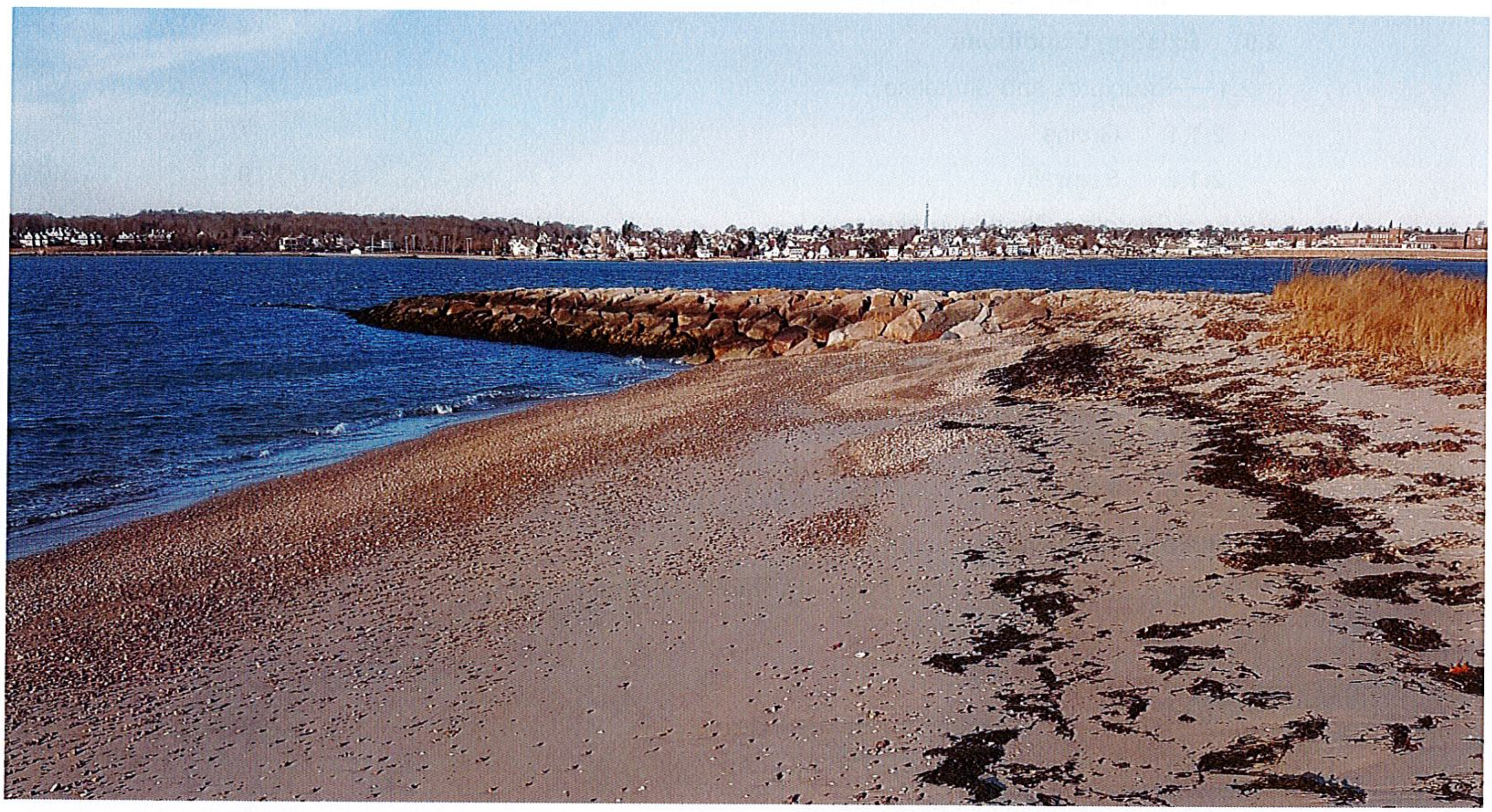
Expanded Environmental Notification Form West Beach

New Bedford, MA

June 2018



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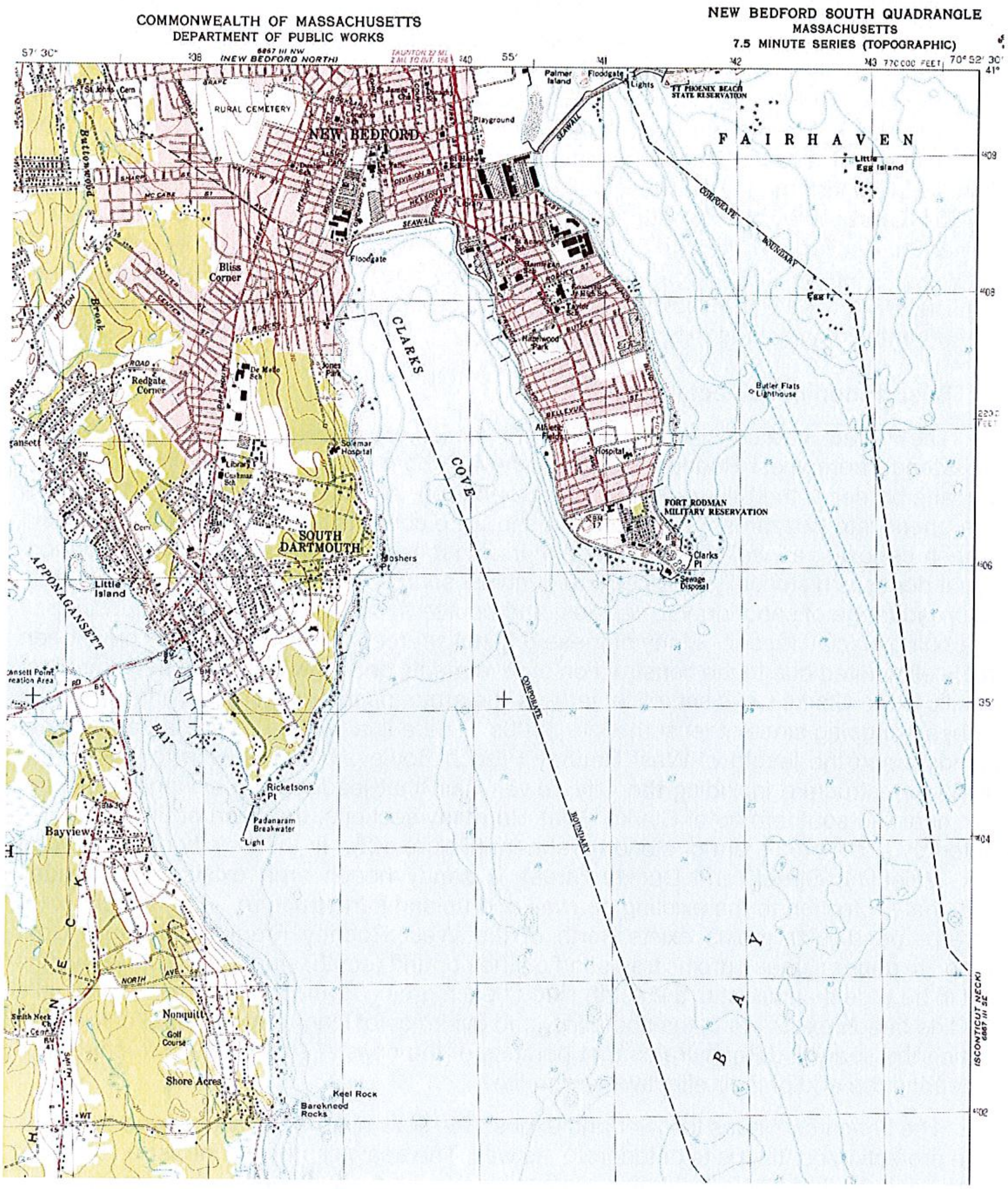
ENVIRONMENTAL NOTIFICATION FORM

See following pages.

USGS MAP


COMMONWEALTH OF MASSACHUSETTS
DEPARTMENT OF PUBLIC WORKS
NEW BEDFORD SOUTH QUADRANGLE
MASSACHUSETTS
7.5 MINUTE SERIES (TOPOGRAPHIC)
(NEW BEDFORD NORTH)
55'
70° 52' 30"
41°
35'
RURAL CEMETERY
NEW BEDFORD
Palmer Island
Floodgate
Lights
Playground
SEAWALL
FAIRHAVEN
Little Egg Island
Buttonwood
Brook
Bliss Corner
Floodgate
CLARKS
COVE
CORPORATE
BOUNDARY
Redgate Corner
Hazelwood Park
Butler Flats Lighthouse
Athletic Field
Hospital
Solemar Hospital
Library
FORT RODMAN MILITARY RESERVATION
Clarks Pt
Sewage Disposal
SOUTH DARTMOUTH
Moshers Pt
Little Island
APPONAGANSETT
BAY
Ricketsons Pt
Padanaram Breakwater
Light
Bayview
NECK
Nonquitt
Golf Course
Shore Acres
Keel Rock
Barekneed Rocks
(SCONTICUT NECK)

1.0 PROJECT OVERVIEW

1.1 Introduction

This document is an Expanded Environmental Notification Form (EENF) for beach nourishment and t-head groin construction on the eastern shoreline of Clarks Cove (the Project) in the City of New Bedford, Massachusetts (see Figure 1.1 for project location). This EENF is submitted on behalf the City of New Bedford Department of Public Infrastructure (DPI) to the Massachusetts Executive Office of Energy and Environmental Affairs (EEA) under the Massachusetts Environmental Policy Act (MEPA), in accordance with 301 Code of Massachusetts Regulations (CMR) 11.00 and with General Laws Chapter 30, Sections 61 through 62H. In accordance with 301 CMR 11.05(4), this EENF includes a concise and accurate description of the Project and its alternatives, identification of review thresholds and agency actions, and an assessment of potential environmental impacts and mitigation measures.

1.2 Description of Project Area

The Project area consists of a 3,830-foot section of West Rodney French Boulevard that extends from West Rodney French Boulevard boat ramp (at the south end) to the hurricane barrier at the Kilburn Mills (at the north end), as indicated in Figure 1.1. Clark's Cove opens into Buzzards Bay, where the shoreline consists of glacial till headlands 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. Specific to the Clarks Cove shoreline, large-scale armoring to protect upland infrastructure has been ongoing since at least the late 1800s. There is a vertical concrete seawall that extends along the length of West Rodney French Boulevard and serves to protect the upland infrastructure including the City sewer main that leads to the sewage treatment plant near the southern tip of Clarks Point. In many sections, the base of the seawall is fronted by a low profile armor stone revetment (Figure 1.2). In the vicinity of Hazelwood Park, Valentine Street, and Dudley Street, a sandy beach area exists that provides additional protection to the existing seawall and upland infrastructure. A series of six (6) shore-perpendicular groins exists north of the West Rodney French Boulevard Boat Ramp. In general, these groins trap sand on their updrift (south) side, where beach widths tend to be widest adjacent to the south side of each groin. South of Hazelwood Park, little high tide beach exists along the shoreline, and evidence of long-term lowering of the area fronting the seawall demonstrates that portions of the coastal engineering structure may be nearing the end of their effective design life.

The beach continues to lower and expose the seawall to wave action, and therefore wave protection continues to deteriorate, as well. The seawall protects infrastructure from failing behind it, but also accelerates erosion by reflecting wave energy, thereby removing sand from the front of the structure. As erosion continues unabated, the beach profile along the wall will continue to lower. As the profile lowers, storm waves impacting the seawall will increase in height due to less breaking in the deeper depths fronting the wall. With larger wave heights, overtopping rates will also increase during storms, resulting in more frequent and severe erosion and damage to the paved and unpaved upland area behind the wall.

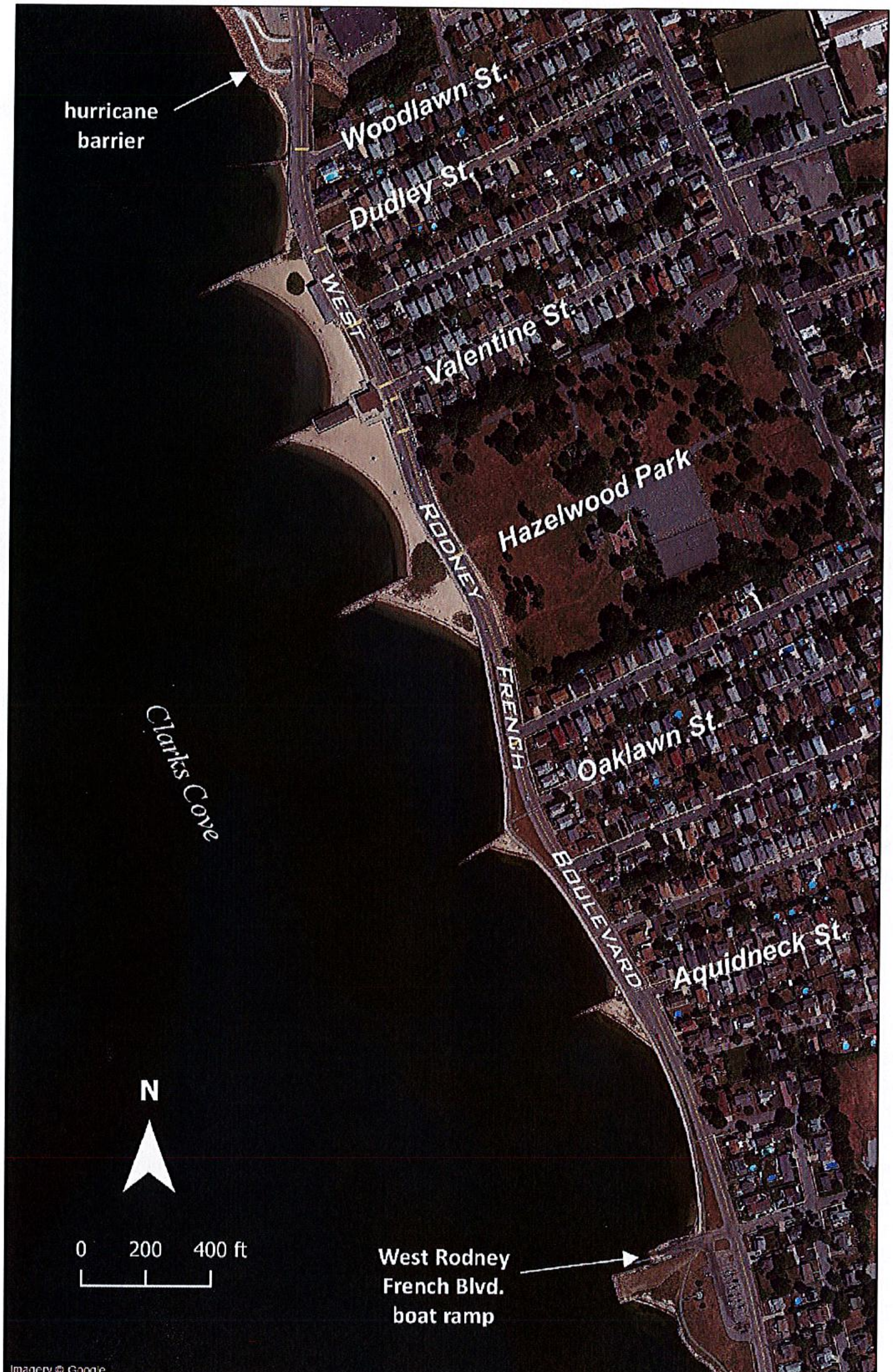


Figure 1.1 August 2016 aerial of the study shoreline between the West Rodney French Blvd. boat ramp and the hurricane barrier at the Kilburn Mills.

Figure 1.2 Photographs of West Rodney French Boulevard seawall taken January 2017

A project is needed to enhance the storm resiliency of the main city sewer line along West Rodney French Boulevard while also being protective of sensitive eelgrass habitat that exists in close proximity to the project shoreline. The goals for this project are twofold: protect existing infrastructure (the city's main sewer line), balanced with the need to protect existing eelgrass adjacent to the project area, resulting in a unique situation that requires an innovative approach. The existence of eelgrass resources in very close proximity to the shoreline has been identified as the main challenge to any design used to improve storm resiliency along West Rodney French Boulevard.

1.3 Project Area History

The West Rodney French Boulevard shoreline has experienced modest erosion of the shoreline in areas that have been not protected by nourishment (nourishment projects along the northern beach areas were constructed in 1958 and 1977). While this beach erosion has not been severe when reviewing shoreline change since 1938 (Figure 1.3), lowering of the beach over time has led to the need for revetment protection along the toe of the exposed seawall sections. The long-term effect of this beach lowering is to expose this shoreline to larger depth-limited waves due to deeper water depths fronting the seawall. During severe conditions, these larger waves can destabilize the seawall protecting the sewer line running its full length. Moreover, the Coastal Structures Inventory (MADCR, 2013) indicates that while the vertical concrete seawall backing the beach is in fairly good condition, the toe revetment that protects against seawall undermining is in poor condition. Due to the loss in beach width and condition of the shore protection, the engineering analysis indicated that critical City infrastructure within West Rodney French Boulevard was at risk to storm-related damage, specifically the sewer main.

During the 1938 Hurricane, substantial damage occurred throughout New Bedford and the shoreline area of Clarks Cove was not spared. Figure 1.4 and Figure 1.5 are photographs showing the condition of the West Rodney French Boulevard area after the storm. Under storm conditions, portions of the seawall failed and upland adjacent to the seawall was substantially scoured. During the peak of the storm, water levels greatly exceeded the low elevation of the seawall and roadway, which limited the damaging effects of waves at the point of maximum water levels.

After Hurricane Carol in 1954, seawall and revetment improvements were made in 1958. As part of this shore protection project, a beach nourishment component was added between Oaklawn and Dudley Streets. Additional repairs and nourishment occurred in 1977. While it appears that a majority of the nourishment from Hazelwood Park north remains, much of the shoreline south of this area contains no high tide beach, except in the immediate vicinity of the groins.

1.4 MEPA Review Thresholds

Under current MEPA review thresholds the Project triggers an ENF and other MEPA review, if the Secretary so requires, as it entails new fill and structures in a velocity zone or regulatory floodway [301 CMR 11.03(3)(b)(1)(e)] and alteration of one half or more acres of other wetland resource areas [301 CMR 11.03(3)(b)(1)(f)]. In addition, the Project also requires a Chapter 91 License [301 CMR 11.03(3)(b)(5)] and involves construction of solid fill structures of 1,000 or more square-feet base area [301 CMR 11.03(3)(b)(6)]. A detailed description of resource area impacts is provided in Section 6.

This EENF application fully describes the project and its alternatives, and assesses its potential environmental impacts and mitigation measures, as described in 301 CMR 11.05(7). Due to the unique nature of this project, where new optimized coastal engineering structures are proposed and portions of existing ineffective coastal engineering structures are dismantled, it is anticipated that a detailed EENF is needed to provide a thorough analysis of alternatives, potential environmental impacts, and mitigation measures. Overall, the Project is focused on improving coastal resiliency to critical City infrastructure in an environmentally sound manner.

Figure 1.3 Aerial photograph from 1938 with the approximate shoreline from January 2017 shown in orange.

Figure 1.4 Damage to the area landward of the West Rodney French Boulevard seawall as a result of the September 1938 Hurricane (source: Spinner Publications, New Bedford, MA, www.spinnerpub.com).

Figure 1.5 Portion of the West Rodney French Boulevard seawall failure that occurred as a result of the September 1938 Hurricane (source: Spinner Publications, New Bedford, MA, www.spinnerpub.com).

2.0 EXISTING CONDITIONS

2.1 Structures and Shoreline

CLE Engineering, Inc. (CLE) performed field inspections of existing structures in February 2017 and documented the detailed findings of their visual (topside) investigation of the existing coastal engineering structures along West Rodney French Boulevard in an April 2017 report. The inspection surveyed the condition of the concrete seawall, stone revetment and stone groins located along the approximate ±3,830 linear feet of study shoreline as shown in Figure 2.1. The complete inspection report summarized here is provided as Appendix A to this report.

The inspection was limited to the topside visual condition evaluation of structures with no below water or subsurface investigations. Based on information provided by the City, the seawall and groins in the project area have been repaired in places, and in some instances rebuilt, with the last major effort dating from 1978. Since then, limited inspections were performed by the Massachusetts Department of Conservation and Recreation (MADCR) in 2006 and 2013 as part of the MA Coastal Inventory and Assessment Report. The 2017 inspection performed for this present project uses the same nomenclature and rating system utilized in the MA Coastal Inventory and Assessment Report to qualitatively assess the conditions of existing coastal infrastructure in the Commonwealth.

A comparison of the conditions identified as part of the MADCR inspections performed in 2006 and 2013 to those recently performed by CLE in 2017 show that, in general, the stone groins remain in Excellent to Good ("A" to "B" rating, respectively) condition and the seawall is in Good to Fair ("B" to "C" rating, respectively; Figure 2.3) condition with observed surface spalling (Figure 2.3) and cracking but no visual signs of global failure (sliding, rotation, settlement, etc.). All of the structures were found to be in a condition which would provide protection to the upland along West Rodney French Boulevard during a major storm event.

However, should these structures be allowed to continue to deteriorate, it may not be possible to repair and/or augment them without a complete replacement. Generally, repairs to the sections of the seawall that are "C"-rated should be made within the next five (5) years to maintain the level of protection that the structures provide. Site inspection plans which detail existing conditions are provided as an attachment to the complete CLE inspection report provided as Appendix A of this report. The condition assessment should be considered preliminary since it is limited to visual inspection of exposed structures.

2.1.1 Groins

The project area includes a total of six (6) stone groins as shown in Figure 2.2. The groins are constructed similarly of 3 to 5 foot diameter stone with 1H:1V side slopes and a 6 to 10 foot wide level bench (Figure 2.3). The structures vary in length from approximately 175 to 400 linear feet (LF). The exact date of construction of the groins is not known, however, they are visible in an aerial photograph from 1945. Groin No. 4 was extended both in 1958 and 1978. The existing groin structures presently appear to have maintained their original slopes and lengths with few signs of displacement, settling, or scour. Groin No. 3 extends from the end of the bathhouse facility and appears to be the only groin which does not extend to West Rodney French Boulevard itself.

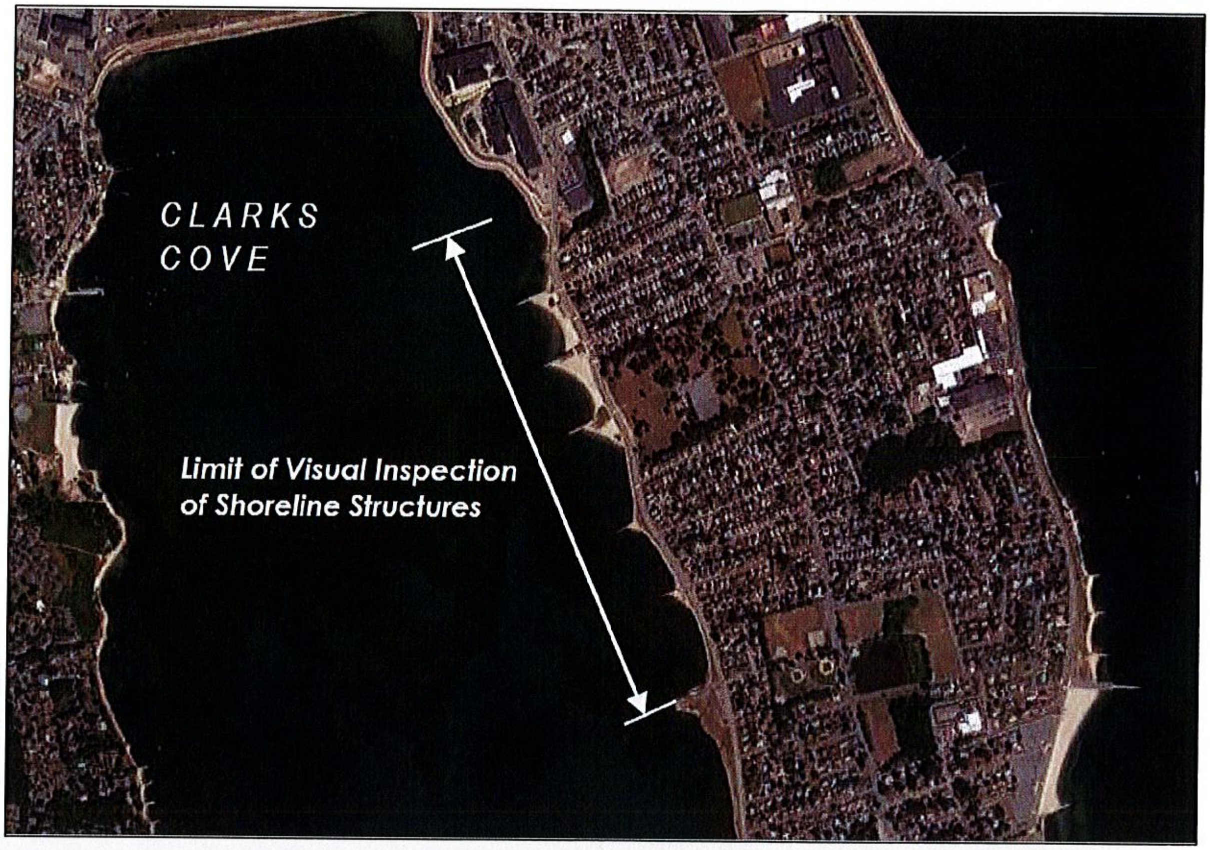


Figure 2.1 Limits of CLE shoreline inspection of coastal structures along West Rodney French Boulevard.

Figure 2.2 Location of Existing Stone Groins within Project Inspection Limits.

