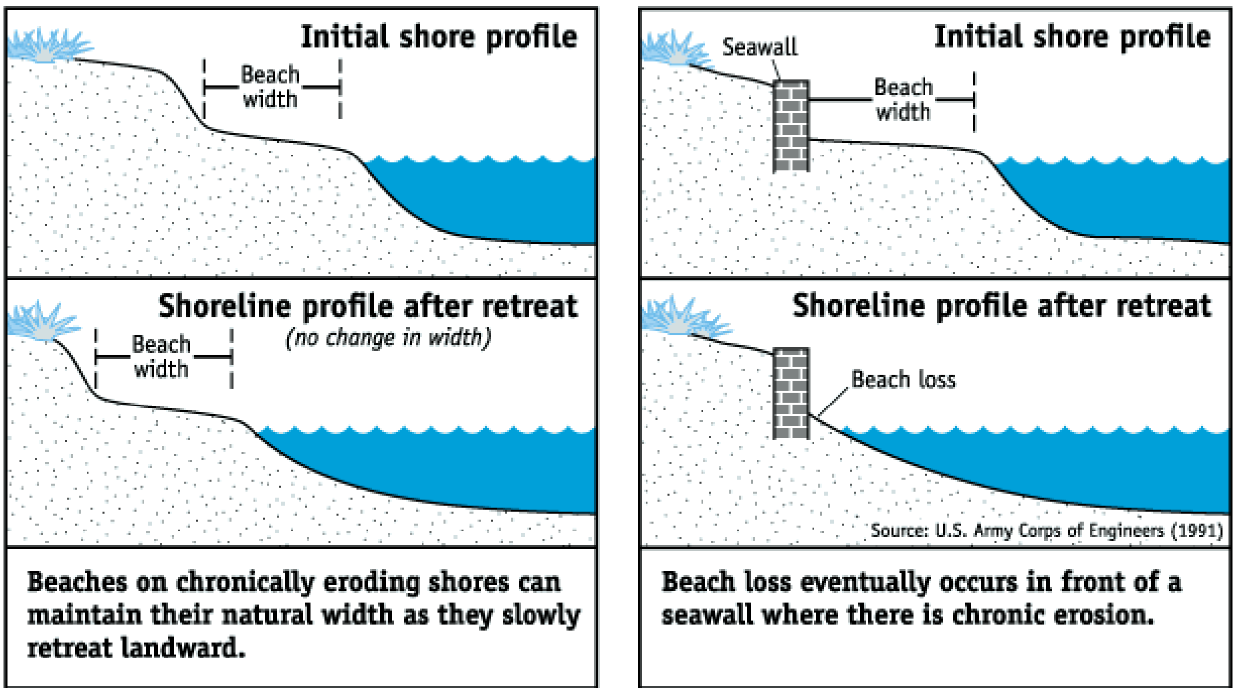
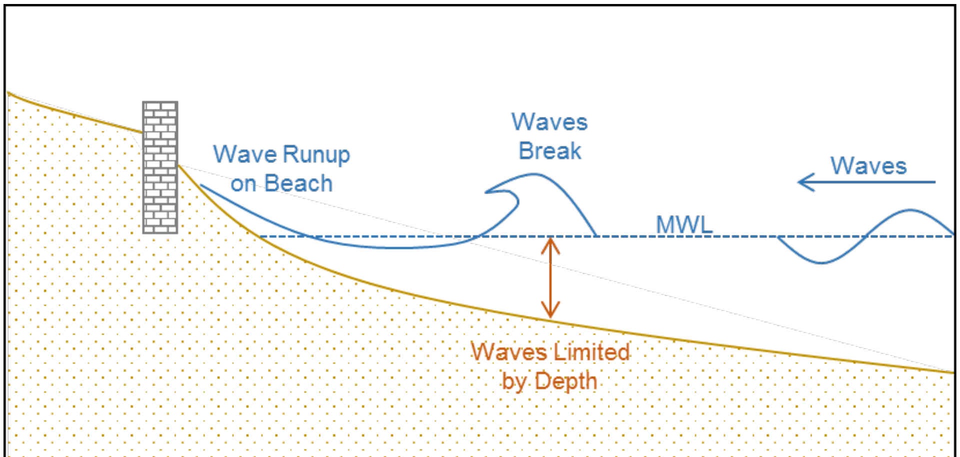
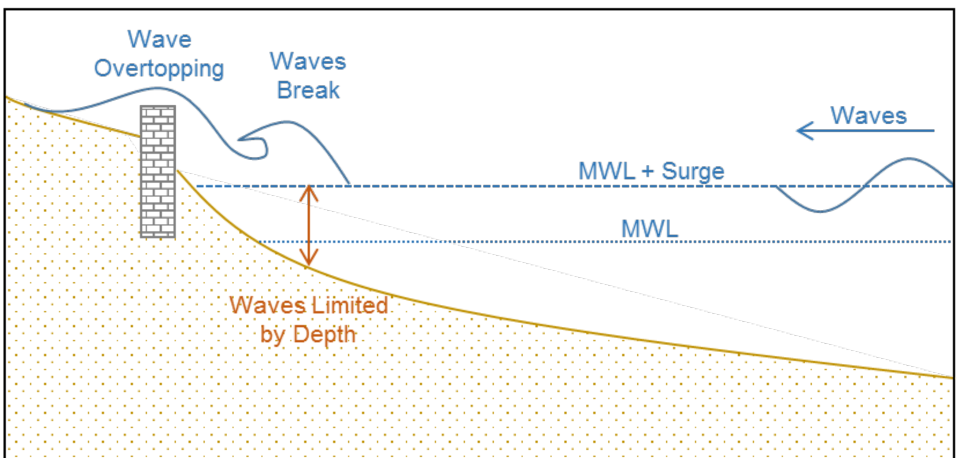


Figure 6.1 Chronic beach erosion on unhardened shores (left) and with seawalls in place (right) (*image credit: U.S. Army Corps of Engineers*).

During calm wave conditions, the waves may runup on the narrow fronting beach. However, as shown in Figure 6.2, the waves break further inland during storm conditions and without sufficient beach width to dissipate the wave energy, the waves will tend to overtop the seawall and lower the beach. This beach lowering is due to the magnified erosion/scour force of the waves as they reflect from the structure, and to the deficiency of bank sediment protected by the wall that otherwise could help replenish the fronting beach (Silvester and Hsu, 1993). In an erosive environment, a seawall or revetment may accelerate the recession rates of adjacent beaches (Shore Protection Manual, 1984). Toe scour and flanking at the ends of the wall may threaten the structure as erosion continues. While the designed revetment may provide storm protection, the lack of high tide beach and low sediment supply along the Scituate shoreline will likely lower the profile in front of the revetment, causing stones to slump and loosen. Therefore, the seawalls and revetments require regular maintenance and repairs to maintain their effectiveness. If the structure and/or beach are not maintained, long-term erosion can often lead to catastrophic failure of the structure, typically during a storm event. Prior to failure, seawalls and revetments often do not exhibit signs of structural inadequacy, which can lead to a "false sense of security" for property owners in areas fronted by these shore protection measures.



Waves break further from shore and dissipate over the beach.



Waves break closer to shore and have less room to dissipate, resulting in waves overtopping the seawall, beach lowering and inland damage.

Figure 6.2 Effect of storm surge on wave and armored shorelines.

From a regulatory stand-point, the seawall and revetment approach is likely only permittable in locations where a structure exists. In addition, state and federal environmental permitting can be difficult, potentially requiring extensive compensatory mitigation, if the existing structure footprint requires expansion to achieve design requirements. The Wetlands Protection Act generally prohibits new coastal engineering structures on barrier beaches or to protect dunes.

An engineering analysis was performed to compare the overtopping rates of the existing seawalls and/or revetments to an enhanced structure design, see Figure 6.3. The enhanced design involves raising the seawall, if it exists, by 2 feet and adding a larger revetment structure that will reduce storm wave overtopping during the 100-year storm to acceptable volumes. Guidance from U.S. Army Corps of Engineers (USACE, 2002) states that damage to the pavement behind the structure is prevented when average overtopping discharge is less than 0.5 $ft^3/s/ft$ (0.05 $m^3/s/m$), see Figure 6.5. An example of pavement damage caused by wave overtopping in Hull, MA is presented in Figure 6.4. The effectiveness of the existing and enhanced structure were evaluated for the 100-year storm and the 100-year storm with 2 feet of sea level rise. The enhanced structure also has the benefit of improving the structural condition of the existing seawalls and revetments along the Scituate shoreline, many of which have a "D - Poor" condition rating based on the 2007 and/or 2013 South Shore Coastal Infrastructure Inventory and Assessment Demonstration Project.

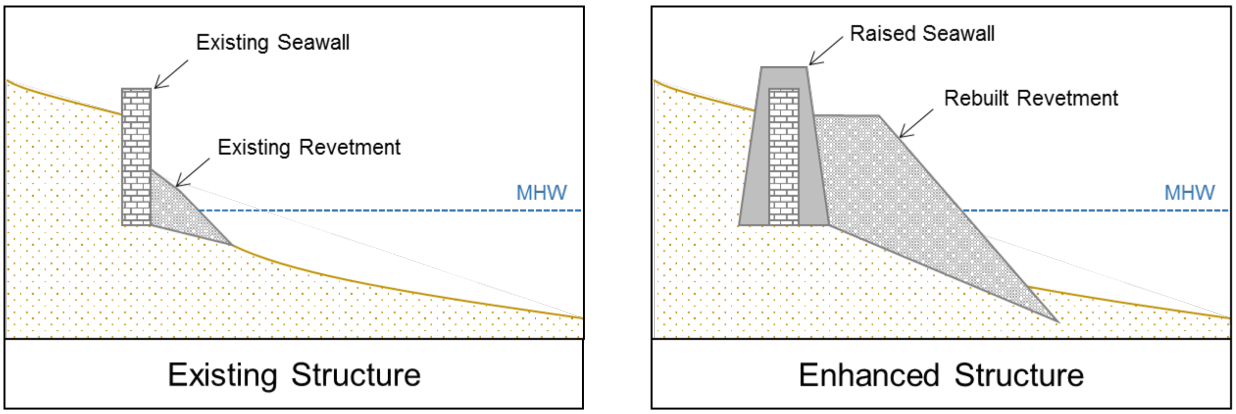


Figure 6.3 Schematic of the existing seawall and revetment structure (left) and the enhanced structure (right) which involves raising the seawall and a larger fronting revetment to dissipate wave energy and reduce overtopping.

For the cases where the water and surge level is higher than the revetment berm, the wave dissipation capacity of revetment was conservatively assumed to be negligible and the empirical equation for overtopping on a vertical wall by Franco and Franco (1999) were utilized. The Franco and Franco equation for average overtopping discharge, *Q*, is:

$$Q = 0.082\sqrt{gH_s^3}\exp\left(-3\frac{R_c}{H_s}\right)$$

where *g* is the acceleration due to gravity, H_s is the significant wave height at the revetment toe, and R_c is the distance between the seawall elevation and the still water level. For the purposes of this study, depth limited wave heights were used ($H_s = 0.78 \times$ *water depth*).

Where the water and surge level was below the revetment berm, overtopping was estimated using the empirical equation by Pedersen (1996). Pedersen is valid for rock-armored permeable slopes with a berm in front of the seawall. The Pedersen equation for *Q* is:

$$Q = 3.2 \cdot 10^{-5}\left(\frac{L_{om}^2}{T_{om}}\right)\left(\frac{H_s}{R_c}\right)^3 \frac{H_s^2}{A_c B cot\alpha}$$

where L_{om} is the deep water wave length with respect to T_{om}, T_{om} is the mean deep water wave period, A_c is the distance between the berm elevation and the still water level, *B* is the berm width, and *cotα* is the revetment slope.

Overtopping on straight and bermed revetments on the Cliffs (no seawall) was calculated using Owen (1980, 1982). The Owen formula for Q is:

$$Q = agH_sT_{om}\exp\left(-b\frac{R_c}{H_s}\sqrt{\frac{s_{om}}{2\pi}}\right)$$

where a and b are coefficients based on the revetment slope and s_{om} is the deep water wave steepness corresponding to the mean wave period.

Lastly, a wave transmission equation by van der Meer and d'Angremond (1991) was used on the design of a revetment dike in Minot Beach. The wave transmission factor, C_t, is calculated by:

$$C_t = \left(0.031\frac{H_s}{D_{n50}} - 0.24\right)\frac{R_c}{D_{n50}} + b$$

$$b = -5.42s_{op} + 0.0323\frac{H_s}{D_{n50}} - 0.0017\left(\frac{B}{D_{n50}}\right)^{1.84} + 0.51$$

where D_{n50} is the median of nominal rock diameter, s_{op} is the deep water wave steepness corresponding to the peak wave period, and B is the width of the dike crest. The wave transmission factor was used to determine the wave height on the landward side of the dike and then the appropriate wave overtopping equation listed above was applied.

Figure 6.4 Damage to pavement caused by wave overtopping in Hull, MA (*photo credit: MCZM*).

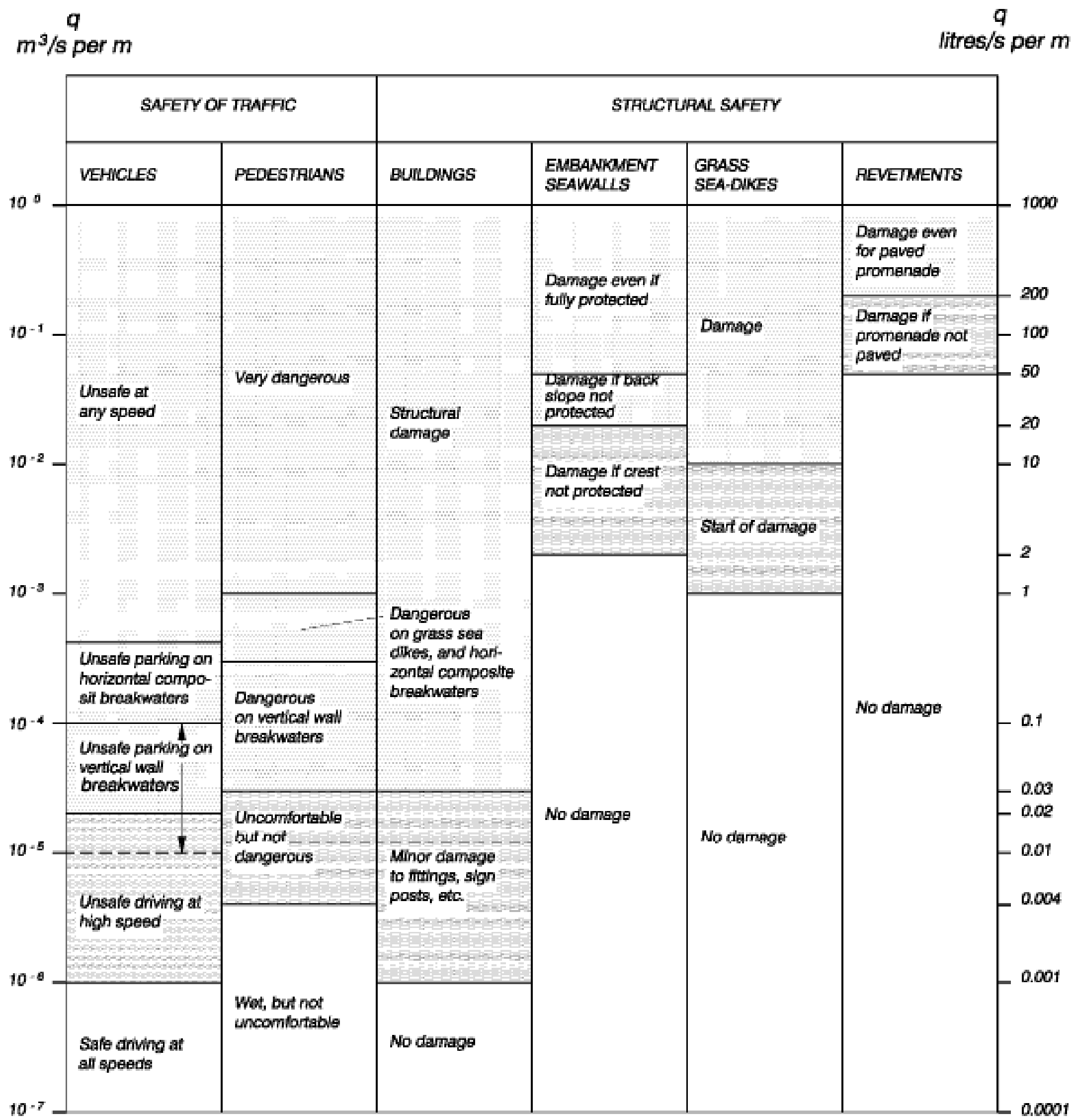


Figure 6.5 Critical values of average overtopping discharges (USACE, 2002).

The cost to raise the seawall and rebuild the stone revetment to an adequate elevation that would provide sufficient storm protection was estimated based on previous construction costs provided by the Town of Scituate during the seawall reconstruction on Oceanside Drive. The cost was estimated at $8,000 per linear feet based on the challenges of working within close proximity to existing buildings and roads. In general, these projects have required installation of temporary steel sheeting prior to seawall construction to ensure stability of landward infrastructure during construction. In some areas, it is not clear whether even this costly approach is possible due to the location of dwellings directly adjacent to the seawall. If publically funded, the project will require public access easements from the adjacent homeowners. See Table 6.3 for a list of the pros and cons associated with the Seawalls and Revetments approach.

Table 6.3 Pros, cons, and challenges of the Seawalls and Revetments approach.	
Pros • Increased wave dissipation, reduced wave overtopping, and increased short-term storm protection • Improved structural condition of seawall • Minimal impacts to nearshore and offshore benthic and aquatic resources if footprint is not expanded	**Cons** • Provides false sense of security • Will not restore beaches • Accelerated beach lowering • Wave overtopping during severe events may still cause damage • Impacts to benthic resources immediately in front of the structure during construction • Impacts to community during construction • Requires regular maintenance
Challenges • Seawalls are very close/attached to buildings • Easements required if publicly funded • Permitting difficult if existing structure footprint expanded • New structures not permittable on barrier beaches/dunes • Significant cost	

6.3 Beach Nourishment

Beach nourishment would add sediment seaward of the seawall/revetment or along a segment of barrier beach to create a wider beach to dissipate wave energy, thereby increasing protection to infrastructure and property currently threatened by overtopping and storm damage. In this case, beach nourishment refers to an engineered beach that is designed to withstand storm conditions including the effects of storm surge and wave action (see an example in Figure 6.6). This large volume of beach compatible sediment is designed to last several years, where the design life is dependent on the local sediment transport dynamics and berm overtopping potential. For the significant storm conditions experienced along the Scituate shoreline, the elevation of the shore protection beach berm likely will be within 2 feet of the existing seawall crest along much of the shoreline, with a mid-tide beach width of at least 150 feet. It should be noted that the engineered beach nourishment projects for shore protection purposes are substantially larger than the Humarock Beach emergency berm placement in 1994. In this study, the beach nourishments are engineered to withstand a 50-year storm event.

Once nourishment material is in place, coastal processes rework the nourishment material to create an equilibrated beach profile, generally with a “flatter” profile the initial placement slope. The ongoing sediment transport will transport the nourishment material both cross-shore and alongshore. The cross-shore component is critical, as nourished material is redistributed to re-establish the nearshore profile shape that typically has become over-steepened as a result of long-term loss in the natural sediment supply. Evaluation of the beach nourishment as a potential approach also must consider the downdrift transport of sediment and the potential impacts including potential to block outfalls or inhibit safe navigational as a result of shoaling. Due to the ongoing migration of sediment to adjacent shorelines as well as offshore, a maintenance plan for re-nourishment and/or backpassing will be necessary for this alternative to be effective as a long-term management strategy. Maintenance should also be anticipated after significant storm events to replenish eroded sections of the beach to ensure stability and provide wave dissipation during future storm events. Repairs and maintenance funds may be provided by FEMA after federally declared disasters if nourishment is consistently monitored and maintained (i.e. a maintenance plan with financial commitments is in place).

Figure 6.6 Placement of large-scale beach nourishment at Winthrop Beach, Massachusetts, where the designed beach berm was approximately 7 feet above the local high tide level (phot by Applied Coastal).

Large-scale beach nourishment would restore the historical beaches along much of the Scituate shoreline. The nourishment would enhance storm protection for the homes and infrastructure landward of the existing revetment and seawall. The beach nourishment considered in this study would have a design life of approximately of 2 to 20 years, depending on the sediment transport rates in the study areas, but would require periodic and regular maintenance and re-nourishment to remain a viable shore protection approach. Beach nourishment may also be coupled with other shore protection approaches, such as a constructed dune, shown in Figure 6.7.

Nourishment is accompanied with some potential adverse impacts that must be carefully minimized and/or mitigated. For example, the nourishment template would cover inter-tidal and sub-tidal habitats which would affect the benthic community and nearshore resources areas. However, the area impacted by nourishment has higher wave energy than areas where offshore structures could be placed to mitigate storm damage; therefore, the benthic community in the beach area tends to be dominated by short-lived species. Beach nourishment addresses the sediment starvation concerns along the shoreline, provides added longevity to the existing shore protection infrastructure, protects the existing development from coastal flooding and storm damage, and in the long-term restores a functional beach system that may enhance wildlife and shellfish habitat.

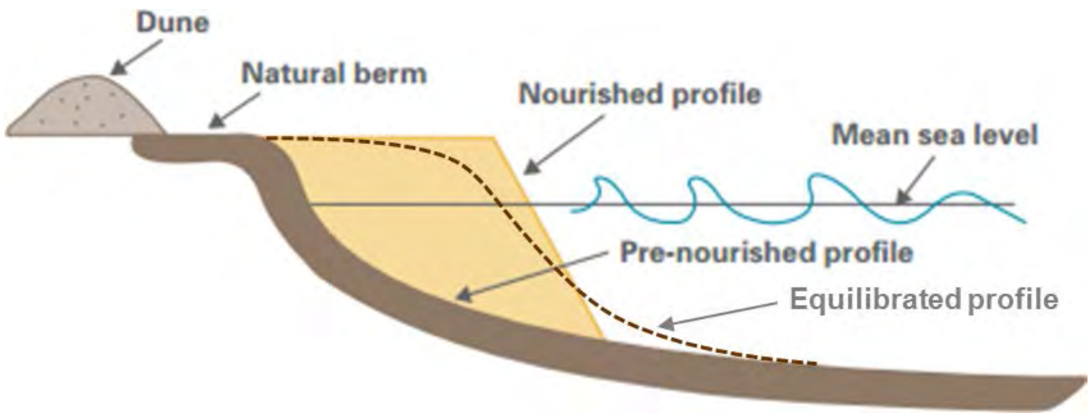


Figure 6.7 Cross-sectional schematic of a beach profiles before nourishment, immediate after nourishment, and after equiibration (*image credit: American Shore and Beach Preservation Association - modifed*).

A beach nourishment template should be constructed with a berm elevation that is high enough to prevent regular overtopping by waves to ensure the optimum design life and protective capacity during storms. The berm elevation is determined by an analysis of wave run up during storms. As waves break on the beach, water will rush up the beach face. The elevation that wave runup will reach is dependent on characteristics of the breaking wave and the slope of the beach. If waves regularly overtop the berm crest, erosion of the cross-shore profile will be accelerated, increasing the risk of storm damage to onshore infrastructure. A method described by FEMA (2007) for calculating the 2-percent incident wave runup ($R_{2\%}$) on natural beaches was used. By this method, runup is calculated as

$$R_{2\%} = 0.6 \frac{m}{\sqrt{H_0 / L_0}} H_0$$

where m is the beach slope, H_0 and L_0 are the offshore significant wave height and wavelength, respectively. For the purposes computing runup on sand and cobble beaches, this method is considered to provide a conservative estimate since the increased porosity of cobble verses sand-only beaches would lead to reduced maximum runup elevations.

The design life of the beach nourishment is estimated using wave and sediment transport modelling of the design template.

Sand/gravel/cobble-mix and cobble nourishments have been considered, if compatible with the in-situ beach material. The nourishment project would require a large volume of sediment that would likely have to come from upland sources. Constructing the nourishment would require the material to be transported by truck to the site. With the volumes anticipated for the nourishment, a significant number of daily deliveries will be required to deliver the nourishment within the project timelines. The truck transport of material to the site could have a significant short-term impact to the community. The impacts would have to be thoroughly investigated, documented, and then mitigated for during the planning, permitting and implementation.

The cost of sourcing and placing the nourishment material is estimated from a large-scale nourishment project utilizing a sand/gravel/cobble borrow in Winthrop, Massachusetts where the cost was approximately $34 per cubic yard. Public access easements will be required from adjacent homeowners if the project is publically funded. See Table 6.4 for a list of the pros and cons associated with the Beach Nourishment approach.

Table 6.4 Pros, cons, and challenges of the Beach Nourishment approach.

Pros	**Cons**
• Restoration of the lost aerial and sub-tidal beach • Nourishment will provide wave dissipation and storm protection • Nourishment will re-establish sediment supply to adjacent beaches • Creation of a recreational resource • Repairs and maintenance funds may be provided by FEMA if nourishment is monitored and maintained	• Impacts from covering of inter-tidal and sub-tidal habitats, benthic communities, and nearshore resources areas • Regular and episodic maintenance and re-nourishment required • Impacts to the community during construction
Challenges • Easements required if publicly funded • Permitting concerns due to large "footprint" • Significant cost – especially if upland source needed	

6.4 Constructed Dunes

Constructed dunes may be appropriate for areas where the existing high-tide beach is relatively wide and space exists to increase dune elevation within the natural planform of the overall beach system. For some areas in Scituate, the existing dune crest has migrated landward of existing homes. In these cases, dune construction would need to be accompanied by beach nourishment to ensure long-term stability of the barrier beach system. Dunes can provide storm damage protection during smaller storms by reducing flooding and overtopping. Regular maintenance and re-nourishment is required to maintain sufficient volume, however, coupled with adjacent beach nourishment, the dune nourishment life can be enhanced (Figure 6.8).

The minimum dune volume required to prevent dune overtopping during a storm is estimated using FEMA's "540 rule" (Figure 6.9). The "540 rule" states that dune volume is sufficient to protect against a 100-year storm when the volume seaward of the dune crest and above the 100-year still water elevation is greater than 540 square feet per linear foot of dune.

Figure 6.8 Mixed sediment dune at Mann Hill Beach, where the existing homes are located seaward of the dune crest (photo taken by Applied Coastal on May 10, 2016).

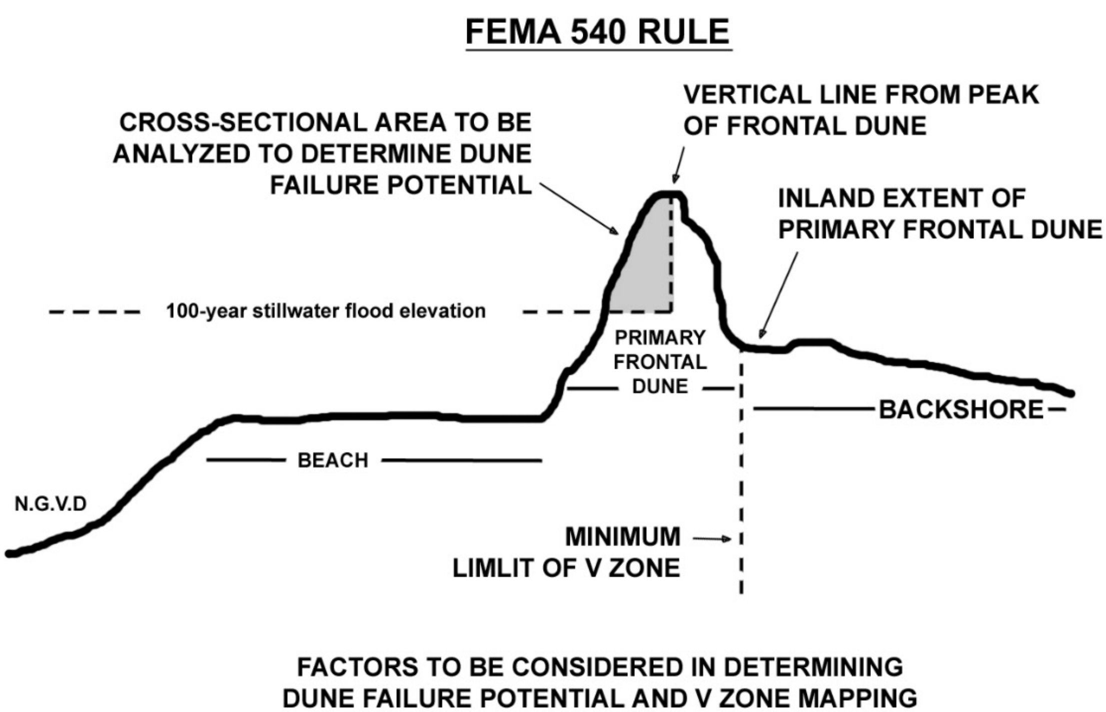


Figure 6.9 FEMA 540 Rule for determining dune failure potential (*image credit: FEMA*).