Table 2.1 below provides a summary comparison of the condition ratings assigned to the 6 groins as part of the MADCR inventory in 2006 and 2013 and as part of the inspection performed by CLE as part of this investigation. It should be noted that Groin No. 1 was not captured during MADCR 2006 or 2013 inspections.

In summary, the groins remain in ***Excellent* or *Good*** ("A" or "B" rating, respectively) condition in all of the inspection years. The structures exhibit minor issues which are considered primarily superficial. Accordingly, the current observed conditions of the groin structures are considered adequate to perform their intended functions under major coastal storm conditions.

Table 2.1. Year/Year condition comparison of groins.

Groin No.	MADCR Inventory No.	MADCR Rating		
		2006	2013	2017
1	N/A	N/A	N/A	B
2	049-009-000-286-200	B	B	B
3	049-011-000-030-400	A	A	A
4, 5 & 6	49-009-000-286-200	B	B	B

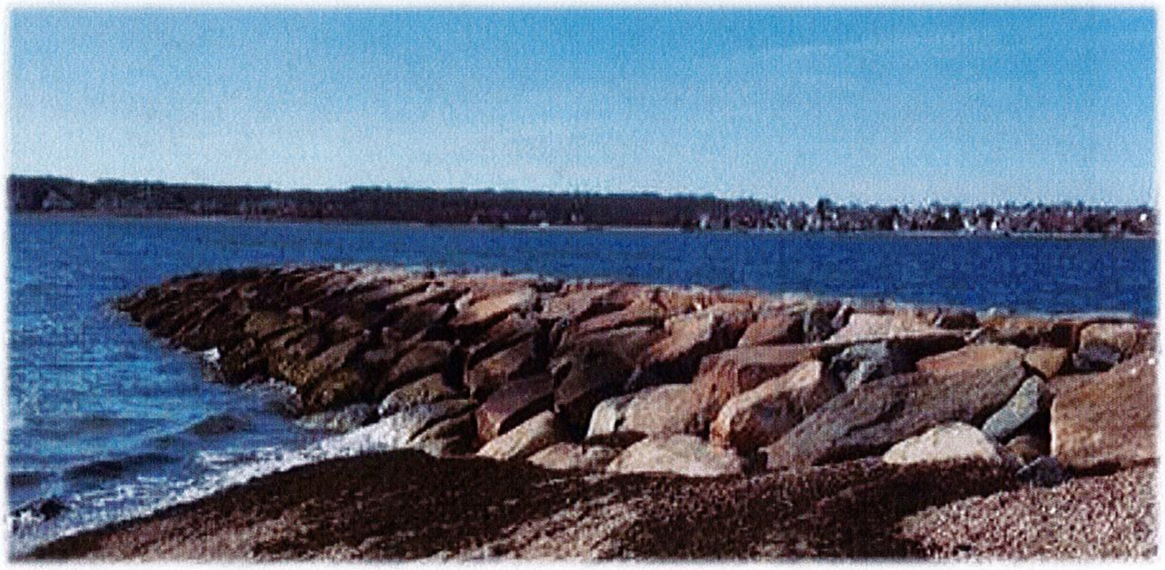


Figure 2.3. Typical groin condition at West Beach.

2.1.2 Seawall

The concrete seawall extends along West Rodney French Boulevard for approximately ±3,820 linear feet (LF). The exposed height of the seawall ranges from 3 to 10 feet above the existing beach elevation. The original date of construction of the seawall is unknown; however, the structure is present in a 1945 aerial photograph and available record documentation shows that the wall was extensively repaired/replaced in 1978. It appears that the original wall structure was comprised of stone which was subsequently overlain with an unreinforced concrete wall at some point in time. The aforementioned unreinforced concrete wall was then overlain with a reinforced concrete layer as part of the 1978 repair effort. Stone protection was also installed along the toe of sections of the seawall as part of this major repair. Figure 2.4 below illustrates the original stone, subsequent concrete overlays and toe stone. For this report, the full length of the wall has been divided into three sections to be similar to the assessments made as part of the MADCR Coastal Inventory: Wall Section 1.1 includes the southernmost ±2,500 LF, Wall Section 1.2 includes the middle ±535 LF and Wall Section 1.3 includes the remaining northernmost ±616 LF. All wall sections are constructed similarly and vary primarily in height. It is noted that toe stone protection was only observed along Wall Section 1.1 in intermittent sections, and the exact limits of the toe stone are limited to what is presently exposed. Additional areas of toe stone protection may be present but covered by windblown/accreted sand.

As shown in Table 2.2 below, the overall condition of the full length of the seawall varied from Good to Fair ("B" or "C" rating, respectively). Wall Section 1.1 remains in Good ("B" rating) condition including the exposed toe stone (Figure 2.5). The conditions observed along Wall Sections 1.2 and 1.3, however, are considered to be Fair ("C" rating) as there are presently visual signs of deterioration, cracking, spalling, etc. (Figure 2.6). Despite these observations, all wall sections still adequately provide flood protection; however, their ability to be reused in the future as a core structure for an elevated wall or to be repaired rather than replaced has been reduced. Sealing the existing cracks and grouting the surface spalls could significantly extend the life of the wall. Provided these measures are implemented within the next 5 years, it is viable that all wall sections could be raised for future sea level rise without a complete reconstructive effort. In addition, it is noted that the existing access ramp located at STA 4+75 exhibits severe deterioration along the wingwalls, and the structures' low elevation presents a risk due to low crest elevation (see Figure 2.7). An evaluation with respect to need for the ramp should be performed before implementing any repairs. The crest elevation of the seawall is shown to vary between 10.0 and 10.8 feet MLW, as shown in Figure 2.4.

Although the seawall itself is in adequate condition, the toe revetment in front of the seawall is in poor condition. Much of the revetment was adjusted or removed during the 1978 seawall reconstruction. Some of the anthropogenic material located on the beach was likely left during combined sewer overflow (CSO) construction and removal of parts of the revetment. The lack of structural support in front of the seawall increases the risk of failure during a storm event.

A total of seven (7) cast iron outfalls pipes are located along the length of the seawall (see Figure 2.8). These pipes extend out to/below Mean Low Water (MLW). Based upon available documentation, it is unclear as to the nature of the flow or associated volumes that presently discharge from these pipes. Further review of these structures should be conducted with the City to determine their current and future need and functionality.

Table 2.2. Year/Year condition comparison wall sections.

Groin No.	MADCR Inventory No.	MADCR Rating		
		2006	2013	2017
Section 1.1	049-007-000-112-100	B	B	B
Section 1.2	049-011-000-030-100	B	C	C
Section 1.3	49-013-000-055-100	C	C	C

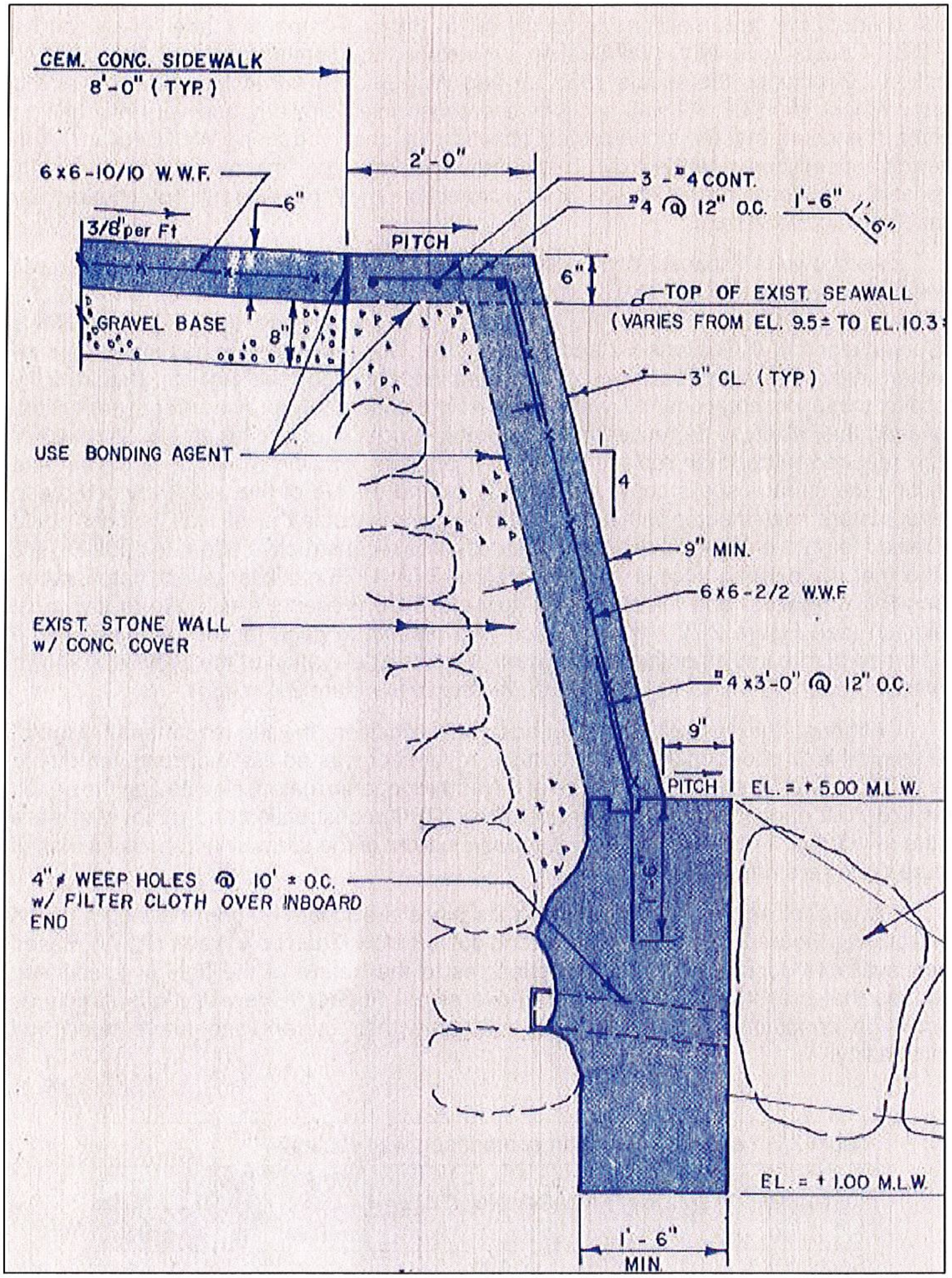


Figure 2.4. Typical seawall section upon completion of repairs in 1978.

Figure 2.5 Typical Good condition of the seawall along the south side (B rated structure).

Figure 2.6 Typical cap spalling at STA 5+00.

Figure 2.7 Ramp heavy spalling STA 4+75.

Figure 2.8 Existing cast iron outfall pipe.

2.2 Shoreline Change Analysis

Based on long-term shoreline information (Figure 2.9), it appears that some limited landward migration of the natural shoreline has occurred since the late 1800s. However, the majority of the observed shoreline change appears to be due to anthropogenic modifications. Specifically, the construction of the seawall and a series of groins (and alterations to these structures over time), as well as placement of beach nourishment in the 1950s and 1970s, have had a significantly larger influence on shoreline change than natural coastal processes. Understanding the influence of these anthropogenic effects was critical for limiting the use of the data for quantifying coastal processes. Therefore, it was determined that a more recent short-term shoreline change analysis would be most appropriate for evaluating local coastal processes. It is understood a priori that due to seawall construction, the shoreline position became fixed over time in areas where erosion reached the coastal engineering structure. Therefore, the long-term influence of coastal erosion that leads to beach lowering along the seawall is not accounted for in the shoreline change analysis.

Use of shoreline change information allows quantification of coastal processes by providing a measure of nearshore accretion or erosion. For the Clarks Cove shoreline, high quality shoreline data sets are available dating back to the mid-1800s. However, as stated above, it was determined that short-term shoreline change subsequent to the most recent groin modifications and major beach nourishment placement would be most appropriate for understanding contemporary coastal processes

Shoreline change is typically minimal along stretches where coastal engineering structures have been built. In many of these areas, notably along un-nourished sections of the West Rodney French Boulevard shoreline, the fronting beaches are submerged at high tide. The shoreline change analysis focused on these areas; however, shoreline change rates for the entire West Rodney French Boulevard coastline were determined for the time period between 1997 and 2009. 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 seawall/revetment, 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 aerial orthophotographs for 1997 and 2009. The high water shoreline position change rates were calculated by casting perpendicular transects to the later input shoreline at each analysis point 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. Figure 2.10 graphically illustrates the short-term shoreline change results.

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 2.3. These calculations estimated the total RMS uncertainty to be ±1.2 feet/year from 1997 to 2009.

Table 2.3. Estimates of Potential Error Associated with Shoreline Position Surveys	
Orthophotography (1997, 2009)	
Delineating high-water shoreline position	±10 ft
Position of measured points	±10 ft
GPS Surveys (2017)	
Delineating high-water shoreline position	±3 to ±10 ft
Position of measured points	±3 to ±10 ft

2.3 Topographic Surveys

Topographic and bathymetric surveys were conducted in January 2017, in a joint effort between CLE Engineering (bathymetry) and the City of New Bedford (topography). The topographic survey included the beach and nearshore areas extending to the approximate 0-ft NAVD contour line and included the beach area and adjacent coastal protection structures, as well as upland public infrastructure along West Rodney French Boulevard.

In addition, a hydrographic survey of the near-shore areas was performed by CLE, extending to a minimum of 300 feet from the shoreline. The survey was performed utilizing a multi-beam fathometer to allow complete bathymetric coverage of the entire survey footprint. This equipment also can provide an initial estimate of submerged aquatic vegetation (SAV) (e.g. eelgrass) coverage based on the 'strength' of the acoustic return from the fathometer. The results of the bathymetric survey are shown in Figure 2.11. The combined results of the topographic and bathymetric surveys will be prepared as part of the existing conditions plans. The initial assessment of the multi-beam data indicated that the eelgrass coverage was limited; however, an underwater video survey was performed to ground-truth this information.

The video survey was conducted in in late February 2017. The results of this survey indicated significant eelgrass coverage in the nearshore region. This coverage may result in the need for augmenting coastal structure design to contain any future beach nourishment to ensure that significant volumes of material do not migrate offshore and smother existing eelgrass beds. The video results were field-verified by Stantec biologists in May of 2017 (Figure 2.12).

Historically, eelgrass within this area appears to be 'rebounding' from historical conditions. As shown in Figure 2.13, very little eelgrass was present in the 1980s, likely as a combination of CSO discharges into this portion of the coast, as well as the status of the wastewater treatment facility. By 1996, it appears that eelgrass had recovered as a result of CSO improvements and wastewater treatment plant upgrades.

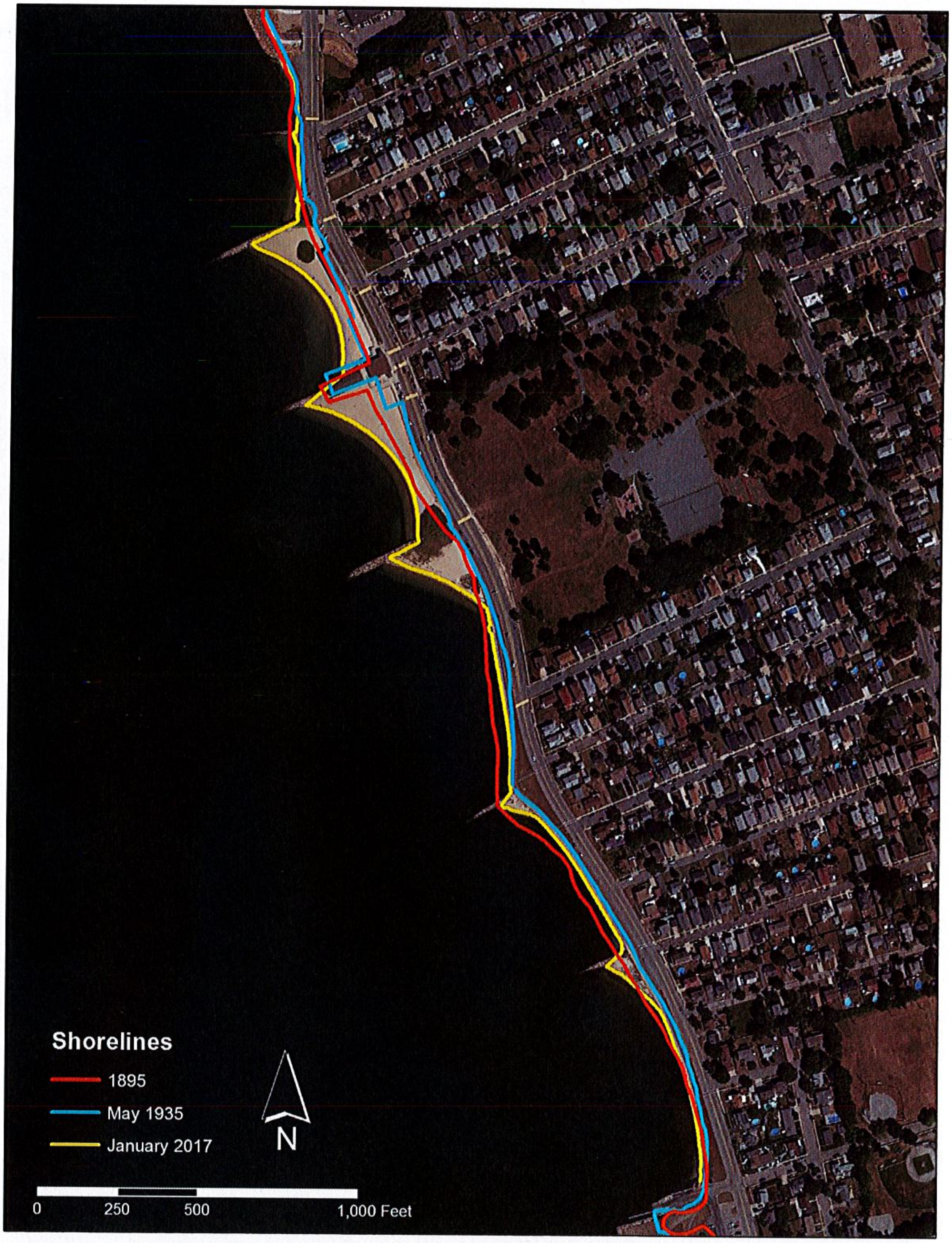


Figure 2.9 Shorelines from 1895, 1935, and 2017, where the shorelines from 1895 and 1935 were derived from the MCZM shoreline database and the 2017 shoreline position was surveyed using RTK-GPS equipment.

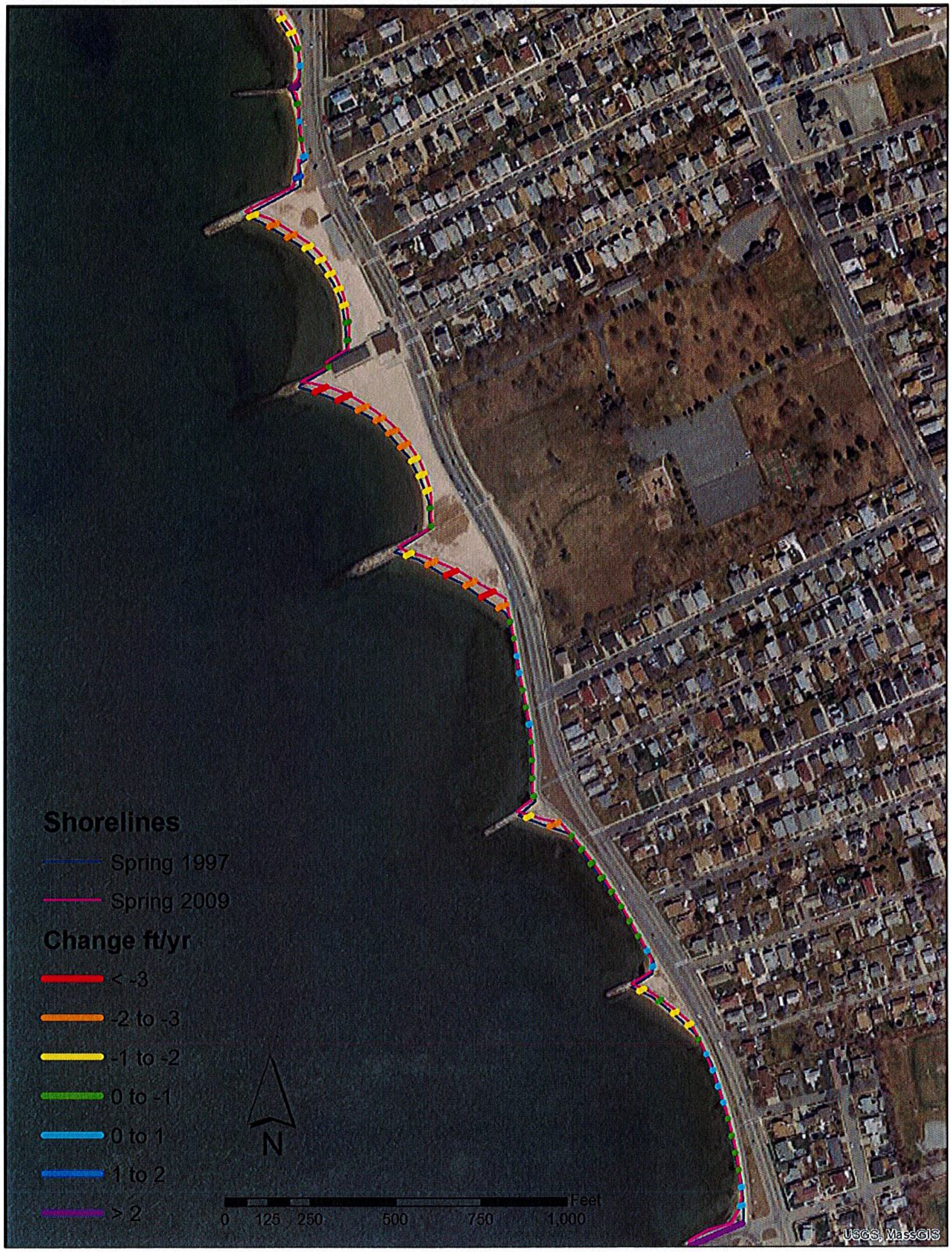


Figure 2.10 Historical shoreline change for West Rodney French Boulevard shoreline from 1997 to 2009.

Figure 2.11 Bathymetric contour data from 2017 multi-beam survey.

Figure 2.12 Eelgrass coverage based upon results of January 2017 reconnaissance video survey by CLE, Inc. and in-season May 2017 diver survey by Stantec.

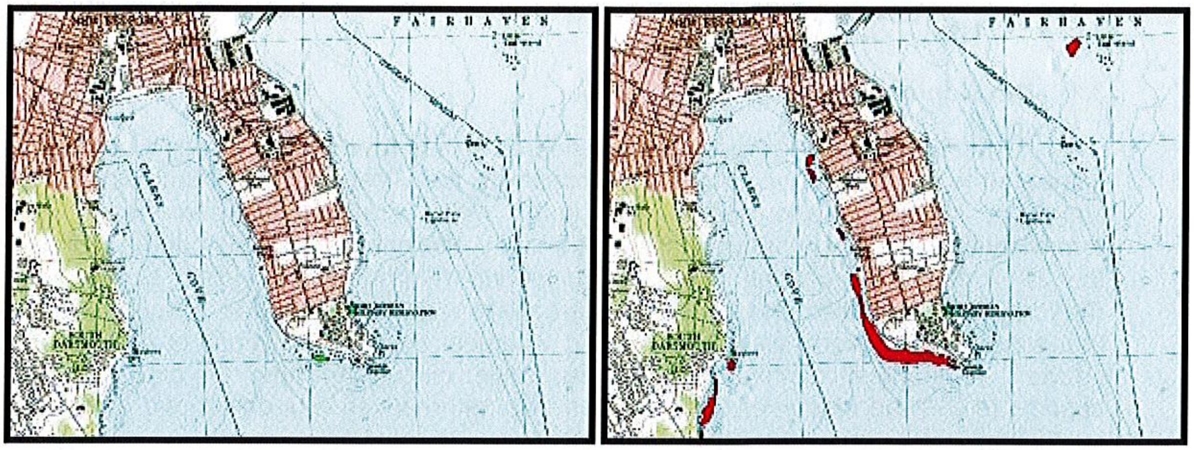


Figure 2.13 Comparison between the Costa 1980s (left) and DEP's 1996 (right) showing increase in eelgrass cover around Clarks Point New Bedford area. This was one of the few areas of increase between the two surveys and may have resulted from the improvements to the wastewater facility, and perhaps more importantly, the elimination of dry weather discharges from CSOs on both sides of Clarks point.

3.0 ALTERNATIVES ANALYSIS

3.1 Development of Alternatives

A number of alternatives were considered to improve storm damage protection compared to the *status quo,* "No Action" option for the West Rodney French Boulevard shoreline, including sand beach fills, groins with T-head breakwater heads and rehabilitating the existing seawall. The preliminary alternatives analysis can be found in the 2017 report *Conceptual Coastal Engineering Alternatives for West Beach New Bedford, MA* by Applied Coastal. The alternatives were considered to develop options that would reduce storm wave overtopping volumes, improve resiliency of the existing seawall, protect critical city infrastructure, while minimizing adverse environmental impacts to wetland resources. A list of potential alternatives was developed for further evaluation:

- Alternative 1 – No Action
- Alternative 2 – Seawall improvements
- Alternative 3 – Beach nourishment
- Alternative 4a – Beach nourishment with structural enhancements (Toe Berm)
- Alternative 4b – Beach nourishment with structural enhancements (T-head Groin)

Example Alternative Comparison Matrix for Practicability

Practicability Category	Factor	Alternative 1 (No Action)	Alternative 2 (Seawall Improvements)	Alternative 3 (Beach Nourishment)	Alternative 4 (Nourishment and Toe Berm)	Alternative 5 Applicant's Preferred (Nourishment and T-head Groin)
Features	Additional Shore Protection Capability	NO	YES Reduction in risk, but minimal improvement.	YES Improvement for a design life	YES Improvement for a design life	YES Improvement for a design life
Environmental Impacts	Habitat Coverage	NO	NO	YES Will cover some eelgrass habitat.	YES Negligible impact to eelgrass	YES Negligible impact to eelgrass
	Footprint Coverage	N/A	N/A	80 ft berm: 11.8 acres 40 ft berm: 8.8 acres	4.7 acres	4.2 acres
Cost (No cost threshold established)	Upfront	NO	YES Least Cost (of the designed structures)	YES	YES Greatest Cost	YES
	Long Term (Post Storm)	YES	YES	YES Negligible	YES Negligible	YES Negligible

3.2 Description of Alternatives

3.2.1 No Action Alternative

Under the No Action alternative, natural processes would occur without any form of human intervention to repair or reconstruct existing shoreline protection. There would be no prevention of continued beach migration and storm damage to existing public infrastructure that occurs during storm events. There are no upfront costs with no action, but future costs to repair or rehabilitate structures will increase as remaining storm protection from the existing beach and seawall system will continue to diminish as a result of ongoing erosion and degradation of the wall.

The beach continues to lower and expose the seawall to wave action, and therefore wave protection is worse than conditions in the early part of the 20th century. The seawall protects infrastructure from failing behind it, but also accelerates erosion by reflecting wave energy, removing sand from the front of the structure. As erosion continues unabated, the beach profile along the wall will continue to lower. As the profile lowers, storm waves impacting the seawall will increase in height due to less breaking in the deeper depths fronting the wall. With larger wave heights, overtopping rates will also increase during storms, resulting in more frequent and severe erosion and damage to the paved and unpaved upland area behind the wall.

Historically, the shoreline within the project site has not seen significant damage since the 1938 hurricane. During the 1938 Hurricane, substantial damage occurred throughout New Bedford and the shoreline along West Rodney French Boulevard underwent heavy damage. A hurricane of equivalent magnitude to the 1938 could have serious implications to the City under the No Action alternative. A storm of that magnitude would destabilize the seawall and expose the sewer main behind it. An aging sewer exposed to waves in hurricane conditions would certainly fail and release raw sewage from the entire city into Clarks Cove. Cleanup costs would be extraordinary, and the sewage would devastate any habitat or marine life for decades. Further discussion of the sewer and implications of a failure are discussed in Section 4.3.

3.2.2 Seawall Improvements

For this alternative, improvements to the seawall are proposed to reduce overtopping during storms and were compared based on storm performance. The first proposed improvement is to increase the elevation of the seawall, increasing the effective height and reducing storm overtopping. The second proposed improvement is strengthening the existing seawall by rebuilding it or installing fronting sheet pile. Neither of these improvements alone address the issues related to the continuing erosion and lowering of the beach fronting the wall.

Increasing the crest height of the seawall was evaluated for storm performance. This evaluation used overtopping rates of the West Rodney French Boulevard seawall for three engineering scenarios during storms, and for a range of water levels. The structural scenarios used in this analysis include 1) the existing conditions of the beach and seawall, 2) the existing beach with a two-foot-high cap wall added to the seawall and 3) the existing seawall with the planned nourishment. The water levels used range from just above MHHW (1.9 feet NAVD) to the point where mean overtopping volume rates become large enough in all scenarios to damage the paved promenade of the seawall. Based on guidance provided by Owen (as presented in Herbich, 2000), overtopping discharges should be limited to 200 liters/meter/second to prevent damage to pavement.

The method used to calculate overtopping rates in this case is presented in the Army Corps Coastal Engineering Manual (CEM, 2002). By this method, the overtopping rate, q, of a vertical barrier is expressed as

$$q = 0.082 \exp\left(-3.0 \frac{R_c}{H_s} \frac{1}{\gamma_\beta \gamma_s}\right) \sqrt{g H_s^3}$$

where H_s is the significant wave height, g is the gravitational constant, R_c is the freeboard of the wall above the still water level, γ_β and γ_s are reduction factors based on wave attack angle and structure geometry, respectively. γ_s has a value of 1.0 for plain, impermeable walls like what is in place at West Beach.

The value of offshore value of Hs used in this analysis is 5.2 feet in the center of Clarks Cove. This was determined using conservative wave modeling results from the wave condition that generated the largest wave offshore of the beach. In the calculation procedure, if the water depth (d) at the toe of the structure was not deep enough to support the offshore wave height, the breaking wave height (H_b) expressed as H_b=0.78d (McCowan, 1891) was used.

Overtopping volumes as a function of still water elevation are presented in Figure 3.1. For existing conditions, overtopping rates that would cause damage to the paved surface of the upland behind the wall occurs for water levels above +6 feet NAVD, which is close to the present FEMA 10% (10-year return period) water level. With both the nourished beach and the case where only a 2-foot crest wall is added to the existing seawall, the overtopping rate exceeds 200 l/m/s at water elevations in excess of +8 feet NAVD, which is between a 20- and 25-year return period water level (between a 4% and 5% percent water level).

The greatest disadvantage of the crest wall scenario is that management of overtopping discharges becomes an issue since upland runoff from overtopping is not able to flow back over the wall, resulting in ponding seawater and channelized flow along the wall. The ponding of seawater accelerates the breakdown of asphalt and may require road improvements. Channelized flow along the road can cause erosion of soil or landscape areas.

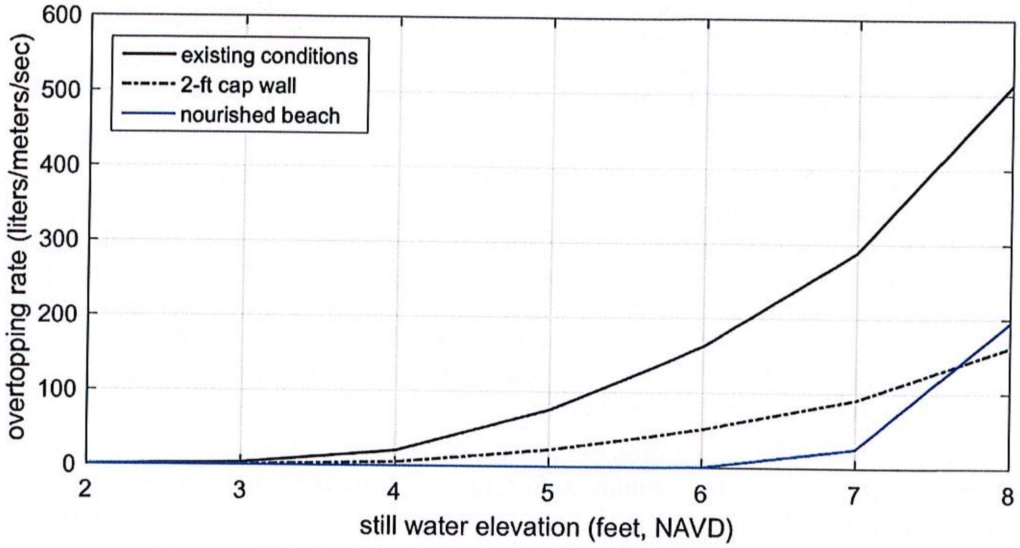


Figure 3.1 Mean overtopping rate of the West Beach seawall as a function of still water elevation, for the three structural alternatives compared in this analysis.

The second improvement option is to buttress the existing wall with sheet pile. This has other disadvantages in addition to those determined for the increase in seawall height. One disadvantage is that sheet pile is often considered as a permanent, no-maintenance option. In practice, however, lack of maintenance leads to degradation of the sheets which eventually leads to failure of the wall, often catastrophically during a storm event. Another disadvantage is that a sheet pile wall without nourishment does not address future accelerated lowering of the beach profile caused by wave reflection against the vertical face of the wall. Sheet pile walls reflect a large percentage of the incident wave energy. The reflected wave energy adds to the incident wave, increasing wave heights near the wall, which in turn mobilizes sediment to greater depths at the wall. This leads to an amplification of the rate of erosion, since reflected waves lead to a lowering of the beach at the wall, which the leads to larger waves being able to impact the wall, and a further increase in reflected wave energy. This amplifying effect of vertical walls on beach erosion is the main disadvantage of sheet pile in an active coastal environment.

The seawall improvements proposed in this alternative do not address the continued erosion and lowering of the beach fronting the seawall. Implementing one or multiple improvements will amplify the rate of erosion due to increased wave reflection and increase beach lowering. This will accelerate the degradation of the wall, the exposing of its foundation, and lead to greatly increased risk to the critical upland infrastructure that the wall protects.

3.2.3 Beach Nourishment

Beach nourishment would add sediment seaward of the existing beach profile to absorb and dissipate wave energy, thereby increasing protection to infrastructure and property currently threatened by overtopping and storm damage. Once nourishment material is in place, coastal processes will rework the nourishment material to create an equilibrated beach profile. The ongoing sediment transport will transport the nourishment material both cross-shore and alongshore. Due to the ongoing transport 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.

The sediment transport potential modeling results indicate that transport rates reach a maximum north-directed magnitude in the vicinity of the Oaklawn Street groin. This is an effect of the varying orientation of the seawall, which forms a point of land at this groin. The gradient in average sediment transport rates along this shoreline stretch provided initial evidence that it would be difficult to maintain beach nourishment along the project shoreline. With no additional measures taken to contain the nourishment, it would be difficult to maintain a beach width along the entire shoreline, and the nourishment could infiltrate into adjacent eelgrass resources.

Building on the insights provided by the sediment transport potential analysis, the shoreline model was used to simulate different nourishment templates, to investigate how they would evolve with time. Two fill templates where modeled, one with a berm width of 40 feet and a second with a berm width of 80 feet. The fill template for each model run was bounded by the boat ramp to the south and the Hazelwood Park groin to the north. The 40-foot beach nourishment would have an estimated fill volume of 34,400 cubic yards, while the 80-foot berm would have a volume of 75,300 cubic yards.

These two nourishment scenarios were run for 10-years (using the WIS wave record starting in 1997). With the 40-foot berm, the nourishment template MLW line begins within the shoreward limit of eelgrass (Figure 3.2) and remains within that limit for the duration of the 10-yer simulation (Figure 3.3). Based only on the percentage of the original fill volume remaining within the limits of the fill template, the 40-foot berm fill has a design life of seven years, which is the length of time that elapses before the percent remaining drops below 30%. The actual usable design life is less than this, since the berm line of the filled beach comes in contact with the seawall in the area north of the Oaklawn Street groin after only three years.

The 80-foot-wide beach nourishment option initially has more than twice the volume of sand than the 40-foot template. The performance of this fill template is much better than the 40-foot fill, based on the percent fill remaining (Figure 3.4), which indicates a design life of more than 10-years. However, the estimated MLW line is seen to encroach on the identified eelgrass areas even with the initial placement of the fill (Figure 3.5). The initial nourishment template completely covered 1.05 acres of seagrass resources. At the end of the 10-year simulation (Figure 3.6), there was continued displacement of the MLW line into the eelgrass areas, while the berm line of the beach was within 15 feet of the seawall in the area north of the Oaklawn Street groin. There was an additional 0.24 acre of eelgrass covered by sand from the nourishment.

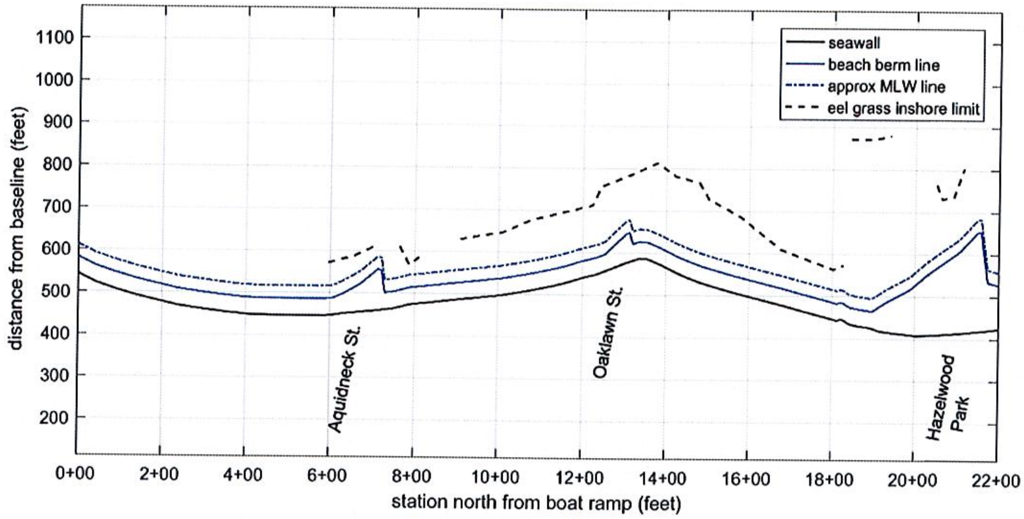


Figure 3.2 Starting shoreline for simulation of a 40-foot-wide nourishment template. Beach berm line, and approximate MLW shoreline are shown with the inshore limit of identified eelgrass resources (from May 2017 survey).

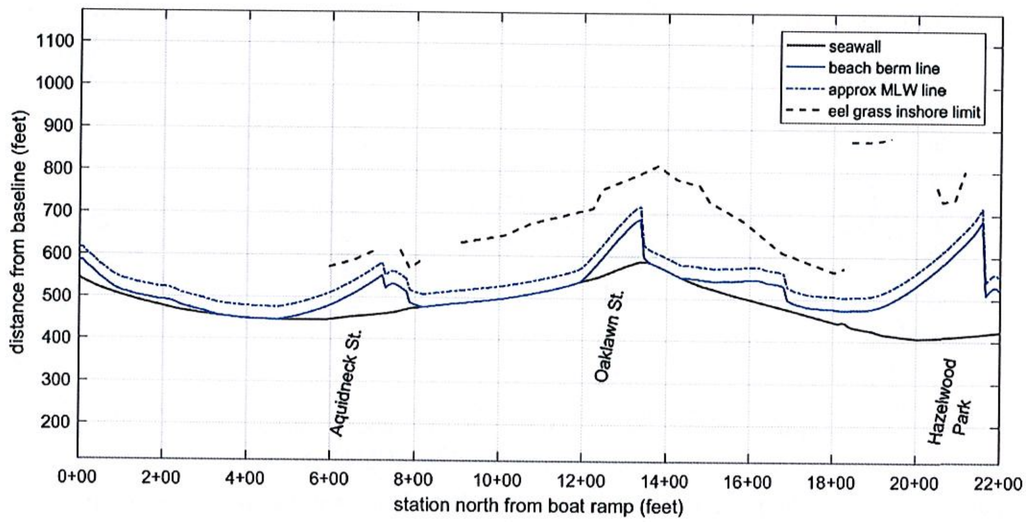


Figure 3.3 Modeled shoreline at the end of the 10-year simulation of a 40-foot-wide nourishment template. Beach berm line, and approximate MLW shoreline are shown with the inshore limit of identified eelgrass resources (from May 2017 survey).

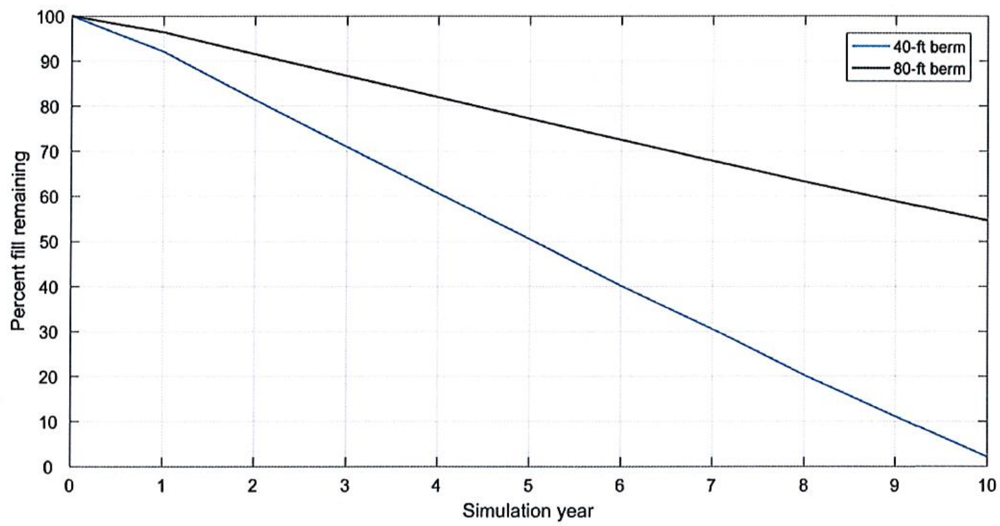


Figure 3.4 Percent fill remaining for the two modeled nourishment scenarios.

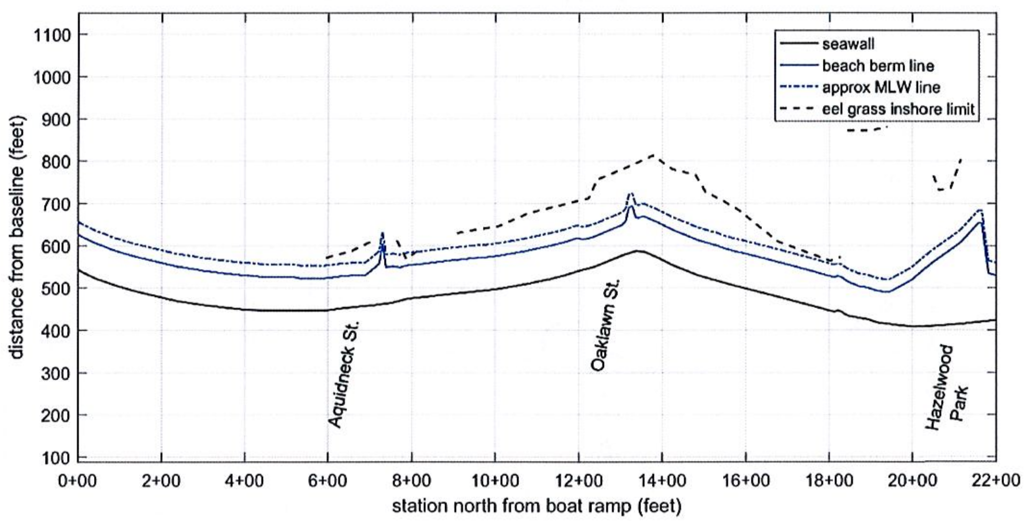


Figure 3.5 Starting shoreline for simulation of the 80-foot-wide nourishment template. Beach berm line, and approximate MLW shoreline are shown with the inshore limit of identified eelgrass resources (from May 2017 survey).

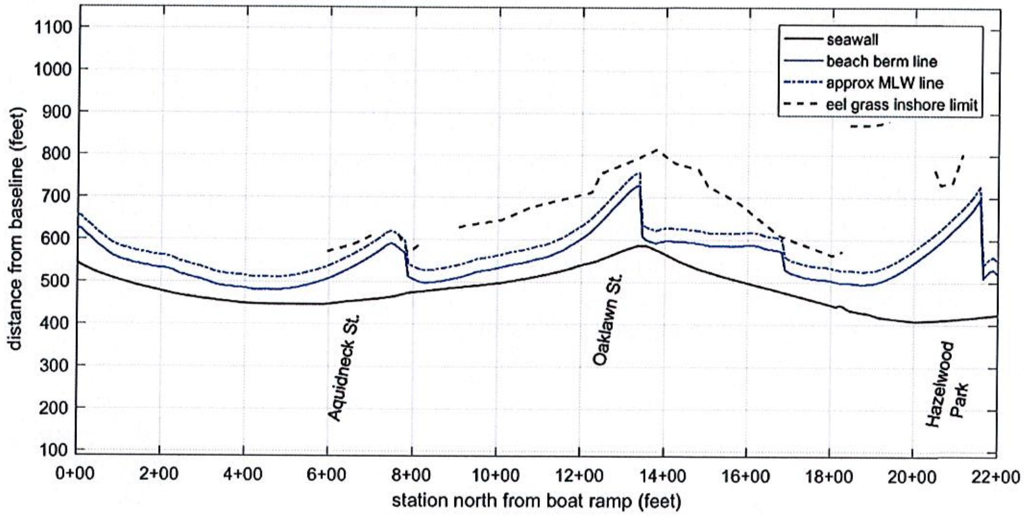


Figure 3.6 Modeled shoreline at the end of the 10-year simulation of an 80-foot-wide nourishment template. Beach berm line, and approximate MLW shoreline are shown with the inshore limit of identified eelgrass resources (from May 2017 survey).

3.2.4 Beach Nourishment with Structural Enhancements

Hard coastal engineering structures can be used to control the cross-shoreline or alongshore movement of beach nourishment. These structures can be constructed to be sand tight barriers that largely block sand movement, or as more permeable impediments to sand transport that do not completely block transport, but instead reduce or inhibit transport rates to control beach width. For West Beach, it has been demonstrated that proposed beach nourishment fill would be highly mobile and therefore would require a system with the primary purpose of containing the fill to increase design life as well as preventing it from infiltrating into the identified eelgrass resources that are found along the project shoreline. Therefore, the structures must effectively control the cross-shore movement of sand. There are two main types of structures that can be utilized to control this cross-shore movement of sand: a toe berm and T-head groins.

Toe Berm. An option that could be used along the project area is a toe berm (structural toe) for the nourishment fill, which is designed to contain the seaward movement of the filled sand volume. The structural toe, or perched beach, concept has a low stone berm placed at some distance offshore of the seawall. For West Beach, this stone berm would be placed at some minimum distance from the identified eelgrass habitat areas and have a crest elevation which is at least a couple of feet above the intersecting profile of the filled nourishment template. As conceived for the shoreline study (Figure 3.7), the submerged berm would have a crest width of 10 feet , side slopes of 1:2.5 (v:h), would follow the -4.0 feet NAVD contour, and would be placed 25 feet shoreward of the eelgrass area (Figure 3.8), at a minimum. The crest elevation would be -0.9 feet NAVD, which is about 2 feet above the fill template elevation at the location where the minimum distance between the seawall and eelgrass occurs. The estimated footprint of this toe-berm is 49,400 square feet.

When compared to the nourishment by itself, the addition of the toe berm does not improve the engineering performance or design life of the fill, since it only helps to limit the cross-shore sand movement into the identified eelgrass resource areas. The cross-shore movement of sand is only an issue for this project due to the presence of eelgrass, since cross-shore losses are not large in the areas of West Beach that have sandy beach.

The toe berm has no effect on the long-shore movement of sediment; therefore, it has no influence on the design life of any placed beach fill or on renourishment frequency. The greatest challenge for the perched toe option is the footprint required to construct it. The estimated area is more than two times more than the footprint area available by removing and reducing existing structures in place along west beach, and therefore not acceptable from a resource impact stand point.

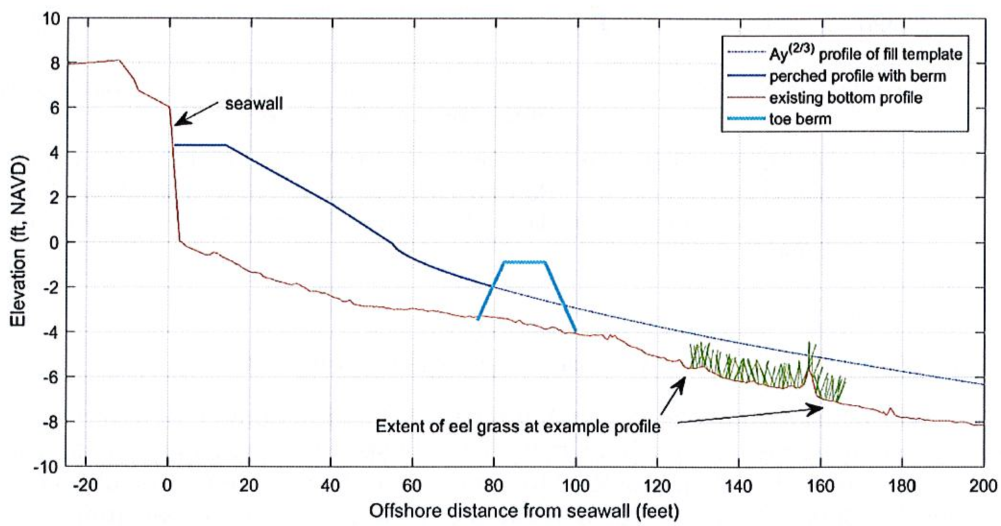


Figure 3.7 Conceptual profile of perched beach alternative, showing the positioning of the toe berm structure and filled nourishment profile relative to the existing eelgrass extent.