More recently, FEMA's Coastal Construction Manual (2000) recommended that the target dune reservoir volume be increased to 1,100 square feet per linear foot of dune based on more recent post-storm surveys. In this study, the 540 Rule is used because the mixed sediment material (cobble, gravel, and sand) prevalent within most dune systems along Scituate are less mobile than the sandy dunes surveyed in developing the 1,100 square feet volume standard.

In Scituate, constructed dunes may be appropriate at Mann Hill Beach, Egypt Beach, Peggotty Beach, and along Humarock (north and south). Like beach nourishment, a publically funded dune nourishment project will require public access easements from the property owners. Educating the public will be important to keep people from walking on the dunes which may lead to lowering of the profile at a specific location, thereby enhancing breaching potential. See Table 6.5 for a list of the pros and cons associated with the Constructed Dunes approach.

Table 6.5 Pros, cons, and challenges of the Constructed Dunes approach.

Pros	**Cons**
• Storm damage reduction during smaller storms • Reduced flooding and overtopping • Dune nourishment life can be enhanced by adjacent beach nourishment	• Regular maintenance and re-nourishment required • Dune alone may not provide enough protection from larger storms
Challenges • Easements required if publicly funded • Education of the public required to keep people off dunes	

6.5 Offshore Breakwaters

Offshore breakwaters could be constructed to dissipate wave energy before it reaches the Scituate shoreline. The breakwater would extend off the bottom into the water column to trigger wave breaking as storm waves approach the shoreline from the Atlantic Ocean. The shadow area behind the breakwater (in the "lee" of the structure) exhibits calmer wave conditions than unprotected areas, potentially allowing for sediment deposition in the sheltered region.

Historically, the Minot area was the subject of an offshore breakwater concept that was developed by a civil engineer out of Quincy, MA (Fred Tupper) in 1911. This structure was intended to be made from armor stone and connect the series of bedrock outcrops offshore of this area. Due to the *in situ* water depths adjacent to these outcrops, the estimated structure height would be approximately 60 feet, with an estimated total cost of $130 million if it were constructed today.

A few criteria must be met for a breakwater to be effective at breaking storm waves and dissipating wave energy. The breakwater must be designed with enough profile (vertical height) off the bottom and large enough crest width relative to wave length (width in offshore direction) to cause storm waves to trip and break. A low and/or narrow structure will not trigger wave breaking and therefore not be a viable shore protection alternative. The profile height of the structure becomes an issue with large tide ranges and/or substantial storm surges. The crest of the structure must be set at a height to cause wave breaking during storms when the water levels are elevated and can be further amplified by high tides.

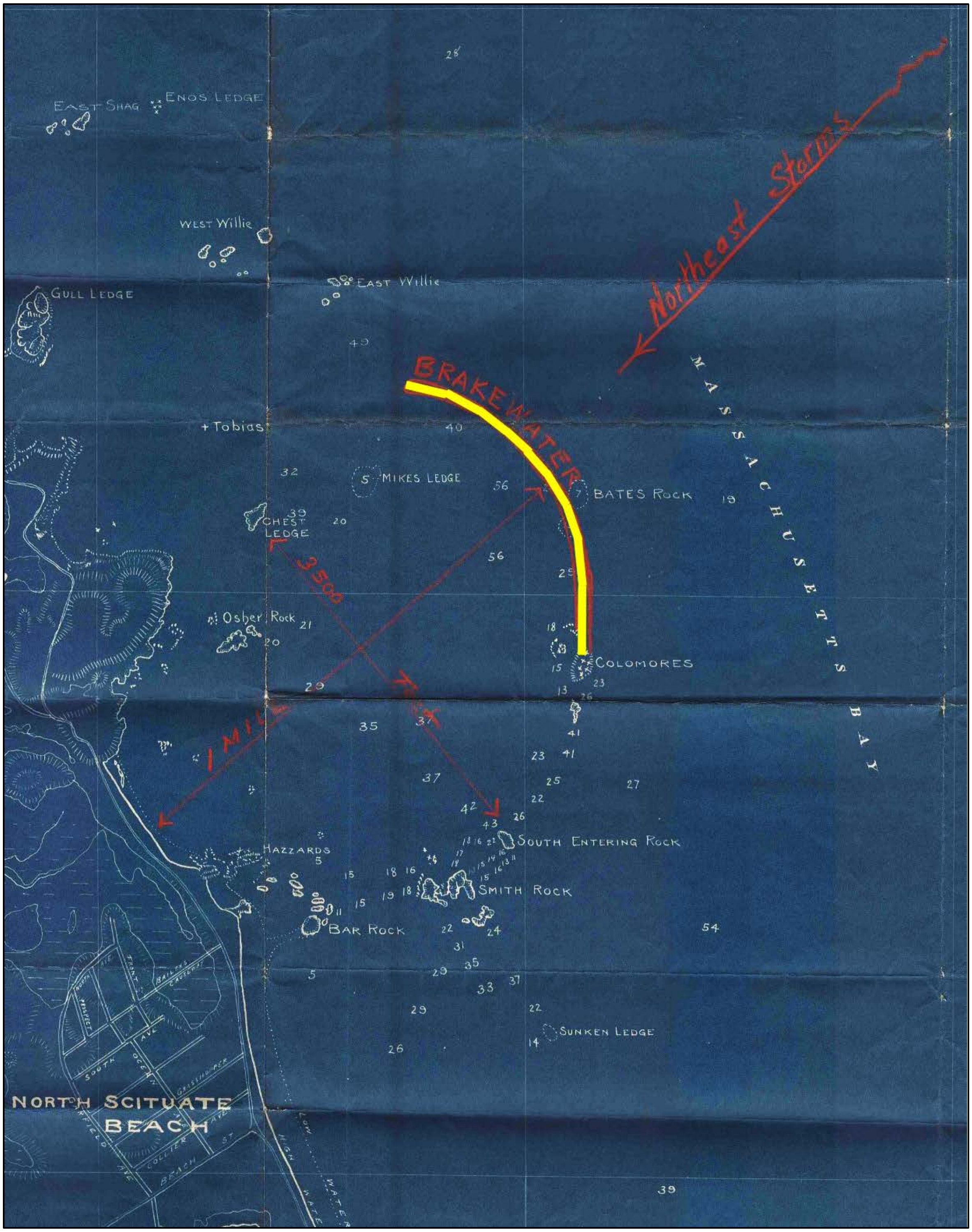


Figure 6.10 Offshore breakwater concept for the Minot Beach area, where the approximate 3,500 feet breakwater would serve to protect about ½ mile of beach (*Source: 1911 Plan from Fred Tupper, C.E.*).

A design estimate for a breakwater at North Scituate Beach and Surfside Road was completed during the permitting phase of the North Scituate Beach Nourishment Project. Based on the dimensions of the emerged stone breakwater at Winthrop Beach, MA, the breakwater would be situated along the -15 foot NAVD88 contour, approximately 400 feet from the shoreline. The approximate length of the breakwater would be 3,300 feet long comprised of 330-foot segments and 100-foot gap widths. A total of 8 segments would be used to span the shoreline. Crest width and elevation are 12 feet and 8.3 feet NAVD88, respectively. Figure 6.11 illustrates the conceptual placement of the breakwaters. The crest would be emerged at all stages of the tidal cycle and submerged by approximately 2 feet during the 100-year storm, allowing for sufficient vertical height to break the storm waves. Lowering the crest of the structure would negate the effectiveness of the structure to perform as an effective breakwater during storm events. The total volume of stone required to construct the breakwater would be approximately 133,000 CY with a structure footprint of approximately 6.3 acres. At approximately $125 per ton to supply and place stone for the breakwater, the estimated cost is $22.2 million.

An effective breakwater would likely require a large emergent rubble-mound breakwater type system. The structure would occupy a large area of the bottom, impacting marine habitat and resources. Attempting to utilize other technologies, such as Wave Attenuation Devices™ and Reef Balls™ (see Section 6.12), is not preferred due to concerns about their effectiveness due to the large tidal range at the site, in addition to significant storm surges and waves encountered during storm events. The structure would also have to be located relatively close to shore due to the sloping offshore bathymetry. Moving the structure into deeper water would increase the size and cost associated with a structure of this type. The sediment-starved nature of the Scituate coastline likely would not provide adequate sediment to produce a deposition area in the lee of the breakwater; however, the breakwater would help to extend the design life of the nourishment.

Overall, the large-scale impact of the structure within a natural area that is designated as habitat for endangered/threatened species and suitable for shellfish will prove unworkable if other viable approaches are available with fewer adverse environmental impacts during the permitting process. Moreover, the substantial cost of the emergent breakwaters, as well as their long-term impacts to the natural sediment transport regime, make this shore protection approach cost-prohibitive relative to other approaches. Therefore, offshore breakwaters were not considered as a viable solution.

Submerged breakwaters would have a similar overall impact as emergent breakwaters; however, the structure “footprint” likely would be somewhat smaller. Due to the 10-foot tide range at the site, structures submerged during all phases of the tide cannot provide meaningful wave height reduction and the associated storm damage prevention.

Figure 6.11 Approximate breakwater placement along the North Scituate Beach and Surfside Road shoreline. Dashed yellow outline shows the approximate footprint of the breakwater. This breakwater configuration was considered as a potential shore protection approach in the alternatives analysis for the North Scituate Beach Nourishment Project.

6.6 Boulder Dike

Along the Scituate shoreline, erosion of glacial deposits has created the observed sediment variability within the regional sediment transport area. In areas where glacial drumlins have eroded, the coarser-grained material tends to stay in place and the finer-grained material is removed by wave action to form down-drift beach and dune systems. In some areas, the coarse-grained cobbles and boulders have formed elevated areas that generally consist of rocky intertidal shorelines, for example, southern Egypt Beach, Cedar Point, and northern Fourth Cliff. These areas provide some natural wave attenuation, especially during periods of normal tidal fluctuations. However, during periods of elevated water levels and increased wave action associated with nor'easters, these natural platforms do not provide effective wave attenuation. An enhancement of these rock intertidal areas could serve to improve wave attenuation and storm protection during more severe storm events, as well as providing vertical structure and complexity which is important to nearshore fisheries.

The proposed concept of boulder dikes consists of 10 to 12 ton boulders placed on a rocky inter-tidal platform in staggered row formations that provides wave dissipation capacity for smaller storms. Boulder dikes may be appropriate for areas where beach nourishment is unacceptable due to adverse environmental impacts (e.g. covering a salt marsh or existing rocky inter-tidal shorelines). The addition of large boulders increases the complexity of the platform to enhance the inter-tidal habitat. Boulder dikes may only be implemented in rocky areas as the large boulders may subside in a sandy and/or silty environment. Figure 6.12 shows the natural rocky platform along the north side of Cedar Point and one of the larger glacial erratics that exist along the shoreline. The proposed boulders for the boulder dike would be similar in scale to this erratic, and be distributed in alternating rows parallel to upland development (Figure 6.13).

Figure 6.12 Rocky platform and a glacial erratic along the north side of Cedar Point (photo taken by Applied Coastal on May 12, 2016).

The environmental permitting process is for boulder dikes is unknown, as enhancement of rocky inter-tidal areas is uncommon, but the concept may be favored by fisheries agencies concerned about impacts of other more intrusive coastal engineering structures. The cost to construct a boulder dike consisting of four rows of evenly spaced boulders is estimated to be

$600 per linear foot. See Table 6.6 for a list of the pros and cons associated with the Boulder Dike approach.

Figure 6.13 Configuration of individual boulders (represented by the yellow circles) forming a boulder dike along Cedar Point.

Table 6.6 Pros, cons, and challenges of the Boulder Dike approach.

Pros	Cons
• Provides wave dissipation and storm protection especially for smaller storm events • Reduces wave overtopping and storm damage along the shoreline • May enhance inter-tidal habitat	• Impacts to sub-tidal and benthic habitats • Only provides limited protection from smaller storms • Minor impacts to the community during construction

Challenges
- Permitting pathway not straight-forward, as enhancement of rocky inter-tidal areas is not common
- Approach is applicable to rocky platforms only

6.7 Elevate Road

Elevating flood-prone roads can improve emergency egress, reduce overwash and the need for debris clearing, and may also offer improved protection from breaching. Many of the identified critical roads have elevations between approximately 2 and 3 feet below the 10-year and 100-year still water elevation, respectively. Figure 6.14 shows that high spring tide along the river side of Central Avenue is less than two feet from the existing road elevation.

Many of the roadways in Scituate were constructed across or immediately adjacent to tidal wetland areas. Over time, it is likely that many of these roadway fills subsided, making them more susceptible to flooding. Regardless, many of these emergency egress routs become impassable during significant storm events.

Figure 6.14 High tide along the river side of Central Avenue on March 31, 2014 (photo by Jason Burtner, *www.mycoast.org*).

Design challenges may be encountered when buildings and access ways (i.e. driveways, doorways, and garages) that were constructed at the present road elevation need to be raised to meet the new road elevation. Utilities along the road (water, gas, electric, etc.) may also need to be raised with the road. Environmental permitting and costly compensatory mitigation may be required when elevating causeways if there are impacts to the salt marsh. The cost to elevate a road and the associated utilities several feet is estimated to be approximately $750 per linear foot based on a study to elevate a marsh-adjacent roadway in Dennis, MA (ESS and Applied Coastal, 2016). There also may be other costs related to adding extensions to “daylight” septic systems or other dwelling-specific requirements. See Table 6.7 for a list of the pros and cons associated with the Elevate Road approach.

Table 6.7 Pros, cons, and challenges of the Elevate Road approach.	
Pros • Improves emergency egress during flood events • Reduces wave overwash and the need for debris clearing • May offer improved protection from breaching	**Cons** • Utilities must also be raised with the road (water, gas, electric, etc.) • Impacts to the community from construction
Challenges • Some homes and access ways may need to be raised to meet the new road elevation	

6.8 Drainage Improvements for the Basins

In addition to still water flooding, wave overtopping of seawall also creates significant coastal flooding problems along the Scituate coast. In many areas, water that overtops the seawall has an efficient return pathway to the ocean. However, the back barrier areas along Oceanside consist of low-lying altered marsh systems that do not have sufficient drainage capacity to handle the volume of water overtopping the seawall during significant storm events.

The Town has applied for a grant from the Massachusetts Emergency Management Agency (MEMA) Hazard Mitigation Grant Program to fund an alternatives analysis, engineering design, and the associated environmental permitting of drainage improvements for the low-lying Basins area (approximately 1st Avenue to 10th Avenue along Oceanside Drive). The improvements will not stop water from overtopping during storm events, but will reduce the duration of flooding to the roads and buildings. Figure 6.15 shows that flood waters cause 7th Avenue to become impassable. Flood waters within the Basins can typically last between 2 to 5 days during moderate storms (e.g. Winter Storm Nemo and Juno) before dissipating into the ocean. During winter months, the retained flood waters may also freeze and pose additional emergency access and safety concerns. See Table 6.8 for a list of the pros and cons associated with Drainage Improvements for the Basins approach.

Environmental permitting will be required due to impacts to wetland resources associated with the proposed outfall(s). However, outfall structures presently exist, ensuring that the regulatory process likely can be stream-lined for the improved drainage system. The cost of construction is estimated to be $4.0 million and construction time is estimated to be 8 months.

Table 6.8 Pros, cons, and challenges of the Drainage Improvements for the Basins approach.	
Pros • Reduce duration of flooding	**Cons** • Does not stop water from overtopping
Challenges • Permitting required due to impacts to wetland resources associated with outfall	

Figure 6.15 Flooding of 7th Avenue during a storm on March 8, 2013 (photo by Jason Burtner, *www.mycoast.org*).

6.9 Protection Improvements for Pump Stations

The operation of the pump stations may be interrupted during storm events from flooding of the power generators. While there are six pump stations located close to the shore in the Town, the stations located at Chain Pond in the Egypt Beach parking lot and at Sand Hills (at the intersection of Otis Road and Scituate Avenue) are prone to flooding due to their relatively low elevation. During Winter Storm Juno, flooding at the Sand Hills pump station was above the first floor level of the pump station as shown by the snow line in Figure 6.16. According to the Town of Scituate Department of Public Works, the power generators in both pump stations are located inside the buildings on the first floors and may benefit from being elevated.

The cost of flood-proofing similar-sized Zone AE pump stations located in Wareham, MA was approximately $570,000 as reported by GHD (2016). These improvements include installing flood doors and watertight hatches, flood-proof painting, raising generators above flood elevation, and installing/raising louvers. These improvements will ensure that the pump stations will remain operational during flood events. See Table 6.9 for a list of the pros and cons associated with Protection Improvements for Pump Stations approach.

Figure 6.16 Flooding elevation is shown by the snow line at the Sand Hill pump station after Winter Storm Juno (photo by Jason Burtner, *www.mycoast.org*).

Table 6.9 Pros, cons, and challenges of the Protection Improvements for Pump Stations approach.	
Pros • Ensure that pump station will remain operational during flood events	**Cons** • None
Challenges • None	

6.10 Managed Retreat

Managed retreat may be a viable approach in areas where the long-term cost of retreat is less than other shore protection approaches. In some cases, buildings may have the opportunity to move landward away from the eroding shoreline within the existing property lines or into suitable Town property. The estimated cost for moving an existing elevated home along a barrier beach system is $300,000, but the cost will vary depending on the size of the home, design of wastewater systems, and complexity of relocation. Environmental permitting assistance may be provided by the Massachusetts Department of Environmental Protection and MCZM in cases where homes are to be relocated towards marsh areas.

In other locations, the space to move landward may be limited by wetlands, land ownership, and safety. For these properties, the option for the Town to buy-out and remove the buildings and restore the land may be possible with mutual agreement between the Town and the property owners. The benefits of managed retreat are that residents are relocated to the safer location, the shoreline and sediment transport will be restored to the natural state, and reoccurring costs for post-storm clean up and repairs can be eliminated or reduced. However, some roads and utilities may need to be maintained to service other parts of the Town. The value of the oceanfront properties carries a significant cost for the Town and buy-outs result in loss of property tax revenue. For the purposes of this study, the assessed values in the 2016 Scituate Assessor's Database was be used to estimate the value of the land and buildings, although the market value of the properties are customarily well above the assessed value in oceanfront locations.

6.11 Elevate Buildings

The Massachusetts State Building Code states that all buildings and structures in a high-hazard zone (i.e. V Zone) shall be elevated so that the bottom of the lowest horizontal structural member supporting the lowest floor is located at least two feet above the base flood elevation (780 CMR 120.G601.2). Elevating a home to the minimum building code elevation or higher reduces the storm damage susceptibility of the building, alleviates flow channelization, and has the additional benefit of reducing flood insurance rates for the property owner. The average cost of elevating a home based on the FEMA Hazard Mitigation Assistance program is \$175,000. The approach of elevating homes is recommended for any building in the floodplain. See Table 6.10 for a list of the pros and cons associated with Elevate Buildings approach.

Table 6.10 Pros, cons, and challenges of the Elevate Buildings approach.

Pros	**Cons**
• Susceptibility of home to storm damage is reduced • Alleviates flow channelization • Reduced flood insurance rate	• None
Challenges • Cost to raise home is, on average, \$175,000 per home	

6.12 Other Innovative Approaches

Other innovative shore protection approaches have been considered in this study including offshore wave break technologies (e.g. Wave Attenuation Devices™, Reef Balls™), floating breakwaters, beach dewatering, etc. Most of these innovative technologies are based on existing shore protection methods, but may involve alternative construction materials, modifications to design configuration, and use of techniques from other industries. The orientation of the Scituate shoreline is highly exposed to large northeast waves and "soft" engineering approaches were not deemed viable; however, other artificial reef approaches that attenuate wave energy could potentially be utilized along the Scituate coast.

Along much of the U.S East and Gulf Coasts, artificial reef technologies that have served as an alternative to the standard rock breakwater to attenuate wave energy are a variety of concrete structures that are intended to promote either shellfish growth or to serve as nearshore fisheries habitat. Two different artificial reef concepts, Reef Balls™ and Wave Attenuation

Devices (WADs^{TM}), were investigated as possible alternatives to the boulder dikes previously described, or as stand-alone wave attenuation along sandier shorelines.

The first technology evaluated were Reef Ball^{TM} units which are typically round-topped concrete balls with holes throughout (Figure 6.17). The purpose of a Reef Ball^{TM} structure would be to create a submerged breakwater that attenuates some of the wave energy while also creating a habitat for mussels and other coastal species. The concrete used to make the units is typically a microsilica-based concrete that is abrasion resistant, high strength, and has a pH similar to that of the natural sea (~8.3) (*reefball.org*). The units can be designed in numerous shapes, sizes, and textures depending on the intended environment. According to *reefball.org*, the largest Reef Ball^{TM} unit available is the "Goliath" which is 6 feet wide and 5 feet tall and costs approximately $450 per unit. The majority of the weight of a Reef Ball^{TM} unit is at the base of the structure to increase stability. In addition, it is possible to increase the height of the Reef Ball^{TM} unit by adding a "booster ring".

It has been stated that Reef Ball^{TM} units are intended to withstand a heavy tropical storm without movement in as little as 20 feet of water without anchors (*reefball.org*). Wave tank stability analyses also have been performed; however, there is no information available regarding the larger units. In a 2003 laboratory study by Armono and Hall, the wave energy attenuation was examined for two different submerged breakwater configurations of Reef Ball^{TM} units (Figure 6.18). From this study, it was determined that the submerged breakwater had an approximate wave transmission coefficient of 0.79 and 0.82 for the 5 row and 3 row configuration, respectively.

The second artificial reef technology considered was WAD^{TM} units. The units are made of precast concrete, are pyramidal in shape, and have pressure release openings on the sides (Figure 6.19). WAD^{TM} units are intended to function as a detached breakwater that minimizes the effect of storm surge through wave energy attenuation and to provide a substrate for shellfish to colonize on. Similar to the other technologies, size and configuration of the WAD^{TM} units is highly project specific. WAD^{TM} units have been used for many projects as detached breakwaters from Maryland to Jamaica.

Two recent studies by Douglass et al. (2012) and Allen (2013) have conducted laboratory wave tests on four-sided apex-truncated square pyramid shaped breakwater units to determine the wave transmission coefficients of different breakwater configurations. In general, these units are similar in configuration to WAD^{TM} units; however, the interlocking is slightly different as a result of the shape (a square base rather than a triangular base). In the Douglass et al. experiments, three different emergent breakwater configurations of armor units were evaluated: one row of units, two rows of closely spaced units, and two rows of widely spaced units. For the two row configurations, the rows were staggered so the units were offset half the width of a unit. Additionally, the water depth, wave period, and wave height was varied. The depths were varied to simulate the tidal variation with depths varying from 18 centimeters, 23 centimeters, and 29 centimeters. These water depths corresponded with the structures being 60%, 75%, and 96% submerged, respectively. Two wave heights, 5 centimeters and 8.6 centimeters, and two wave periods, 1.34 seconds and 1.54 seconds, were evaluated. The wave transmission ranged from 0.4 and 0.9 depending on the emergence of the unit, configuration of the units, and wave period. Douglass et al. (2012) concluded that emergent armor unit breakwater structures had higher wave transmission than a submerged rock structure (Figure 6.20).

Figure 6.17 Individual Reef Ball^{TM} unit *(image credit: reefball.org).*

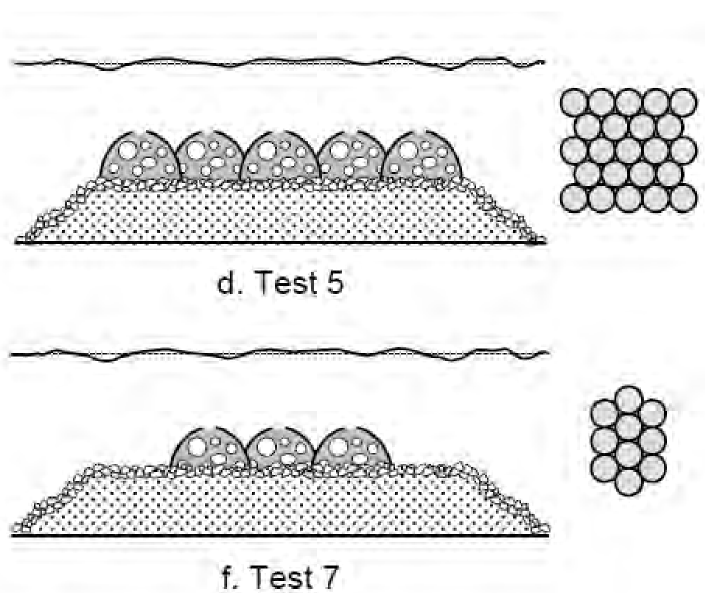


Figure 6.18 Submerged Reef Ball^{TM} breakwater configurations (modified from Armono and Hall, 2003).

Allen's (2013) experiments focused on two 4-sided apex-truncated square pyramid shaped breakwater configurations: one row of units and two staggered row of closely spaced units. Similar to the Douglass' study the water depth, wave height, and wave period were varied. The wave height and wave period ranged from 0.02 to 0.27 meters (0.08 and 0.89 feet) and 1.1 to 2.29 seconds, respectively. Importantly, this study included tests where the structure was completely submerged, unlike Douglass' study. Five different water depths were used: 0.2

meters (0.66 feet), 0.25 meters (0.82 feet), 0.3 meters (0.98 feet), 0.36 meters (1.2 feet), and 0.41 meters (1.3 feet). The individual concrete units used in this experiment were 0.3 meters (0.98 feet) tall signifying that the breakwater was submerged for the three deeper water depth cases. The wave transmission for the submerged breakwaters experiments ranged from 0.4 to 1.10 for both configurations and for the varying water depths and wave parameters. However, most configurations of submerged units indicated transmission coefficients higher than 0.75. Based on both the work of Douglass et al. (2012) and Allen (2013), the WAD™ units did not perform as well at attenuating wave energy as a similar-sized traditional stone breakwater.

Figure 6.19 Photograph of the Shark Island WAD™ project completed in New Iberia, Louisiana *(photo credit: www.livingshorelinesoultions.com)*.

Along the Scituate shoreline, the spring tide range is in excess of 10 feet; therefore, artificial reef units submerged at low tide would effectively provide no wave attenuation at high tide, similar to, but less effective than an armor stone submerged breakwater (see Figure 6.19). If the structure is utilized in an emergent situation, where it is exposed for much of the tide, subsidence and scour become major concerns, similar to other coastal engineering structure in this dynamic environment. Similar to the boulder dike, these concrete units could be placed on the natural boulder platforms (e.g. Cedar Point or Egypt Beach); however, concerns regarding construction materials remain in this coarse-grained sediment environment. Specifically, during storm conditions, gravel and cobble are highly mobile, causing rapid degradation of exposed concrete surfaces. An example of this effect is shown for a concrete outfall structure along Oceanside Drive is shown in Figure 6.20. Therefore, artificial reef structures were not deemed appropriate for the Scituate shoreline due to both the inability to attenuate waves when submerged and the lack of durability of the concrete construction material.

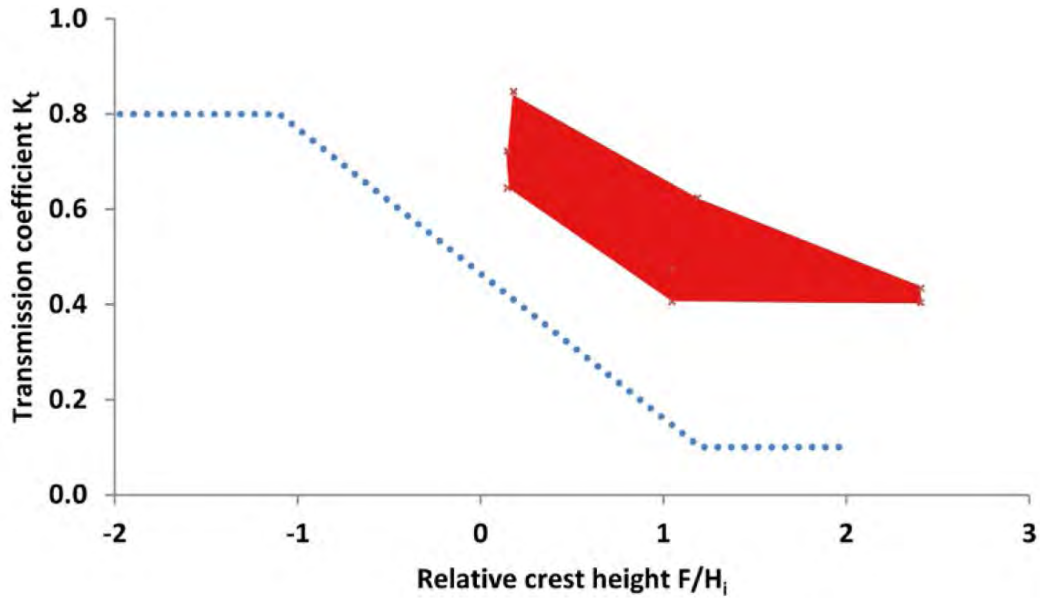


Figure 6.20 Figure from Douglass et al. (2012) showing the wave transmission coefficients of the laboratory experiments for a breakwater consisting of 4-sided apex-truncated square pyramid shaped breakwater units (red data envelope show) compared to coefficients of a traditional stone breakwater in blue.

Figure 6.21 Observed scouring of concrete surface of outfall structure along Oceanside Drive. It is anticipated that degradation of concrete would be more rapid along the cobble/boulder strewn platforms at Cedar Point or Egypt Beach.

7.0 SHORE PROTECTION APPROACHES BY STUDY AREA

Once the approaches were assessed relative to their applicability to shore protection (Section 6.0), screening of these options was performed to determine the most appropriate approaches for each shoreline section. In general, discretionary criteria were utilized to assess the applicability of different options, considering aspects of each alternative including engineering, economics, long-term viability, and potential environmental impacts. Once the approach screening process was completed, a matrix of potential shore protection options was developed for each shoreline section based upon the assessment of vulnerability and "need" from the overall economic parameters. This scheme included both "hard" and "soft" shore protection measures, based on project need within each of the shoreline sections identified. In general, economic drivers were critical to this prioritization process; however, coastal resiliency also was addressed, as future shore protection expenditure planning required that a sustainable outcome will be achieved based upon a 50-year planning horizon. In some cases, the economics indicated that managed retreat is the most feasible alternative; however, other considerations and/or policy decisions by the Town might alter the selection of shoreline management approach. The outcome of the prioritization assessment of shore protection management strategies based on both "need" and economic drivers is aimed at providing guidance for future Town planning efforts.

The intent of providing recommended approaches to shore protection and/or shoreline management by study area was to indicate potential options that likely represent the most economically viable alternative, considering both environmental impacts and sustainability of the Scituate coastal development. Understanding the long history of "hard" shore protection along much of the developed coastline of Massachusetts, the analysis attempted to address many of the concerns about reduced littoral sediment supply. Where appropriate, combinations of "hard" and "soft" measures also were considered. The recommended approaches do not provide a detailed engineering-level analysis that is intended for design purposes, but rather provide conceptual-level information to assist with Town planning efforts. In this manner, the recommended approaches (as well as approaches that were not initially recommended) can be vetted by the Town as they move forward to address the town-wide coastal sustainability issues.

In the sections below, the shore protection approaches for each study area are listed and the conceptual design details are presented. A construction cost estimate is provided along with lifecycle costs for 50-years, if applicable. A breakdown of the lifecycle costs are presented in Appendix D. For non-structural coastal engineering measures (e.g. beach and/or dune nourishment), the design life generally is on the order of 5 to 15 years; therefore, designs could be readjusted as sea-levels increased in the future. These design modifications would become part of the ongoing maintenance requirement for the project and there would be no need to incorporate sea-level rise directly into the design.

For all study areas, elevating homes and buildings in high hazard flood areas above base flood elevations is recommended, but has not been listed specifically. The cost to elevate a home is approximately $175,000. As of June 2016, 14 grant applications to elevate homes under the FEMA Hazard Mitigation Grant Program are underway and additional 7 applications are pending. Additional details are presented in Section 6.11.

With all the approaches presented, there will be some impacts associated with construction. Beyond direct impacts to the coastal environment, these may include other concerns such as air quality impacts from construction equipment emissions, traffic impacts from material-carrying trucks, and noise impacts from heavy vehicles. Additional project-specific impacts would be identified during the permitting process and the appropriate mitigation measures would become part of the overall project. Potential permitting issues were identified for each approach. Difficult environmental permitting challenges are expected to arise in situations where coastal structures (i.e. seawalls and revetments) require expansion seaward and where the approach may adversely affect benthic flora and fauna.

In all the shore protection approaches, appropriate public access easements will need to be acquired from the involved property owners if the project is publically funded.

7.1 Minot Beach

Utilizing information developed from the coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, appropriate shore protection strategies were developed for the Minot Beach shoreline segment. The shoreline change assessment (Appendix A) indicates that much of the shoreline has been relatively stable over the past 60+ years, with modest erosion observed at the southern end of the beach. Structural manipulation of the nearshore area was conducted through construction of the nearshore revetment dike within this area, essentially eliminating landward migration of the high water line. Due to the relatively short stretch of beach in this area, as well as the exposure to open ocean wave conditions (Appendix B), effective beach nourishment seaward of the coastal armoring is not feasible. Instead, potential improvements to the existing shoreline armoring were evaluated, along with potential nourishment within the area between the existing seawall and the revetment dike. Additionally, elevating Bailey's Causeway was considered to provide emergency egress during periods of combined high tide and storm surge. The shore protection approaches for Minot Beach are summarized in Table 7.1.

Table 7.1 Shore protection approaches and costs for Minot Beach.

Shore Protection Approach	Cost	50-Year Lifecycle Cost
Seawall North section - 330 feet	$2.7 million	$6.7 million
Seawall South section - 1,200 feet	$9.5 million	$23.6 million
Revetment Dike 1,200 feet	$5.0 million	$12.4 million
Beach Nourishment Perched cobble beach - 1,200 feet	$600,000	$2.2 million
Elevate Bailey's Causeway For emergency access - 900 feet	$675,000	Not applicable

Seawall

The condition of the existing seawalls along Minot Beach are rated as "good" by Bourne Consulting as part of the 2015 Massachusetts Coastal Infrastructure Inventory and Assessment Report Update (inspection was performed by CLE Engineering in 2013). There are two sections of seawall – a shorter 330-foot section at the north end that is slightly lower in elevation (13 feet NAVD88) than the longer 1,200-foot south section (17.5 feet NAVD88). The south section is further protection with a revetment dike that is located approximately 50 fee seaward of the seawall.

The wave overtopping rates during a 100-year storm along the north and south sections are capable of damaging the paved roads behind the seawalls. If sea levels rise by hypothetically by 2 feet, wave overtopping during the 100-year storm may increase by a factor of 5. Increasing the height of the seawall by 2 feet can reduce wave overtopping rates to levels that will not damage pavement under existing climate conditions but the improved design does prevent damage under the hypothetical sea level rise (SLR) scenario. Schematics of the north and south seawall designs and wave overtopping rates are summarized in Figure 7.1, Figure 7.2, Table 7.2, and Table 7.3.

The cost of raising the seawalls at Minot Beach are $2.7 million and $9.5 million for the north and south sections, respectively. Regular inspections and repairs are required to maintain the structural integrity of the seawall.

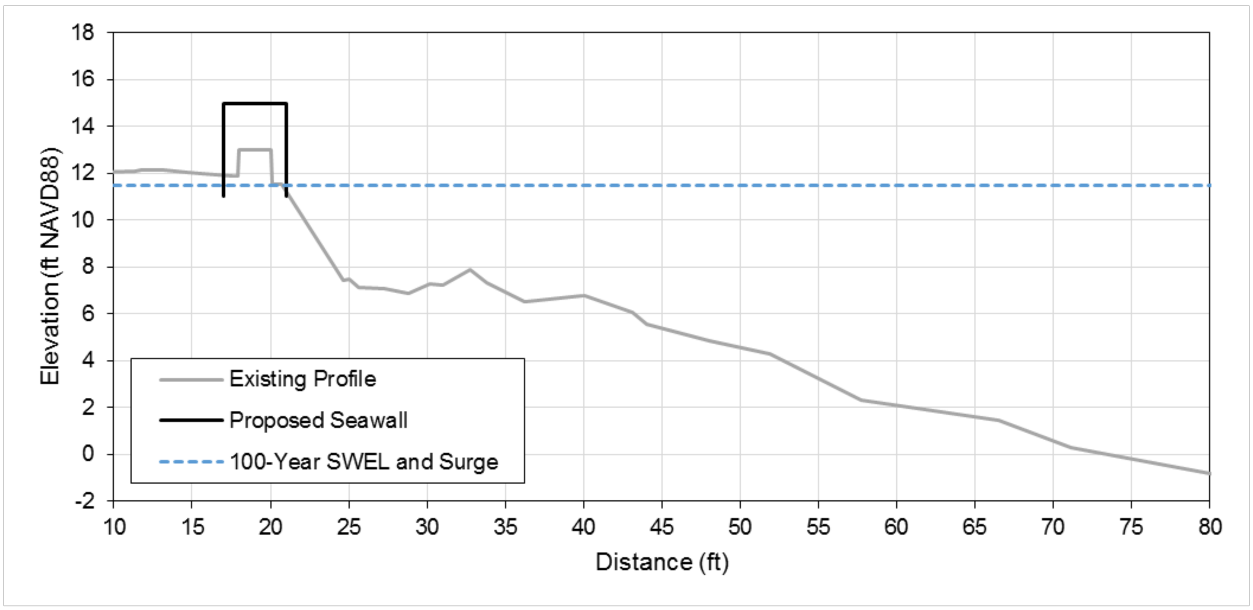


Figure 7.1 Schematic of the proposed seawall design along Minot Beach (north wall section) where the seawall height is increased by 2 feet.

Table 7.2 Wave overtopping and predicted damage at Minot Beach (north wall section) during 100-year storm conditions. The proposed design involves increasing the existing seawall height by 2 feet.

Scenario	Wave Overtopping ($ft^3/s/ft$)	Predicted Damage
Existing Design	1.8	Yes
Existing with 2 feet of SLR	9.2	Yes
Proposed Design	0.5	No
Proposed with 2 feet of SLR	3.4	Yes

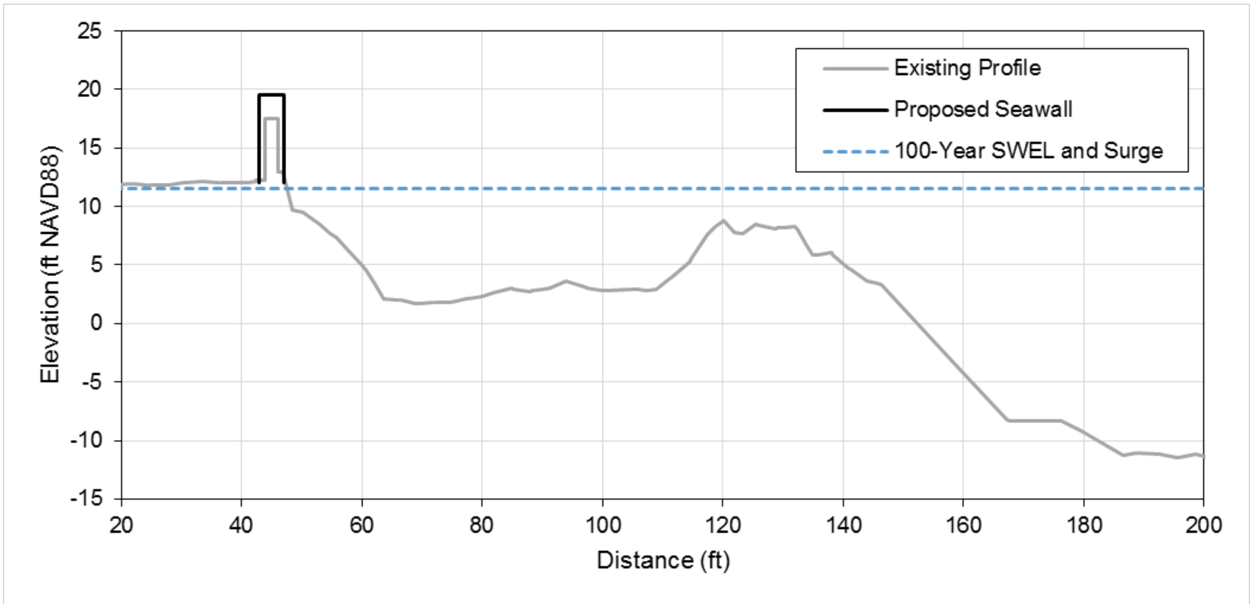


Figure 7.2 Schematic of the proposed seawall design along Minot Beach (south wall section) where the seawall height is increased by 2 feet.

Table 7.3 Wave overtopping and predicted damage at Minot Beach (south wall section) during 100-year storm conditions. The proposed design involves increasing the existing seawall height by 2 feet.

Scenario	Wave Overtopping ($ft^3/s/ft$)	Predicted Damage
Existing Design	1.0	Yes
Existing with 2 feet of SLR	4.7	Yes
Proposed Design	0.5	No
Proposed with 2 feet of SLR	2.2	Yes

Revetment Dike

A roughly 1,200-foot long revetment dike is situated approximately 50 feet seaward of the south seawall section at Minot Beach. The crest elevation of the existing revetment dike, shown in Figure 7.3, is approximately 9 feet NAVD88 and the crest is approximately 14 feet wide. To increase the wave dissipation capacity of the dike, the proposed design suggests increasing the crest of the dike well above the 100-year still water and surge elevation to 13 feet NAVD88 and increase the crest width to 20 feet. The proposed revetment dike would decrease the wave transmission by 15% and lowers the wave overtopping rates to below pavement damage levels during the 100-year storm. Unfortunately, the proposed design does not prevent damage during a hypothetical 2-foot sea level rise (SLR) scenario. A schematic of the proposed design is presented in Figure 7.4 and the wave overtopping rates are summarized in Table 7.4.

The cost of constructing the proposed revetment dike is estimated to be $5.0 million plus the associated long-term maintenance costs.

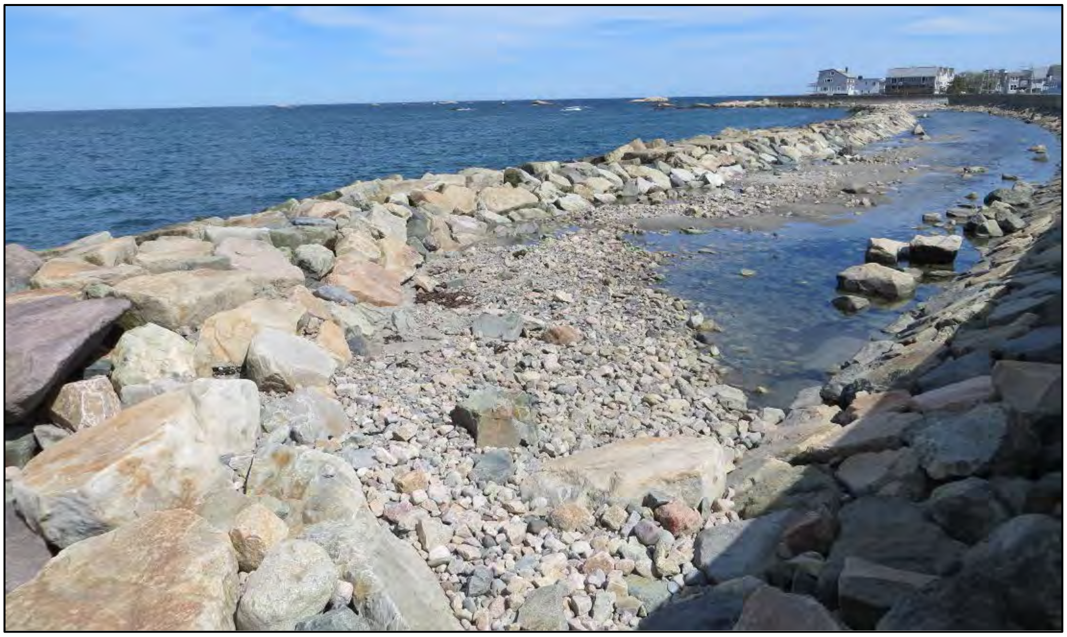


Figure 7.3 Existing revetment dike at Minot Beach (photo taken by Applied Coastal on May 10, 2016).

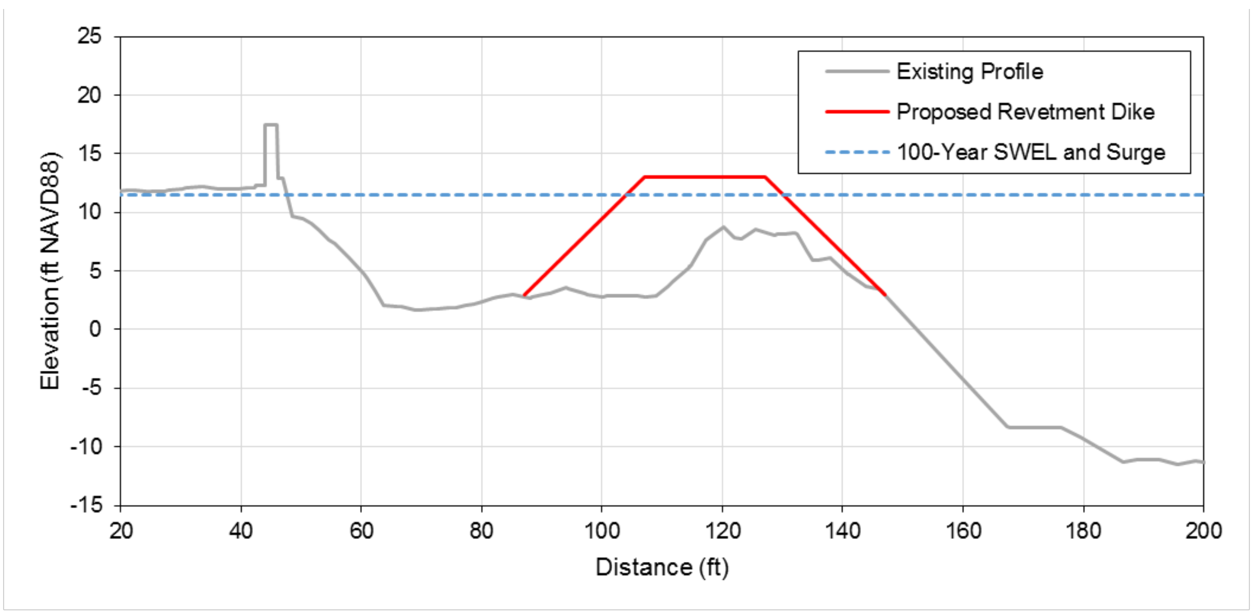


Figure 7.4 Schematic of proposed revetment dike design along Minot Beach where the crest of the existing revetment dike is raised and widened and the seawall is unaltered.

Table 7.4 Wave overtopping and predicted damage at Minot Beach during 100-year storm conditions. The proposed design involves raising and widening the existing dike crest.		
Scenario	**Wave Overtopping ($ft^3/s/ft$)**	**Predicted Damage**
Existing Design	1.0	Yes
Existing with 2 feet of SLR	4.7	Yes
Proposed Design	0.3	No
Proposed with 2 feet of SLR	1.9	Yes

Beach Nourishment – Perched Cobble Beach

As much of the Minot Beach shoreline is fronted by a revetment dike (Figure 7.3), it is possible to utilize this existing structure to "perch" beach nourishment on the landward side of the structure to attenuate incoming storm wave energy. This perched beach system would be completely contained by the revetment dike and could be constructed by using the existing 1,200-foot revetment dike as a sill to support the lower portion of the beach profile. A schematic of the proposed nourishment template is presented in Figure 7.5. The cobble beach is expected to reshape immediately after construction and the crest of the beach, initially at 12 feet NAVD88, will adjust to match the runup elevation to minimize overtopping (CIRIA/CUR, 1991).

Approximately 18,000 cubic yards of cobble are required for the nourishment at a construction cost of $600,000. The revetment dike is expected to "hold in" the cobble to prevent loss of material, however, monitoring and maintenance would be required to ensure that the material is well distributed along the south seawall.

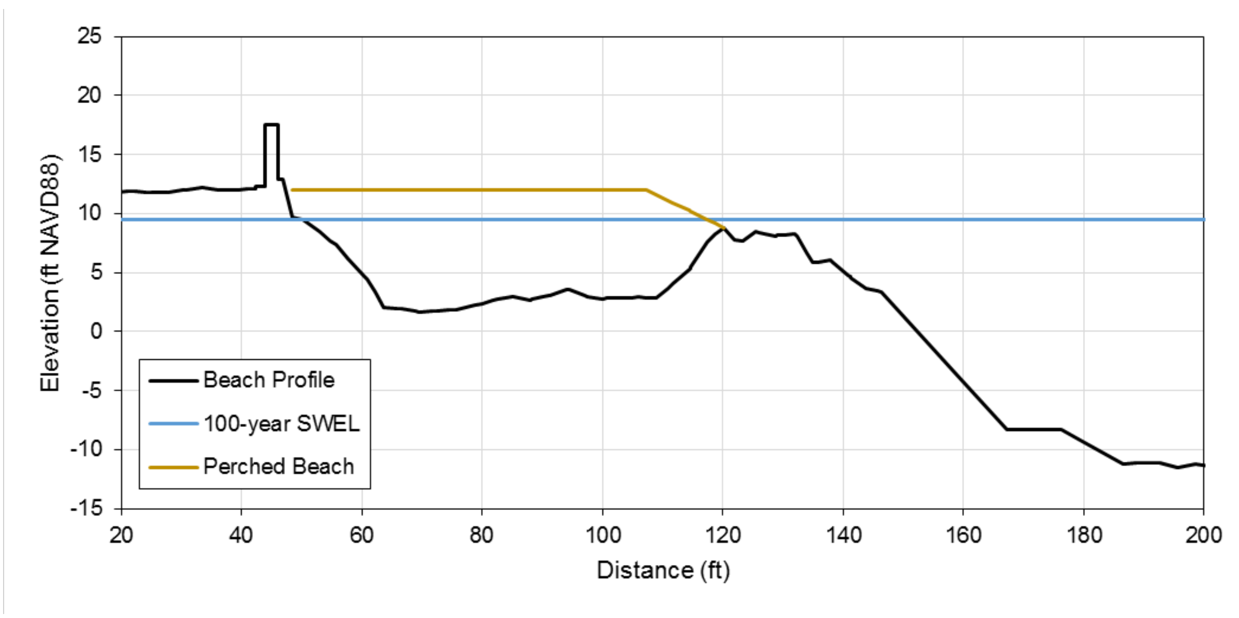


Figure 7.5 Nourishment template profile for a perched cobble beach at Minot Beach.

Elevate Bailey's Causeway

Bailey's Causeway serves as an emergency access route for Scituate Neck, Minot Beach, and North Scituate Beach when Glades Road becomes impassable due to overtopping waves. Figure 7.6 and Figure 7.7 show that the entire length of Bailey's Causeway, approximately 900 feet long from Buttonwood Lane to Glades Road, is submerged under the 10-year still water elevation. On average, the road is 0.7 feet and 1.7 feet below the 10- and 100-year still water elevation, respectively.

The number of homes that have driveways connected to Bailey's Causeway is small, however the proximity to the marsh may cause permitting concerns depending on the design. The estimated construction cost of elevating the road and associated utilities is $675,000.

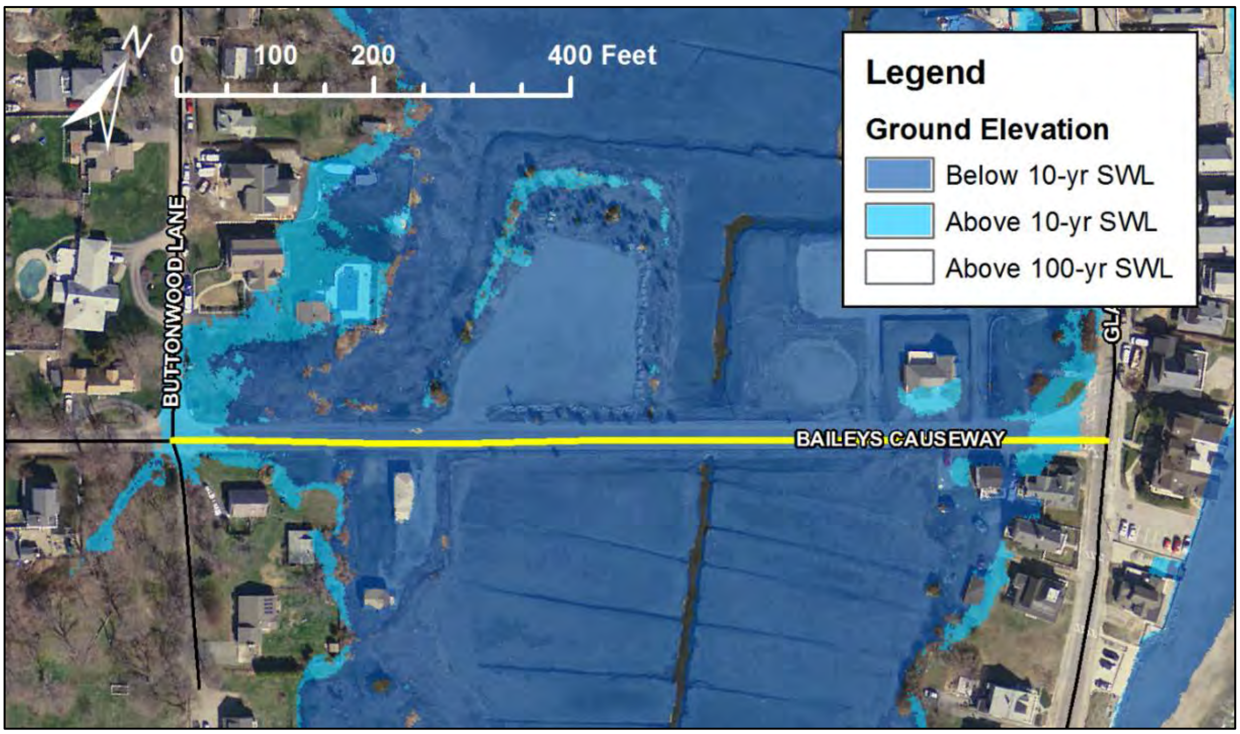


Figure 7.6 Flooding extents of the 10-year and 100-year still water elevation across Bailey's Causeway.

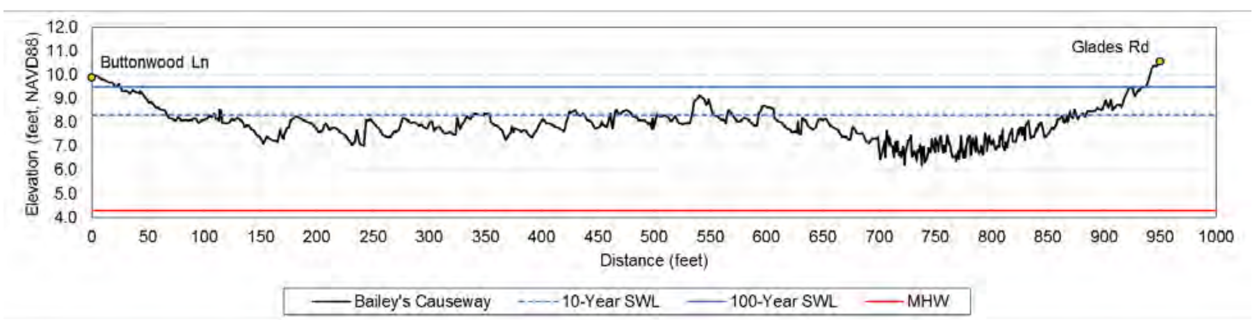


Figure 7.7 Road elevation along Bailey's Causeway from Buttonwood Lane to Glades Road.

Recommended Approach for Minot Beach

The recommended shoreline protection approach for Minot Beach consists of nourishment in the form of a perched cobble beach. The nourishment is estimated to cost $600,000 in initial construction costs and a total of $2.2 million over a 50-year lifecycle. In addition, the north portion of Minot Beach would need seawall improvements requiring a total of $6.7 million over a 50-year lifecycle. The need to raise Bailey's Causeway likely is dependent upon the shore protection developed along North Scituate Beach, as shore protection along Glades Road will allow emergency access to Minot Beach without utilizing Bailey's Causeway. Therefore, elevating the causeway has not been recommended at this time. Comparatively, the cost to maintain the status quo along Minot Beach over 50 years is estimated to be $31.3 million.

7.2 North Scituate Beach

The shoreline change and coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, guided development of appropriate shore protection strategies for the North Scituate Beach shoreline segment. Although North Scituate and Surfside were considered as separate shoreline segments as a result of storm damage vulnerability, the two segments form a contiguous low elevation beach backed by combined seawall and revetment protection. The shoreline change assessment (Appendix A) indicates that much of the shoreline has been relatively stable over the past 60+ years; however, this presumed stability is a result of fixing the position of the shoreline with the seawall, preventing landward migration of the beach. Over time, the beach has continued to lower and no high tide beach exists along this shoreline stretch. Although this shoreline is exposed to open ocean wave conditions, relatively modest longshore sediment transport rates (Section 3) indicate that beach nourishment could be effective if implemented as a large-scale program. Potential improvements to the existing shoreline armoring also was evaluated. Additionally, elevating Bailey's Causeway was considered to provide emergency egress during periods of combined high tide and storm surge. The shore protection approaches for North Scituate Beach are summarized in Table 7.5.