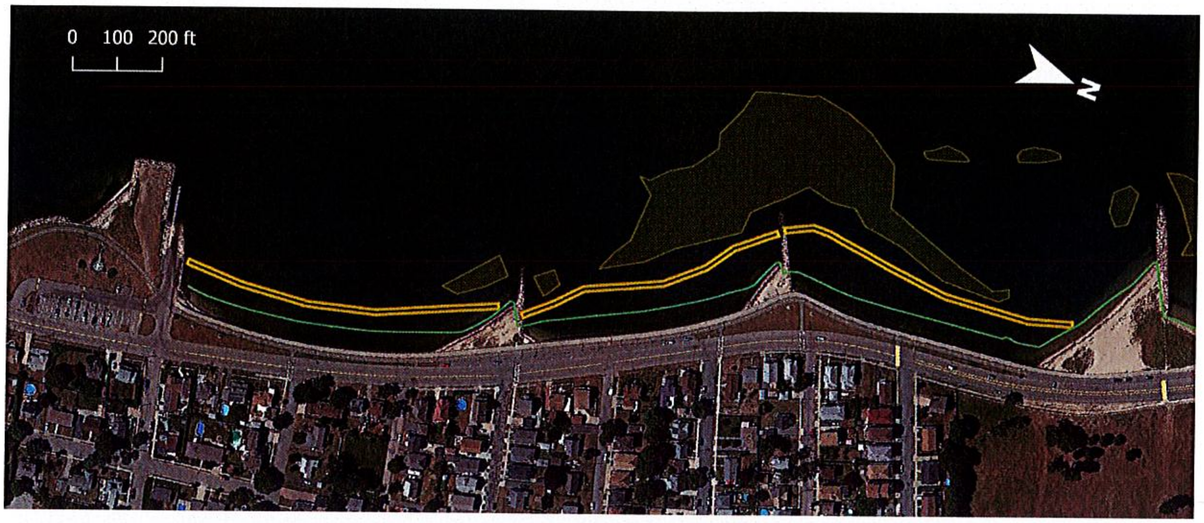


Figure 3.8 Conceptual plan for perched beach alternative, showing toe berm layout (orange-black line), and MHW line (green solid line) of the 40-foot wide beach. Areas of eelgrass identified in the May 2017 survey are also provided.

T-head Groins. The construction of a T-head groin field along the project shoreline is another alternative that utilizes structures together with nourishment in order to improve the storm survivability and resiliency of the seawall while protecting the identified nearshore eelgrass resource areas that exist along West Rodney French Boulevard. T-head groins are essentially short offshore breakwater sections that are connected to the offshore tip of a conventional wooden shore-perpendicular groin, as shown in Figure 3.9. The positioning of the tips of the T-head sections shapes the resulting equilibrated shoreline by the diffraction of waves as they enter the groin compartments between the T-head sections. The groins and T-heads act together to hold the beach fill material in place, which increases its engineering design life. The shore-perpendicular groin trunks are used to control the along-shore movement of beach sediment, while the T-head breakwaters act both to control cross-shore and along-shore movement. The T-heads also are positioned strategically to prevent the infiltration of the fill into the identified eelgrass resource areas

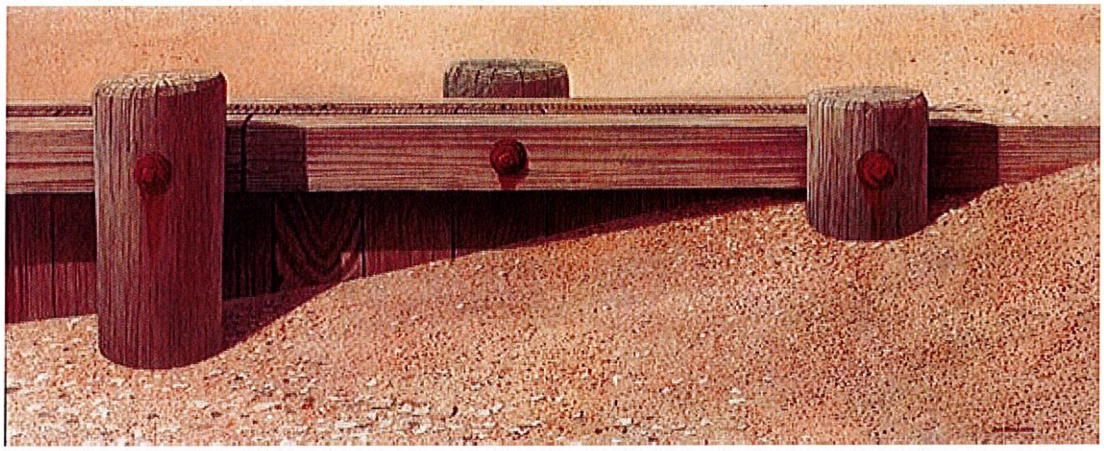


Figure 3.9 Example of a typical wooden groin that would be utilized as the trunk section of the T-head structures, where the armor stone shore parallel breakwater section would join the shore perpendicular wood section to form the 'T". The trunk initially will be covered by sand, with a small portion near the breakwater uncovered. The coverage of the wooden trunks will fluctuate as the beach establishes an equilibrium profile.

Design guidance is provided by Bodge (1998 and 2003) and Hansen and Krause (2001). By the method developed by Bodge, the MLW shoreline position is estimated using the known gap width between T-head sections and the average wave approach angle (Figure 3.12). The design MLW shoreline is taken as the average of a parabolic spiral that is described by Hsu et al. (1993) for shorelines under the influence of headlands (natural and man-maid) and a simple line that is drawn using a straightforward fraction of the gap distance (G/3, or one third of the gap distance). Besides the gap width, an average wave angle is used in the calculation of the parabolic spiral shoreline. For West Beach, a wave-energy-weighted, vector-averaged wave angle was determined using the output of the wave model of Clarks Cove, including all modeled compass sectors and wind bands. The average wave angle relative to the average shoreline orientation is 16.4 degrees by this method, where the detailed modeling analysis is described in Section 4, below.

Figure 3.10 Aerial imagery of a T-head groin field along the James River in Virginia. Imagery is from USGS 2013 satellite observations.

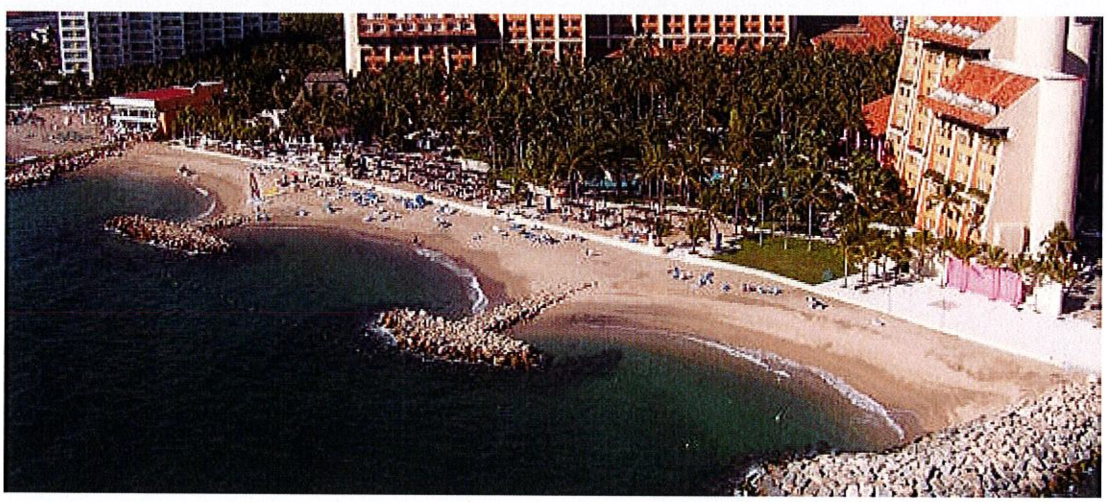


Figure 3.11 Example of a T-head groin project by Olsen and Associates (Photo by Olsen and Associates).

After the development of two initial T-head layouts as part of the original 2016 MCZM grant study (Applied Coastal, 2017), a refined layout was designed with the following goals: reduce structure footprint to within the area that could be taken from existing structures along West Beach, and provide a nourishment template with sufficient volume for the purpose of enhancing the storm performance of the existing seawall. Based on the results of this refined layout which minimized the overall project 'footprint' and ensured that offshore eelgrass resources were protected, the T-head groins in combination with beach nourishment was determined to be the preferred alternative.

The final, optimized T-head layout has a minimum MLW gap distance of 20 feet, while the most common MLW gap distance is 30 feet. The beach fill is designed to have an equilibrated slope of 1:10 (v:h), and a minimum berm width (the distance between the seawall and the +3.5 ft NAVD contour) of 30 feet. The construction template would initially be filled along a line parallel to the seawall (dashed green line in Figure 3.13), have a crest elevation of +4.5 ft NAVD, and a foreshore slope of 1:6 (v:h). The completed construction template would, with time, evolve into a crenulated beach (as shown by the berm crest line shown in Figure 3.12) as waves influenced by the T-heads shape the beach.

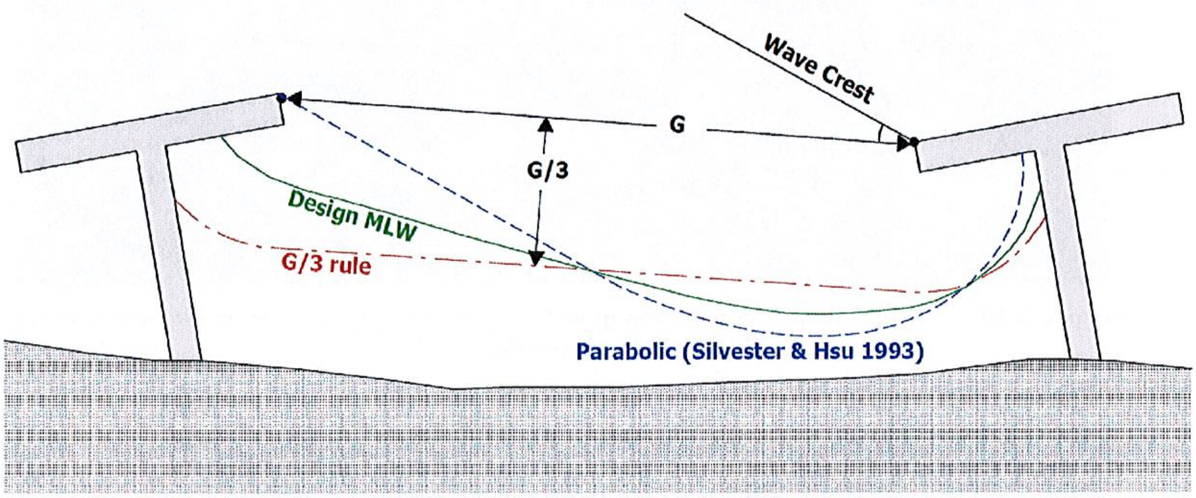


Figure 3.12 Bodge method prediction of MLW shoreline in T-head groin compartments (from Hansen and Krause (2001).

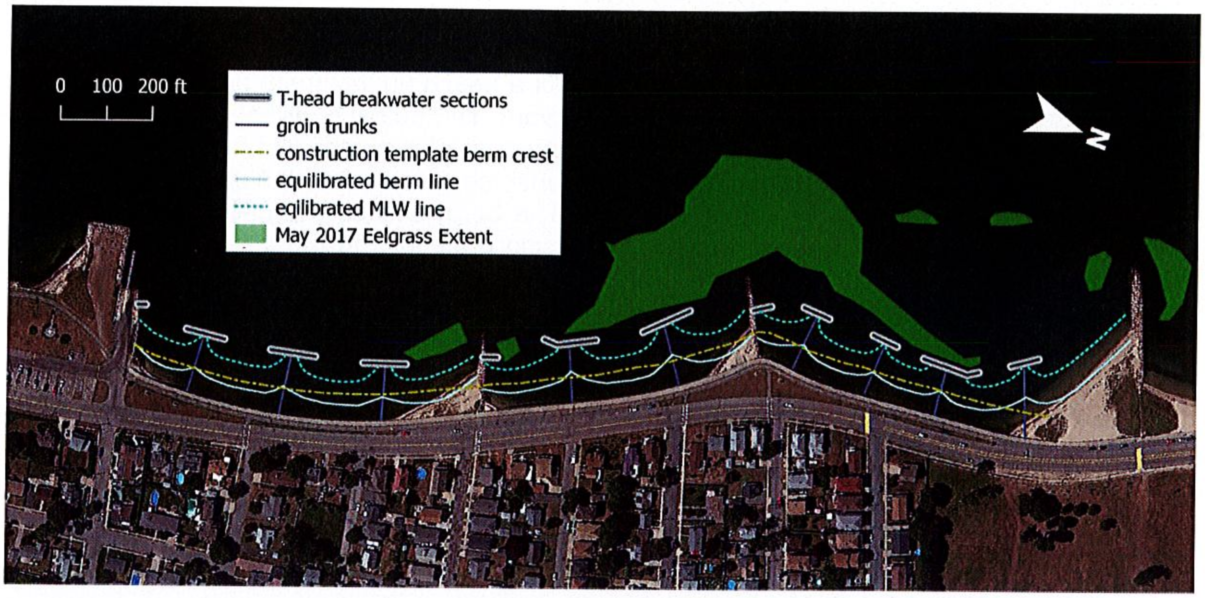


Figure 3.13 Conceptual layout of T-head groin alternative with a 30-foot-wide beach. T-heads are indicated by the gray-black lines, the beach berm line (+3.5 feet NAVD) is the solid gray line, the estimated MLW shoreline is the dashed line, and the construction template berm line (+4.5 feet NAVD) is the green dot-dashed line. May 2017 eelgrass survey results are shown.

The plan for this alternative includes the removal of the Woodlawn Street groin and portions of the groins located at Valentine Street and Hazelwood Park to offset the area taken up by the new T-head breakwaters. The Valentine Street and Hazelwood Park structures were extended in the 1970s, at the time West Beach was nourished. These extensions would be removed to existing (beach) grade level, leaving the older main groin trunks in place. Additionally, the tips of the groins at Oaklawn and Aquidneck Streets would be reconstructed in order to provide short "L" breakwater ends to better retain the sand within the adjoining, downdrift groin compartment. Existing CSOs along the project shoreline would remain and be reconstructed. Each would be configured to run along the proposed groin trunk and through the T-head breakwater. A one-way check valve would be installed in the line to ensure unidirectional flow from the upland to the ocean.

4.0 PROJECT DESCRIPTION

The recommended project is to construct a beach nourishment project seaward of the seawall along West Rodney French Boulevard. The beach fill will be contained with a series of T-head groins, consisting of shore perpendicular trunks, and heads that parallel the orientation of the shoreline. The beach nourishment will extend the berm seaward and provide additional sediment to the system. The berm can be designed to absorb and dissipate storm wave energy, thereby increasing protection to the infrastructure behind the seawall. The additional shore protection will reduce risk of seawall destabilization during a large event (e.g., hurricane). Once beach nourishment material is in place, coastal processes will rework the nourishment material to create an equilibrated beach profile. The additional construction of the T-head field will provide environmental mitigation to contain nourishment sediment from migrating offshore into eelgrass habitat. While expansion of coastal engineering structures is generally discouraged by environmental regulatory agencies, recommendations to "trade" structures, where there is no overall increase in the cumulative "footprint" of coastal engineering structures, may have merit to maximize shore protection goals. This can be accomplished by dismantling portions of existing structures and "trading" them for optimized new structures.

There are several environmental concerns with beach nourishment including altered water quality and natural habitat disturbance from depositing of material. Adherence to regulations and temporal considerations can help mitigate adverse impacts by avoiding vegetative, shellfish, and shorebird activity. With careful design and planning, a beach nourishment with a T-head groin field is a practicable alternative for the shoreline along West Rodney French Boulevard in New Bedford, MA to protect infrastructure. Evaluations of nourishments must have clear performance expectations, as they are designed to exist within the project area only for a specific period of time. These projects are meant to manage coastal erosion, and do not prevent it entirely. Damage by exposure to the ocean and waves to infrastructure upland is postponed by the nourishment for a designed length of time, after which renourishment must be anticipated to maintain proper shore protection. The frequency of renourishment is dependent on the initial design of the project. Permit level plans of the proposed project are provided in Appendix B.

To optimize the design life of this shore protection methodology, the performance of the beach nourishment under severe storm wave conditions, as well as typical long-term 'average' wave conditions was evaluated by conducting a detailed modeling and analysis from Groin 1 south to the boat ramp. Analyses of the nearshore wave environment, alongshore sediment transport, and cross-shore equilibration were utilized to inform the design process for the beach nourishment alternative, as described in the following sections.

4.1 Historical Analysis

A historical analysis of shoreline changes to the West Rodney French Boulevard shoreline was assembled to assist in evaluating alternatives. The earliest documented shoreline position from the MCZM database is from 1895. Aerial imagery from 1938 provides insight in the shoreline position relative to the seawall (Figure 1.3), prior to nourishments placed in 1958 and 1977. The West Rodney French Boulevard shoreline has experienced modest erosion of the shoreline in areas that have been not protected by these two nourishments. While this beach erosion has not been severe when reviewing shoreline change rates since 1938 (See Figure 1.3), lowering of the beach over time has led to the need for revetment protection along the face of the exposed seawall sections.

The long-term effect of this beach lowering is to expose this shoreline to larger depth-limited waves due to deeper water depths fronting the seawall. During severe conditions, these larger waves can destabilize the seawall protecting the sewer line behind the seawall. Moreover, the Coastal Structures Inventory indicates that while the vertical concrete seawall backing the beach is in fairly good condition, the toe revetment that protects against seawall undermining is in poor condition. Due to the loss in beach width and condition of the shore protection, concerns have been raised by the City regarding critical infrastructure within West Rodney French Boulevard, specifically the sewer main.

4.2 Wave and Sediment Transport Modeling

In Applied Coastal 2017, wave and shoreline evolution modeling were conducted to determine general and storm-induced physical conditions in the Project Area. These models were used to develop the conceptual design and project an expected lifespan and maintenance schedule for the preferred alternative.

The sediment transport calculations 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 Waterways Experiment Station (WES). The WIS program has generated hindcast wave data for waves propagating from open ocean, through the use of computer simulations, for many sites along the U.S. coast.

For the shore along West Rodney French Boulevard, the direct open ocean exposure is prevented by its orientation in Clarks Cove and by the Elizabeth Islands that form the southeastern boundary of Buzzards Bay (Figure 4.1). It is still possible that some offshore wave energy can propagate to West Beach by refraction and diffraction of waves, which are processes that redirect waves. Because it was not initially known what the contribution offshore waves made to sediment transport at the study shoreline, the wave climate was estimated using a method that incorporated offshore waves and locally generated wind waves.

In this study, a three-part 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.

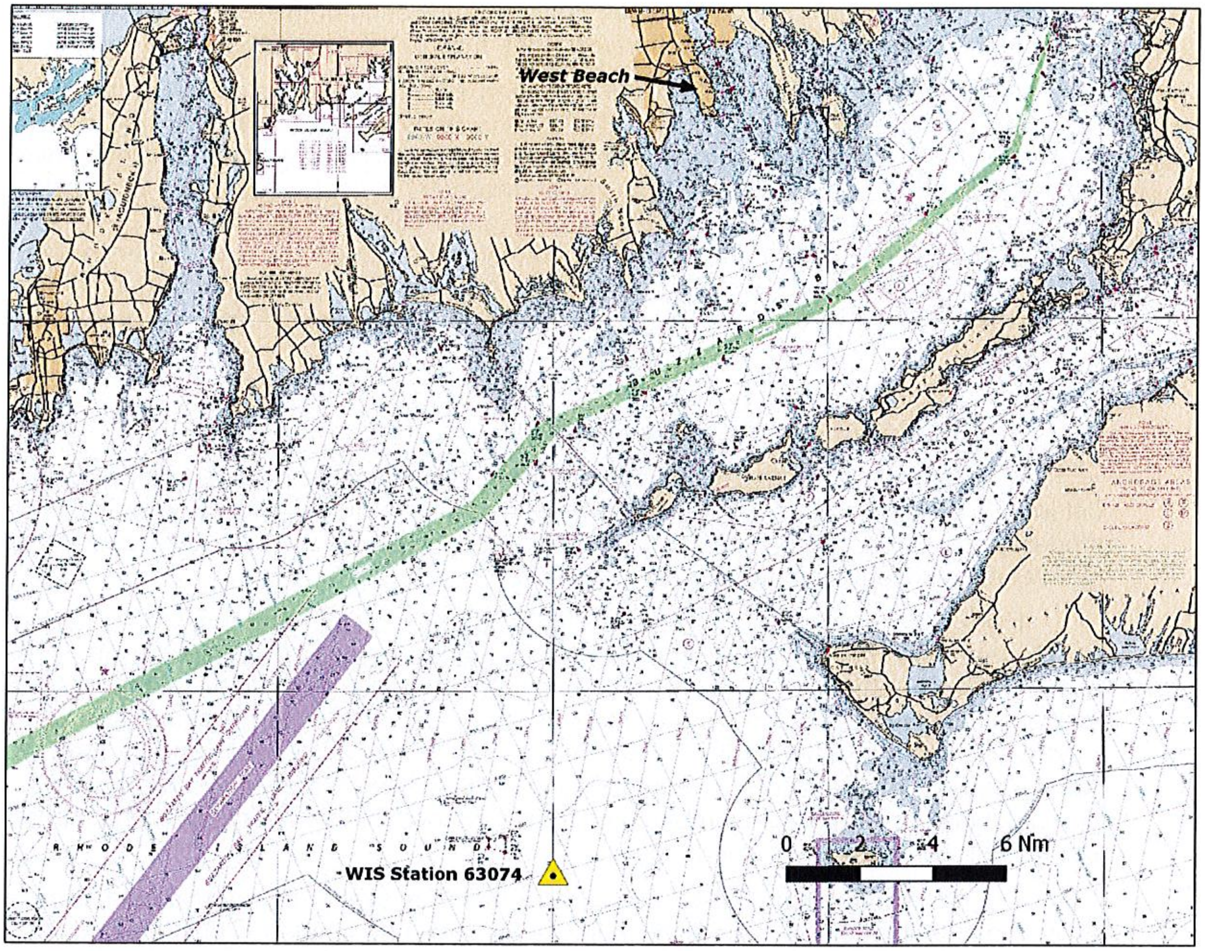


Figure 4.1 Detail of NOAA chart 13218 (Martha's Vineyard to Block Island) showing the locations of the WIS hindcast station (63074) and the West Beach study shoreline.

4.2.1 Wave and Wind Data

For this study, wave conditions were generated using the wind and wave data available from the US Army Corps of Engineers (USACE) WIS hindcast database, at station 63074 located 20.9 NM south of the mouth of Clarks Cove and 10 NM south of Cuttyhunk (Figure 4.1). The WIS data were used to develop offshore wave boundary conditions as well as the winds applied to the surface of Rhode Island Sound and Buzzards Bay. The WIS has a record that spans the 33-year period between January 1980 through December 2012.

The entire wave and wind records from the WIS hindcast are presented in Figure 4.2 and Figure 4.3, respectively, as compass rose plots which show magnitude and percent occurrence by compass sector. From the hindcast, winds most frequently blow from the SW, with a percent occurrence of 10.0%. For sectors approaching the West Beach shoreline (SSE through NNW) winds blow 63.6% of the time, with winds greater than 25 knots blowing 4.4% of the total 33-year span of the record. From all direction sectors, wind speeds are greater than 10 knots 65.0% of the record and greater than 25 knots for 6.7% of the record. The greatest wind speed of the entire record (64 knots) occurred during Hurricane Bob (August 1991).

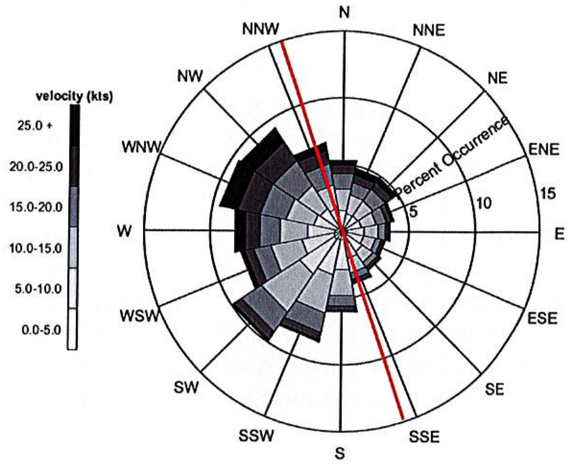


Figure 4.2 Wind rose of data from the WIS hindcast station 63074 (Rhode Island Sound), for the 33-year period between January 1980 and December 2012. Direction indicates from where wind was blowing. Grey tone segments indicate magnitude of wind speeds. Radial length of each segment indicates percent occurrence over the total duration of the data record. The red diametric line indicates the approximate orientation of the West Beach shoreline.

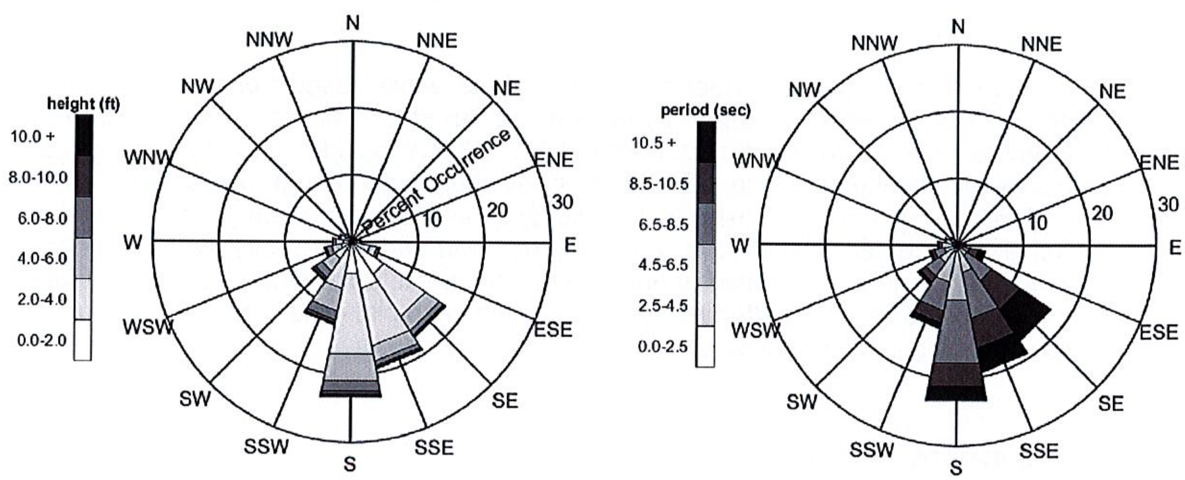


Figure 4.3 Wave height and period for hindcast data from WIS station 63074 (Rhode Island Sound) for the 33-year period between January 1980 and December 2012. Direction indicates from where waves were traveling, relative to true north. Radial length of gray tone segments indicates percent occurrence 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 record, the predominant sector is from due south. Waves propagate from this direction 23.6% of the time. The second-most frequently occurring sector at this station is SSE, which occurs 19.4% of the time. Most of the waves from the south sector (approximately 9% of the total record) have an amplitude between 2 and 4 feet. The 6.5 to 8.5 second wave period band from the south has the greatest occurrence (7% of the record) of all sectors.