Table 7.5 Shore protection approaches and costs for North Scituate Beach.

Shore Protection Approach	**Cost**	**50-Year Lifecycle Cost**
Seawall and Revetment 1,800 feet	$14.8 million	$36.8 million
Beach Nourishment Phase 1 - 2,900 feet	$8.2 million	$58.9 million
Elevate Bailey's Causeway For emergency access - 900 feet	$675,000	Not applicable

Seawall and Revetment

The top of the seawall along North Scituate Beach has an elevation of 17 feet NAVD88 and the fronting revetment consists of slumped and scatter stones that are generally too low to effectively dissipate waves. The existing seawall and revetment is approximately 1,800 feet long and the condition is shown in Figure 7.8. The condition of the structure was reported to be “fair” and “poor” by CLE Engineering in 2013.

As it exists, the dimensions of the structure does not provide adequate wave overtopping protection during a 100-year storm. To reduce wave overtopping rates enough to prevent pavement damage, it is proposed that the seawall be raised a minimum of 2 feet while the fronting revetment is expanded by raising the revetment crest to above storm surge levels, as shown in the schematic in Figure 7.9. As a result of increasing the height of the revetment, the structure footprint would need to extend further seaward. Although pavement damage is not expected to occur with the proposed design under existing conditions, damage is predicted under a hypothetical 2-foot sea level rise (SLR) scenario. Table 7.6 summarizes the predicted wave overtopping rates under the various scenarios.

The cost to construct the proposed seawall and revetment is estimated to be $14.8 million. Regular maintenance and inspections are required after construction to preserve the integrity of the structure.

Figure 7.8 Existing seawall and revetment along North Scituate Beach (photo by CLE Engineering).

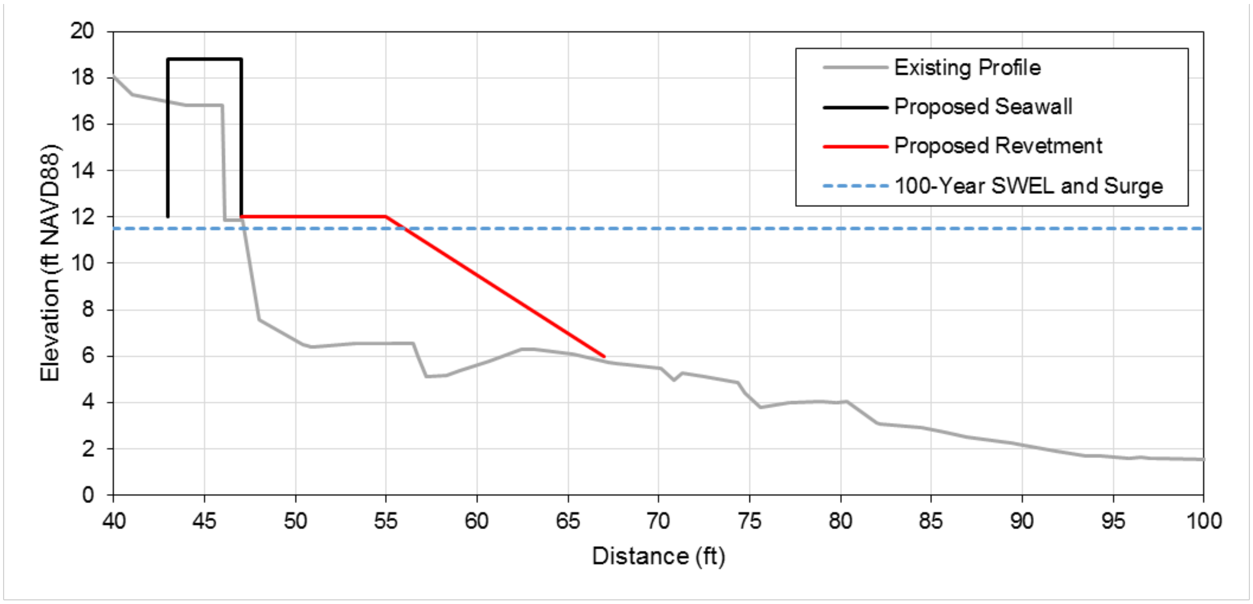


Figure 7.9 Schematic of proposed seawall and revetment design along North Scituate Beach where the seawall height is increased by 2 feet and the fronting revetment is expanded.

Table 7.6 Wave overtopping and predicted damage at North Scituate Beach during 100-year storm conditions. The proposed design consists of increasing the existing seawall height by 2 feet and expanding the fronting revetment.

Scenario	Wave Overtopping ($ft^3/s/ft$)	Predicted Damage
Existing Design	0.7	Yes
Existing with 2 feet of SLR	3.2	Yes
Proposed Design	0.5	No
Proposed with 2 feet of SLR	1.5	Yes

Beach Nourishment

A beach nourishment project is currently in the environmental permitting stage for North Scituate Beach. In the initial phase, the nourishment would be placed along the northern 2,900 feet of the beach. The project would require approximately 240,000 cubic yards of material to construct a nourished beach with a renourishment interval of approximately 9 years. Renourishment is required when the volume of fill remaining within the initial fill limits falls below 30%. The nourished beach crest would be approximately 100 feet wide with a berm height of 12 feet NAVD88 to correspond to the runup during a 50-year storm. The seaward face of the nourishment would then slope downward on roughly 1V:10H slope to meet the existing bottom. The beach nourishment would extend from Station 0+00 to 29+00 along a distance of approximately 2,900 feet, including tapered sections at both ends, shown in Figure 7.10. The nourishment footprint is approximately 20 acres. Sediment from the nourishment footprint, also shown in Figure 7.10, will naturally migrate towards the north and south to provide some storm protection to the adjacent properties. The approximate cost of the initial phase is $8.2 million.

The full 4,900-feet length project for North Scituate Beach extending from Station 0+00 to 49+00 will be both permitted and constructed as funding becomes available. The second phase of nourishment requires an additional 152,000 cubic yards of sediment with a total project footprint of 30.0 acres. The approximate cost of full-length project is $13.4 million. Based on input from public meetings associated with the project, there is strong property owner interest in signing the appropriate easements that will be required for public funding of the project. The Town plans to request easements from all property owners within this additional nourishment area prior to seeking regulatory approval.

Figure 7.10 Proposed initial phase of the North Scituate Beach nourishment project.

Elevate Bailey's Causeway

Details of this approach is described in Section 7.1.

7.3 Surfside Road

The shoreline change and coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, guided development of appropriate shore protection strategies for the Surfside Road shoreline segment. As described above, although North Scituate and Surfside were considered as separate shoreline segments as a result of storm damage vulnerability; however, the two segments form a contiguous low elevation beach backed by combined seawall and revetment protection. The shoreline change assessment (Appendix A) indicates that much of the shoreline has been relatively stable over the past 60+ years; however, this presumed stability is a result of fixing the position of the shoreline with the seawall, preventing landward migration of the beach. Over time, the beach has continued to lower and no high tide beach exists along this shoreline stretch. Although this shoreline is exposed to open ocean wave conditions, relatively modest longshore sediment transport rates (Section 3) indicate that beach nourishment could be effective if implemented as a large-scale program. Potential improvements to the existing shoreline armoring also was evaluated. The shore protection approaches for Surfside Road are summarized in Table 7.7.

Table 7.7 Shore protection approaches and costs for Surfside Road.

Shore Protection Approach	Cost	50-Year Lifecycle Cost
Seawall and Revetment 2,700 feet	$21.8 million	$54.3 million
Beach Nourishment Phase II - 2,000 feet	$4.9 million	$35.2 million

Seawall and Revetment

Similar to the seawall and revetment along North Scituate Beach, the condition of the 2,700-foot long structure along Surfside Road were rated as "fair" and "poor" by CLE Engineering in 2013. Figure 7.11 shows the deteriorating seawall and loose revetment stones. The footing of the seawall is also exposed from on-going beach erosion and beach lowering. During a 100-year storm, wave overtopping rates large enough to cause pavement damage are predicted under the existing design conditions. The proposed seawall and revetment design is presented in Figure 7.12. The proposed design involves increasing the seawall elevation from approximately 17 to 19 feet NAVD88 and extending the revetment footprint to accommodate a more robust revetment. As summarized in Table 7.8, the proposed design reduces wave overtopping during the 100-year storm to acceptable levels but is unable to reduce wave overtopping sufficiently in a hypothetical 2-foot sea level rise (SLR) scenario.

The estimated cost to construct the seawall and revetment along Surfside Road is $21.8 million. Costs associated with maintenance and repairs are required post-construction for the life of the structure.

Table 7.8 Wave overtopping and predicted damage at Surfside Drive during 100-year storm conditions. The proposed design involves of increasing the height of the existing seawall by 2 feet and expanding the fronting revetment.

Scenario	Wave Overtopping ($ft^3/s/ft$)	Predicted Damage
Existing Design	0.9	Yes
Existing with 2 feet of SLR	3.7	Yes
Proposed Design	0.3	No
Proposed with 2 feet of SLR	1.8	Yes

Figure 7.11 Existing seawall and revetment along Surfside Road (photo by CLE Engineering).

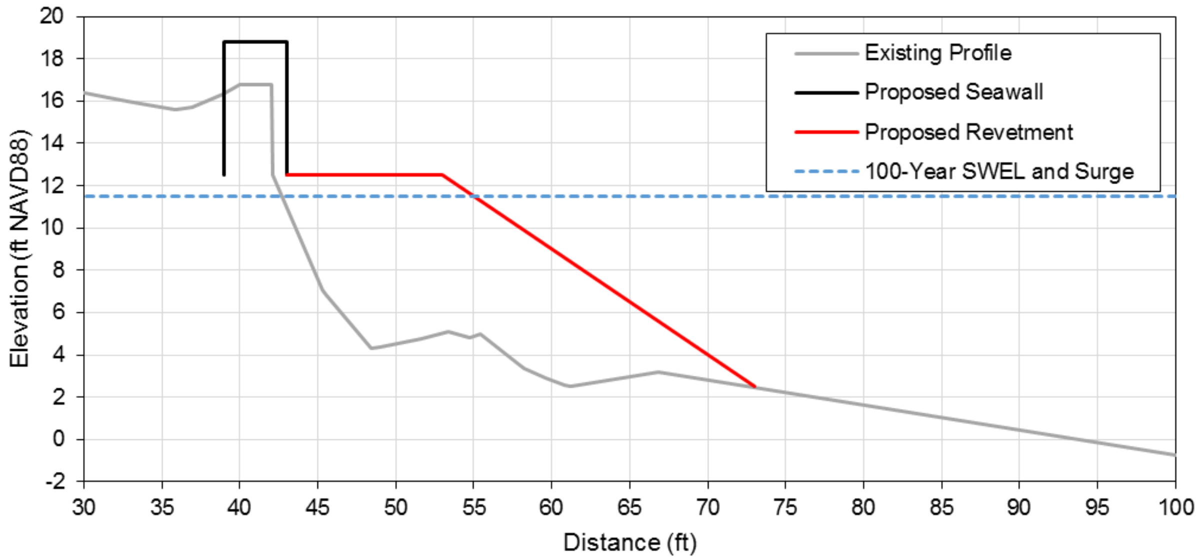


Figure 7.12 Schematic of proposed seawall and revetment design along Surfside Drive where the seawall height is increased by 2 feet and the fronting revetment is expanded.

Beach Nourishment

Details of this approach is described in Section 7.2.

Recommended Approach for North Scituate Beach and Surfside Road

Overall, the recommended shore protection approach for North Scituate Beach and Surfside Road is large-scale beach nourishment. Compared to the cost to reconstruct and maintain the seawalls and revetments along the two study areas over 50 years ($94.1 million), the 50-year lifecycle cost of nourishment is not significantly higher at $95.6 million, but the nourishment has the benefit of providing improved storm protection, providing a sediment source for the adjacent shorelines (i.e. likely improvement in shore protection to areas further south including Mann Hill and Egypt Beaches), and creating a recreational resource. Compared to the cost to maintain the status quo along the two study areas over 50 years ($105.9 million), the 50-year lifecycle cost of both nourishment and seawall/revetment improvements have a lower cost. However, it should be noted that as presented, the improved seawall and revetment design alone will not protect upland development under future sea level rise conditions; therefore, there likely would be additional costs associated with this alternative due to future increased storm damage to upland dwellings/infrastructure.

7.4 Mann Hill Beach

The shoreline change and coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, guided development of appropriate shore protection strategies for the Mann Hill Beach shoreline segment. The Mann Hill Beach shoreline represents the area immediately south of the North Scituate and Surfside Road areas that are fronted by coastal engineering structures, consisting of a mixed sediment beach and dune.

Based on observed dune and beach migration, the long-term shoreline erosion is relatively moderate (Appendix A); however, storm overwash has caused periodic migration of the dune crest in the landward direction. Specifically, the southern end of Mann Hill Beach, where dune restoration has not been performed, the dune exhibits both landward migration and lowering since 2000. Due to the performance of the cobble dune along the northern end of the beach, restoration of the entire dune is a potential option for protecting dwellings in jeopardy. The shore protection approaches for Mann Hill Beach are summarized in Table 7.9.

Table 7.9 Shore protection approaches and costs for Mann Hill Beach.

Shore Protection Approach	**Cost**	**50-Year Lifecycle Cost**
Constructed Dunes Stand-alone - 730 feet	$2.0 million	$16.3 million
Managed Retreat Move landward, all homes	>$1.5 million	Not applicable
Managed Retreat Buyout, all homes	>$2.3 million	Not applicable

Constructed Dunes

Figure 7.13 shows the location of the dune crest and high water shoreline along Mann Hill Beach and Egypt Beach. The five homes located on Stanton Lane are shown to be situated on or in front of the dune crest which largely increases the storm damage susceptibility of the home. To construct a dune seaward of these homes to satisfy the "540 rule" (discussed in Section 6.4), 60,000 cubic yards of cobble nourishment (to match the in situ material) spanning a length of 730 feet is required at a cost of approximately $2.0 million. A schematic of the proposed constructed dune profile is shown in Figure 7.14. The crest of the constructed dune is designed to be 22.5 feet NAVD88 to match the cobble dune along the north portion of Mann Hill Beach. Monitoring and periodic renourishment of the dunes is necessary to maintain the dune volume and storm damage protection.

Figure 7.13 2001 shoreline and dune crest along Mann Hill Beach and Egypt Beach.

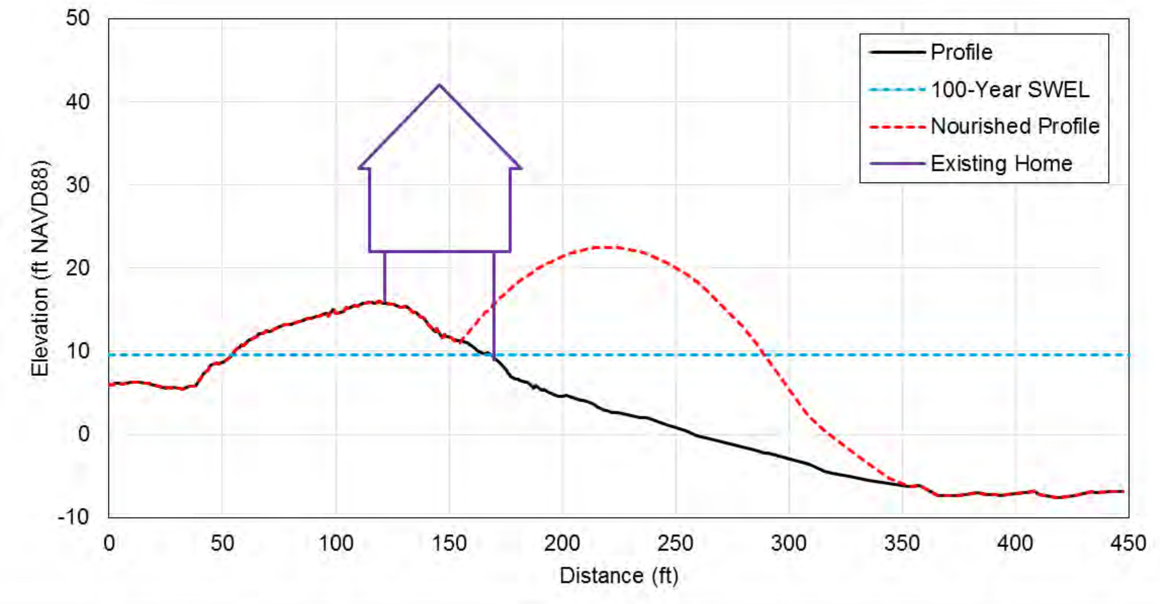


Figure 7.14 Schematic of the dune profile required to satisfy the "540 rule" along Mann Hill Beach.

Managed Retreat

In order to move landward, the properties on Mann Hill Beach may be able relocate their homes to the landward side of Stanton Lane. However, the land is privately owned and would require an agreement between the property owners in order to enable the move. The cost to relocate the 5 homes along Mann Hill Beach would be $1.5 million ($300,000 per home) plus the costs to facilitate an arrangement between property owners.

Buy-out of the homes by the Town is also an option if it is agreed upon by the Town and the property owners. The cost to buy-out all 5 homes is at least $2.3 million (assessed value) as the market value of the homes is historically greater than the assessed value.

Recommended Approach for Mann Hill Beach

The recommended shore protection approach for Mann Hill Beach is managed retreat either in the form of moving the homes landward or buy-outs (>$1.5 million). If beach nourishment is constructed along North Scituate Beach and Surfside Road, the longevity of development along Mann Hill Beach could be improved; however, continued erosion of the cobble dune landform will be difficult and/or cost-prohibitive to maintain in the long-term, especially if 2 feet of potential sea level rise is realized over the next 50 years. The cost of maintaining the status quo over the next 50 years is $4.2 million which includes the cost of FEMA repetitive loss claims and assuming the complete loss of property values due to continued erosion and increasing water levels.

7.5 Egypt Beach

The shoreline change and coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, guided development of appropriate shore protection strategies for the Egypt Beach shoreline segment. The Egypt Beach shoreline represents the area immediately south of Mann Hill Beach, consisting of a mixed sediment beach, as well as a “boulder platform” that is a lag deposit formed from erosion of glacial till by wave action. Based on observed beach migration, the long-term shoreline erosion is minimal (Appendix A), where the natural boulder platform stabilizes the shoreline position. Similar to Mann Hill Beach, the coarse-grained dune deposit along Egypt Beach becomes overwashed during significant storm events. Therefore, constructed dunes likely could be effective storm damage protection. The shore protection approaches for Egypt Beach are summarized in Table 7.10.

Table 7.10 Shore protection approaches and costs for Egypt Beach.

Shore Protection Approach	**Cost**	**50-Year Lifecycle Cost**
Constructed Dunes Stand-alone - 1,100 feet	$782,000	$6.4 million
Protection Improvements for Pump Station Egypt Beach Pump Station	$560,000	Not applicable
Boulder Dike 2,300 feet	$1.4 million	Not applicable

Constructed Dunes

Based on the “540 rule” (discussed in Section 6.4), the existing 1,000-foot cobble dune along Egypt Beach has a volume of 250 cubic feet per foot. The cobble dune would require an additional 23,000 cubic yards of cobble (to match the in situ material) to provide sufficient storm protection based on the “540 rule”. Figure 7.15 shows that the dune crest is increased from 20 to 22.5 feet NAVD88 in order to meet the “540 rule”. The dune will required regular monitoring and periodic renourishment, especially after large storms, to maintain the volume required to continue providing adequate storm protection. The estimated cost to construct the dunes is $782,000.

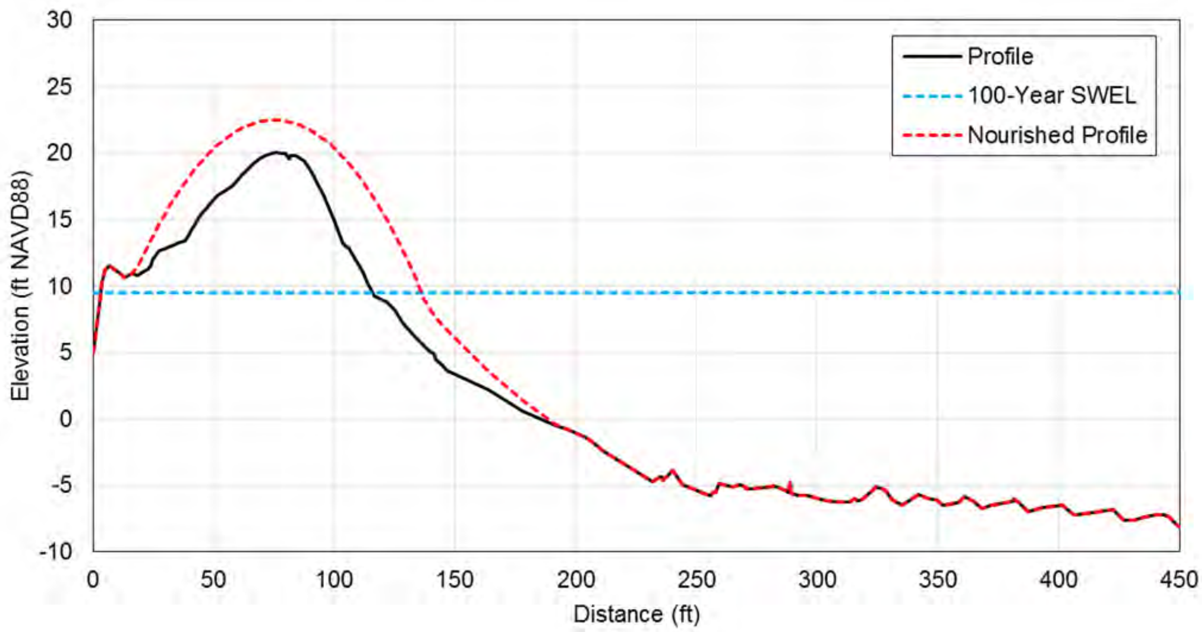


Figure 7.15 Schematic of the dune profile required to satisfy the “540 rule” along Egypt Beach.

Protection Improvements for Pump Station

Details of this approach is described in Section 6.9.

Boulder Dike

A rocky inter-tidal platform extends along Egypt Beach from Bay Street to Standish Avenue. There are patches of salt marsh located along the shore of the northern section, as shown in Figure 7.16. The elevation of the platform is approximately 3 feet NAVD88. The proposed boulder dike, presented in Figure 7.17, would be constructed along approximately 2,300 feet of Egypt Beach, from Bay Street to Standish Avenue, and at least 100 feet from the shore to avoid disturbing the existing salt marsh. Boulders weighing approximately 10 to 12 tons and approximately 6 feet in diameter would be placed in four staggered rows to increase wave energy dissipation over the platform during small storms (e.g. low return period storms like Winter Storm Nemo and Juno). The boulder spacing would be approximately 10 feet center-to-center. During significant storms (e.g. high return period storms like the Blizzard of 1978), the boulders would likely be submerged and less effective at dissipating wave energy.

The cost to construct the boulder dike is approximately $1.4 million. The project could easily be phased to accommodate funding constraints.

Figure 7.16 Salt marsh along the Egypt Beach (photo taken by Applied Coastal on May 12, 2016).

Recommended Approach for Egypt Beach

The recommended shore protection approach for Egypt Beach is to construct a boulder dike ($1.4 million) and to implement protection improvements for the Egypt Beach pump station ($560,000). The cost of maintaining the status quo along the study area is $7.5 million, which accounts for the projected FEMA repetitive loss claims over 50 years. It should be noted that the boulder dike alone does not provide protection from severe storm events; however, it is anticipated that a rejuvenated sediment supply via nourishment provided further to the north will allow long-term accretion along the landward side of the dike. In this case, the overall effect will be improved coastal resiliency over existing conditions.

Figure 7.17 Configuration of individual boulders (represented by the yellow circles) forming a boulder dike along Egypt Beach from Bay Street to Standish Avenue.

7.6 Oceanside Drive

The shoreline change and coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, guided development of appropriate shore protection strategies for the Oceanside Drive shoreline segment. The Oceanside Drive shoreline is fronted by a concrete seawall, with limited armor stone fronting the concrete structure. Based on the analysis of long-term shoreline change (Appendix A), the beach in this area has been stable over the past 60+ years. However, lowering of the beach along this entire shoreline stretch has been observed and several sections of failing seawalls have been replaced. Due to the relatively low elevation of development landward of the seawall, an increase in structure elevation and/or other measures to reduce wave energy were deemed worthy of consideration. Beach nourishment was considered but concerns regarding the fate of nourishment material remains, as the net south-directed littoral drift along Oceanside Drive could exacerbate shoaling within Scituate Harbor. The shore protection approaches for Oceanside Drive are summarized in Table 7.11.

Table 7.11 Shore protection approaches and costs for Oceanside Drive.		
Shore Protection Approach	**Cost**	**50-Year Lifecycle Cost**
Seawall and Revetment 10,000 feet	\$80.2 million	\$199.6 million
Beach Nourishment (50 foot berm) 7th Avenue to Scituate Avenue - 3,800 feet	\$7.2 million	\$89.0 million
Beach Nourishment (100 foot berm) 7th Avenue to Scituate Avenue - 3,800 feet	\$10.3 million	\$74.0 million
Drainage Improvements for the Basins	\$4.0 million	To be determined based on final design
Protection Improvements for Pumping Stations Sand Hills Pump Station	\$560,000	Not applicable

Seawall and Revetment

The existing seawall along Oceanside Drive varies in elevation and structural condition. Rehabilitation of the seawall and revetment is underway along 4th Avenue to 6th Avenue and construction is planned for 11th Avenue to Kenneth Road. Construction for the seawall area across from 7th Avenue is currently pending. Rehabilitation plans by CLE Engineering indicated that the seawall was raise from 17 feet to 19 feet NAVD88 and the fronting revetment was replaced with new stone while staying within the existing structure footprint. The proposed design is based on the implemented design by CLE Engineering; a simplified schematic of the proposed design is presented in Figure 7.18. The condition of the structure was reported to be “fair” and “poor” by CLE Engineering in 2013.

Table 7.12 summarizes the wave overtopping rates of the pre-rehabilitation structure and the proposed structure during a 100-year storm. The proposed design reduces wave overtopping by nearly 60%, preventing pavement damage behind the seawall. However, the proposed design is unable to reduce overtopping sufficiently under a hypothetical 2-foot sea level rise (SLR) scenario.

The construction cost of rehabilitating the entire seawall and revetment along the Oceanside Drive study area (10,000 feet) is \$80.2 million plus ongoing maintenance costs.

Table 7.12 Wave overtopping and predicted damage at Oceanside Drive during 100-year storm conditions. The proposed design involves increasing the height of the existing seawall by 2 feet and expanding the fronting revetment.		
Scenario	**Wave Overtopping ($ft^3/s/ft$)**	**Predicted Damage**
Existing Design	0.9	Yes
Existing with 2 feet of SLR	3.8	Yes
Proposed Design	0.4	No
Proposed with 2 feet of SLR	1.8	Yes

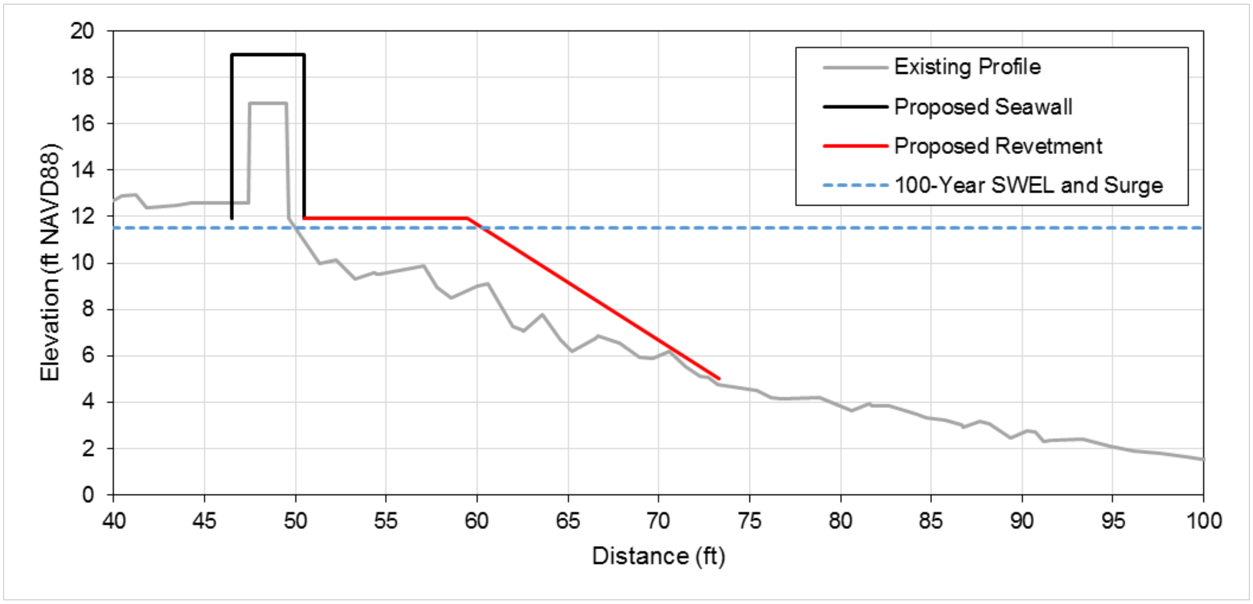


Figure 7.18 Schematic of proposed design along Oceanside Drive where the seawall height is increased by 2 feet and the fronting revetment is expanded.