To develop the wind input conditions for the wave model, the wind data from the WIS record were binned by 22.5-degree compass sector and by magnitude, as presented in Table 4.1. For each separate compass sector, the hourly events from the wind record were divided into top, middle, and bottom bins, based on wind speed. To determine which bin each wave case belonged, the maximum wind speed for each sector was found. The bin limits then were set at one-third and two-thirds of the maximum wind speed. For each separate sector, this binning method resulted in two more frequently occurring wind cases (bottom and middle bins) that represent more common conditions and one less frequently occurring bin (top bin) that represents rarer storm conditions. A total of 289,295 total hourly time steps of the WIS record were sorted in this fashion.

The WIS hindcast record also was used to determine the offshore wave input conditions. Each hourly WIS record includes parameters that describe the wave conditions (i.e., wave period, T_p; wave height, H_s; and direction, θ). Wave conditions for each wind case were determined by the wave data concurrent with the wind records. Average wave heights for each wind case were computed as the square root of the mean squared wave heights. Wave direction was determined as the vector average direction of all wave cases occurring with each particular wind case. This method of sorting the wave data determines the average wave conditions that correspond to each binned wind case.

Thirty-three separate model cases (i.e., three wave cases from each of eleven compass sectors) were developed by this processing of the wind and wave data of the WIS record. The 11 compass sectors from ESE to NNW include all winds that generate waves to drive sediment transport along the study shoreline. Though winds from the ESE and SE sectors do not blow onshore at the study shoreline, waves from these sectors can refract as they enter Clarks Cove, and may result in sediment movement along West Beach. The percent occurrence of each separate case is determined using the number of hourly records from the WIS hindcast that fall into each bin, divided by the total number of wave records in the entire 33-year record.

4.2.2 SWAN Model Development

As locally generated and offshore wave components propagate into shallower water near shore, the height of the shoaling waves will change, and they will gradually change direction to conform to the bathymetry in that area. In order to estimate how waves will change as they grow under the influence of winds blowing across the surface of Buzzards Bay and move toward West Beach, the two-dimensional wave transformation program SWAN was used. As discussed previously, wind data from the NOAA buoy and wave data from the WIS hindcast were used as boundary input to the runs of SWAN.

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 (grid increments in the x and y directions are equal), though other options are available (including a finite difference formulation using an unstructured mesh). An 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 wind and 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 (smaller refined grids with greater detail), the directional and frequency resolution of the wave spectrum, and wave physics (e.g., breaking, wave-wave interactions).

The SWAN model developed for West Beach used a coarse grid with 200-meter spacing for the region including the offshore area of Rhode Island Sound beyond Buzzards Bay and the Elizabeth Islands (Figure 4.4), and a nested fine-scale 2.5-meter grid that covers all of Clarks Cove (Figure 4.5). The National Ocean Service (NOS, 2017) was the main source of bathymetric data used to create the grids. A nearshore bathymetry survey performed by CLE and an upland survey by the New Bedford Department of Public Infrastructure (DPI) (both performed in Jan 2017) provided recent nearshore bathymetry and beach profile data for the fine grid along the West Beach shoreline. Additional high-resolution elevation data were available from a 2013 LiDAR flight of the area by the USGS which includes the upland of the study area. All elevation data were transformed to the NAVD88 datum.

Table 4.1. Wave model input parameters, listed by compass sector and wind velocity bin (i.e., bottom, middle and top thirds). Listed offshore wave parameters include compass direction θ_o, peak wave period T_o and wave height $H_{s,o}$. Angles are given in the meteorological convention (i.e., from where the wind blows, in compass degrees).

sector		percent occ.	wind angle (degrees)	wind speed (knots)	θ_o (degrees)	$H_{s,o}$ (feet)	T_o (seconds)
ESE	bot 1/3	3.1	112.5	10.1	148.3	3.2	8.2
	mid 1/3	0.3	111.5	25.4	133.4	7.6	7.4
	top 1/3	9.8×10^{-4}	109.0	51.6	156.3	24.9	13.8
SE	bot 1/3	2.6	135.3	8.1	153.9	2.7	8.3
	mid 1/3	0.9	135.6	18.3	147.1	5.1	7.1
	top 1/3	0.1	135.2	30.8	151.5	10.4	8.0
SSE	bot 1/3	3.3	158.0	8.7	158.0	2.8	8.2
	mid 1/3	0.7	157.9	20.1	160.9	6.1	6.8
	top 1/3	1.9×10^{-2}	159.2	34.2	168.0	14.2	9.3
South	bot 1/3	4.9	180.7	8.9	163.0	2.8	8.0
	mid 1/3	1.3	181.0	19.0	175.5	5.5	6.5
	top 1/3	0.0	179.3	33.2	177.7	14.0	9.2
SSW	bot 1/3	8.0	203.1	11.3	173.6	3.4	7.4
	mid 1/3	0.5	203.0	24.6	192.8	8.1	7.0
	top 1/3	3.3×10^{-4}	193.0	64.0	194.0	34.3	15.9
SW	bot 1/3	8.3	224.8	10.3	177.2	3.2	7.4
	mid 1/3	1.7	225.1	20.9	205.5	6.4	6.6
	top 1/3	6.2×10^{-3}	224.2	37.3	196.1	18.5	11.5
WSW	bot 1/3	5.1	247.1	8.8	175.4	3.0	7.7
	mid 1/3	2.6	247.2	18.7	210.3	5.8	7.0
	top 1/3	0.2	249.8	31.8	213.2	12.1	9.2
West	bot 1/3	4.3	269.8	8.8	178.4	3.1	7.9
	mid 1/3	3.5	271.1	19.5	218.5	5.8	7.5
	top 1/3	0.4	272.5	31.1	230.4	10.0	8.5
WNW	bot 1/3	4.4	292.4	9.7	183.4	3.1	8.0
	mid 1/3	4.7	292.9	21.3	236.6	5.6	7.3
	top 1/3	0.4	291.8	32.7	253.1	8.9	7.6
NW	bot 1/3	4.5	314.7	10.3	183.9	3.1	8.1
	mid 1/3	4.6	314.3	21.7	260.8	4.9	7.0
	top 1/3	0.2	314.4	35.0	285.8	8.1	6.8
NNW	bot 1/3	3.6	337.3	10.1	173.1	3.0	8.4
	mid 1/3	2.8	336.8	20.9	294.3	4.3	7.2
	top 1/3	0.1	336.8	35.3	334.7	7.9	7.7

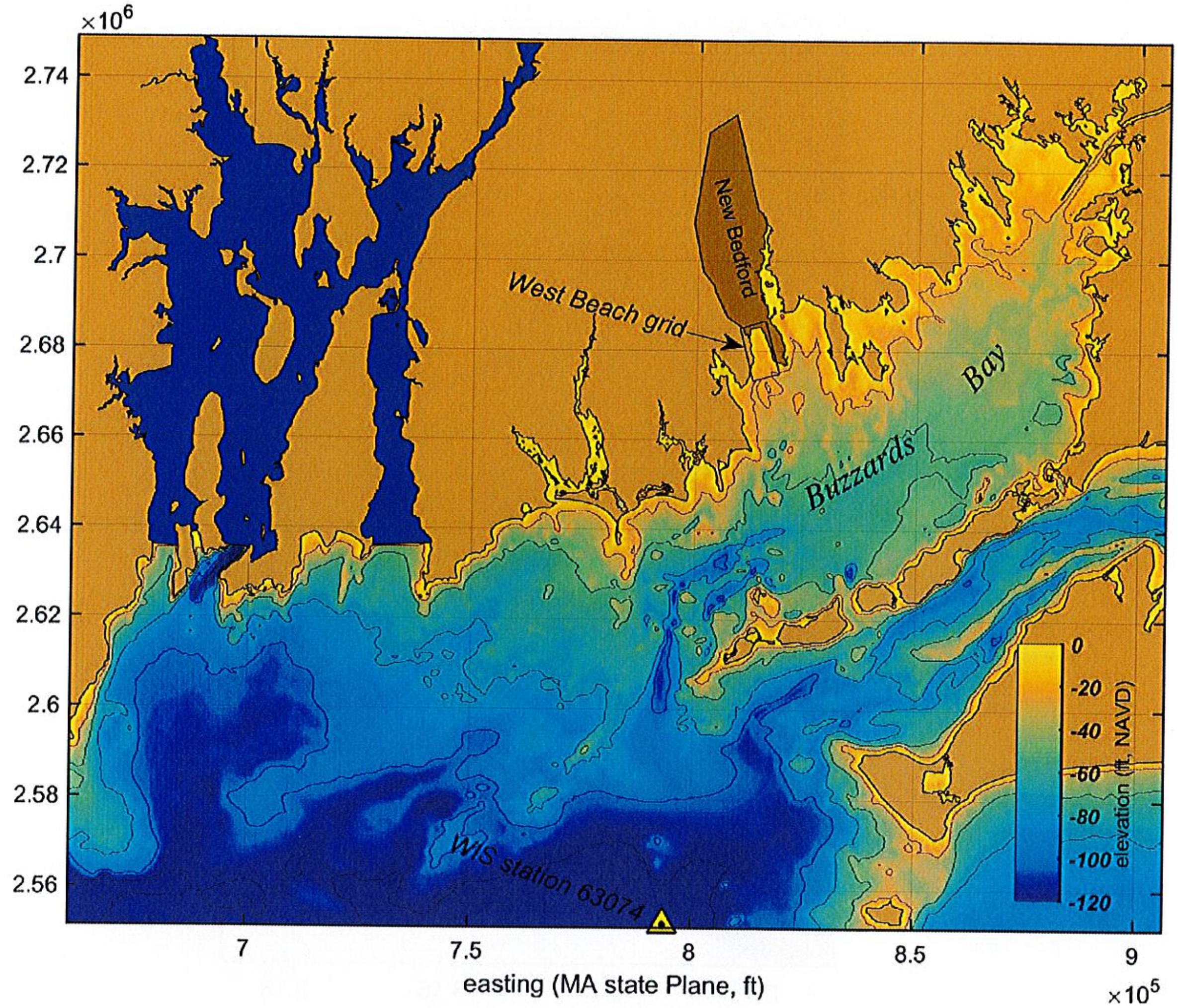


Figure 4.4 Map showing wave model grid limits and bathymetry, for the coarse model grid of Buzzards Bay, and the limits of the fine-scale model grid for Clarks Cove and West Beach. Contour lines are also provided at 20-foot intervals.

The coarse grid was used to propagate offshore waves developed from the analysis of the WIS hindcast record, and also generate wind-waves within Buzzards Bay. The nested fine-scale mesh serves to provide highly-detailed wave information at the shoreline of West Beach, which were used as input conditions for the shoreline change model of the study shoreline. As executed, spatially varying model output from the coarse grid (at points that correspond to nodes along the fine grid open boundary) is used as the boundary condition for the fine scale grid model runs, therefore the refined grid results are truly nested within the coarse grid simulations.

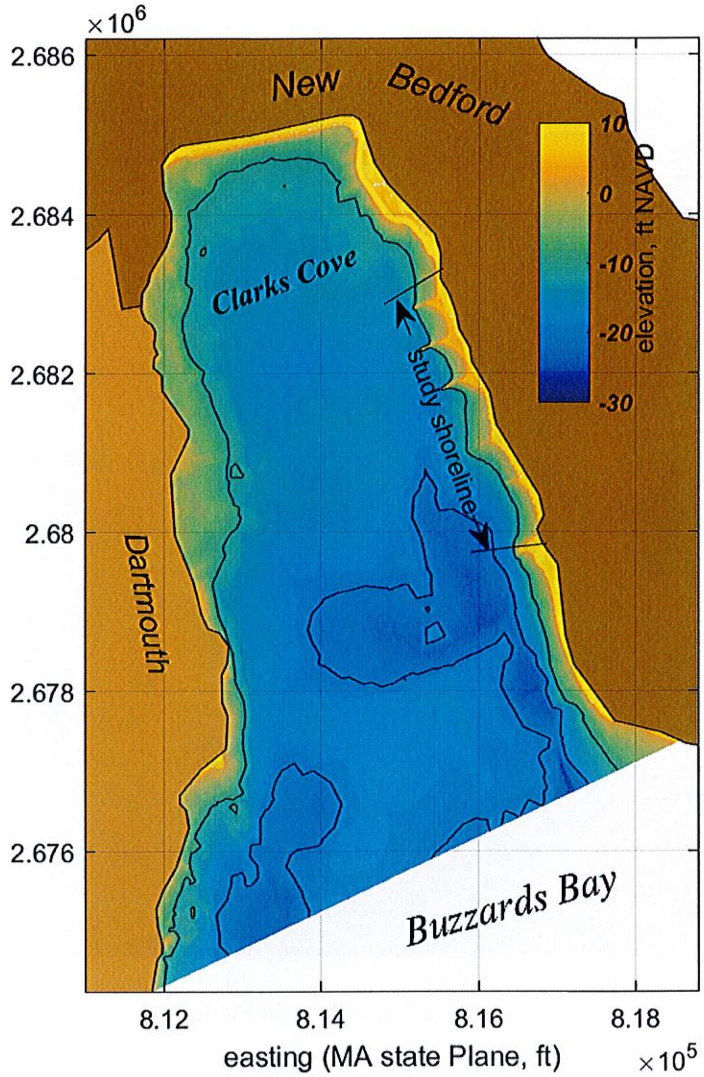


Figure 4.5 Map of the fine-scale 2.5-meter model grid of Clarks Cove. Contour lines are shown at 10-foot intervals.

The coarse grid is made up of 112,875 computational cells with a spacing of 656 feet (200 meters). The x-axis of the grid is 40.5 nautical miles (75.0 km) or 375 cells wide. The y-axis of the grid is 32.5 nautical miles (60.2 km) or 301 cells long. The y-axis is oriented due north. The greatest depth in the coarse grid domain is -156 feet NAVD (-48 meters).

The fine-scale 2.5-meter grid is made up of 1,074,856 total cells. The x-axis is oriented along the West Beach shoreline of Clarks Cove, and has a total length of 1.6 nautical miles (3.0 km). The y-axis is oriented along the compass heading of 245 degrees. The y-axis has a total cross-shore length of 1.2 nautical miles (2.3 km). The maximum depth (26.3 feet NAVD) of the fine grid occurs along the southeastern edge of the grid, at the Cove's mouth to Buzzards Bay.

The wave spectrum resolution specified for the model runs both coarse and fine model meshes included 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).

Examples of wave model output are presented in Figure 4.6 and Figure 4.7, from the coarse and fine grid runs of the top South case (Table 4.1). In these plots the color contours indicate wave height and vectors are used to indicate the direction of wave propagation.

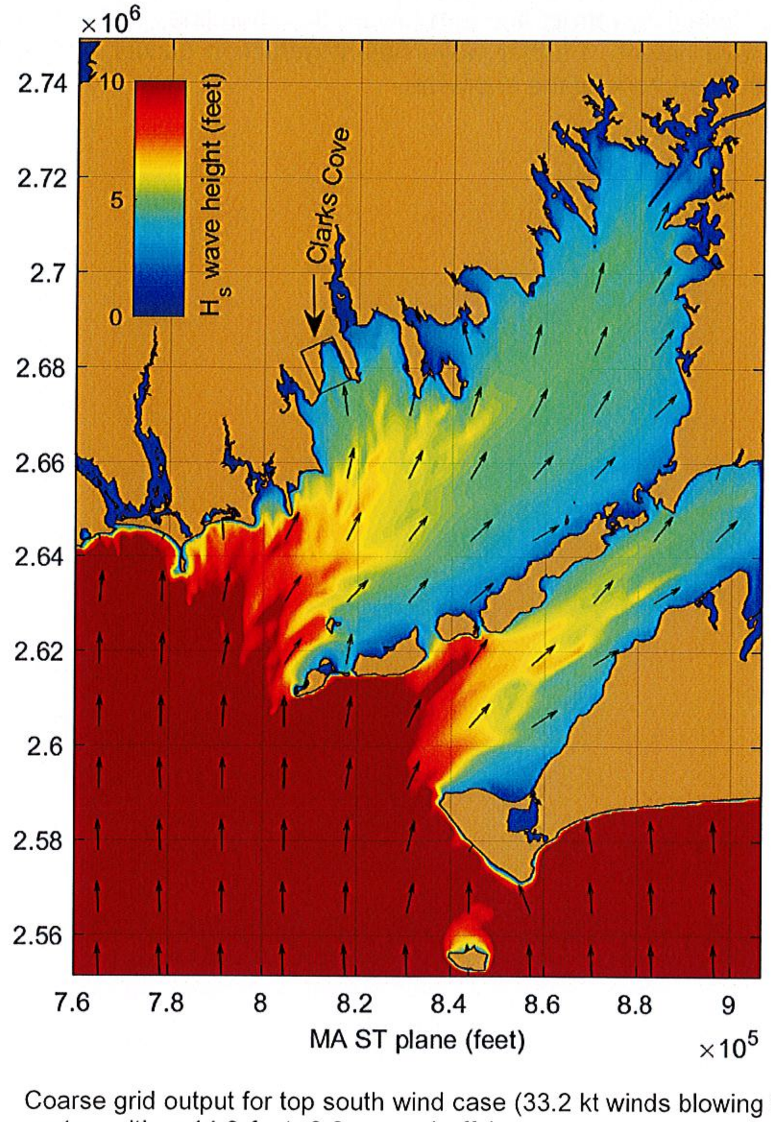


Figure 4.6 Coarse grid output for top south wind case (33.2 kt winds blowing from the South sector, with a 14.0 foot, 9.0 second offshore wave approaching from the South

sector). Color contours indicate wave heights and vectors show peak wave direction.

In Figure 4.6, offshore waves with heights of 14.0 feet approach the entrance to Buzzards Bay in the course grid. The sheltering effect of the Elizabeth Islands along the southeastern boundary of the Buzzards Bay is evident in this plot. Wave heights at shoreline areas that are more exposed, such as Horseneck Beach in Westport and at Cuttyhunk, experience wave heights that are greater than 12 feet. At the entrance to Clark Cove, even with a wind of 33 knots, waves are less than half the offshore wave height due to the sheltering provided by the mainland and Elizabeth Islands.

Results plotted for the fine-scale grid of Clarks Cove (Figure 4.7) show that waves entering the cove area are oriented along its long axis. Wave heights of about 4 feet occur over most of the surface of the Cove, but as waves enter the shallower water along the perimeter of the Cove they refract and turn toward the shoreline. Refraction also causes a reduction in wave height. Further wave height reduction occurs by breaking as the waves roll in to the surf zone at the shoreline.

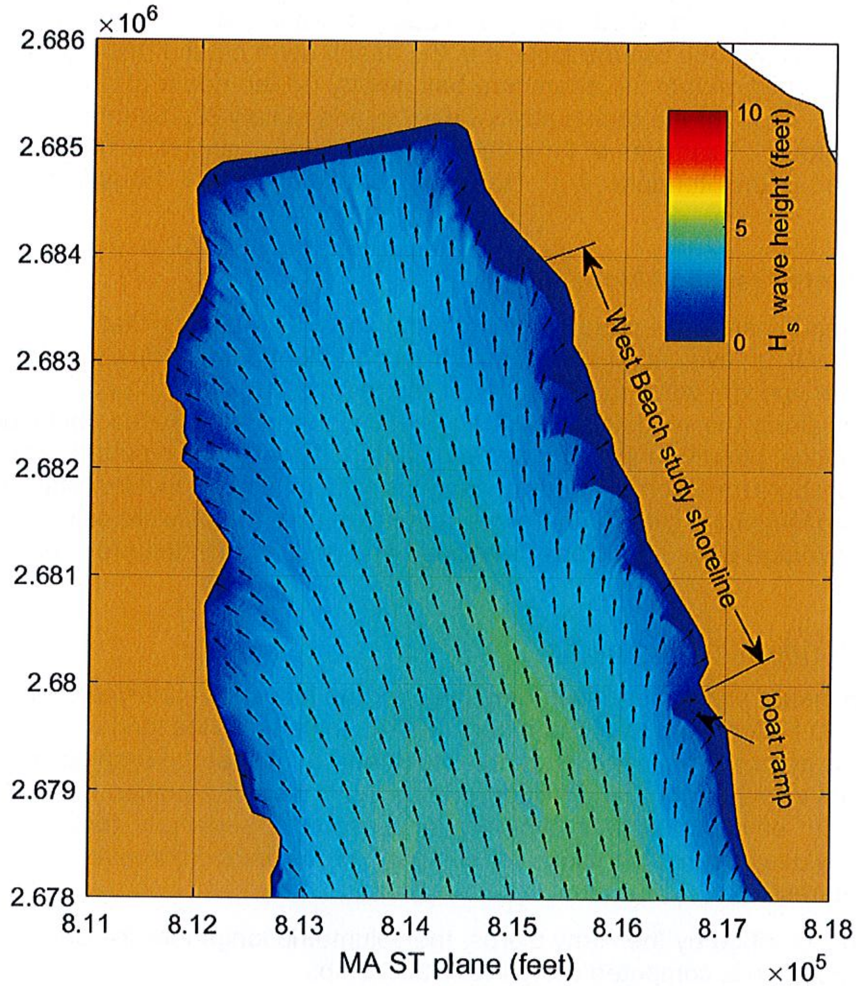


Figure 4.7 Nested fine-scale wave model output for the top South wind case (Table 4.1). Color contours indicate wave heights and vectors show peak wave direction.

4.2.3 Shoreline Evolution Modeling

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 coastal processes along the West Beach shorelines falls in the middle of this technical range. While simplified analytical models ignore many of the important principles governing shoreline change, 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 this analysis.

Shoreline evolution modeling at West Beach 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 beach berm crest or 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. Examples of formulations of this type of shoreline model are very well documented in the literature (e.g., Dean and Dalrymple, 1991; Hansen and Kraus 1989).

4.2.3.1 Sediment Transport Modeling

As an integral part to the understanding of the coastal processes that are at work to shape West Beach, an evaluation of sediment transport along the shoreline is necessary. Results from the spectral wave modeling effort formed the basis for computed sediment transport rates along the modeled beach segment 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 developed and described in the previous section.

Formulation Of Transport Calculations

The sediment transport equation employed for the longshore analyses is based on the work of the U.S. Army Corps of Engineers (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.

For this study, immersed weight longshore sediment transport, I, was computed using a method based on the so-called "CERC formula",

$$I = KP_{ls}$$

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

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

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. A value of K=0.39 is designated for use with significant wave heights (as output from SWAN).

The actual method used to compute immersed weight longshore sediment transport for this study 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

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

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

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

where the coefficient K^* = 0.0013. The value of transport potential derived using this method 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 empirical expressions of sediment transport potential, a computer code was developed which computed sediment transport potential along the West Beach shoreline. 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. For this application, transport potential calculations were performed using a 8.2 foot (2.5 meter) grid spacing, which corresponds to the grid spacing of the fine wave grid. The January 2017 shoreline, determined by the RTK-GPS survey, was used as the input shoreline. The modeled shore segment is approximately 3,600 feet long, and includes the shoreline between the West Rodney French Boulevard boat ramp to the south and the hurricane barrier at the Kilburn Mills to the north.

Inputs into the sediment transport potential calculations include beach slope and sediment grain size. A 0.46 mm representative grain size was determined based on mid-tide sediment samples collected at the beach (Sediment sampling details provided in Section 4.5). Beach slope was set to 0.04 (1:25 v:h) for the sandy portions of the shoreline based on the profile data available from the 2017 CLE and New Bedford DPI surveys.

Present Sediment Transport Rates

The computed net average annual sediment transport potential for present conditions is mapped in Figure 4.8 as vectors. The results of the transport potential calculations indicate that the modeled West Beach shoreline segment acts as a single littoral cell with transport directed north. Net transport rates peak at about 8,000 cubic yards (cu. yds) per year (Figure 4.9) at the Oaklawn Street groin, about 1,370 feet north of the boat ramp. Potential rates then decrease moving north to the Hazelwood Park groin. Across the sandy beach between the Hazelwood Park and Dudley Street groins, transport potential rates are low (around 500 cu. yds/year), an indication that this sub-segment of West Beach is well equilibrated to the presence of the groins. Though the net transport is to the north, there is a southerly component along the whole shoreline. The southerly component is largest (about 1,000 cu. yds/year) at the southern end of the modeled shoreline segment and smallest at the hurricane barrier. The only area of net southerly transport is at the boat ramp, where net rates are less than 1,000 cu. yds/year to the south.

Figure 4.8 Computed average net sediment transport potential along the Sandy Neck shoreline. Arrows indicate the direction of transport, while the color and size of the arrows corresponds to transport magnitude.

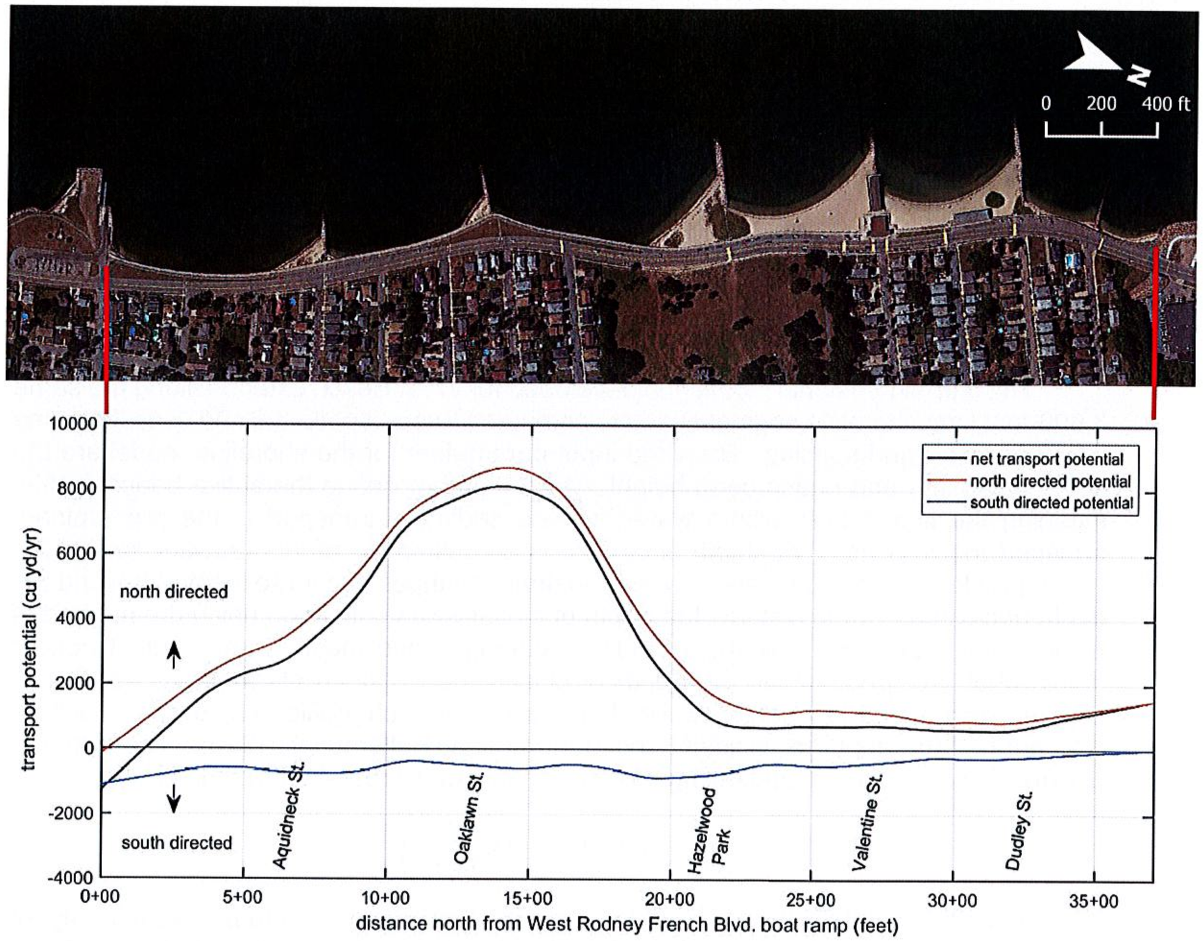


Figure 4.9 Annualized average sediment transport potential (positive north-directed; negative south-directed) computed for the shoreline of West Beach. The net transport (solid black line) is the resultant of the north-directed (red line) and the south-directed (blue line) components of transport. The August 2016 aerial photo provided for reference is via Mass GIS and Google Images (©2017).

4.2.4 Shoreline Model Development

Using this expression 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 longshore sediment transport rates, and subsequently use the model to evaluate beach management alternatives for West Beach.

The model code incorporates the ability to simulate the effects of seawalls (and coastal dikes) and groins on shoreline evolution. 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 term, 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 in the model simulation.

The one-dimensional model grid developed for West Beach extends along the same 3,600-foot long shoreline segment used to compute transport potential and uses the same 8.2 (2.5 meter) grid spacing. Required input parameters for the shoreline model are the depth of closure and beach berm height, which together define the active beach profile, meaning the littoral area where wave induced sediment transport is the predominant transport mechanism. The depth of closure is an estimation of the seaward limit of the beach profile. By definition, areas where no depth changes occur are located beyond the depth of closure. For this study, the depth of closure was estimated using the method of Hallermeier (Dean and Dalrymple, 2001). Although sand motion can occur at bottom depths that are greater than the depth of closure (e.g., during storms), the net flux of sediment is not great enough to cause changes in the beach profile. The depth of closure is about half the depth for incipient sediment motion (Hallermeier, 1978). The depth of closure (h_c) can be computed using the relationship developed by Birkemeier (1985),

$$h_c = 1.75H_e - 57.9\left(\frac{H_e^2}{gT_e^2}\right)$$

which uses the significant wave height and period that is expected to be exceed only for 12 hours each year, H_e and T_e. A useful approximation to this is given by $h_c = 1.57H_e$, where H_e is computed as $H_e = \overline{H} + 5.6\sigma_H$, and $\overline{H}$ and $\sigma_H$are the mean wave height and standard deviation of the wave record, respectively. Using a 33-year wave hindcast from WIS station 63074, H_e is computed to be 4.5 feet (1.4 meters), which results in a depth of closure of 10.3 feet (3.1 meters). Therefore, the depth of incipient sand motion is 20.6 feet (6.2 meters).

Similar to the computation of sediment transport potential, output from the wave modeling analysis is used to drive the shoreline evolution model. A time series of wave conditions was created using the Rhode Island Sound WIS hindcast so that the 33 wave cases (Table 4.1) representing mean annual conditions occurring from different compass sectors could be used in a time dependent simulation of shoreline movements. At each model time step (15 minutes) during the course of the seven-year model calibration period, a wave case from the 33 modeled cases was selected based on each separate wave record from the WIS hindcast. For hourly periods where waves were not propagating onshore from any of the ten compass sectors of Table 4.1, no waves were applied to the model shoreline for that time step.

Coastal engineering structures along the modeled shoreline segment are included in this model. Six groins are included (e.g., Figure 4.10), as is the seawall along Rodney French Boulevard (Figure 4.11). The groins act to impound sand, and are included in the model by introducing a permeability factor that reduces the transport rate across the grid cell where each groin exists. Permeability ranges between 0.0 and 1.0, where 0.0 would be a completely impermeable block to transport and 1.0 represents a structure that has no sand holding capacity (e.g., completely unraveled or filled to bypassing). For the groins along West Beach, the permeability factor was set at 0 to represent the solid condition and large size of the groins, which effectively hold the available sediment. If at any point during the simulation the shoreline accretes past the tip of the groin, the permeability is set to 1 and sand is allowed to move across the structure uninhibited.

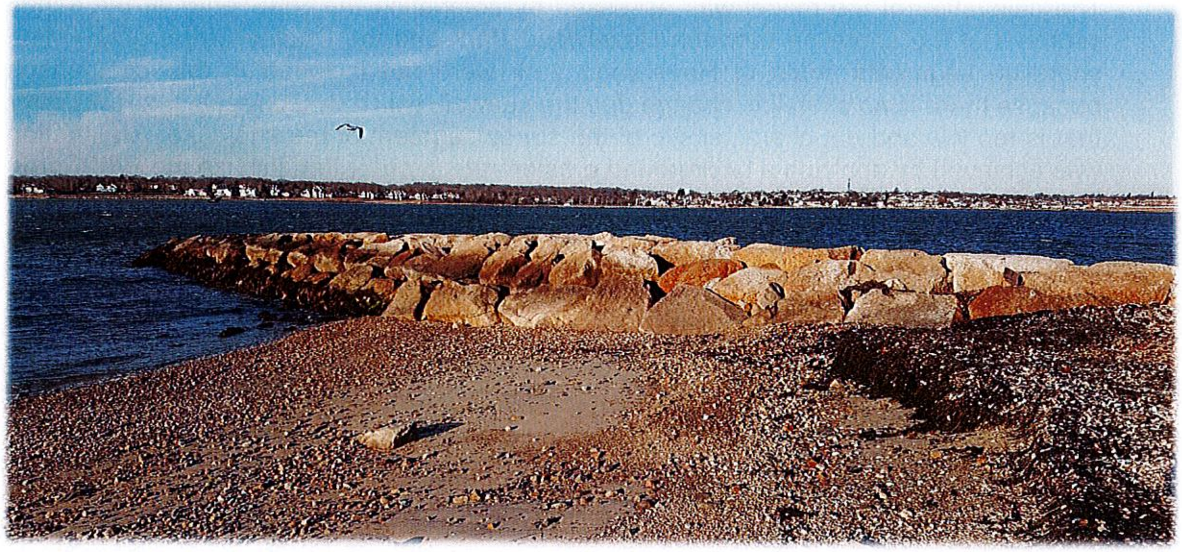


Figure 4.10 View of the Oaklawn Street groin, looking northwest (Jan 2017).

Figure 4.11 Photo a section of the seawall along West Rodney French Boulevard, viewed from the boat ramp, looking north (Jan 2017).

The seawall acts to limit the shoreward movement of the shoreline as it moves during the course of the simulation. If the shoreline at any grid cell erodes to the point where it comes into contact with the seawall, the shoreline is not allowed to move farther shoreward. Unlimited accretion is allowed in front of the seawall.

Model performance was calibrated by running the model between 1997 and 2009, which is a time period bookended by aerial orthophotographs available through MassGIS. The model input shoreline was digitized from the 1997 aerial set. The model was run for 12 years using the wave cases indicated by the WIS record, which has complete coverage of this time period. The computed shoreline at the end of the 12-year simulation was compared to the shoreline digitized using the 2009 aerial orthophoto set (Figure 4.12). The calibration of the model was assessed by computing the RMS error of the sandy segment of the shoreline between Hazelwood Park and the Dudley Street groin. The shoreline segments south of Hazelwood park were not included in this comparison because there is no shoreline change due the seawall, which would result in a RMS error that is too low and not characteristic of the sandy segment of the shoreline. Calibration was achieved by applying a background erosion rate in order to minimize the RMS error. The error of the final calibration run was 12.9 feet, which is comparable to the uncertainty associated with the aerial photo analysis (14.1 feet). The erosion rate applied to the calibrated model was 1.7 feet/year. Results are shown in Figure 4.12.

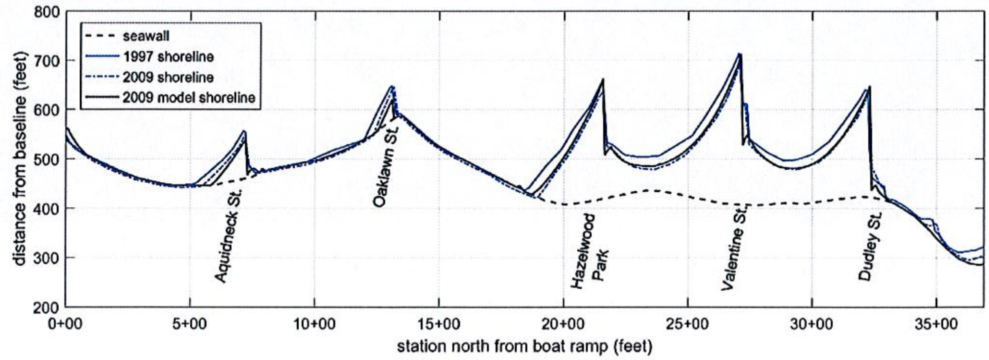


Figure 4.12 Comparison on modeled and measured shorelines for the shoreline model calibration period between 1997 and 2009. The calculated RMS error for the sandy segment of the shoreline (between stations 18+00 and 33+00) is 12.9 feet, and the R^2 correlation coefficient is 0.96. The groin positions are indicated by reference from Figure 4.9.

4.3 Sewer Main

Behind the seawall, the sewer line runs to the wastewater treatment for the city of New Bedford (Figure 4.13). The sewer line consists of a 7' x 7'8" concrete pipe that runs parallel to West Rodney French Boulevard. Approximately 98% of the population of New Bedford (93,100) are connected to the sewer line. The number of housing units serviced by the sewer system is estimated to be 38,800 homes based on an average of 2.4 persons per housing unit (CDM, 1989). The treatment plant receives an average flow of 21.3 million gallons per day (mgd) and a maximum daily flow (averaged monthly) of 50 mgd ("Pumping Stations - Public Infrastructure - City of New Bedford Official Website," n.d.). Both stormwater and sewage are treated by the plant. During large rainfall events, the sewer main is designed to release the combined stormwater and sewage into Clarks Cove and New Bedford Harbor by means of a CSO system as shown in Figure 4.14. The release of untreated combined sewage historically has threatened habitat and marine life. More specifically, eelgrass population and shellfish populations are starting to improve in Clarks Cove with an effort to increase treatment capabilities and a reduction in CSOs. However, these events still happen, and the potential for large events to release untreated sewage exist.

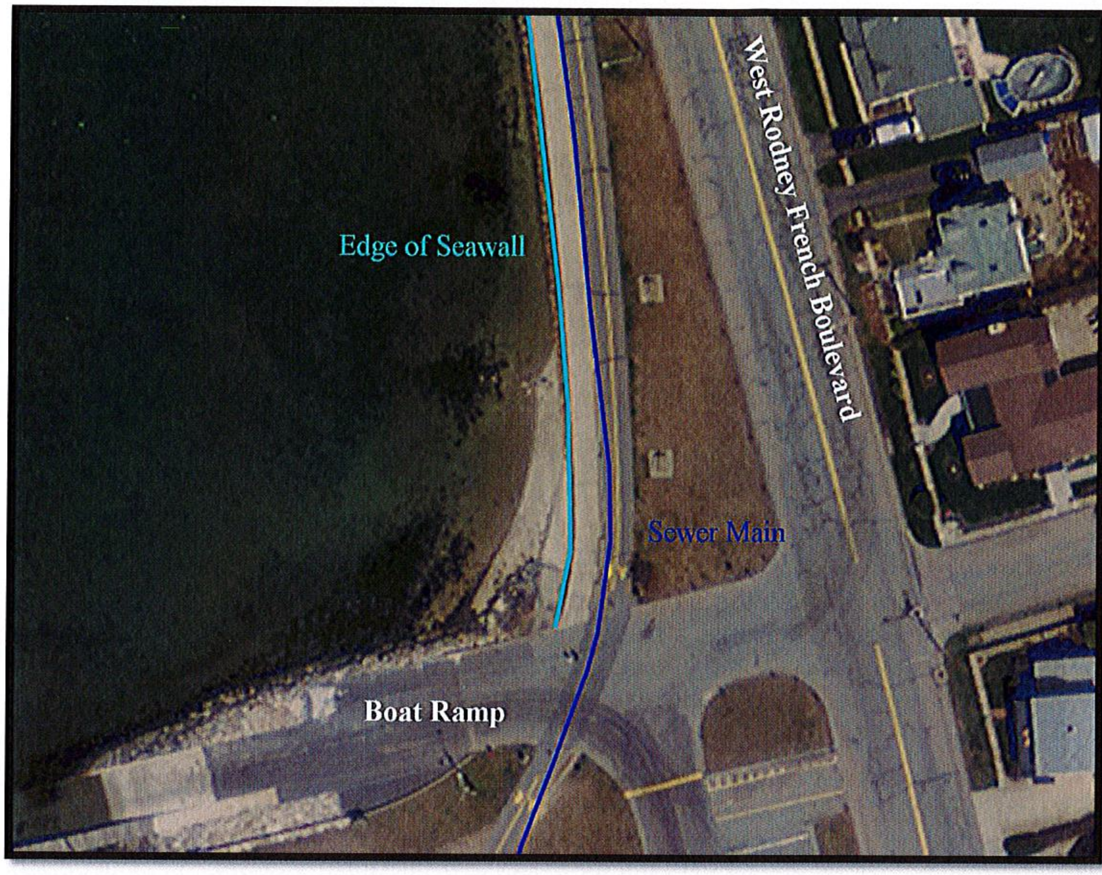


Figure 4.13 Aerial image of the approximate separation of the sewer main and the seawall. Just north of the boat ramp in the first groin compartment, the sewer main centerline (navy blue) is within 10 ft of the seaward edge of the seawall. This section of the seawall is exposed to the ocean during high tides.

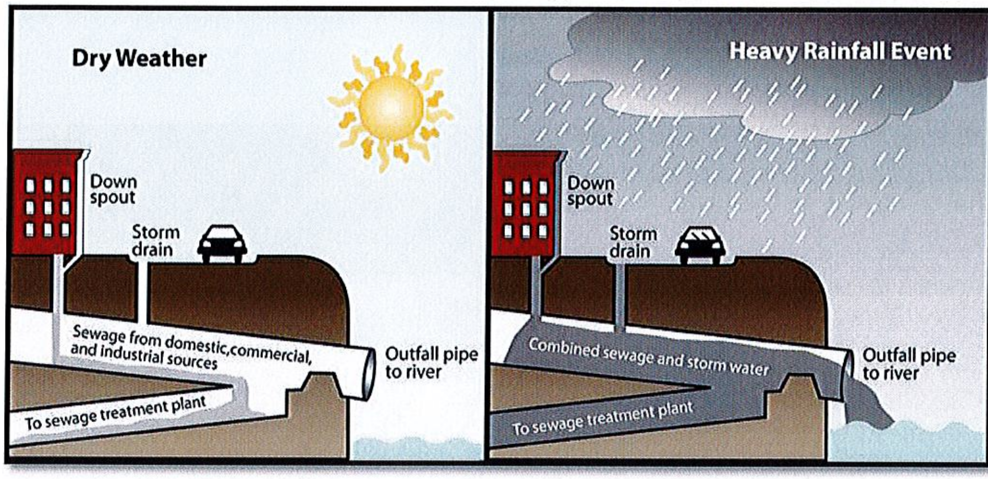


Figure 4.14 Example of a combined sewer system in dry and wet weather conditions (Photo: Akron Waterways).

4.4 Assessment of Proposed T-Head Groins and "Structure Trading"

The final proposed plan for West Beach is the result of an iterative engineering design process which evaluated several options based on defined project goals. The project needed to enhance the storm resiliency of the main city sewer line along West Rodney French Boulevard while also being protective of sensitive eelgrass habitat that exists in close proximity to the project shoreline. Through the course of the design process, the best features of particular designs were carried forward to later iterations, while design features that did not achieve the project goals were modified or replaced.

The critical need to protect existing infrastructure (the city's main sewer line) is balanced with the need to protect existing eelgrass adjacent to the project, resulting in a unique situation that requires an innovative approach. The existence of eelgrass resources in very close proximity to the shoreline was identified as the main challenge to any design used to improve storm resiliency along West Rodney French Boulevard.

A beach fill with T-head groins was selected as the best option in the previous 2016-2017 West Beach CZM grant study. The eelgrass proximity to the shoreline presents a challenge to accomplish both protection of the sewer main and the habitat, both of which had significant influence on the design of the project. The T-head structures are necessary to increase the engineering design life of the beach fill while preventing the fill from infilling nearshore areas of eelgrass. From the results of the previous study, without the T-head groins, it is impossible to construct a beach fill that has a reasonable design life (greater than about 5 years) without infilling eelgrass areas. Even the addition of a toe berm to manage the infilling of eelgrass beds with sand at the time of construction does not address infilling caused by the movement of sand out of the original template area due to alongshore transport. The T-head structures help to hold the beach fill in place by minimizing both the alongshore and cross-shore movement of sand, thus achieving both project goals.

40- and 30-foot-wide (based on the position of the MHW elevation contour) variations of T-head project were initially developed. The 30-foot-wide version became the selected option since it met the project goals with the smallest cumulative structural footprint. As originally estimated, the footprint of this option was within the allowable footprint that is available from existing structures along West Beach that could be either shortened or removed altogether. The structure trading allowed for nearly no net construction of hard structures within the project site.

Further optimization of the T-head layout was performed for the more recent 2017-2018 CZM West Beach grant. It was designed for a minimum width of 30 feet at the berm crest elevation of +3.5 feet NAVD to minimize overtopping of the beach during more frequently occurring wave conditions. In addition, the construction template would initially be constructed with a berm crest of +4.5 feet NAVD. This construction fill template would be gradually worked over by waves to develop into a crenulate beach due to the wave blocking influence of the T-head breakwaters. The volume of sand needed to construct the berm initially is estimated to be 31,150 cubic yards, constructed on a 1:6 (v:h) slope, which results in an initial berm width of slightly more than 50 feet at +4.5 feet NAVD, and a fill toe-of-slope that is within the line of breakwaters. The resulting footprint of the new T-head breakwater sections was determined to be 24,271 square feet, which is within the footprint available from existing structure trading (24,270 square feet) .

Typically, the design life of a beach nourishment is determined as the point in time when the volume of sand remaining in the original fill template drops below 30% of the original construction volume. Design life for this project is defined differently since the goal is not just to maintain a certain volume of sand along the project shoreline, but rather maintain a minimum beach width in order to protect the existing seawall and critical upland infrastructure behind it. In this case, the design life was determined as the length of time when the minimum berm width is more than 30% of the original equilibrated width. Using erosion rates from the existing sandy portion of West Beach, the design life for this optimized T-head design is estimated to be between 9 and 12 years, using erosion rates that range between 0.54 and 0.72 feet per year.

The various conceptual designs evaluated for the West Rodney French Boulevard considered the public shoreline north of the existing boat ramp as a contiguous littoral system that required a regional sediment management and shore protection approach. With this consideration in mind, the evaluation considered the net overall potential environmental impacts associated with the shore protection alternatives. Specifically, modification and/or reconfiguration of coastal engineering structures was considered a viable alternative, as long as it could be demonstrated that this approach provided improved shore protection along the shoreline without increasing overall environmental impacts. To the extent possible, the design sought to improve the quality of the nearshore environment. Presently, much of the intertidal and beach area is covered with stones, brick, and concrete from previous armoring efforts for the shoreline. In addition, a series of cast iron outfall pipes (CSOs) exist along the shoreline (Figure 4.15).

Figure 4.15 Example of existing cast iron CSO pipe that projects from the West Rodney French Boulevard seawall.

To provide a holistic approach to region-wide shore protection, existing environmental regulations were evaluated within the context of the overall project's urban shoreline. The project shoreline is fronted by a vertical concrete seawall, as well as a series of stone groins. "Pocket beaches" have been established within some of the groin cells trough pro-active beach nourishment activities in the 1970s. The entirety of coastal armoring along the West Rodney French Boulevard shoreline was viewed as a single project from the aspect of evaluating potential environmental impacts. From an environmental regulatory perspective, the Massachusetts Wetlands Protection Act (M.G.L. c. 131, § 40) identifies eight "public interest" functions that wetland areas provide and performance standards to protect these functions. Any activity that will potentially affect a wetland area is to be regulated in order to contribute to the following interests:

1. Protection of public and private water supply
2. Protection of groundwater supply
3. Flood control
4. Storm damage prevention
5. Prevention of pollution
6. Protection of land containing shellfish
7. Protection of fisheries, and
8. Protection of wildlife habitat

In accordance with the Massachusetts Wetlands Protection Act (WPA), review is required for any activity that will remove, fill, dredge, or alter any wetland resource area, with "alter" being defined to include (among other things) the changing of drainage characteristics, flow and/or sedimentation patterns, or flood retention areas, and/or the destruction of vegetation. The WPA regulations contain extensive damage prevention standards that are organized according to: (1) the type(s) of coastal wetland resource area in which a project is located; and (2) the statutory interests that are declared (or presumed) to be significant within each area (i.e., storm damage prevention, flood control, or protection of wildlife habitat and/or marine fisheries). The regulations also identify the characteristics of the respective resource areas that, if changed by a proposed project, may result in adverse effects on interests protected by the wetlands statute. For the proposed work associated with the West Rodney French Boulevard shore protection project, the project requires that best available measures be used to minimize adverse effects. "Best available measures" mean the most up-to-date technology or the best designs, measures, or engineering practices that are commercially available. In general, non-structural alternative approaches to coastal hazards reduction are preferred over structural alternatives. Structural flood and erosion control alternatives should not interfere with the ability of a coastal landform to erode (providing material to adjacent beaches, dunes, and nearshore areas) and respond to wind, tide, and wave activity, if these landforms contribute to storm damage prevention or reduction and/or flood control. Beaches and dunes must also be allowed to naturally (re)build and migrate and/or grow landward, seaward, and laterally.

At present, the urban shoreline along West Rodney French Boulevard does not allow natural migration of beach or nearshore sediments to migrate, due to the existence of shore-perpendicular groins. As described above, the entire shoreline is fronted by a vertical concrete seawall that protects the sewer main; therefore, long-term stability of this structure is critical to the resiliency of critical city infrastructure. Development of appropriate shore protection along this shoreline region involved utilizing best available measures to create an overall project that minimized adverse effects to resource areas. To the extent practicable, the project involved using non-structural alternatives (beach nourishment) to provide protection to the seawall foundation. However, to maintain project longevity and prevent migration of nourishment sand into nearshore eelgrass beds, it was necessary to incorporate a series of T-head groins into the project. As designed, the T-head structures work in a similar manner to the series of existing stone groins along the project shoreline; therefore, the new structures would not create additional barriers to alongshore sediment movement. In addition, the series of T-head structures would be filled to entrapment capacity with beach-compatible sand and this beach planform would be maintained to ensure long-term shore protection of the seawall and associated critical infrastructure. As required by regulatory standards, the T-head structures are designed to be the minimum length and height to maintain the beach profile required for long-term shore protection at this site.