Beach Nourishment

Two beach nourishment options were considered for Oceanside Drive extending from 7th Avenue to Jericho Road (3,800 feet) as presented in Figure 7.19 with berm widths of 50 and 100 feet. The berm elevation in both options was 13 feet NAVD88 to correspond with the 50-year runup elevation. Table 7.13 summarizes the required nourishment volume, footprint area, and construction cost for each option.

The placed sediment generally migrates south towards Cedar Point. Sediment is transported from the north end of the nourishment template within the first year of nourishment and does not provide sufficient protection for the properties north of Kenneth Road. Figure 7.20 shows the modeled volume of fill remaining within the fill limits over time. Renourishment is required when the volume of fill remaining within the fill limits falls below 30% and is required at approximately 6.5 and 10 years for the 50- and 100-foot berm nourishments, respectively. While the estimated design life for the options are suitable, there are several concerns associated with placing nourishment along Oceanside Drive that need to be further considered. There is potential for sediment to migrate to and enter Scituate Harbor which may require dredging to keep navigational pathways open. There are also two storm water outfalls on the beach along Oceanside Drive, one south of 7th Avenue and another at 11th Avenue, which may become blocked with nourishment material.

Figure 7.19 Nourishment options for Oceanside Drive extending from 7th Avenue to Jericho Road.

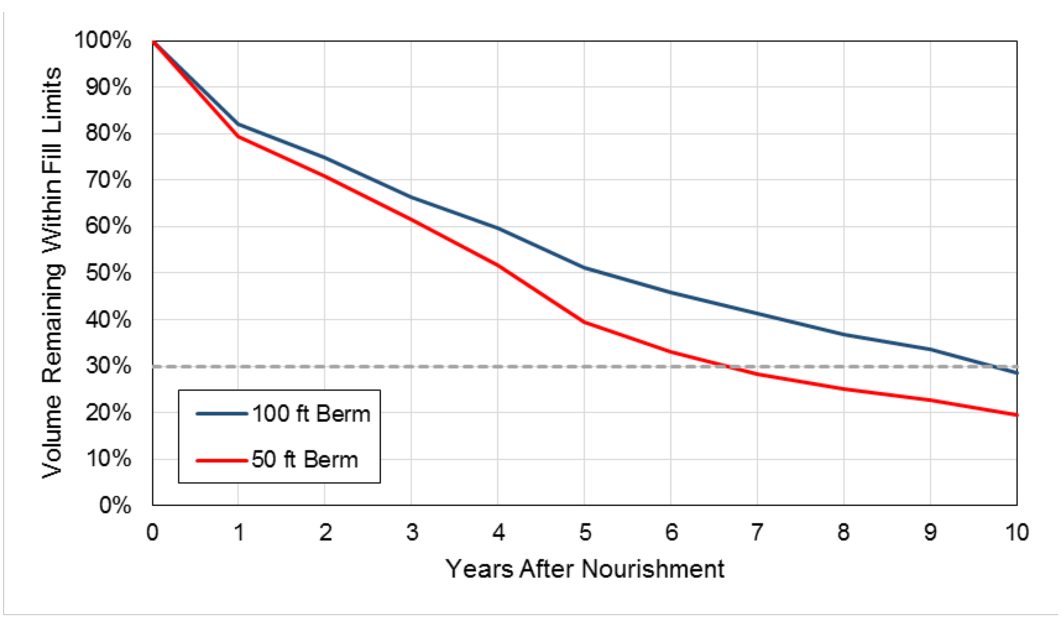


Figure 7.20 Volume of fill remaining over the course of the model simulation for two nourishment options along Oceanside Drive. The dashed gray line indicates the 30% remaining design threshold.

Table 7.13 Nourishment options for Oceanside Drive.

Berm Width (feet)	Nourishment Volume (cubic yards)	Nourishment Footprint (acre)	Cost
50	211,000	14.8	$7.2 million
100	302,000	18.9	$10.3 million

Drainage Improvements for the Basins

Details of this approach is described in Section 6.8.

Protection Improvements for Pump Station

Details of this approach is described in Section 6.9.

Recommended Approach for Oceanside Drive

The recommended shore protection approaches for Oceanside Drive are to rehabilitate the seawall and revetments, improve drainage of the basins, and to improve the protection to the Sand Hills pump station. The greatest cost is the seawall and revetment; the initial construction cost is $80.2 million with a total 50-year lifecycle cost of $199.6 million, which is lower than the cost of maintaining the status quo over 50 years ($246.8 million). While beach nourishment can be implemented along Oceanside Drive at a lower cost, there are obstacles in providing lasting protection for the northern portions of study area and the possibility of inhibiting and/or blocking navigational pathways into Scituate Harbor and outfalls from the basins. If beach nourishment is revisited as a potential alternative for this area, additional analyses of groins to reduce down-drift losses of sediment, as well as a thorough analysis of possible harbor shoaling concerns, should be performed.

7.7 Cedar Point

The shoreline change and coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, guided development of appropriate shore protection strategies for the Point shoreline segment. The Cedar Point shoreline is fronted by a concrete seawall, with some placed armor stone fronting the series of concrete structures. Based on the analysis of long-term shoreline change (Appendix A), the beach in this area has been stable over the past 60+ years. However, the beach fronting Cedar Point consists of a "boulder platform", similar to Egypt Beach. Due to the relatively low elevation of development landward of the seawall, an increase in structure elevation and/or other measures to reduce wave energy were deemed worthy of consideration. Due to the proximity to the Scituate Harbor entrance, beach nourishment only was considered in the area north of the boulder platform. Potential use of a boulder dike to reduce wave energy along the area fronted by the boulder platform also was considered. The shore protection approaches for Cedar Point are summarized in Table 7.14.

Table 7.14 Shore protection approaches and costs for Cedar Point.		
Shore Protection Approach	**Cost**	**50-Year Lifecycle Cost**
Seawall and Revetment Rebecca Road - 1,300 feet	$10.4 million	$25.9 million
Beach Nourishment Cobble berm - 1,200 feet	$4.6 million	$17.1 million
Boulder Dike 1,200 feet	$720,000	Not applicable

Seawall and Revetment

Approximately 1,300 feet of the existing seawall and revetment along Cedar Point is publically owned with the limits from 11 Lighthouse Road to 3 Rebecca Road and 61 to 91 Rebecca Road. The seawalls are generally rated as “fair” by CLE Engineering in 2013, however one section of seawall west of 17 Rebecca Road is considered to be “failing”. Under a 100-year storm, wave overtopping exceeds the damage threshold for pavement behind the seawall. With a 2-foot increase in the seawall height and expansion of the revetment, shown in Figure 7.21, the structure adequately protects against a 100-year storm. Table 7.15 shows that under a hypothetical 2-foot sea level rise (SLR) scenario neither the existing or proposed structure would provide adequate protection against pavement damage.

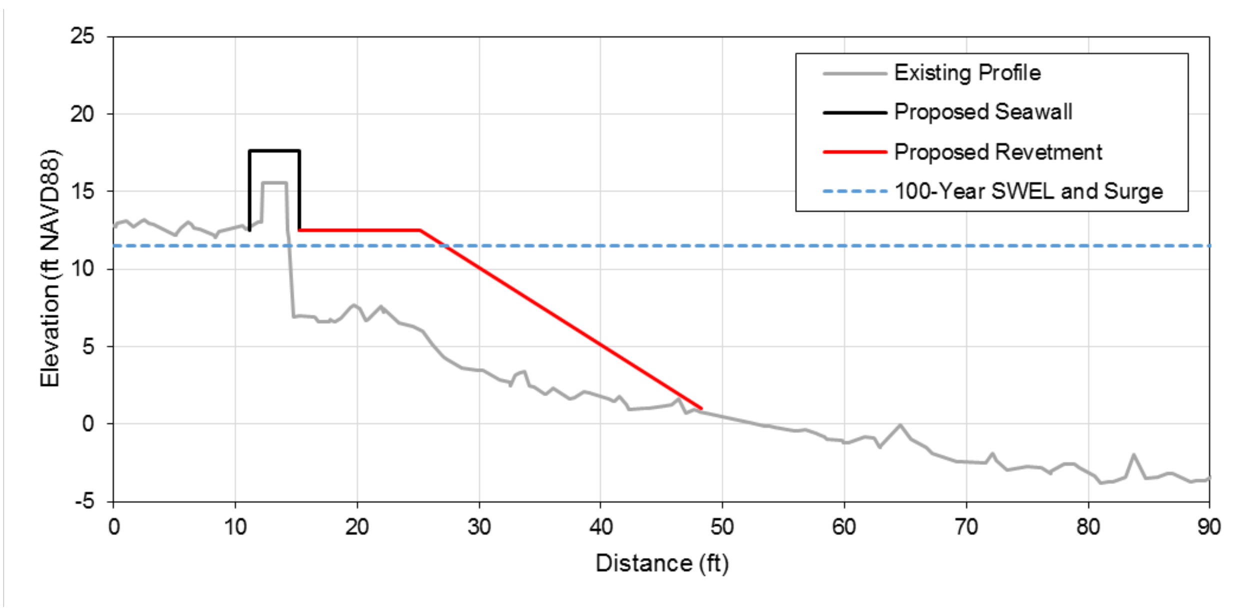


Figure 7.21 Schematic of proposed seawall and revetment design along Cedar Point (Rebecca Road) where the seawall height is increased by 2 feet and the fronting revetment is expanded.

There are no coastal structures in place from 17 to 37 Rebecca Road. From 39 to 59 Rebecca Road, private coastal structures exist. While condition ratings were not completed by CLE Engineering for private structures, a site visit showed that the walls were in poor condition.

The cost to reconstruct the public seawall structure is approximately $10.4 million not including ongoing maintenance costs. Reconstruction of the private portion of the seawall would cost an additional $4.3 million.

Table 7.15 Wave overtopping and predicted damage at Cedar Point during 100-year storm conditions. The proposed design involves increasing the height of the existing seawall by 2 feet and expanding the fronting revetment.

Scenario	Wave Overtopping ($ft^3/s/ft$)	Predicted Damage
Existing Design	1.4	Yes
Existing with 2 feet of SLR	5.4	Yes
Proposed Design	0.5	No
Proposed with 2 feet of SLR	2.7	Yes

Beach Nourishment

A cobble nourishment is proposed from 153 Turner Road to the parking area on Rebecca Road (1,200 feet) to reduce wave overtopping and to provide breaching protection for Lighthouse Road. Cobble beaches tend to reshape over time to dissipate waves and reduce overtopping while providing a more stable beach relative to sand. Figure 7.22 shows a natural cobble beach at Mann Hill Beach. The nourishment, with a berm width of 60 feet and a crest elevation of 12 feet NAVD88, equilibrates over time as shown in Figure 7.23. From the berm, the beach slopes seaward at a 1V:4H slope until it intersects with the ocean bottom. During equilibration, see Figure 6.7, the majority of the cobble is transported to the southeast and builds up along the west-facing portion of Rebecca Road. The profile of the cobble nourishment is expected to reshape immediately after construction and the berm crest will adjust to match the runup level to minimize overtopping (CIRIA/CUR, 1991). The reshaped profile along the narrowest section of Lighthouse Road is shown in Figure 7.24. Based on Powell (1990), the crest of the cobble berm after a 50-year storm is expected to reach 17 feet NAVD88 which is approximately 2 feet higher than the existing seawall.

Approximately 137,000 cubic yards of cobble material is required at a cost of $4.6 million. Preliminary shoreline modeling shows that the nourishment equilibrates and stabilizes without renourishment, however maintenance and monitoring is required to document the movement of the material over time.

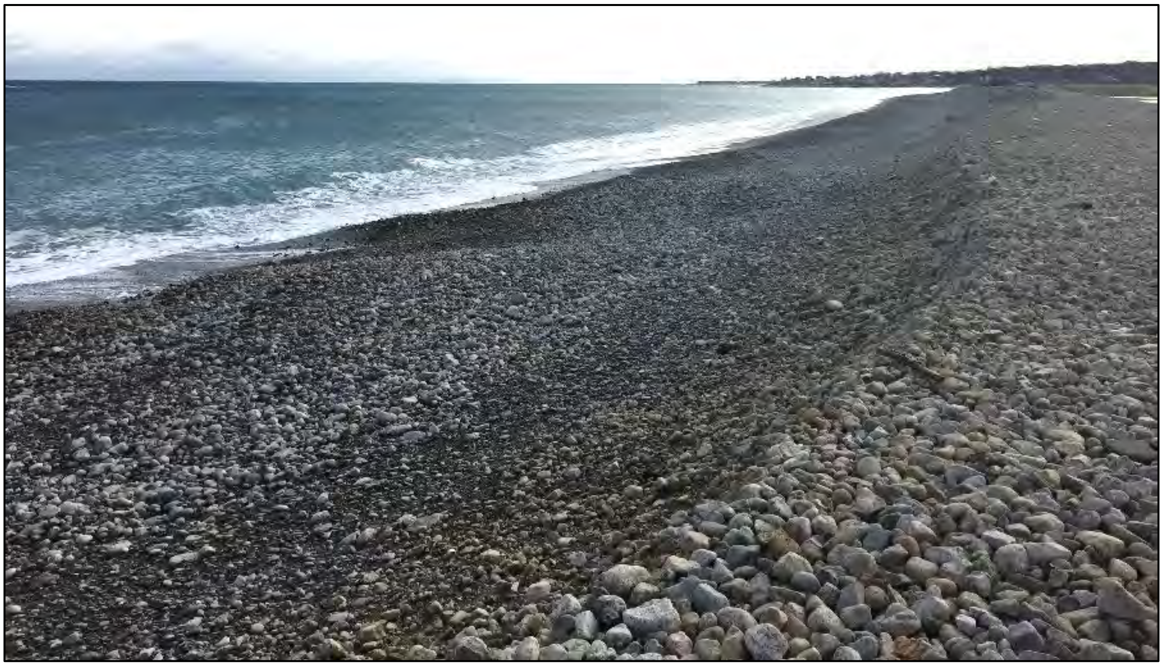


Figure 7.22 Natural cobble beach at Mann Hill Beach in Scituate, MA.

Figure 7.23 Cobble-nourished and equilibrated shoreline along Turner Road and Lighthouse Road.

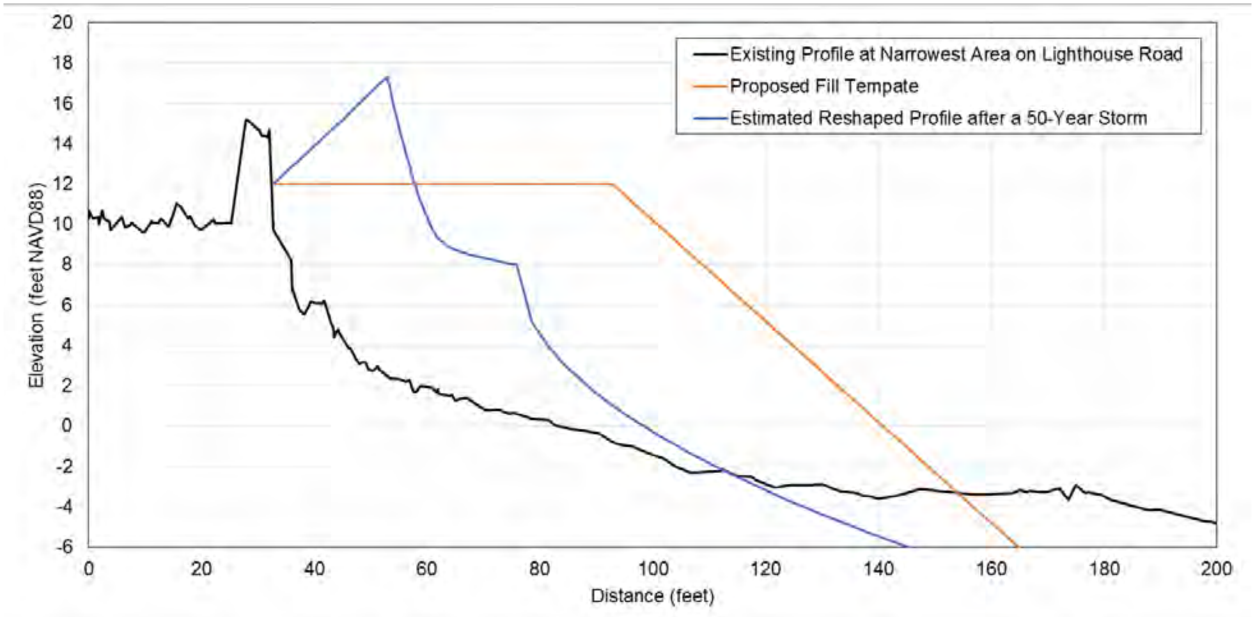


Figure 7.24 Existing profile, proposed fill template, and reshaped profile after a 50-year storm at the narrowest area on Lighthouse Road.

Boulder Dike

A rocky inter-tidal platform scattered with large glacial erratics runs along the north-west shore of Cedar Point. The proposed boulder dike would extend 1,200 feet along the platform from the area with no seawall starting at 17 Rebecca Road to the east end of the private seawall at 59 Rebecca Road. The elevation of the platform is approximately 3 feet NAVD88. Four staggered rows of 10 to 12 ton boulders measuring approximating 6 feet in diameter would be placed to increase wave energy dissipation over the platform during small storms. The boulder spacing would be approximately 10 feet center-to-center. Figure 7.25 shows a schematic of the boulder placement along Cedar Point. During significant storms, the boulders would likely be submerged and less effective at dissipating wave energy.

The cost to construct the boulder dike is approximately $720,000. The project could be constructed in phases to accommodate to funding restraints.

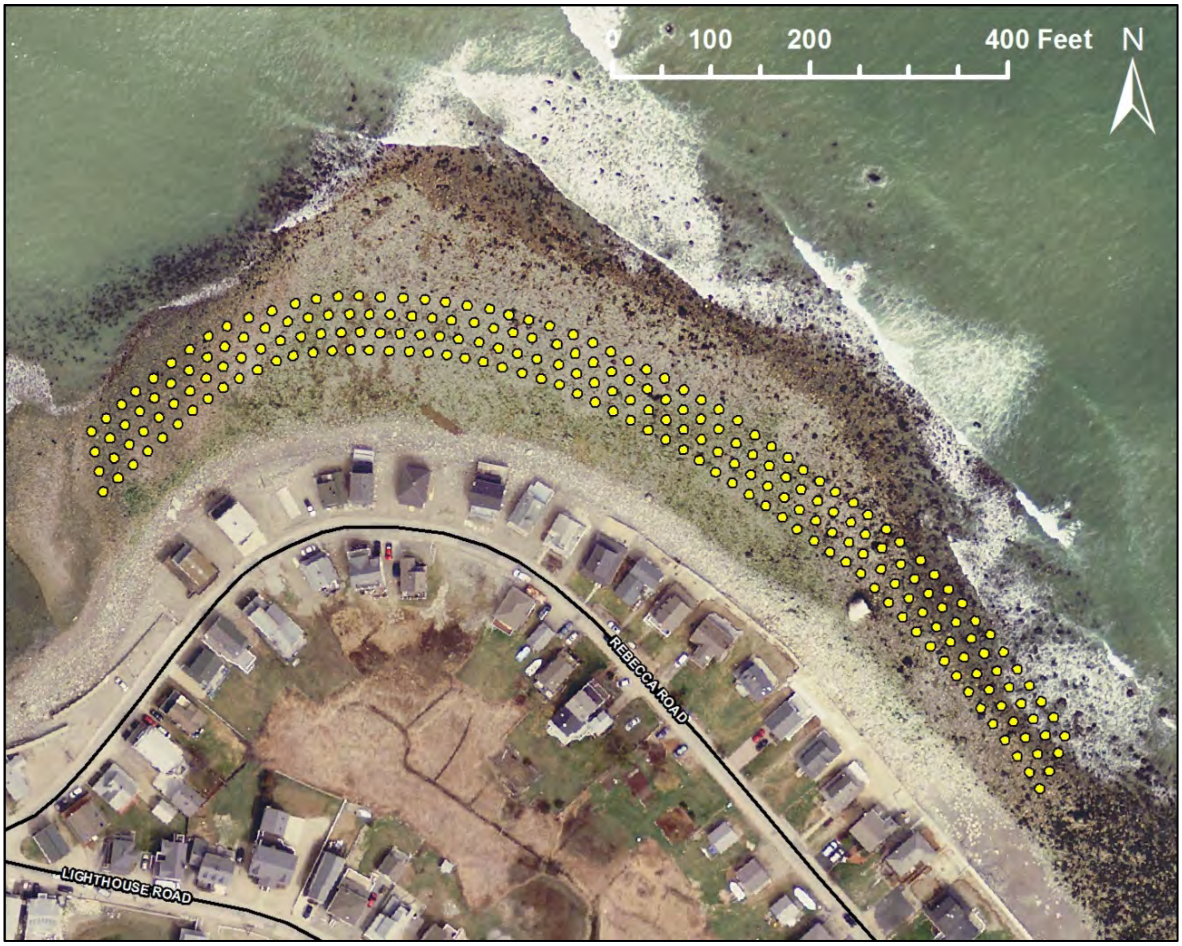


Figure 7.25 Configuration of individual boulders (represented by the yellow circles) forming a boulder dike along Cedar Point.

Recommended Approach for Cedar Point

The recommended shore protection approach for Cedar Point is to rehabilitate the existing seawall and revetments, place cobble nourishment along the narrow section of Lighthouse Road, and install a boulder dike. The 50-year lifecycle cost of these approaches is approximately $43.7 million. While the cost is higher than the cost to maintain the status quo ($36.4 million), the benefits include increased storm protection, upgraded condition of the existing coastal engineering structures, and improved emergency egress. In the case of Cedar Point, a major portion of the existing dwellings are well below the 100-year still water elevation and any increase in sea level will have a marked effect on this highly vulnerable area.

7.8 First Cliff

Utilizing information developed from the coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, appropriate shore protection strategies were developed for the First Cliff shoreline segment. The shoreline change assessment (Appendix A) indicates that the shoreline has been stable over the past 60+ years, as a result of the armoring fixing the position of the shoreline. Overall, the existing revetment dimensions appear to be adequate for shore protection based upon the local wave analysis. Additionally, elevating portions of Edward Foster Road was considered to provide emergency egress during periods of combined high tide and storm surge. The shore protection approaches for First Cliff are summarized in Table 7.16.

Table 7.16 Shore protection approaches and costs for First Cliff.

Shore Protection Approach	Cost	50-Year Lifecycle Cost
Revetment 1,700 feet	Maintenance costs only	$10.3 million
Elevate Edward Foster Road For emergency access - 800 feet	$600,000	Not applicable
Elevate Edward Foster Road (Causeway) For emergency access - 1,800 feet	$1.8 million	Not applicable

Revetment

Under both existing and a hypothetical 2-foot sea level rise (SLR) scenarios, the 1,700-foot revetment at First Cliff is shown to provide adequate protection against pavement scour during the 100-year storm as summarized in Table 7.17. The wide revetment berm is located nearly 5 feet above the storm surge level, as shown in Figure 7.26, and effectively dissipates waves. The condition rating of the revetment is “poor” based on inspections by CLE Engineering in 2013, therefore repairs and continued maintenance of the structure is required.

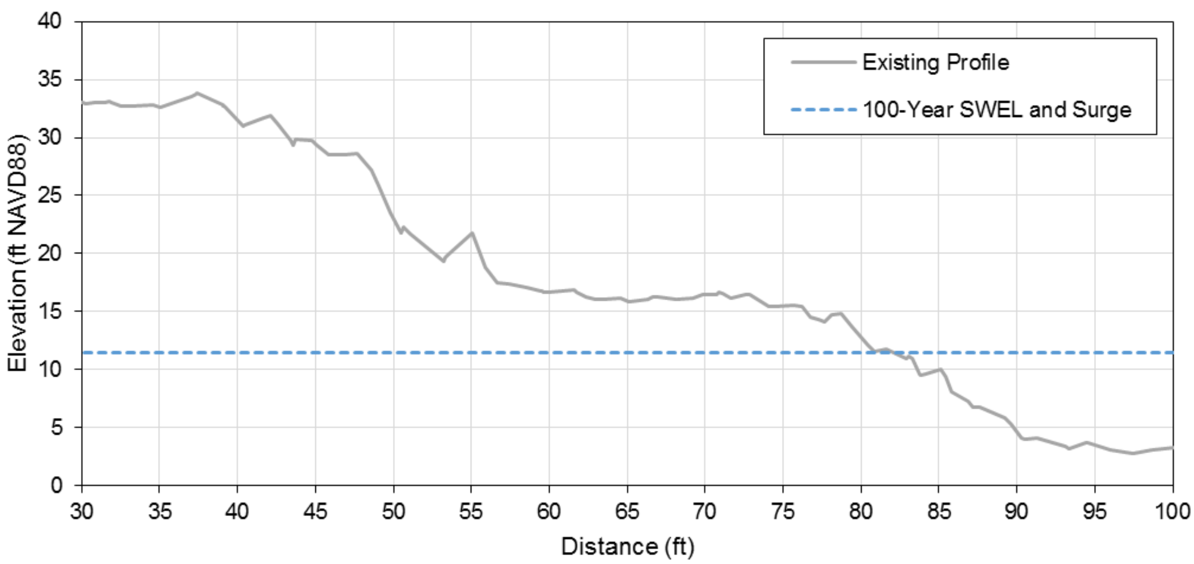


Figure 7.26 Profile of the existing revetment along First Cliff.

Table 7.17 Wave overtopping and predicted damage at First Cliff during 100-year storm conditions.

Scenario	Wave Overtopping ($ft^3/s/ft$)	Predicted Damage
Existing Design	0.1	No
Existing with 2 feet of SLR	0.4	No

Elevate Edward Foster Road

Details of this approach is described in Section 7.9.

Elevate Edward Foster Road (Bridge)

Details of this approach is described in Section 7.9.

Recommended Approach for First Cliff

The recommended shore protection approach for First Cliff is to maintain the status quo ($10.3 million over 50 years). Plans for repair to First, Second, and Third Cliff are underway to address damages incurred from over the last several years from Hurricane Sandy, Winter Storm Nemo, and Winter Storm Juno.

7.9 Edward Foster Road

Utilizing information developed from the coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, appropriate shore protection strategies were developed for the Edward Foster Road shoreline segment. The shoreline change assessment (Appendix A) indicates that the shoreline has been stable over the past 60+ years, as a result of the armoring fixing the position of the shoreline, although the area contains a narrow beach fronting the armored shoreline. Improvements to the revetment/seawall were considered to ensure adequate long-term shore protection. Additionally, elevating portions of Edward Foster Road was considered to provide emergency egress during periods of combined high tide and storm surge. The shore protection approaches for Edward Foster Road are summarized in Table 7.18.

Table 7.18 Shore protection approaches and costs for Edward Foster Road.		
Shore Protection Approach	**Cost**	**50-Year Lifecycle Cost**
Seawall and Revetment 970 feet	$7.8 million	$19.4 million
Elevate Edward Foster Road For emergency access - 800 feet	$600,000	Not applicable
Elevate Edward Foster Road (Causeway) For emergency access - 1,800 feet	$1.4 million	Not applicable

Seawall and Revetment

Seawall rehabilitation plans by CLE Engineering proposed that the northernmost portion of the structure fronting 138 Edward Foster Road be replaced with seawall approximately 4.7 feet higher than the existing wall and a new revetment. As of June 2016, the rehabilitation of the seawall at 138 Edward Foster Road has been completed. A simplified schematic of the design is shown in Figure 7.27. The design effectively decreases pavement damage from wave overtopping to below critical levels under current and hypothetical 2-foot sea level rise (SLR) scenarios as summarized in Table 7.19.

The condition of the seawall along Edward Foster Road is generally rated as “good” by CLE Engineering in 2013. The cost of increasing the height of the entire structure (970 feet) and adding a new revetment along Edward Foster Road is approximately $7.8 million. Regular maintenance and repairs will also be required throughout the design life to ensure the integrity of the structure.

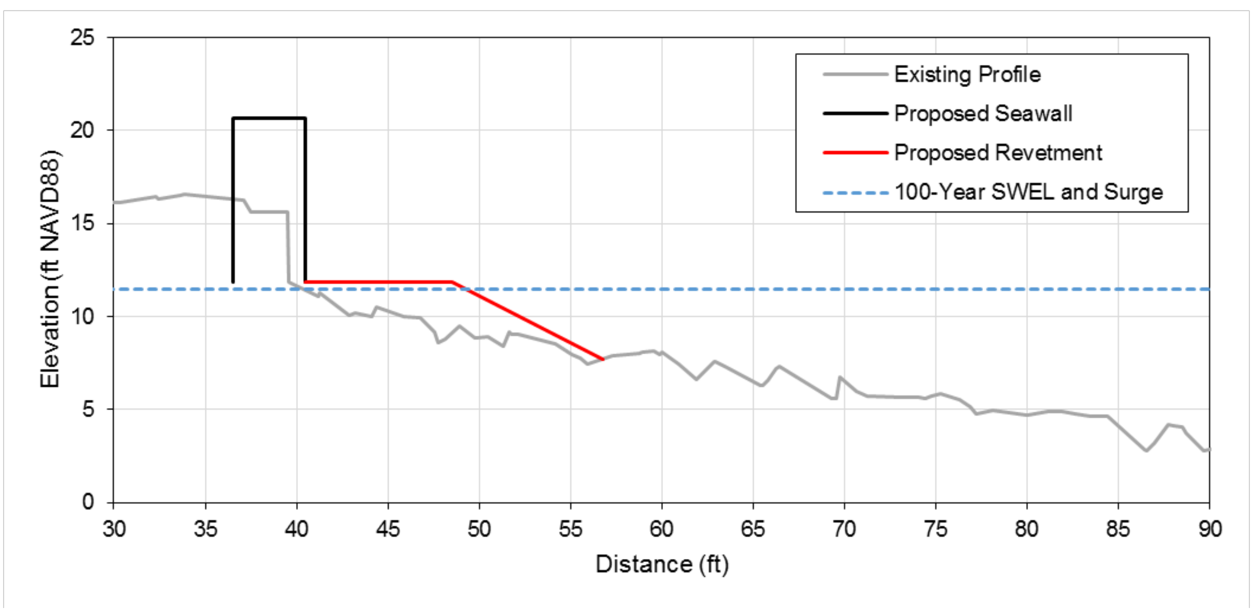


Figure 7.27 Schematic of the proposed seawall and revetment design along Edward Foster Road where the seawall height is increased by 4.7 feet and the fronting revetment is expanded.

Table 7.19 Wave overtopping and predicted damage at Edward Foster Road during 100-year storm conditions. The proposed design involves raising the height of the existing seawall by 4.7 feet and expanding the fronting revetment.		
Scenario	**Wave Overtopping ($ft^3/s/ft$)**	**Predicted Damage**
Existing Design	0.5	No
Existing with 2 feet of SLR	2.9	Yes
Proposed Design	0.1	No
Proposed with 2 feet of SLR	0.4	No

Elevate Edward Foster Road

The section of Edward Foster Road between 100 Edward Foster Road and 138 Edward Foster Road (800 feet) serves the only emergency access route for First Cliff. Flooding of the road generally occurs through flooding from the harbor side of the road, as shown in Figure 7.28. On average, the road is 0.7 feet and 1.2 feet below the 10- and 100-year still water elevation, respectively.

A total of 8 homes along the road section have driveways connected to Edward Foster Road and the driveways will need to be raised to meet the new road elevation. The cost to elevate Edward Foster Road and the associated utilities is approximately $600,000.

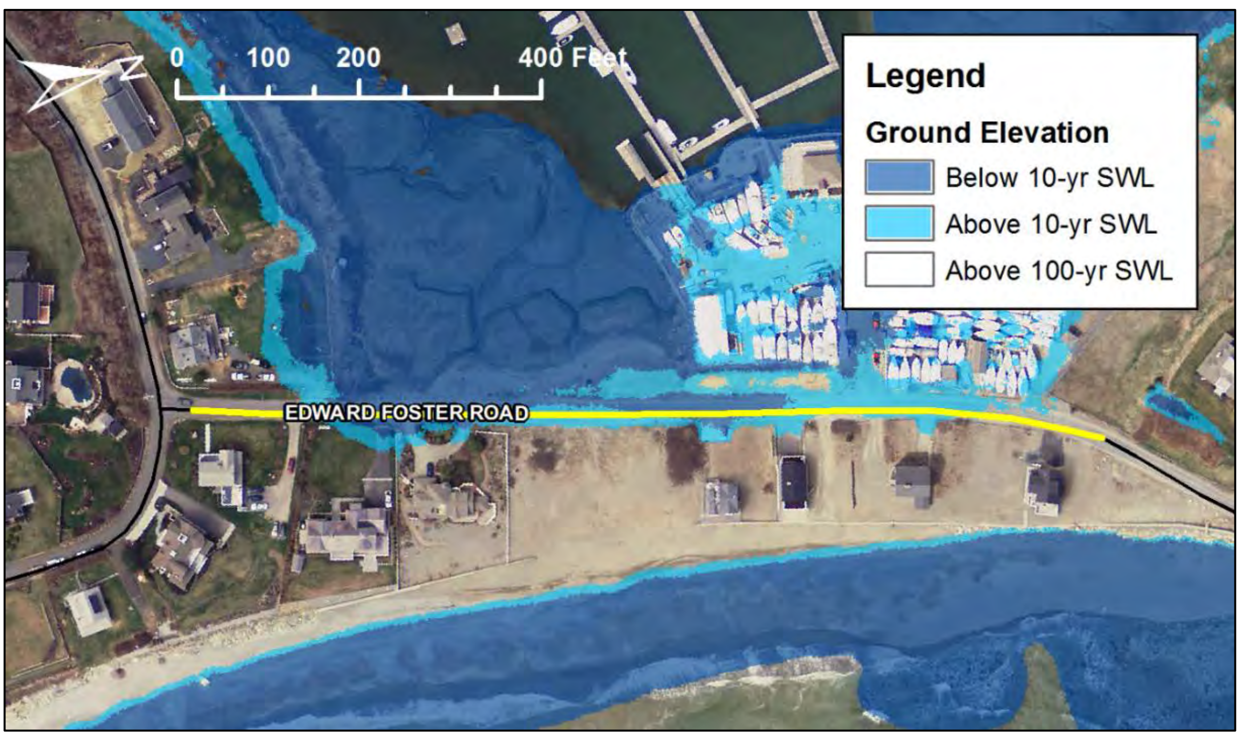


Figure 7.28 Flooding extents of the 10-year and 100-year still water elevation across Edward Foster Road.

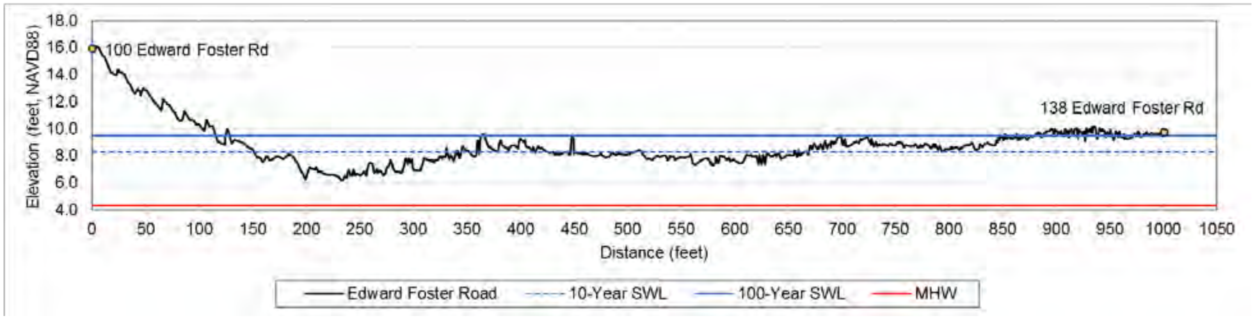


Figure 7.29 Road elevation from 100 Edward Foster Road to 138 Edward Foster Road.

Elevate Edward Foster Road (Causeway)

The Edward Foster Road causeway, located between 17 First Parish Road and Peggotty Beach Road (1,800 feet) provides emergency access to First and Second Cliff. The Town has noted that the causeway is often flooded during high tide. The lowest section of the causeway is located about 500 feet from the bridge and is only about 2 feet above mean high water. On average, the road is 0.8 feet and 1.6 feet below the 10- and 100-year still water elevation. The flooding extents and elevation of the road is presented in Figure 7.30 and Figure 7.31.

The cost to elevate Edward Foster Road and the associated utilities is approximately $1.4 million. The elevation of the road may be completed in conjunction with repairs to the existing revetment along the bridge. The condition of the revetment was rated as “poor” by CLE Engineering in 2013. The condition rating does not include the structural condition of the Edward Foster Road Bridge.

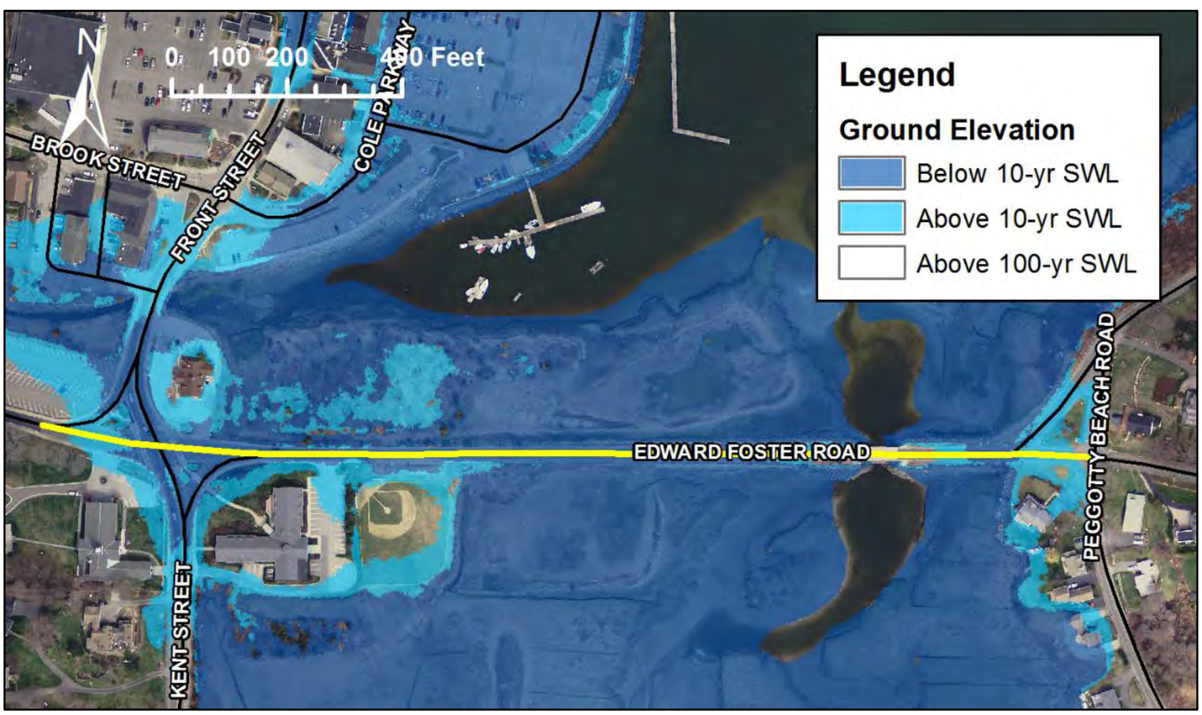


Figure 7.30 Flooding extents of the 10-year and 100-year still water elevation across Edward Foster Road (causeway).

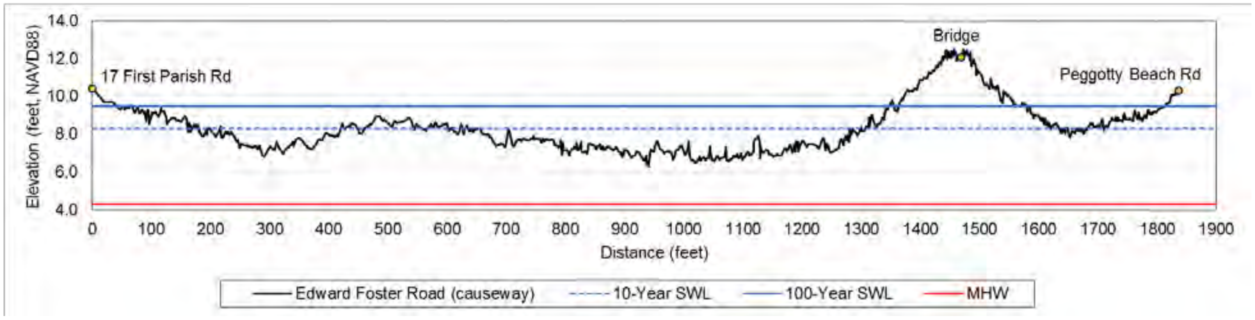


Figure 7.31 Road elevation along Edward Foster Road from 17 First Parish Road to Peggotty Beach Road.

Recommended Approach for Edward Foster Road

The recommended shore protection approach for Edward Foster Road is to rehabilitate the seawall and revetment at a cost of $19.4 million over a 50-year lifecycle ($7.8 million initial cost). While the cost of maintaining the status quo is lower at $11.7 million over 50 years, the rehabilitated structure can protect against increased wave overtopping due to potential sea level rise and maintain the emergency egress between First and Second Cliff.

7.10 Second Cliff

Utilizing information developed from the coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, appropriate shore protection strategies were developed for the Second Cliff shoreline segment. The shoreline change assessment (Appendix A) indicates that the shoreline has been stable over the past 60+ years, as a result of the armoring fixing the position of the shoreline. Improvements to the revetment were considered to ensure adequate long-term shore protection. Additionally, elevating portions of Edward Foster Road was considered to provide emergency egress during periods of combined high tide and storm surge. The shore protection approaches for Second Cliff are summarized in Table 7.20.

Table 7.20 Shore protection approaches and costs for Second Cliff.

Shore Protection Approach	**Cost**	**50-Year Lifecycle Cost**
Revetment 2,200 feet	$8.9 million	$22.2 million
Elevate Edward Foster Road For emergency access - 800 feet	$600,000	Not applicable
Elevate Edward Foster Road (Causeway) For emergency access - 1,800 feet	$1.8 million	Not applicable

Revetment

The condition of the 2,200-foot revetment along Second Cliff was generally rated as "poor" by CLE Engineering in 2013. Under existing conditions, wave overtopping during the 100-year storm is shown to damage pavement. However, the history of storm damage on Second Cliff does not indicate that wave overtopping is an issue. Figure 7.32 shows that the lack of damage to private and public infrastructure may be attributed to the distance at which the homes are set back from the top of the revetment and the vegetation fronting the homes which aids in dissipating the energy of the overtopped waves.

Figure 7.33 illustrates a potential revetment project that may be undertaken to reduce storm damage, especially under a hypothetical 2-foot sea level rise (SLR) scenario. The revetment is modeled after the structure at First Cliff which includes a wide, above-surge berm. Table 7.21 summarizes the calculated wave overtopping rates. The cost of the project is approximately $8.9 million and ongoing maintenance and repairs costs.

Figure 7.32 Existing revetment along Second Cliff (photo by Kevin Ham).

Table 7.21 Wave overtopping and predicted damage at Second Cliff during 100-year storm conditions. The proposed design involves extending the revetment to a higher elevation and adding a berm.

Scenario	Wave Overtopping ($ft^3/s/ft$)	Predicted Damage
Existing Design	3.6	Yes
Existing with 2 feet of SLR	8.4	Yes
Proposed Design	0.1	No
Proposed with 2 feet of SLR	0.5	No

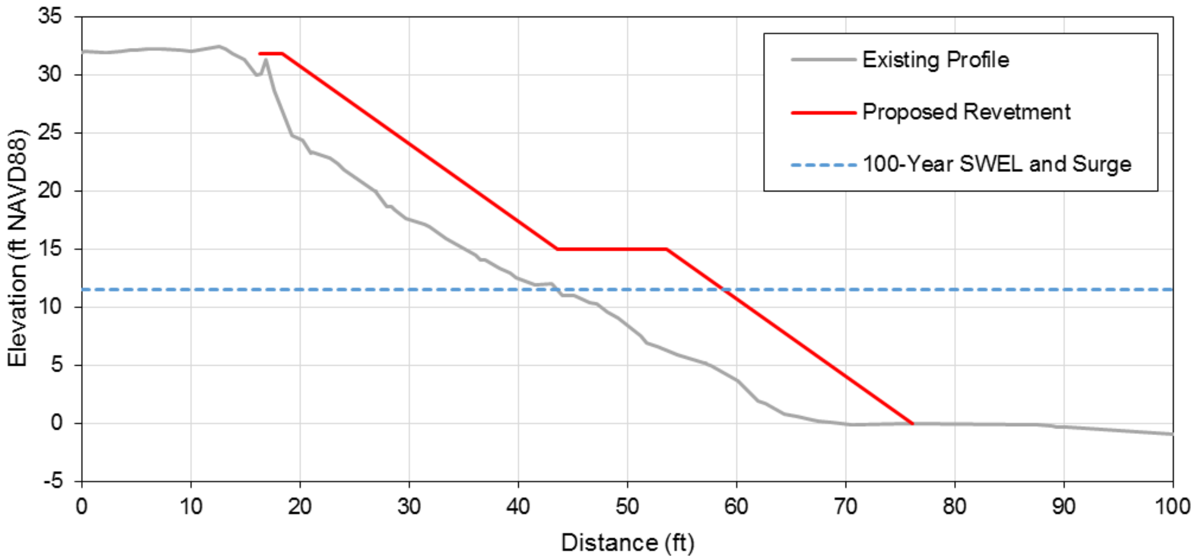


Figure 7.33 Schematic of the proposed revetment design along Second Cliff where the revetment is extended to a higher elevation and a berm is added to dissipate wave energy.

Elevate Edward Foster Road

Details of this approach is described in Section 7.9.

Elevate Edward Foster Road (Bridge)

Details of this approach is described in Section 7.9.

Recommended Approach for Second Cliff

The recommended shore protection approach for Second Cliff is to maintain the status quo ($13.3 million over 50 years). Plans for repair to First, Second, and Third Cliff are underway to address damages incurred from over the last several years from Hurricane Sandy, Winter Storm Nemo, and Winter Storm Juno.

7.11 Peggotty Beach

The shoreline change and coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, guided development of appropriate shore protection strategies for the Peggotty Beach shoreline segment. The Peggotty Beach shoreline represents the area immediately south of the Second Cliff segment and immediately north of the Third Cliff segment that are both fronted by extensive coastal engineering structures. Based on observed dune and beach migration, the long-term shoreline erosion is the highest of any developed section of the Scituate shoreline, with retreat rates between 2 and over 4 feet per year (Appendix A). Due to the relatively high shoreline erosion rates and the generally low elevation of the existing dune system, potential alternatives included both beach and dune nourishment.

Managed retreat also was explored as an option, due primarily to the challenges associated with nourishment options. The shore protection approaches for Peggotty Beach are summarized in Table 7.22.

Table 7.22 Shore protection approaches and costs for Peggotty Beach.

Shore Protection Approach	Cost	50-Year Lifecycle Cost
Beach Nourishment (50 foot berm) North Peggotty Beach only - 780 feet	$1.6 million	$62.7 million
Beach Nourishment (100 foot berm) North Peggotty Beach only - 780 feet	$3.0 million	$62.0 million
Beach Nourishment (50 foot berm) Entire beach - 1,800 feet	$2.9 million	$57.0 million
Beach Nourishment (100 foot berm) Entire beach - 1,800 feet	$5.8 million	$54.1 million
Constructed Dunes Stand-alone, north Peggotty Beach only 780 feet	$918,000	$7.5 million
Constructed Dunes Stand-alone, south Peggotty Beach only 1,000 feet	$2.7 million	$22.0 million
Managed Retreat Move landward, all homes	>$4.8 million	Not applicable
Managed Retreat Buy-out, all homes	>$8.7 million	Not applicable

Beach Nourishment

Four nourishment options were considered for Peggotty Beach. Details of the nourishment volume, footprint area, and construction cost for each option is summarized in Table 7.23. In all the options, the proposed berm elevation is 13 feet NAVD88 which corresponds to the wave runup elevation during a 50-year storm. The first two options, as shown in Figure 7.34, consist of a 50- and 100-foot berm nourishment along the northern area where the beach is used more regularly by public beachgoers. From the berm, the beach slopes seaward at a 1V:10H slope until it intersects with the ocean bottom. Renourishment is required when less than 30% of the initial volume of fill remains within the initial fill limits. Shoreline change modeling over a 10-year period predicted that renourishment will be required at 2- and 4- year intervals for the 50- and 100-foot berm options, respectively. Where the nourishment is extended across the entire length of the Peggotty Beach, as shown in Figure 7.35, the renourishment interval is extended: approximately 4.5 year for the 50-foot berm and 9 years for the 100-foot berm. The predicted volume remaining over time is presented in Figure 7.36 and Figure 7.37.

Monitoring of the beach nourishment performance is recommended to determine when renourishment is required, to assess accretion or erosion of adjacent beach, and to identify any potential “hot-spot” erosion (areas of anomalously high erosion).

Figure 7.34 Two nourishment options for the northern area of Peggotty Beach.

Figure 7.35 Two nourishment options for the entire length of Peggotty Beach.

Figure 7.36 Volume of fill remaining over the course of the model simulation for a nourishment along the northern area of Peggotty Beach. The dashed gray line indicates the 30% remaining design threshold.

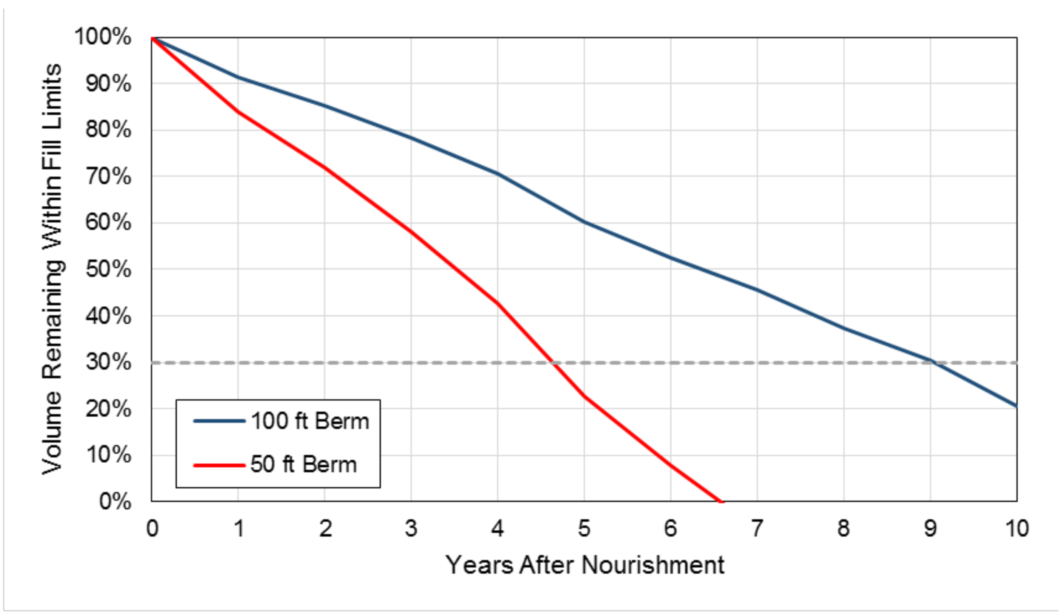


Figure 7.37 Volume of fill remaining over the course of the model simulation for a nourishment along the entire length of Peggotty Beach. The dashed gray line indicates the 30% remaining design threshold.

Table 7.23 Beach nourishment options for Peggotty Beach.

Option	Berm Width (feet)	Nourishment Volume (cubic yards)	Nourishment Footprint (acre)	Cost
Northern area 780 feet	50	46,000	6.2	$1.6 million
	100	88,000	7.3	$3.0 million
Entire beach 1,800 feet	50	84,000	12.8	$2.9 million
	100	172,000	15.5	$5.8 million

Constructed Dunes

Figure 7.38 shows the location of the dune crest and high water shoreline along Peggotty Beach. Along the northern portion of the beach (780 feet), the seven homes are situated landward of the dune crest while along the southern portion (1,000 feet) the homes are generally location on or seaward of the dune crest. The susceptibility of storm damage is increased when homes are located on the seaward side of the dune. Historic FEMA repetitive loss claims spanning from 1978 to 2015 indicate that total value of claims for the homes along Town Way Extension (homes located seaward of the dune crest) was more than 40 times greater than those along Inner Harbor Road (homes located landward of the dune crest).

The existing volume of the northern dunes based on the “540 rule” (discussed in Section 6.4) is 151 cubic feet per foot. Approximately 27,000 cubic yards of compatible material is required to raise the dune crest to 22.5 feet NAVD88 and to provide sufficient dune volume for storm protection. A schematic of the constructed dune profile is shown in Figure 7.39. The cost to construct the dune is approximately $918,000.

Along the south portion of Peggotty Beach, the homes are situated seaward of the dune crest. To construct a dune that satisfies the “540 rule” and that is situated seaward of the homes, 78,000 cubic yards of compatible material are required at a construction cost of $2.7 million. A schematic of the proposed dune profile is presented in Figure 7.40.

For both the north and south portions, monitoring and renourishment of the dunes will be necessary to maintain adequate dune volume. Constructed dunes may be implemented with beach nourishment to provide further storm protection and design life longevity.

Figure 7.38 Location of the dune crest and high water shoreline along Peggotty Beach.

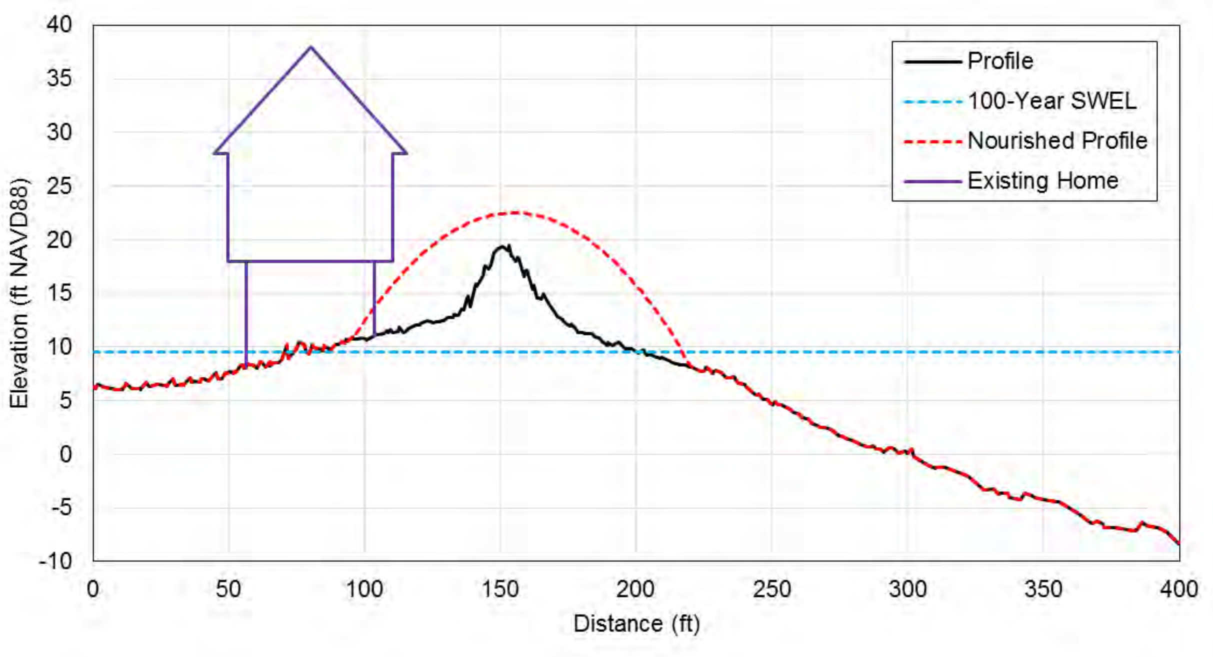


Figure 7.39 Schematic of the proposed dune profile required to satisfy the "540 rule" along the northern portion of Peggotty Beach.