The repurposing of existing structure footprint (or "structure trading") is proposed as a way that the nine new T-head groin breakwater sections of the proposed West Beach project (Figure 4.16) will be constructed with no net increase in area permanently occupied by coastal engineering structures along West Rodney French Boulevard. In this manner, the overall impact of the proposed structures would by offset by removal of unneeded portions of existing structures. As proposed, portions of the Hazelwood Park and Valentine Street groins and the entire Woodlawn Street groin would be removed to the approximate elevation of the adjacent ocean bottom. It is anticipated that structure removal will leave the base layer of armor stone (i.e. boulders) along the seafloor that will provide enhanced fisheries habitat within the area of structure removal. The footprint area removed from these structures (as indicated in Figure 4.17 through Figure 4.19) is roughly equivalent to the area occupied by the new, more effective, breakwater sections.

In addition to the footprint removed from existing structures, groins at Oaklawn and Aquidneck Streets would be reshaped by turning their outer-most 50 foot to the north so that the ends are configured in an L-shape. Repurposing the groin tips in this fashion allows these structures to be better integrated into the proposed project, and to hold the sand nourishment more effectively.

Footprint relocation. The sections of the Hazelwood Park and Valentine Street groins that are proposed to be removed were added in the late 1970's as part of a maintenance project that placed sand in the two groin compartments at the public bath house at Valentine Street and placed a smaller tapered fillet of sand against the south side of the Hazelwood park groin. The groin extensions added 17,664 square feet to the footprint of these structures. Based on analysis and observations of the present condition of the beach within the two groin compartments at the bath house, the extensions have a negligible influence on the width/stability of the beach, which indicates that the groins are longer than they need to be to effectively hold sand in place along the shoreline. The modified length of the groins would be equivalent to the existing length of the Dudley Street groin, north of the bath house.

Along with the circa 1980 groin extensions, the complete length of the Woodlawn Street groin would be removed. This last groin at the northern end of west beach is in an area with a narrow to non-existent high-tide beach. The sediment-starved condition of the compartments on either side of this groin is primarily the cause of the updrift Valentine Street groin, which is effectively a complete block to sand transport to this area and allows minimal sand to travel north. Removal of this groin would have negligible impacts on the condition of this shoreline segment between the Valentine Street groin and the hurricane barrier. The footprint of this groin is 6,585 square feet.

For the groin removal sections, stones would be removed to the level of the adjacent ocean bottom. The remaining embedded material would be left in place as rocky/boulder habitat. It is anticipated that this creation of fisheries habitat would mitigate for any concerns potentially related to placement of beach nourishment over the anthropogenically derived material (concrete, cobbles, and brick) along the existing beach area.

Stone removed from the existing structures that meets the specifications of the proposed T-head breakwater sections would be recycled and utilized in the new structures.

Repositioned groins. The footprint area of reshaped groins at Aquidneck and Oaklawn Street will be reduced by a small amount, as the groin tips will be repositioned into shallower water and; therefore, their bases would not be as wide as they presently are constructed, given the same crest elevation and side slopes. Like the removed groin sections north of the nourishment area, for the portion of the groins tips that are repositioned, material will be removed to the level of the surrounding ocean bottom. Any remaining material that is embedded below that level will be left as rocky substrate. This is intended to mitigate for any potential loss of man-derived coarse cobble- and boulder-sized material that may presently serve as nearshore fisheries habitat. The existing man-derived material in the intertidal and nearshore areas will be covered by the planned beach nourishment. The footprint of the relocated groin tips together is 2,480 square feet. The combined footprint of all structures planned for removal is 27,729 square feet

Figure 4.16 Aerial photo of the existing structure footprint (shaded in red) to be traded for the new T-head structures (shaded in blue).

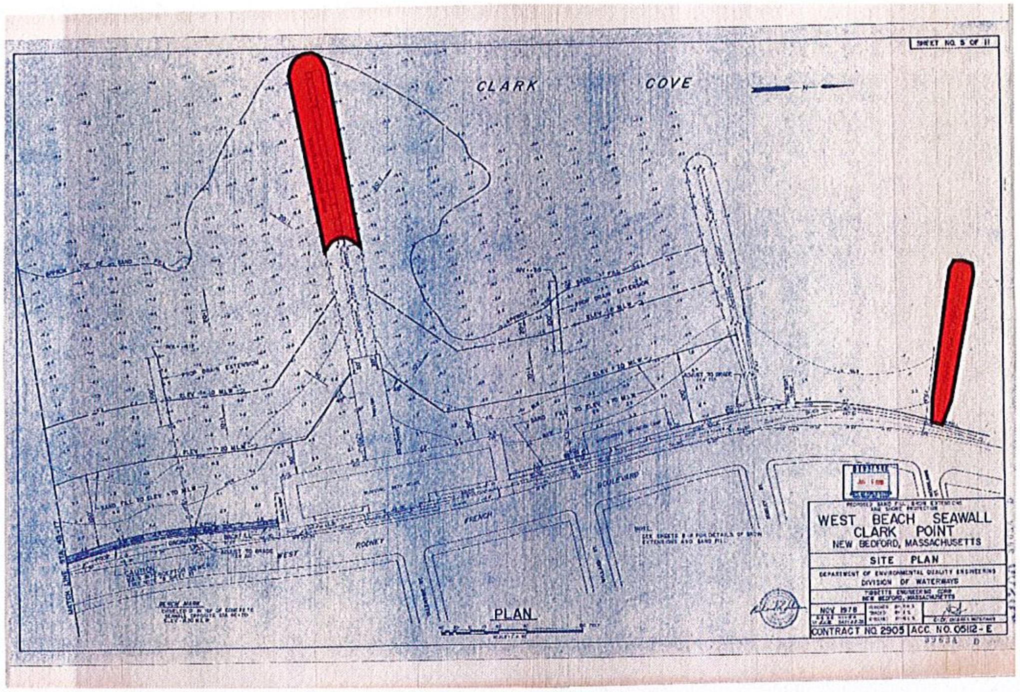


Figure 4.17 Tibbetts Engineering 1978 plan of West Beach. The footprint of Groins 1 (right) and 2 (left) are shaded in red.

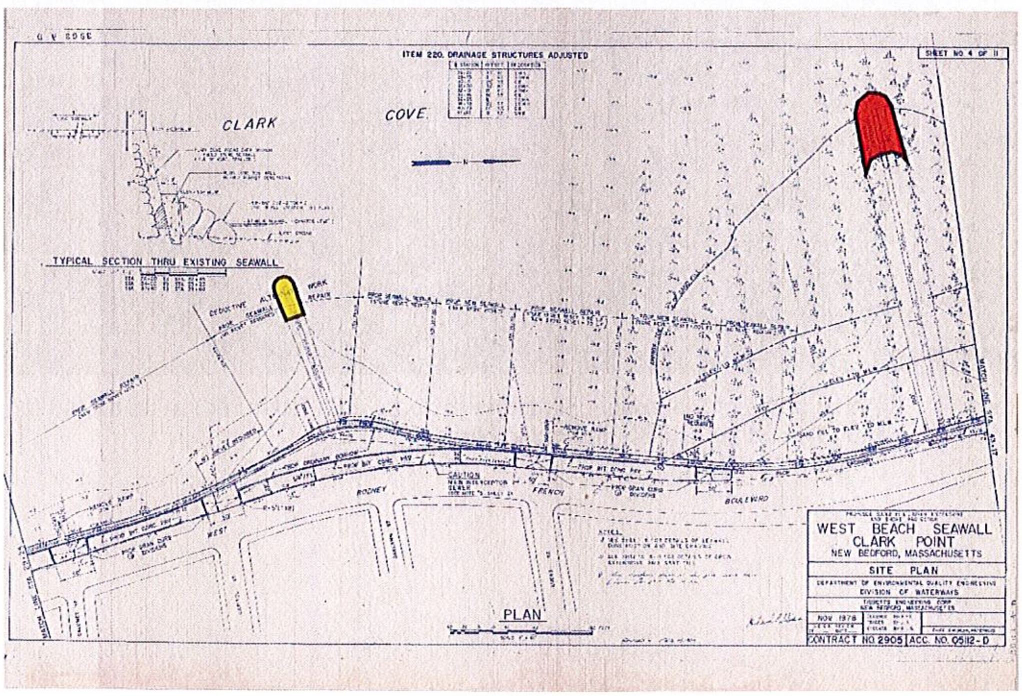


Figure 4.18 Tibbetts Engineering 1978 plan of West Beach. The footprint of the Groin 4 (right) extension to be removed is shaded in red. The tip of Groin 5 (left) shaded in yellow will be adjusted to the adjacent breakwater T-head section.

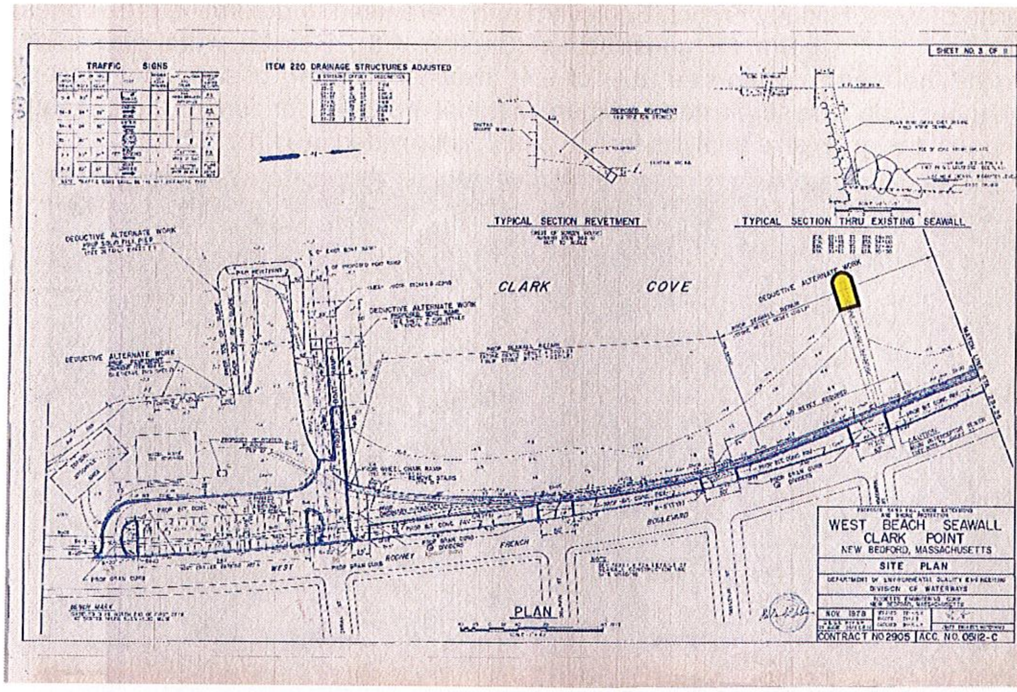


Figure 4.19 Tibbetts Engineering 1978 plan of West Beach. The tip of Groin 6 (right) shaded in yellow will be adjusted to the adjacent breakwater T-head section.

4.5 Sediment Sampling

To determine sediment distributions of the existing beach to nourish the beach effectively and protect infrastructure behind the seawall, a sampling and sediment analysis program was completed. A successful beach nourishment project relies on compatibility between the existing sediment and the chosen supply. To assess compatible sediment distributions to target potential borrow sites, grain size data were obtained from beach grab sampling conducted by Briggs Engineering and Testing. Beach grain size samples were taken along the existing beach to characterize the range of sediments "native" to the beach system between approximate mean low water and the seawall/revetment. One sample was collected from a lower beach location at the shoreline just above mean low water (MLW) and another sample was collected from the upper beach just above mean high water (MHW). There two samples were collected along three shore-perpendicular transects in the groin compartments to the north of the project area, as the material is characteristic of "native" sediment to the beach (Figure 4.20). The material residing in the nourishment footprint contains a mix of sand, gravel, stones, and leftover anthropogenic material or non-native material, likely deposited during seawall repair in 1978.

Each of the six sediment samples were analyzed for grain size utilizing standard ASTM analysis techniques. Individual sample grain size distributions are provided in Appendix C.

Sediment data were evaluated by finding the maximum and minimum grain sizes in each dataset, then calculating a mean value. Figure 4.22 and Figure 4.23 show the maximum/ minimum and mean data for the lower and upper beach samples, respectively.

These values best represent the current natural composition of the berm along the shoreline of West Rodney French Boulevard and were used to determine the suggested composition of the berm for construction (Section 4.5). Results were also used to determine the median grain size (d_{50}) of 0.46 millimeters (mm) used in the conceptual design models and the suggested sediment material gradation for construction. A tabular summary of the grain size analysis for the beach is provided in Table 4.2.

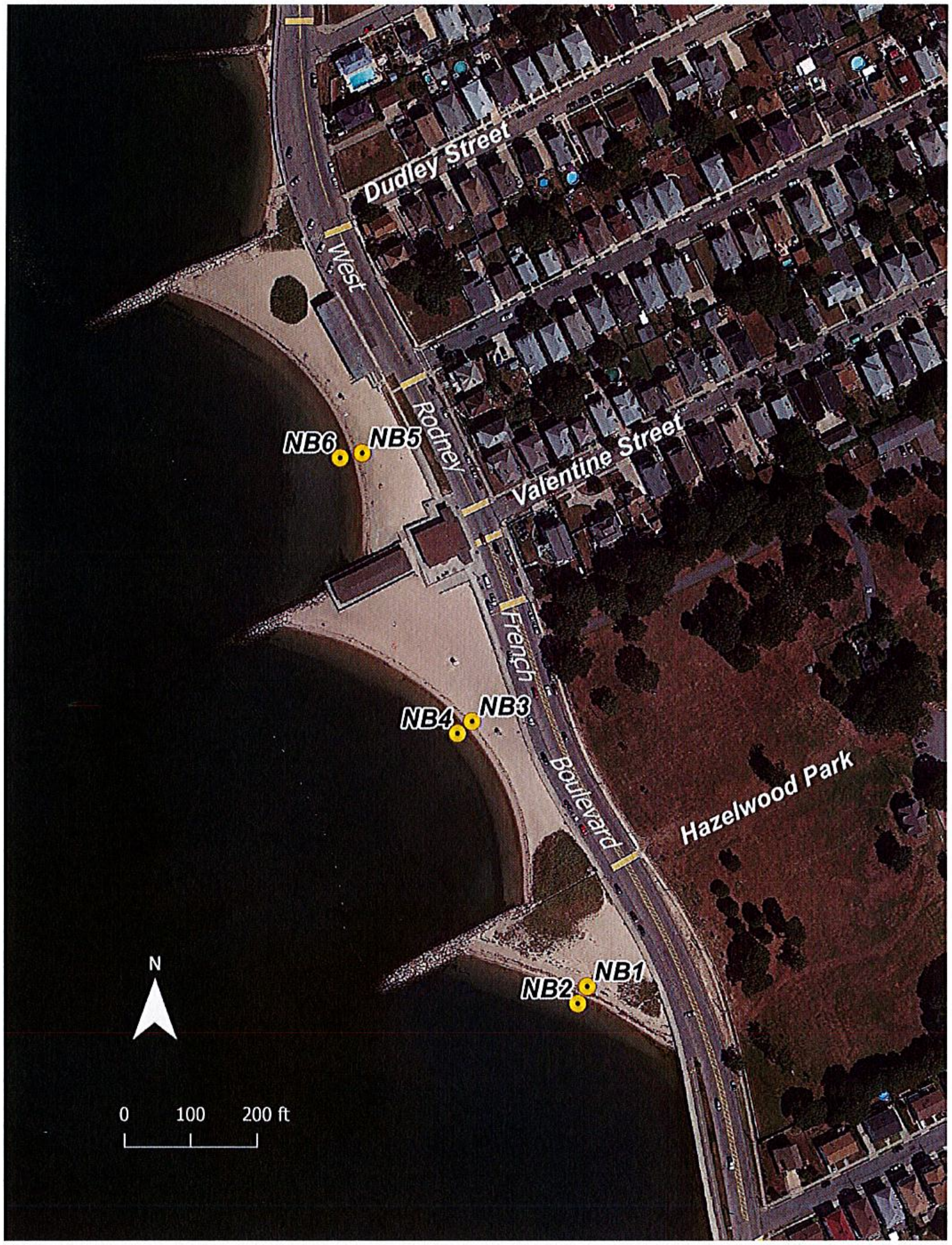


Figure 4.20 Sediment sampling locations for the six samples used in the sediment evaluation. Aerial photo from GoogleEarth.

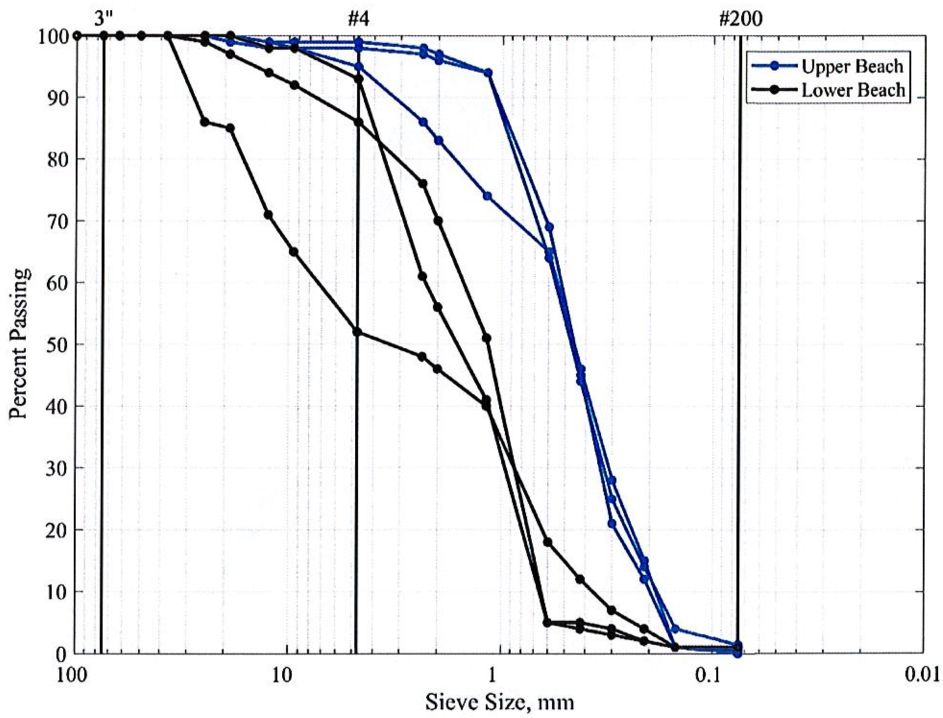


Figure 4.21 Grain size distributions for the upper (N1, N3, N5) and lower (N2, N4, N6) beach locations.

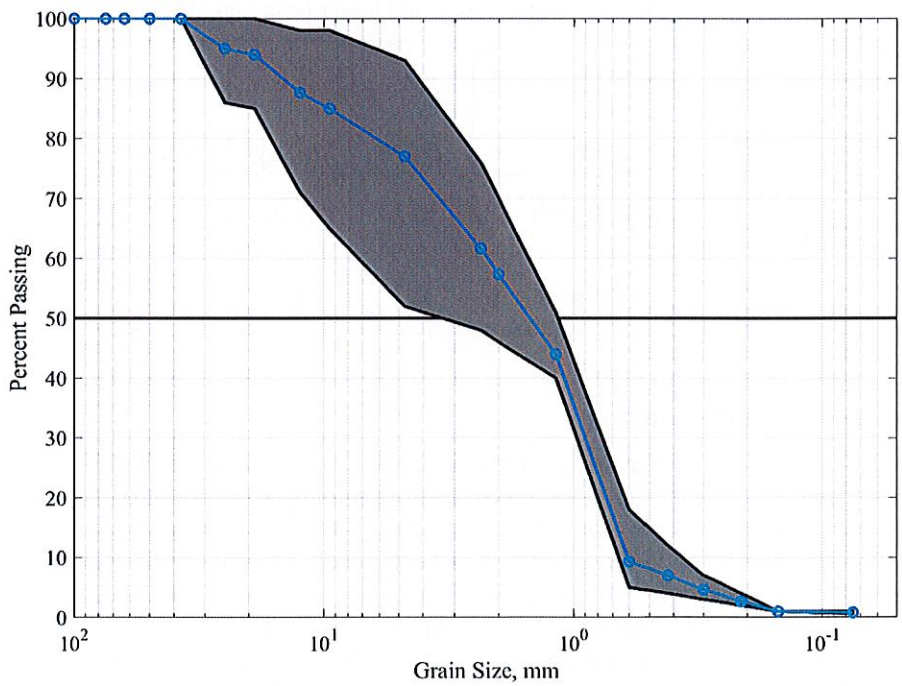


Figure 4.22 Lower beach (NB2, NB4, N6) grain size min/max range (shaded) and mean grain size (blue) for Briggs Engineering and Testing samples.

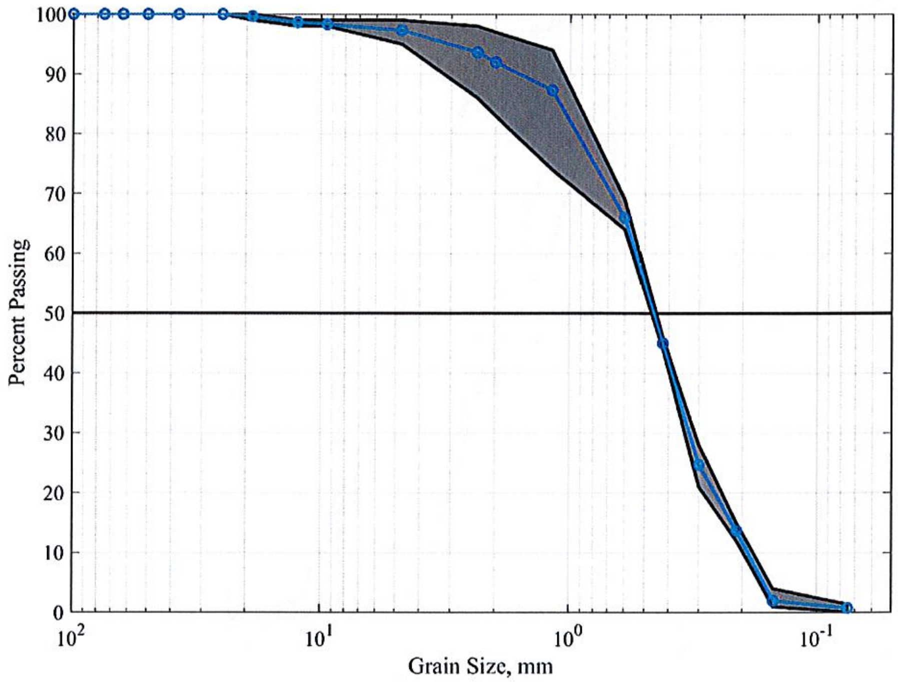


Figure 4.23 Upper beach (NB1 ,NB3, N5) grain size min/max range (shaded) and mean grain size (blue) for Briggs Engineering and Testing samples.

Table 4.2 D_{16}, D_{50}, and D_{84} values for the mean upper and lower beach locations.

	Location	D_{16} (mm)	D_{50} (mm)	D_{84} (mm)
Sample	Upper Beach	0.2	0.46	1.1
	Lower Beach	0.7	1.5	8.9

4.6 Potential Sediment Sources

An initial analysis of potential sediment sources to provide compatible material for the beach nourishment at Clarks Cove in New Bedford was performed utilizing available information from upland borrow sources. There are several options for acquiring the material for beach nourishment. Sand can be dredged from an offshore sand borrow site or an adjacent harbor. Or the sand may be mined from a terrestrial location and transported to the site. The success of the project is dependent on the grain size distribution, and the placement of the fill. To lengthen the lifespan of the nourishment, the grain size distribution should be consistent with the native distribution (or coarser).

Due to the difficulty in securing regulatory approval for potential offshore mining sites (both in time delays and cost), the initial assessment was restricted to land-based sediment sources that are available within approximately 20-30 miles of the site. It should be noted that the construction cost required by utilizing an upland source is approximately double that of using an offshore source for a mid- to large-scale project. The total volume proposed for nourishment is 31,150 cubic yards, where the approximate cost of upland nourishment would be between $780,000 and $935,000 ($25 to $30 per cubic yard, respectively).

4.7 Suggested Gradation

Based on an analysis of the material present at Clarks Cove in New Bedford, the suggested gradation for the nourishment material is presented in Table 4.3; however, some modifications to the gradation may be allowed based upon available borrow source.

Table 4.3. Suggested sediment gradation for nourishment material.
Nothing greater than #4 (4.75 mm)
Less than 85% by weight passing the #18 sieve (1 mm)
Less than 35% by weight passing the #30 sieve (0.6 mm)
Less than 25% by weight passing the #50 sieve (0.3 mm)
Less than 5% by weight passing the #200 sieve (0.075 mm)

4.8 Relative Sea Level Rise

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 Woods Hole, Massachusetts has been rising generally in a linear fashion (see Figure 4.24). Utilizing monthly mean sea level data, the long-term average relative sea level rise in Boston has been 2.86 mm per year or 0.938 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 4.25. 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.