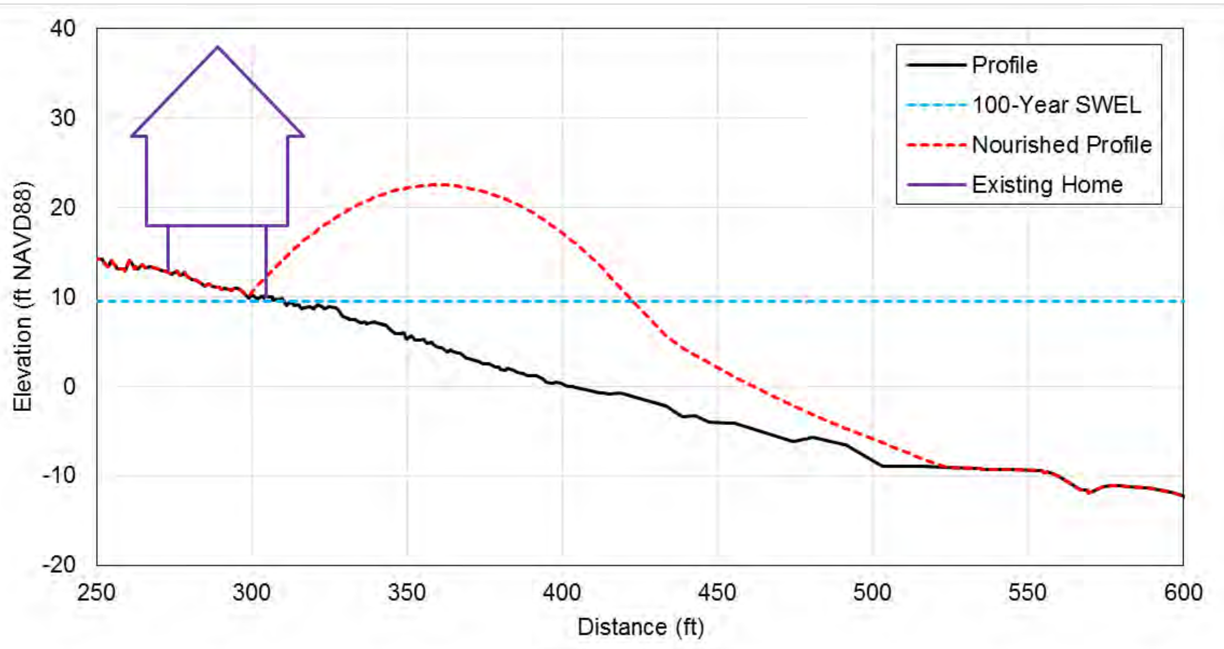


Figure 7.40 Schematic of the dune profile required to satisfy the "540 rule" along the southern portion of Peggotty Beach.

Managed Retreat

Retreat from the shore along Peggotty Beach is an option for the homes along Inner Harbor Road where the properties extend across the road into the marsh. These homes may be relocated approximately 100 feet landward. With a mutual agreement between the Town and the property owners, the homes along Town Way Extension may also be relocated approximately 200 feet landward onto the parcel of town property behind the homes. For the homes along Town Way Extension, moving landward to the backside of the dune crest will help to reduce storm damage.

The estimated cost to relocate all the homes is approximately $4.8 million at $300,000 per home ($1.8 million and $3.0 million for the homes on Inner Harbor Road and Town Way Extension, respectively). From 1978 to 2015, the total value of FEMA repetitive loss claims received by the 16 homes along Peggotty Beach is $2.3 million. Table 7.24 summarizes the historic FEMA damage claims and assessed value of the homes. The cost to buy-out all 16 homes is at least $8.7 million (assessed value) as the market value of the homes is historically greater than the assessed value.

Table 7.24 Historic repetitive loss claims and assessed value of homes along Peggotty Beach.

Road	Total Repetitive Loss Claims 1978-2015	Assessed Value (2016)
Inner Harbor Road (6 homes)	$54,469	$5,083,500
Town Way Ext. (10 homes)	$2,262,212	$3,614,000
Total	$2,316,681	$8,697,500

Figure 7.41 Properties along Inner Harbor Road and Town Way Extension on Peggotty Beach.

Recommended Approach for Peggotty Beach

The recommended shore protection approach for Peggotty Beach is managed retreat either in the form of moving the homes landward or buy-outs from the town (>$4.8 million). The cost of maintaining the status quo over the next 50 years is $16.9 million which includes the cost of FEMA repetitive loss claims and assuming the complete loss of property values due to continued erosion and increasing water levels. Peggotty Beach represents one of the most highly erosional areas along the Scituate coast, where overwash of the low-lying barrier beach has caused readily observable landward migration of the barrier beach into the salt marsh system along its landward limit. While this effect may have an adverse impact on salt marsh resources, this overwash process is natural and existing environmental regulations acknowledge and accept this natural process. The overwash is also necessary for the barrier beach to adapt to sea level rise. While regulations encourage beach and dune stabilization through nourishment, it may prove difficult and/or cost-prohibitive to maintain the Peggotty Beach shoreline in its present position; therefore, it is likely that some type of managed retreat will be necessary over the next 50 years, even if proactive nourishment is performed along the beach.

7.12 Third Cliff

Utilizing information developed from the coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, appropriate shore protection strategies were developed for the Third Cliff shoreline segment. The shoreline change assessment (Appendix A) indicates that the shoreline has been relatively stable over the past 60+ years, as a result of the armoring fixing the position of the shoreline. Improvements to the revetment were considered to ensure adequate long-term shore protection. Additionally, elevating Gilson Road was considered to provide emergency egress during periods of combined high tide and storm surge. The shore protection approaches for Third Cliff are summarized in Table 7.25.

Table 7.25 Shore protection approaches and costs for Third Cliff.

Shore Protection Approach	**Cost**	**50-Year Lifecycle Cost**
Revetment 4,800 feet	$19.2 million	$47.8 million
Elevate Gilson Road For emergency access	$750,000	Not applicable

Revetment

Similar to the revetment on Second Cliff, high wave overtopping rates were calculated for the 4,800-foot revetment along Third Cliff but overtopping damage has not been historically documented. The homes on Third Cliff are set back about 150 feet from the top of the revetment. While reconstruction of the structure is not likely required for storm protection, the revetment requires maintenance and repairs based on the “poor” condition rating given by CLE Engineering in 2013.

A schematic of the proposed design for the Third Cliff revetment is shown in Figure 7.42 if addition storm protection is desired in hypothetical 2-foot sea level rise (SLR) scenario.

Modeled after the revetment at First Cliff, the revetment is extended to a higher elevation and an above-surge berm is constructed. Wave overtopping rates under the sea level rise scenarios are shown in Table 7.26. The cost to construct the revetment along the entire length of Third Cliff is $19.2 million. Ongoing maintenance and repairs will be required to ensure the integrity of the structure over its design life.

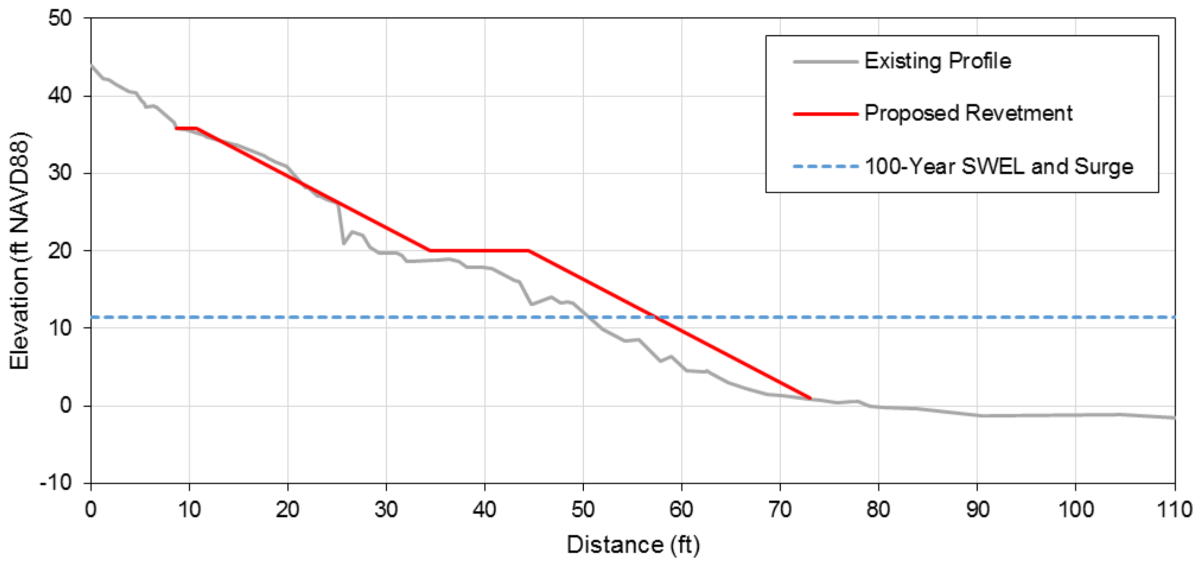


Figure 7.42 Schematic of proposed revetment design along Third Cliff where the revetment is extended to a higher elevation and a berm is added to dissipate wave energy.

Table 7.26 Wave overtopping and predicted damage at Third Cliff during 100-year storm conditions. The proposed design involves extending the revetment to a higher elevation and adding a berm.

Scenario	Wave Overtopping ($ft^3/s/ft$)	Predicted Damage
Existing Design	2.8	Yes
Existing with 2 feet of SLR	6.3	Yes
Proposed Design	0.0	No
Proposed with 2 feet of SLR	0.3	No

Elevate Gilson Road

Gilson Road serves as one of the emergency access routes for the residents of Third Cliff and is also identified by the Town as a frequently flooded road. Figure 7.43 and Figure 7.44 shows that road between Kent Street and Town Way is generally at or below the 10-year still water elevation. On average, the road is located 0.5 feet and 1.3 feet below the 10- and 100-year still water elevation, respectively, with the lowest point in the road located at the marsh culvert.

The number of homes that have driveways connected to Gilson Road is small, however the proximity to the marsh may cause permitting concerns. The estimated construction cost of elevating the road and associated utilities is $750,000. The road elevation may also be constructed in conjunction with culvert improvements to enhance the marsh function and drainage.

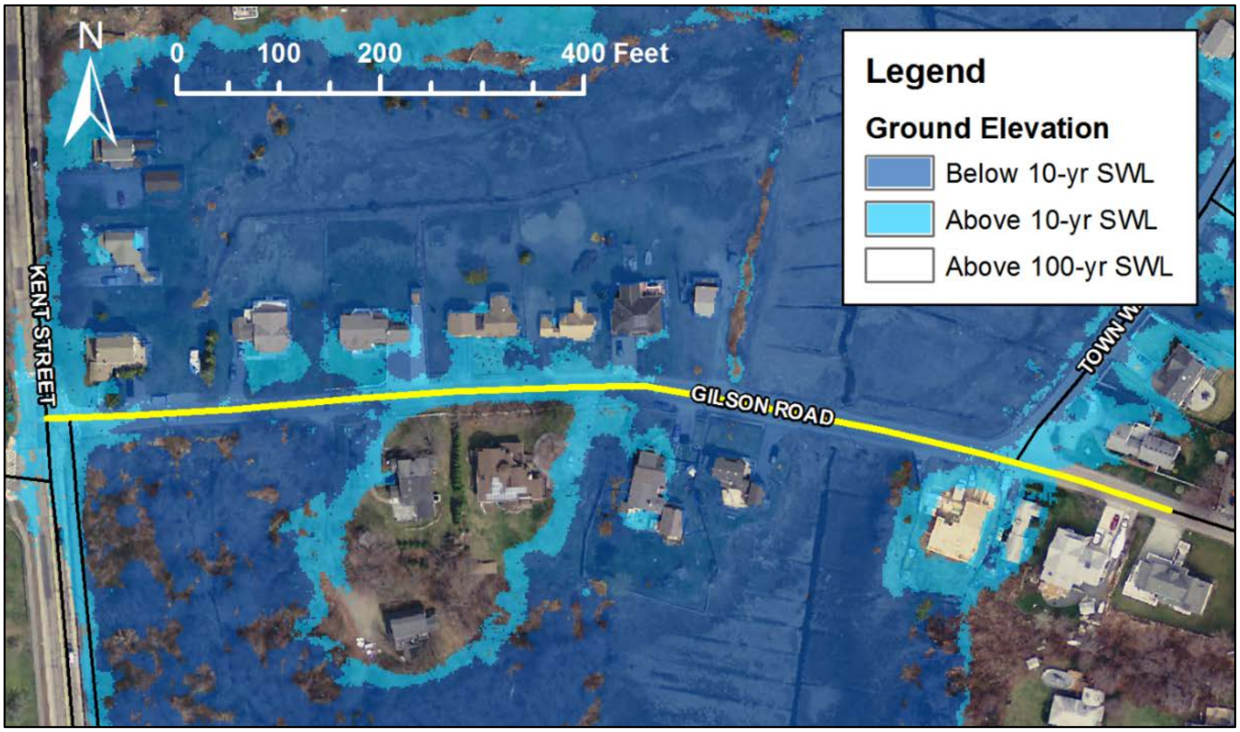


Figure 7.43 Flooding extents of the 10-year and 100-year still water elevation across Gilson Road.

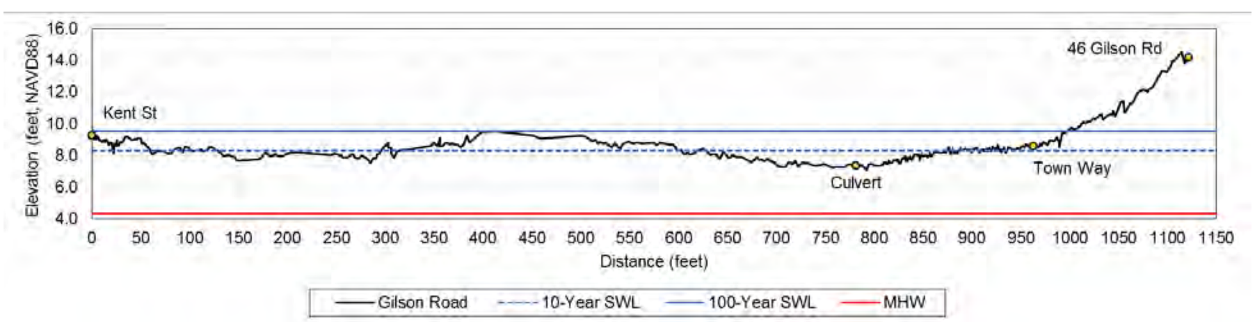


Figure 7.44 Road elevation along Gibson Road from Kent Street to 46 Gilson Road.

Recommended Approach for Third Cliff

The recommended shore protection approach for Third Cliff is to maintain the status quo ($26.8 million over 50 years). Plans for repair to First, Second, and Third Cliff are underway to address damages incurred from over the last several years from Hurricane Sandy, Winter Storm Nemo, and Winter Storm Juno.

7.13 Fourth Cliff

Utilizing information developed from the coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, appropriate shore protection strategies were developed for the Fourth Cliff shoreline segment, which does not include the U.S. Air Force property adjacent to the inlet. The shoreline change assessment (Appendix A) indicates that the shoreline has been moderately erosional (approximately 2 feet per year) over the past 60+ years. A narrow high tide beach remains seaward of the existing revetment toe, but continued landward migration of the beach may lead to future shore protection concerns. Improvements to the revetment were considered to ensure adequate long-term shore protection. Additionally, elevating Central Avenue was considered to provide emergency egress during periods of combined high tide, storm surge, and dune overwash. The shore protection approaches for Fourth Cliff are summarized in Table 7.27.

Table 7.27 Shore protection approaches and costs for Fourth Cliff.

Shore Protection Approach	**Cost**	**50-Year Lifecycle Cost**
Revetment 720 feet	$2.9 million	$7.2 million
Elevate Central Avenue For emergency access	$3.6 million	Not applicable

Revetment

The 720-foot long revetment at the base of Fourth Cliff was rated to be in "poor" condition by CLE Engineering in 2013. Under existing conditions, the revetment prevents wave overtopping from causing damage to pavement. However, under a hypothetical 2-foot sea level rise (SLR) scenario, overtopping waves are predicted to cause damage. To reduce overtopping, the revetment may be reconstructed with a higher and wider berm while extending the revetment up the face of the cliff. A schematic of the proposed design is shown in Figure 7.45. Table 7.28 summarizes the wave overtopping rates under the various scenarios.

The cost to reconstruct the revetment is approximately $2.9 million. Costs associated with maintenance and repairs are required post-construction for the design life of the structure.

Table 7.28 Wave overtopping and predicted damage at Fourth Cliff during 100-year storm conditions. The proposed design involves extending the revetment to a higher elevation and adding a berm.

Scenario	**Wave Overtopping ($ft^3/s/ft$)**	**Predicted Damage**
Existing Design	0.4	No
Existing with 2 feet of SLR	1.7	Yes
Proposed Design	0.0	No
Proposed with 2 feet of SLR	0.0	No

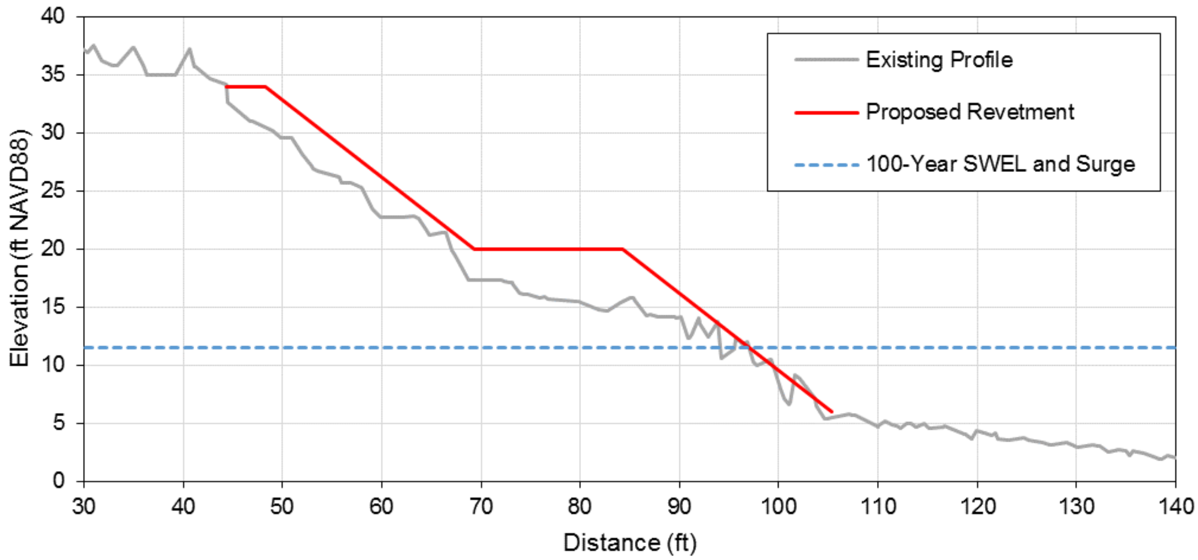


Figure 7.45 Schematic of proposed design along Fourth Cliff where the revetment is extended to a higher elevation and a berm is added to dissipate wave energy.

Elevate Central Avenue

Details of this approach is described in Section 7.14.

Recommended Approach for Fourth Cliff

The recommended shore protection approach for Fourth Cliff is to maintain the status quo ($4.3 million over 50 years).

7.14 Humarock North

The shoreline change and coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, guided development of appropriate shore protection strategies for the Humarock North shoreline segment. Based on observed beach migration, the long-term shoreline erosion is relatively moderate (Appendix A); however, storm overwash has caused periodic migration of the dune crest in the landward direction. This overwash can be severe, leading to substantial blockage of Central Avenue with several feet of gravel and cobble. The combination of shoreline erosion and long-term management of cobble dune material without augmenting the volume of material available has led to a general sediment deficit within the beach system. In general, the existing beach/dune elevation and volume is not sufficient to withstand even modest nor'easters. Beach and dune nourishment were considered potential alternatives to provide shore protection. Specifically, the relatively low net sediment transport in this region (Appendix B and Section 3) indicate large-scale nourishment can provide viable long-term protection. Additionally, elevating Central Avenue was considered to provide emergency egress during periods of combined high tide, storm surge, and dune overwash. The shore protection approaches for Humarock North are summarized in Table 7.29.

Table 7.29 Shore protection approaches and costs for Humarock North.		
Shore Protection Approach	**Cost**	**50-Year Lifecycle Cost**
Seawall and Revetment 4,800 feet	\$38.0 million	\$94.6 million
Beach Nourishment (50 foot berm) Humarock North only - 3,500 feet	\$4.1 million	\$160.7 million
Beach Nourishment (100 foot berm) Humarock North only - 3,500 feet	\$6.3 million	\$130.2 million
Beach Nourishment (50 foot berm) Humarock North and South - 10,500 feet	\$12.3 million	\$70.8 million
Beach Nourishment (100 foot berm) Humarock North and South - 10,500 feet	\$26.4 million	\$120.0 million
Constructed Dunes Stand-alone - 4,800 feet	\$9.6 million	\$78.2 million
Elevate Central Avenue 4,800 feet	\$3.6 million	Not applicable
Managed Retreat Buy-out, all homes along Humarock North	>\$57.0 million	Not applicable

Seawall and Revetment

The 4,800 feet of existing seawalls and revetments constructed along Humarock North are private structures and the condition has not been evaluated. A typical existing seawall section is presented in Figure 7.46 where the top of the seawall is at 15 feet NAVD88 and a small revetment exists at the toe of the wall. With a 2-foot increase in the seawall height and construction of a robust revetment, the wave overtopping can be reduced to rates that will prevent pavement damage under existing conditions but not under a hypothetical 2-foot sea level rise scenario. Table 7.30 summarizes the wave overtopping rates.

The cost to construct a seawall and revetment along the length of Humarock North would be approximately \$38.0 million. Regular maintenance and repairs will also be required throughout the design life to ensure the integrity of the structure. Permitting will likely be a significant obstacle to the proposed design; the majority of the seawall would be considered as new construction which is not permittable on barrier beaches.

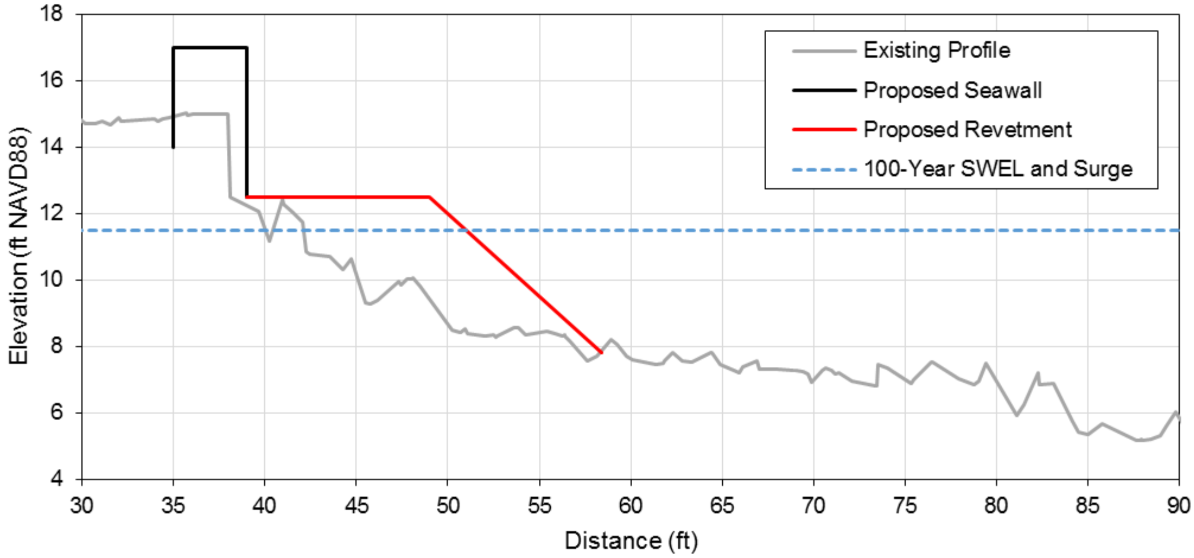


Figure 7.46 Schematic of the proposed seawall and revetment design along Humarock North where the seawall height is increased by 2 feet and the fronting revetment is expanded.

Table 7.30 Wave overtopping and predicted damage at Humarock North during 100-year storm conditions. The proposed design involves raising the seawall height by 2 feet and expanding the fronting revetment.

Scenario	Wave Overtopping ($ft^3/s/ft$)	Predicted Damage
Existing Design	1.2	Yes
Existing with 2 feet of SLR	5.3	Yes
Proposed Design	0.3	No
Proposed with 2 feet of SLR	2.4	Yes

Beach Nourishment

Four options for beach nourishment were considered for Humarock Beach. For all options a berm elevation of 13 feet NAVD88 was selected by determining the 50-year runup elevation along the beach. The required nourishment volume, footprint area, and construction costs are summarized in Table 7.31. Two options were considered wherein nourishment is placed along the most storm damaged and breach susceptible portion of Humarock North, from River Road to Seaview Avenue (“short-length” nourishment). Two additional options were considered that would span all of Humarock Beach, from River Road to Julian Street (“long-length” nourishment). Along each section, the beach berm was extended 50 and 100 feet from the existing high water shoreline. From the berm, the beach slopes seaward at a 1V:10H slope until it intersects with the ocean bottom. Extent of the nourishment options are shown in Figure 7.47 and Figure 7.48.

Table 7.31 Beach nourishment options for Humarock Beach.

Option	**Berm Width (feet)**	**Nourishment Volume (cubic yards)**	**Nourishment Footprint (acre)**	**Cost**
Humarock North 3,500 feet	50	122,000	13.3	$4.1 million
	100	184,000	16.1	$6.3 million
Humarock North and South 10,500 feet	50	362,000	60.1	$12.3 million
	100	778,000	77.6	$26.4 million

The sediment in all four scenarios generally migrates south over time and provides some protection to the southern section of Humarock and Rexhame Beach. Renourishment is required when the volume of fill remaining within the initial fill limits is less than 30%. For the “short-length” options, renourishment is required at approximately 2 and 4 years for the 50- and 100-foot berm design, respectively. The “long-length” options have a much longer design life: 13 and 20 years for the 50- and 100-foot berm designs, respectively. The modeled volume remaining over a 10-year period is presented in Figure 7.49 and Figure 7.50. The volume of the “long-length” nourishment options did not reach 30% at the end of the 10-year modeling period, therefore the renourishment interval was linearly interpolated from the model data.

Monitoring of the beach nourishment performance is recommended to determine when renourishment is required, to assess accretion or erosion of adjacent beach, and to identify of any potential “hot-spot” erosion (areas of anomalously high erosion). Phasing of nourishment was not considered in this study, however phased nourishments should generally be placed within 2 years to prevent excessive loss of sediment.

Figure 7.47 Two nourishment options for Humarock North extending from River Road to Seaview Avenue ("short-length" nourishment).

Figure 7.48 Two nourishment options for Humarock North and South extending from River Road to Julian Street ("long-length" nourishment).

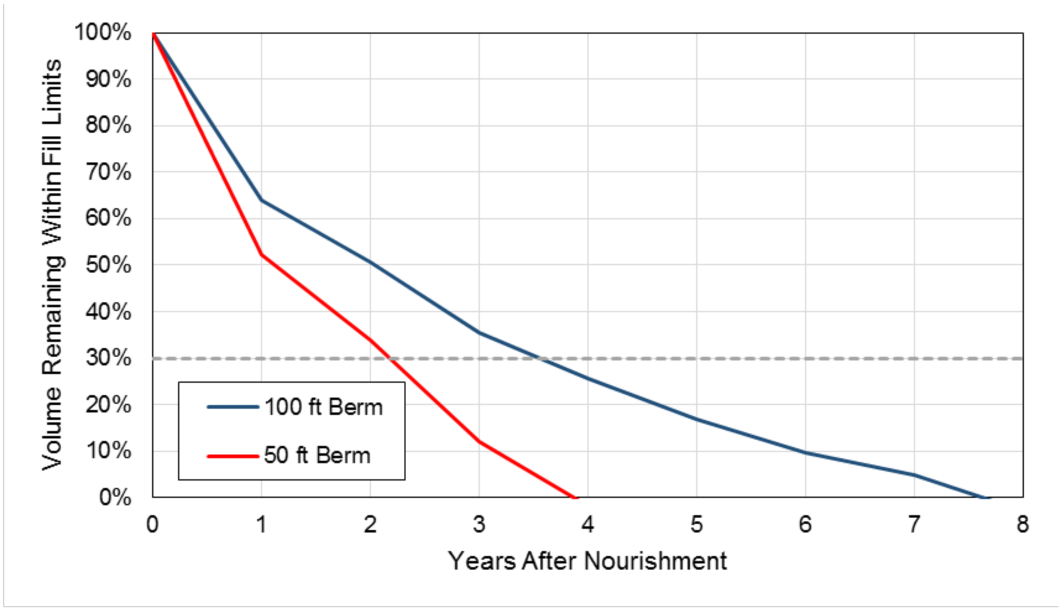


Figure 7.49 Volume of fill remaining over the course of the model simulation for the "short-length" nourishment (Humarock North only). The dashed gray line indicates the 30% remaining design threshold.

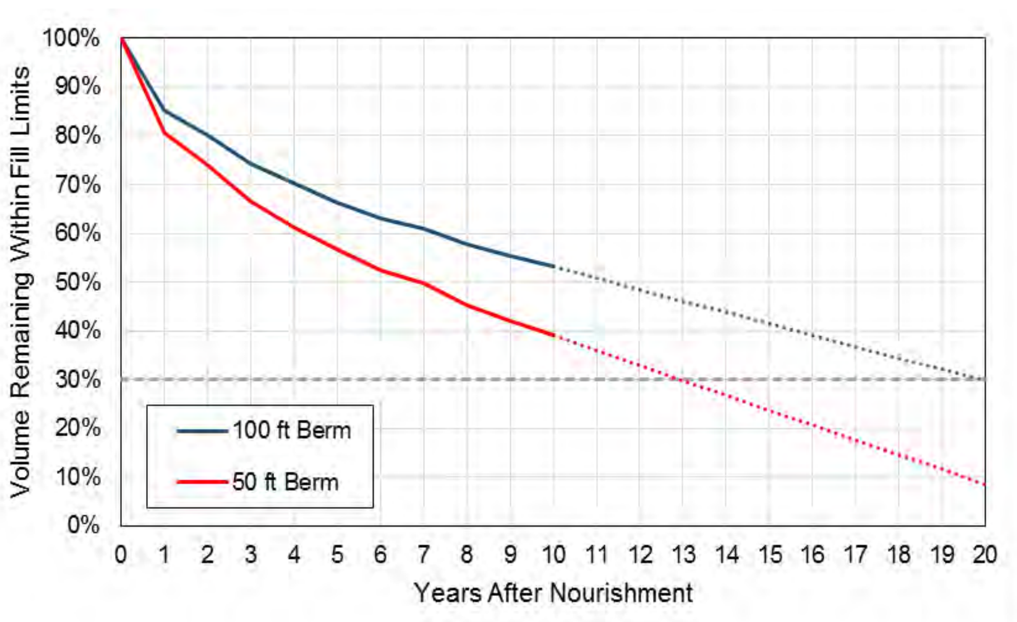


Figure 7.50 Volume of fill remaining over the course of the model simulation for the "long-length" nourishment (Humarock North and South). The dashed gray line indicates the 30% remaining design threshold.

Constructed Dunes

The existing 4,800-foot dune crest along Humarock North is situated under the homes. Based on the "540 rule" (discussed in Section 6.4), the existing dune volume is merely 97 cubic feet per foot. To construct dunes along the entire shoreline that will satisfy the “540 rule”, a new dune with a crest elevation of 22.5 feet NAVD88 would be constructed seaward of the existing homes, as shown in Figure 7.51. The dunes could help to reduce the breaching susceptibility of the Humarock North area by reducing overwash of sediment across Central Avenue.

The dunes would require approximately 282,000 cubic yards of compatible sediment at a cost of $9.6 million. The dunes requires regular maintenance and renourishment to maintain sufficient volume to protect against a 100-year storm. Constructed dunes maybe implemented with beach nourishment to increase the level of storm protection.

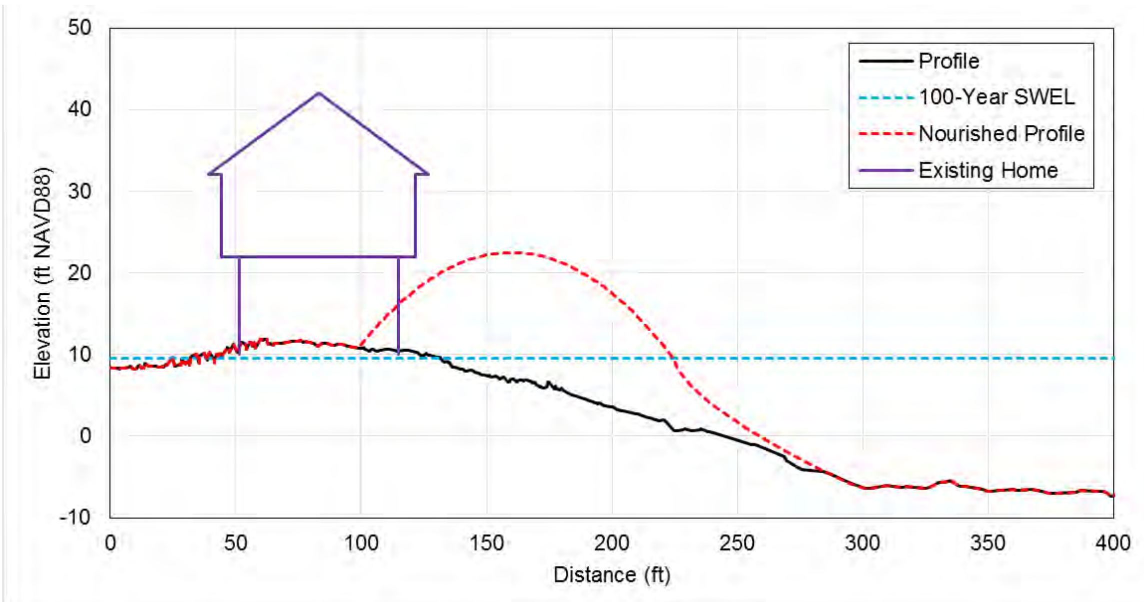


Figure 7.51 Schematic of the proposed dune profile required to satisfy the “540 rule” along Humarock North.

Elevate Central Avenue

Elevation of Central Avenue between River Road and Barratt Street (4,800 feet) would serve to maintaining emergency egress for both Humarock North and Fourth Cliff. As shown in Figure 7.52 and Figure 7.53, the majority of the road is submerged under 10- and 100-year storm conditions. On average, the road is submerged by 0.8 feet and 1.7 feet of water during the 10- and 100-year still water elevation, respectively. Elevation of the road would also help to decrease the breaching susceptibility of the area by reducing overwash across the barrier beach.

A large number of homes on Central Avenue have solid foundations and living space on the ground floor which may pose access challenges if the road is raised by several feet. Water and utility lines will also need to be elevated. The cost to elevate the road and the associated utilities is approximately $3.6 million. The Town has applied for a Coastal Resiliency Grant from MCZM to develop a conceptual plan for elevating Central Avenue and nourishment of the fronting beaches.

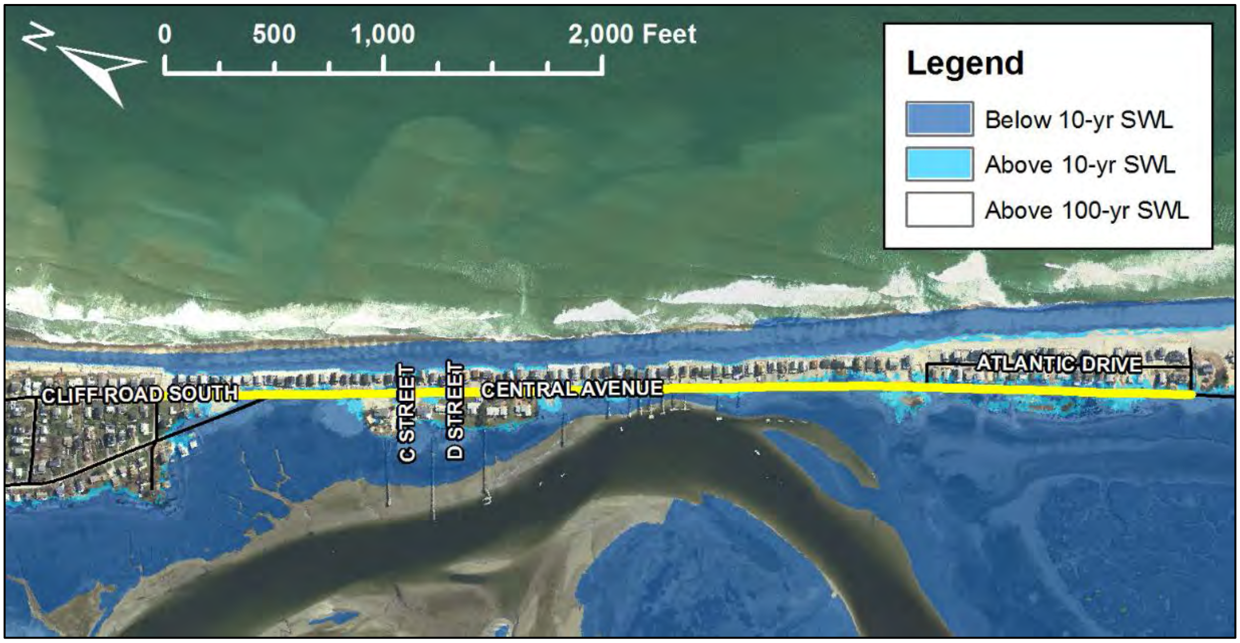


Figure 7.52 Flooding extents of the 10-year and 100-year still water elevation across Central Avenue.

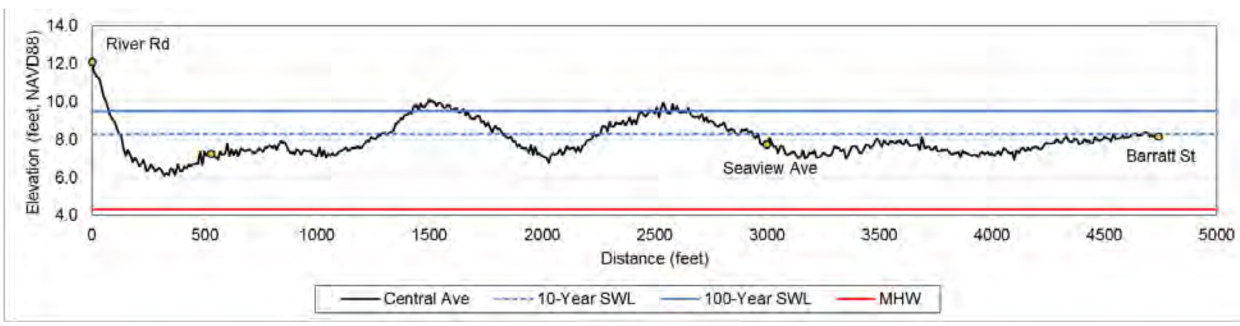


Figure 7.53 Road elevation along Central Avenue from River Road to Barratt Street.

Managed Retreat

There are 113 homes located along Central Avenue, Cliff Road South, and Atlantic Drive in North Humarock. From 1978 to 2016, these homes have claimed approximately $6.7 million in FEMA repetitive loss damage claims. The option to move landward is not possible for the majority of the homes as flooding occurs on both sides of the barrier beach, therefore, the managed retreat approach for North Humarock would consist of buying-out the homes by the Town. However, Central Avenue would continue to require maintenance and storm debris clearing in order to provide access and utility service to Fourth Cliff. Breach repair would also still be required in the event of a breach along Central Avenue to maintain access to and from Fourth Cliff. Table 7.32 summarizes the historic FEMA damage claims and assessed value of the homes. The cost to buy-out all 113 homes is at least $57.0 million as the market value of the homes is historically greater than the assessed value.

Table 7.32 Historic repetitive loss claims and assessed value of homes along Humarock North.

Road	Total Repetitive Loss Claims 1978-2015	Assessed Value (2016)
Central Avenue (91 homes)	$4,618,876	$43,701,700
Cliff Road South (5 homes)	$1,168,300	$3,208,300
Atlantic Drive (17 homes)	$918,088	$10,068,800
Total	$6,705,264	$56,978,800

Recommended Approach for Humarock North

The recommended shore protection approach for Humarock North is to elevate Central Avenue, construct dunes along the Humarock North, and nourish the beach along the entire Humarock North and South. The total cost for both North and South Humarock would be approximately $152.6 million over a 50-year lifecycle ($25.5 million initial cost). Compared to the cost of maintaining the status quo for both Humarock North and South over 50 years ($103.6 million), the recommended approaches have the benefits of increasing storm protection, eliminating the need for post-storm roadway clearing along Central Avenue, providing an increased littoral sediment supply to protect down-drift beaches, providing a greater recreational resource, and preventing a breach between Humarock and Fourth Cliff. Again, similar to other areas with extensive historical storm damage, the estimates utilized to project potential storm damage for the 50-year projections related to the status quo scenario are conservative and likely underestimate future damage costs, especially if sea level rise accelerates as projected.

7.15 Humarock South

The shoreline change and coastal processes analysis (Sections 2 and 3), as well as the existing conditions of the shoreline, guided development of appropriate shore protection strategies for the Humarock South shoreline segment. Based on observed beach migration, the long-term shoreline erosion is relatively moderate (Appendix A); however, storm overwash has caused periodic overtopping of the dune and seawalls in this shoreline stretch. In general, the existing beach/dune elevation and volume is not sufficient to withstand significant nor'easters. Beach and dune nourishment were considered potential alternatives to provide shore protection. Specifically, the relatively low net sediment transport in this region (Appendix B and Section 3) indicate large-scale nourishment can provide viable long-term protection. To provide maximum longevity, nourishment of both Humarock North and South was considered. The shore protection approaches for Humarock South are summarized in Table 7.33.

Table 7.33 Shore protection approaches and costs for Humarock South.		
Shore Protection Approach	**Cost**	**50-Year Lifecycle Cost**
Seawall 8,300 feet	\$66.4 million	\$165.2 million
Beach Nourishment (50 foot berm) Humarock North and South - 10,500 feet	\$12.3 million	\$70.8 million
Beach Nourishment (100 foot berm) Humarock North and South - 10,500 feet	\$26.4 million	\$120.0 million
Constructed Dunes Stand-alone - 8,300 feet	\$10.7 million	\$87.2 million

Seawall

Figure 7.54 shows the existing public seawall along Humarock South which spans from Newell Street to Palfrey Street. The fronting beach is at approximately the same elevation as the seawall and provides a sufficient level of protection against overtopping during the 100-year storm. However, CLE Engineering rated the condition of the seawall as “poor” in 2013 and the existing structure is unable to provide adequate protection in a hypothetical 2-foot sea level rise (SLR) scenario.

Figure 7.54 Existing seawall along Humarock South from Newell Street to Palfrey Street (photo by CLE Engineering).

Increasing the height of the existing seawall by 2 feet, from 15.5 to 17.5 feet NAVD88 as shown in Figure 7.55, reduces the wave overtopping in a sea level rise scenario by nearly 3 times, however damage to pavement is still expected, see Table 7.34. To further reduce overtopping, the seawall will require a fronting revetment. Given that no existing revetment exists, permitting is be expected to be challenging.

To construct a seawall along the entire length of Humarock South, from Barratt Street to Old Mouth Road, (8,300 feet) would cost $66.4 million. The existing public seawall accounts for approximately 25% of the total study area length, therefore the majority of the seawall would be new construction, which is not permittable on barrier beaches. Reconstruction of the existing public seawall only would cost $16.2 million. After construction, all coastal structures will require long-term maintenance and repairs. Increasing the height of a seawall along the barrier beach will likely redirect floodwater to adjacent areas; mitigation would likely be required to minimize impacts to others in this approach.

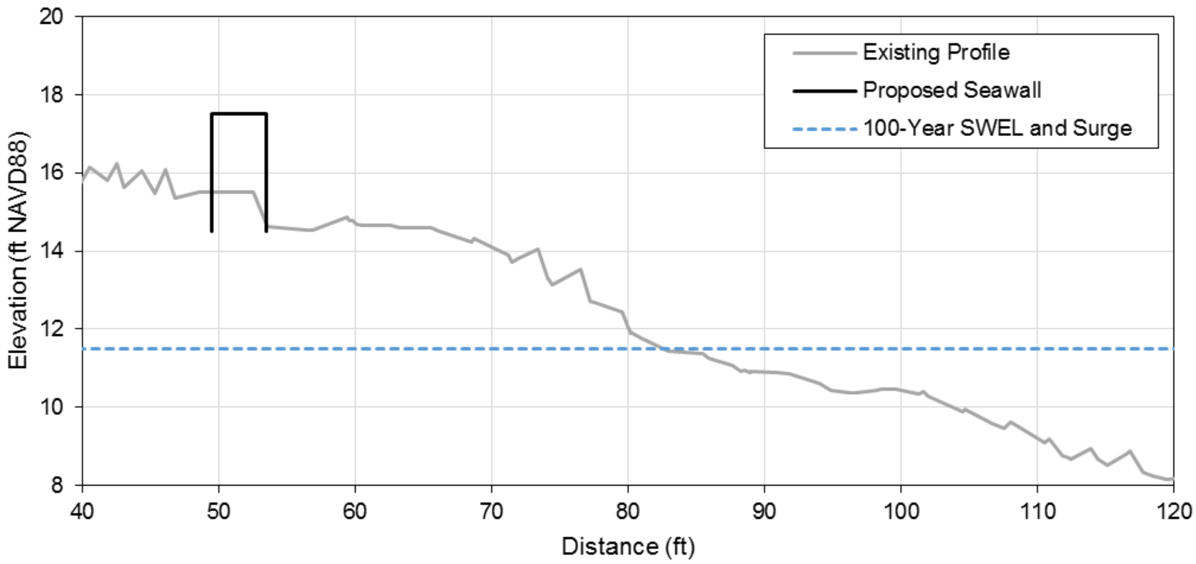


Figure 7.55 Schematic of the proposed design along Humarock South where the seawall height is increased by 2 feet.

Table 7.34 Wave overtopping and predicted damage at Humarock South during 100-year storm conditions. The proposed design involves increasing the seawall height by 2 feet.

Scenario	Wave Overtopping ($ft^3/s/ft$)	Predicted Damage
Existing Design	0.3	No
Existing with 2 feet of SLR	2.7	Yes
Proposed Design	0.1	No
Proposed with 2 feet of SLR	1.0	Yes

Beach Nourishment

Details of this approach is described in Section 7.14.

Constructed Dunes

A schematic of the 8,300-foot constructed dune required to protect the landward infrastructure based on the “540 rule” (discussed in Section 6.4) is presented in Figure 7.56. Approximately 314,000 cubic yards of material is required to construct the dune along the entire length of Humarock South.

The construction cost is $10.7 million with monitoring and renourishment of the dune required to maintain sufficient volume for storm protection. The constructed dune approach may be combined with beach nourishment to create a wider beach with additional longevity.

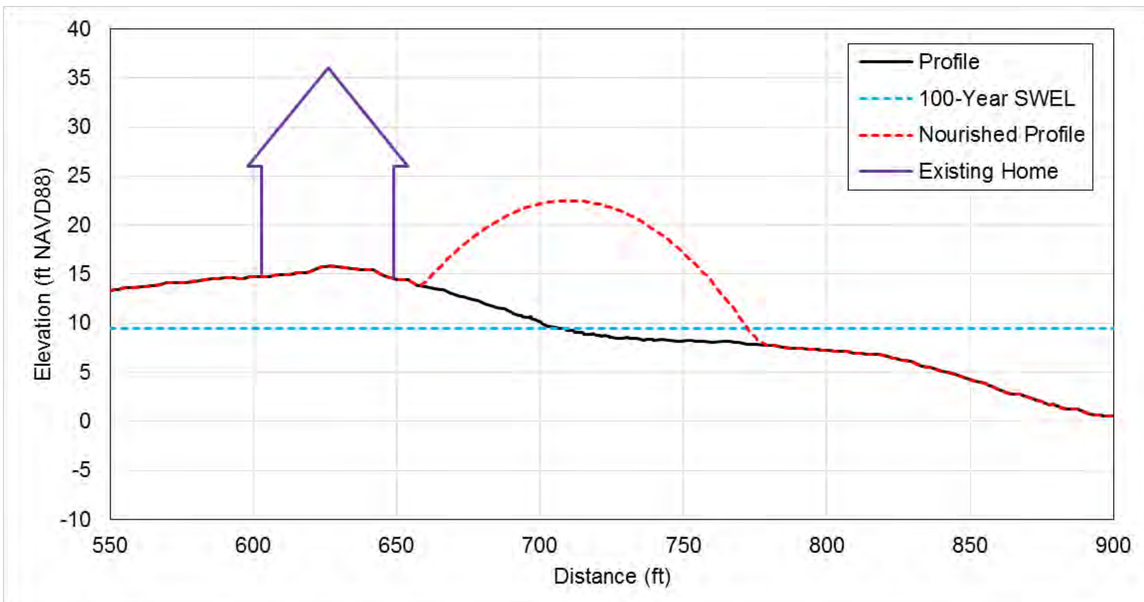


Figure 7.56 Schematic of the proposed dune profile required to satisfy the “540 rule” along Humarock South.

Recommended Approach for Humarock South

The recommended shore protection approach for Humarock South is to nourish the beach along the entire Humarock North and South, as the contiguous nourishment provides a design life that is substantially greater than nourishing either Humarock North or Humarock South as stand-alone projects. This would be performed in conjunction with raising Central Avenue and reconstructing the dune along Humarock North. The total cost for both North and South Humarock would be approximately $152.6 million over a 50-year lifecycle ($25.5 million initial cost). Compared to the cost of maintaining the status quo for both Humarock North and South over 50 years ($103.6 million), the recommended approaches have the benefits of increasing storm protection, eliminating the need for post-storm roadway clearing along Central Avenue, providing an increased littoral sediment supply to protect down-drift beaches, providing a greater recreational resource, and preventing a breach between Humarock and Fourth Cliff.

Again, similar to other areas with extensive historical storm damage, the estimates utilized to project potential storm damage for the 50-year projections related to the status quo scenario are conservative and likely underestimate future damage costs, especially if sea level rise accelerates as projected.

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GLOSSARY OF TERMS

540-RULE
Primary frontal dunes will not be considered as effective barriers to base flood storm surges and associated wave action where the cross-sectional area of the primary frontal dune, as measured perpendicular to the shoreline and above the 100-year still-water flood elevation and seaward of the dune crest, is equal to, or less than, 540 square feet.

ACCRETION
The accumulation of (beach) sediment deposited by natural fluid flow processes.

ALONGSHORE
Parallel to and near the shoreline; same as longshore.

BARRIER BEACH
A bar essentially parallel to the shore, the crest of which is above normal high water level. Also called offshore barrier and barrier island.

BATHYMETRY
The measurement of depths of water in oceans, seas, and lakes; also the information derived from such measurements.

BEACH BERM
A nearly horizontal part of the beach or backshore formed by the deposit of material by wave action. Some beaches have no berms, others have one or several.

BEACH EROSION
The carrying away of beach materials by wave action, tidal currents, littoral currents, or wind.

BEACH FILL
Material placed on a beach to re-nourish eroding shores, usually pumped by dredge but sometimes delivered by trucks.

BEACH NOURISHMENT
The process of replenishing a beach by artificial means; e.g., by the deposition of dredged materials, also called beach replenishment or beach feeding.

BEACH PROFILE
A cross-section taken perpendicular to a given beach contour; the profile may include the face of a dune or sea wall, extend over the backshore, across the foreshore, and seaward underwater into the nearshore zone.

BEACH WIDTH
The horizontal dimension of the beach measured normal to the shoreline and landward of the higher-high tide line (on oceanic coasts) or from the still water level (on lake coasts).

BREACHING
Formation of a channel through a barrier spit or island by storm waves, tidal action, or river flow. Usually occurs after a greater than normal flow, such as during a hurricane.

BREAKWATER
A man-made structure protecting a shore area, harbor, anchorage, or basin from waves.

COASTAL PROCESSES
Collective term covering the action of natural forces on the shoreline and the nearshore seabed.

COASTAL ZONE
The land-sea-air interface zone around continents and islands extending from the landward edge of a barrier beach or shoreline of coastal bay to the outer extent of the continental shelf.

DUNES
Accumulations of windblown sand on the backshore, usually in the form of small hills or ridges, stabilized by vegetation or control structures.

ELEVATION
The distance of a point above a specified surface of constant potential; the distance is measured along the direction of gravity between the point and the surface.

EOEEA
Executive Office of Energy & Environmental Affairs

ERDC
U.S. Army Engineer Research and Development Center

EROSION
Wearing away of the land by natural forces. On a beach, the carrying away of beach material by wave action, tidal currents or by wind action.

FEMA
Federal Emergency Management Agency

FIRM
Flood Insurance Rate Map

HARD DEFENSES
General term applied to impermeable coastal defense structures of concrete, timber, steel, masonry, etc., which reflect a high proportion of incident wave energy.

HEADLAND
A land mass having a considerable elevation.

HINDCASTING
In wave prediction, the retrospective forecasting of waves using measured wind information.

INTERTIDAL
The zone between the high and low water marks.

LIDAR
Light Detection and Ranging, is a remote sensing method that uses light in the form of a pulsed laser to measure ranges (variable distances) to the Earth.

MARSH
Soft, wet area periodically or continuously flooded to a shallow depth, usually characterized by a particular subclass of grasses, cattails and other low plants.

MCZM
Massachusetts Office of Coastal Zone Management

MEAN HIGH WATER (MHW)
The average elevation of all high waters recorded at a particular point or station over a considerable period of time, usually 19 years. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.

MEAN LOW WATER (MLW)
The average height of the low waters over a 19-year period. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.

NAVD 88
The North American Vertical Datum of 1988 is the vertical control datum of orthometric height established for vertical control surveying in the US based upon the General Adjustment of the North American Datum of 1988.

NEARSHORE
(1) In beach terminology an indefinite zone extending seaward from the shoreline well beyond the breaker zone. (2) The zone which extends from the swash zone to the position marking the start of the offshore zone, typically at water depths of the order of 20 meters.

NFIP
National Flood Insurance Program

NOAA
National Oceanic and Atmospheric Administration

OFFSHORE
(1) In beach terminology, the comparatively flat zone of variable width, extending from the shoreface to the edge of the continental shelf. It is continually submerged. (2) The direction seaward from the shore.

OUTCROP
A surface exposure of bare rock, not covered by soil or vegetation.

OVERTOPPING
Passing of water over the top of a structure as a result of wave runup or surge action.

OVERWASH
The part of the uprush that runs over the crest of a berm or structure and does not flow directly back to the ocean or lake.

PERCHED BEACH
A perched beach is a beach retained above the otherwise normal profile level by a submerged structure parallel to the coast.

POCKET BEACH
A beach, usually small, in a coastal reentrant or between two littoral barriers (often rocky headlands).

REVETMENT
A facing of stone, concrete, etc., to protect an embankment, or shore structure, against erosion by wave action or currents.

RUN-UP
The rush of water up a structure or beach on the breaking of a wave. The amount of run-up is the vertical height above still water level that the rush of water reaches.

SEA LEVEL RISE (SLR)
The long-term trend in mean sea level.

SEAWALL
A structure built along a portion of a coast primarily to prevent erosion and other damage by wave action. It retains earth against its shoreward face.

SEDIMENT TRANSPORT
The main agencies by which sedimentary materials are moved are: gravity (gravity transport); running water (rivers and streams); ice (glaciers); wind; the sea (currents and longshore drift). Running water and wind are the most widespread transporting agents. In both cases, three mechanisms operate, although the particle size of the transported material involved is very different, owing to the differences in density and viscosity of air and water. The three processes are: rolling or traction, in which the particle moves along the bed but is too heavy to be lifted from it; saltation; and suspension, in which particles remain permanently above the bed, sustained there by the turbulent flow of the air or water.

SOFT DEFENSES
Usually refers to beaches (natural or designed) but may also relate to energy-absorbing beach-control structures, including those constructed of rock, where these are used to control or redirect coastal processes rather than opposing or preventing them.

STILL-WATER LEVEL (SWL)
The surface of the water if all wave and wind action were to cease. In deep water this level approximates the midpoint of the wave height. In shallow water it is nearer to the trough than the crest.

STORM SURGE
A rise above normal water level on the open coast due to the action of wind stress on the water surface. Storm surge resulting from a hurricane also includes that rise in level due to atmospheric pressure reduction as well as that due to wind stress.

SWAN
The SWAN (Simulating Waves Nearshore) model is a spectral wave model.

USGS
United States Geological Survey

WAVE DIRECTION
The direction from which the waves are coming.

WAVE HEIGHT
The vertical distance between the crest (the high point of a wave) and the trough (the low point).

WAVE PERIOD
(1) The time required for two successive wave crests to pass a fixed point. (2) The time, in seconds, required for a wave crest to traverse a distance equal to one wave length.

WIS
Wave Information Study