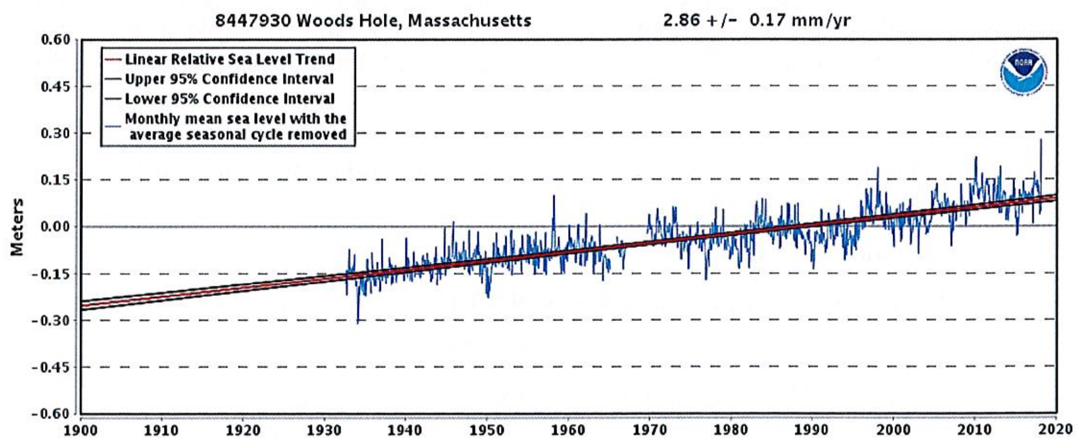


Figure 4.24 Long-term mean sea level data for NOAA Woods Hole tide gauge station with linear trend and confidence interval.

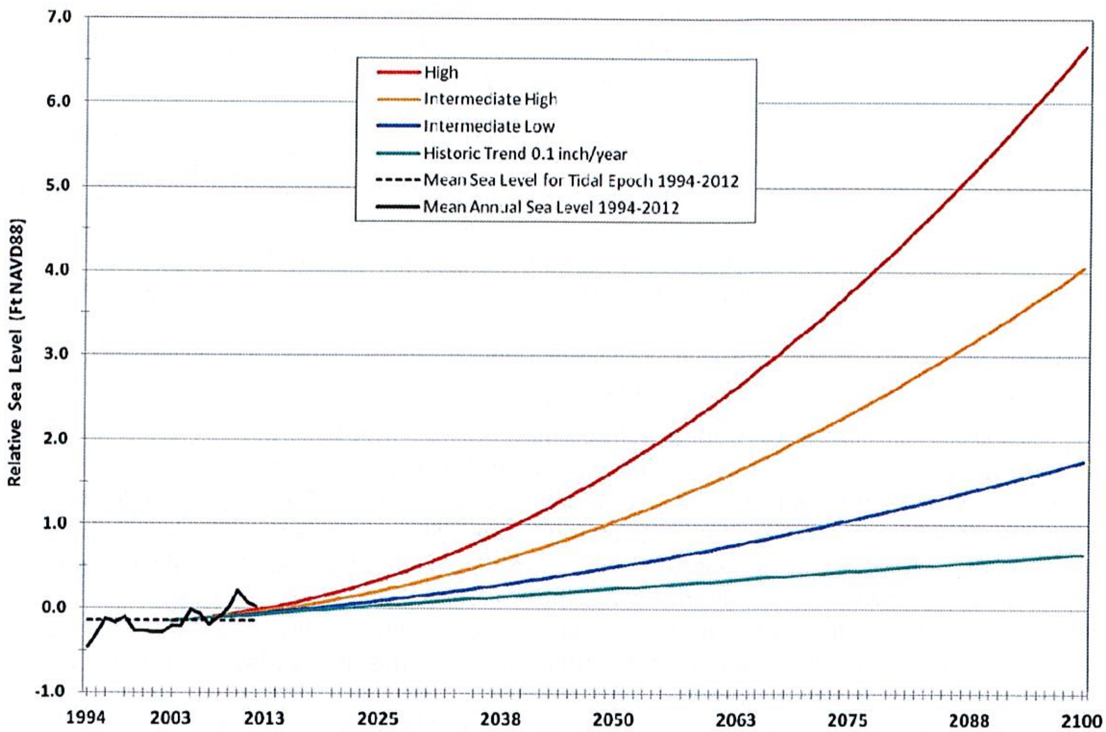


Figure 4.25 Relative sea level rise scenarios estimates (in feet NAVD88) for Boston, MA. Global scenarios from were adjusted to account for local vertical land movement with 2003 as the beginning year of analysis (*figure credit: MCZM, 2013*).

4.9 Storm Susceptibility

Due to the unique geographic location of Massachusetts, both tropical (originate in the tropics) and extra-tropical (originate in mid-latitudes) storm events are important to the characterization of potential coastal hazards. For the shorelines in Buzzards Bay, including New Bedford, as well as along the south shore of Cape Cod, Martha's Vineyard, and Nantucket, hurricanes typically are considered the storms of record. However, storm damage along the remainder of the Massachusetts coast is dominated by extra-tropical storm events (northeasters). Tropical storms and hurricanes generally move across Massachusetts rapidly (often in a few hours); however, their storm surge can be substantial, especially in large semi-enclosed basins oriented toward the direction of storm approach (e.g., Buzzards Bay). In addition to their rapid passage, significant hurricane events are relatively infrequent, with only two Category 1 Hurricanes making official landfall (where the center of the Hurricane eye crosses the shoreline) in Massachusetts during the past 100 years (1916 and 1954). Hurricane landfalls in the Massachusetts region are shown on Figure 4.26. However, extensive damage has been caused by more powerful hurricanes that made landfall west of Massachusetts, including hurricanes in 1991 (Bob), 1944, and 1938. In contrast, extratropical storms (out of the northeast, east or southeast) typically occur several times per year, generally between late October and April. Although the sustained winds are typically less than hurricane-strength, the duration of these storms can be problematic, causing coastal flooding situations for upwards of two-to-three days for severe storms. Although storm surge elevations associated with these easterly storms are not as severe as major hurricanes, their relatively frequent occurrence and duration create significant coastal hazards along Buzzards Bay and the south shore of Cape Cod. To evaluate the susceptibility of the project area to the full range of storms, historical storm surge elevations were evaluated.

Figure 4.27 illustrates the FEMA 100-year storm surge levels along the Massachusetts coast. Due to the limited data available, it is not possible to determine an accurate FEMA 100-year storm surge level along the undeveloped shoreline of outer Cape Cod, or along the south shores of Martha's Vineyard or Nantucket. The FEMA 100-year storm surge elevation represents the stillwater elevation without the local influence of waves. The highest storm surge levels experienced in Massachusetts occur in Buzzards Bay, where the 1938 hurricane caused a storm surge in excess of 13 feet NAVD. However, it should be noted that most of the Massachusetts coast has FEMA 100-year storm surge levels in excess of 10 feet NAVD.

As shown in Figure 4.28, for the shorelines of Buzzards Bay and the south shore of Cape Cod, the difference between the 1-year and 100-year storm surge elevations is generally between 6-8 feet (1.8-2.4 m), with areas where the difference is greater than 8 feet (2.4 m) in the upper reaches of Buzzards Bay. This indicates quite clearly that the annual winter storms of the region result in storm surge elevations significantly lower than those associated with a rare, severe tropical storm, such as a hurricane. In short, a severe hurricane impacting the area will be accompanied by historic flooding and associated damage. Although hurricanes can cause damage along the entire Massachusetts Coast, areas most susceptible to this damage are the south-facing shoreline including the Buzzards Bay coast, the southern shore of Cape Cod, Nantucket, and Martha's Vineyard.

More recent flood inundation mapping has been performed for New Bedford (SeaPlan, 2014) using the NOAA Sea, Lake, and Overland Surges from Hurricanes (SLOSH) model (Figure 4.29). For the West Rodney French Boulevard area, some inundation along the roadway can be anticipated during a Category 2 or higher Hurricane. However, due to the relatively high elevations that exist on Clarks Point, the flooded area is limited. In addition, the west-facing shoreline further protects this area from receiving the direct impact of Hurricane wave forces that typically have the greatest impact on south-facing shorelines. This natural protection is shown by the relatively modest alongshore sediment transport rates and long-term stability of the West Rodney French Boulevard shoreline, as described in Section 2.0.

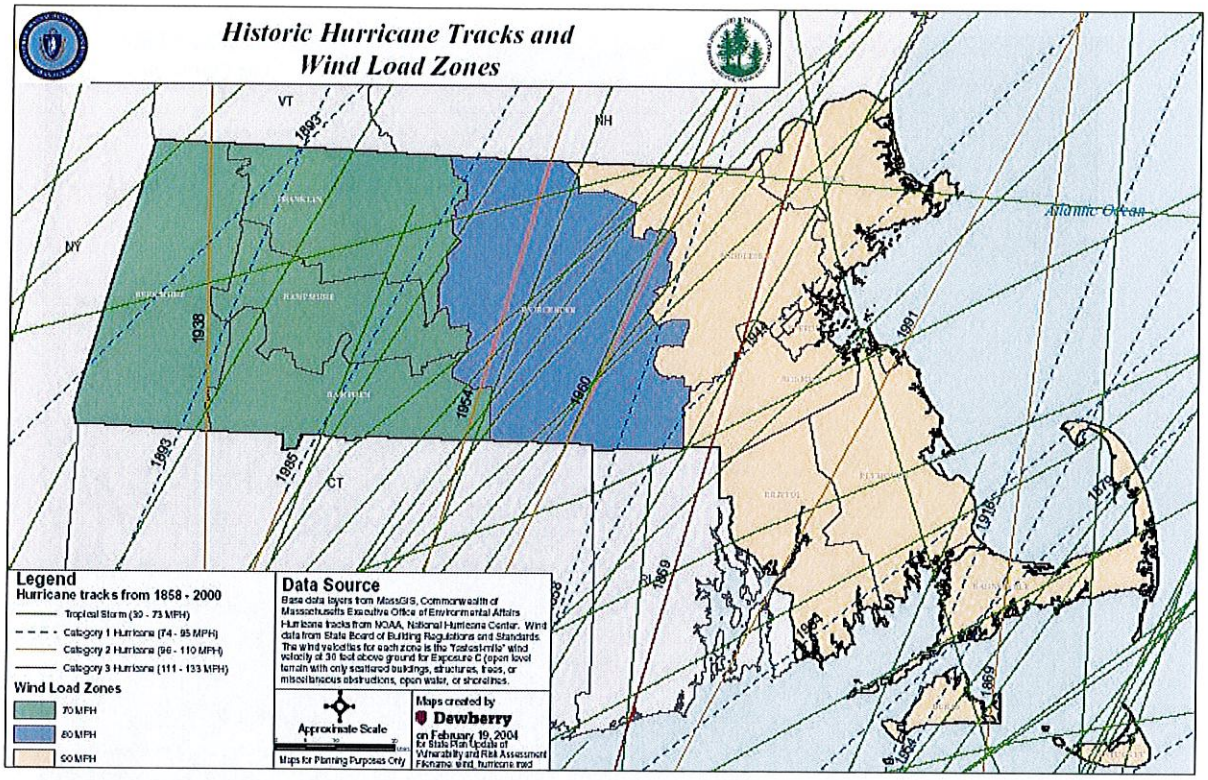


Figure 4.26 Historical hurricane tracks impacting Massachusetts from 1858 to 2000.

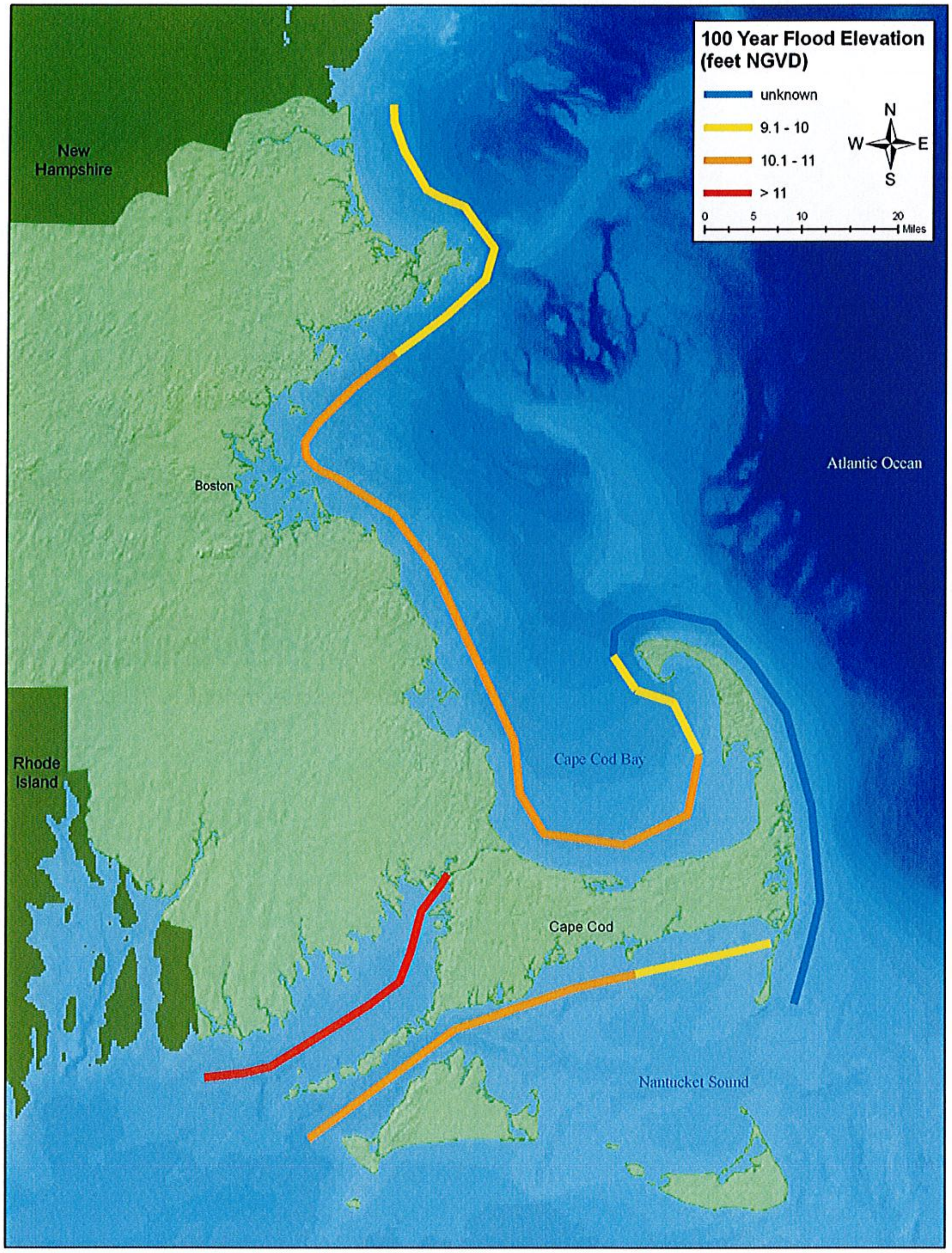


Figure 4.27 100-year coastal storm surge elevations along the Massachusetts shoreline (derived from Tidal Flood Profiles, New England Coastline. U.S. Army Corps of Engineers, New England Division, September, 1988).

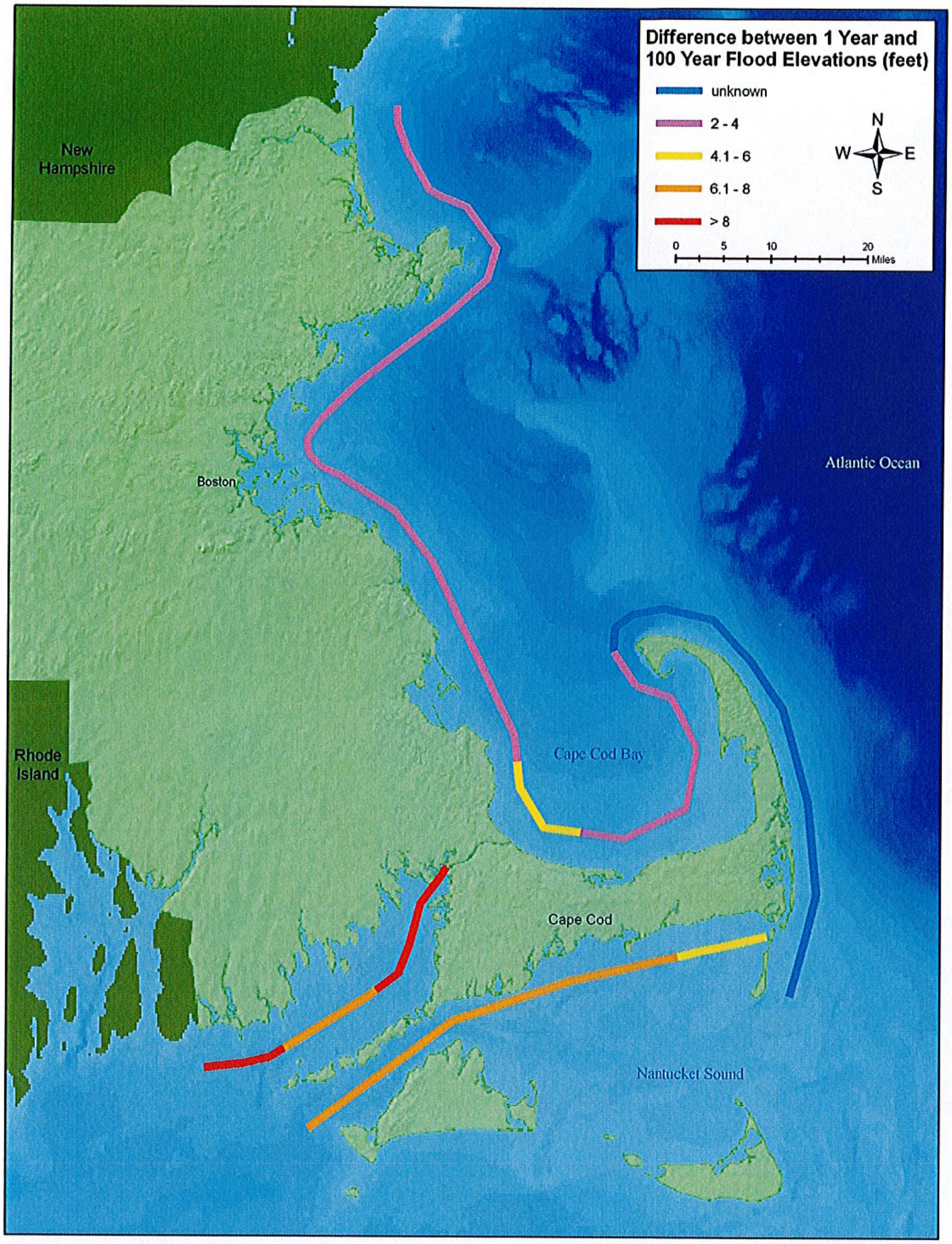


Figure 4.28 Difference between 1-year and 100-year coastal storm surge elevations along the Massachusetts shoreline (derived from Tidal Flood Profiles, New England Coastline. U.S. Army Corps of Engineers, New England Division, September, 1988).

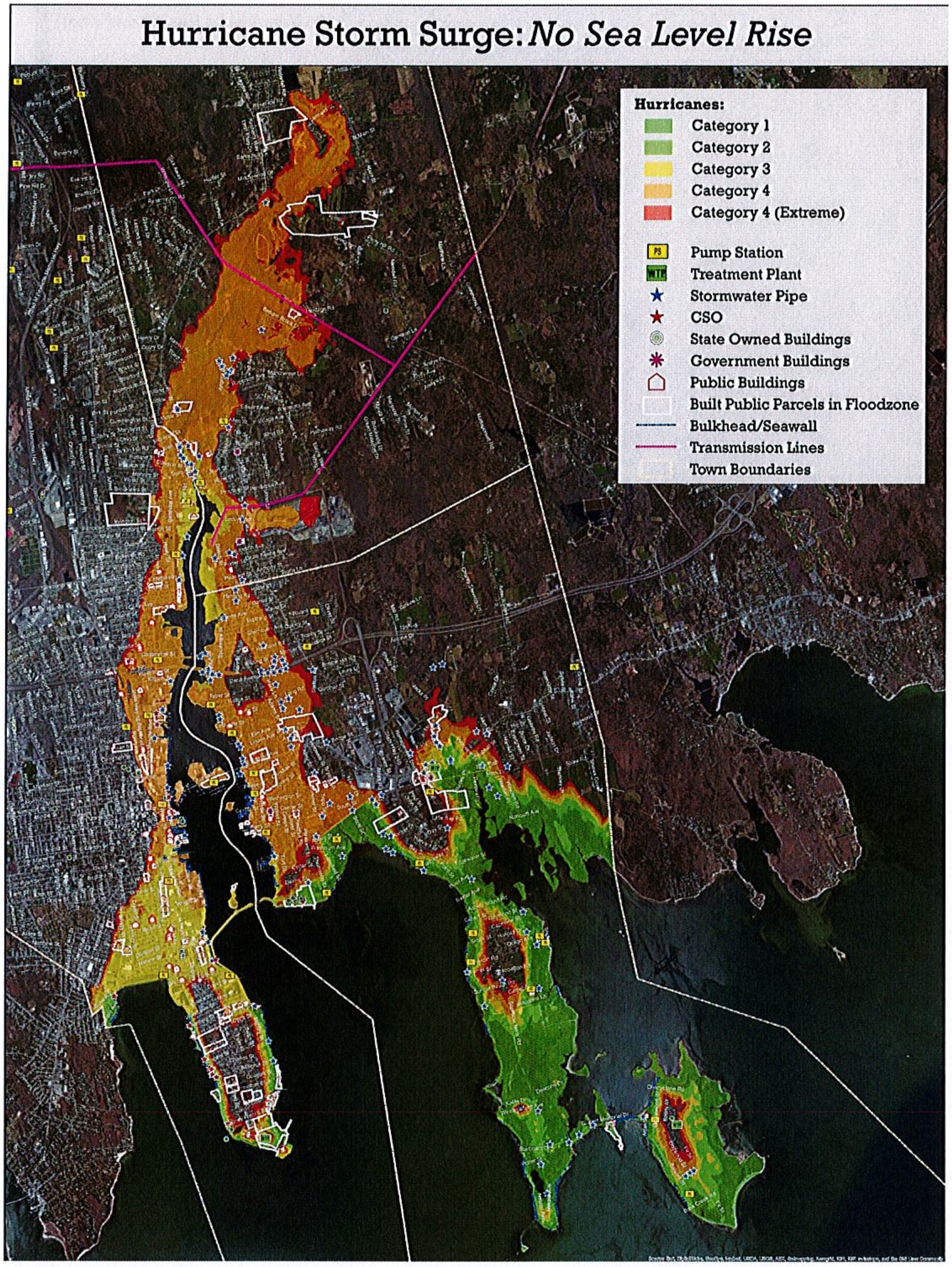


Figure 4.29 Hurricane inundation areas under existing sea-level conditions predicted from SLOSH model (SeaPlan, 2014)

4.10 Preliminary Construction Details

MADMF stated that the beach is most biologically active during the summer and therefore a November to April construction time would be most appropriate. With the assumption that construction could begin in the fall of 2019, the project is estimated to take approximately 6 months to complete..

The beach nourishment contract will include obtaining the beach nourishment material from an upland source, delivering it to the beach along West Rodney French Boulevard, and spreading it. A total of approximately 1,420 to 1,560 truck trips will be required to bring the nourishment material to the project site. The designated truck route will likely travel to the project site from I195 and follow MA18 down to the parking lot adjacent to the boat ramp (Figure 4.30). No mitigation measures are being proposed at this time. The final shaping of the fill material will be completed by bulldozing or other approved means. The nourishment material will be used to build a platform tongue from which the breakwater stones may be placed. Once the breakwater is in place, some of the sand will be removed behind the breakwater so that the trunk portion of the groin may be placed. All earth-working equipment will operate above the tide line.

Temporary, short-term impacts from construction activities would be mitigated to the extent practicable. Appropriate construction mitigation measures would be incorporated into the contract documents and specifications governing the activities of contractors and subcontractors working on elements of the project. On-site resident engineers and inspectors will monitor all construction activities to ensure that mitigation measures are properly implemented.

4.11 Preliminary Maintenance and Monitoring Plan

Since the purpose of the beach nourishment program is to re-establish the local sediment supply and provide storm and flood protection for West Rodney French Boulevard and the infrastructure behind it, an evaluation of long-term needs will be required to assist the City with future maintenance. An essential aspect to this project will be the monitoring of the beach nourishment performance. This monitoring information will aid the City of New Bedford by determining:

- The position of the berm relative to the seawall.
- Accretion or erosion along adjacent beaches.
- The longshore variability in berm width indicative of potential "hot spot" erosion.
- Future nourishment need required to maintain shore protection and berm width.

The rapid introduction of such a large volume of sediment to the nearshore area will result in the material moving in both the cross-shore and to a lesser extent the alongshore to reach equilibrium with the waves and currents in the area. Monitoring will provide a means to measure the berm position in relation to the seawall and shifting of the fill. This can be determined by measuring the elevation along a series of shore perpendicular control transects (or cross-sections) along the length of the fill, as well as performing a differential GPS survey of the observed high water mark.

There will be two transects in each of the groin compartments. The transects will straddle the centerline of the groin compartment and be spaced 25 ft apart. A pre-construction survey will be performed immediately preceding the project. The post-construction survey will be performed as soon as is practical after the completion of the fill and will extend offshore to the -1 ft below MLW. In the first-year post-construction, surveys will be performed quarterly, wading out to the -1 ft below MLW. The second-year post-construction will have a 12-month and an 18-month survey again to -1 ft below MLW. Starting in the third-year post-construction, surveys will be conducted once annually out to -1 ft below MLW.

To ensure consistency between surveys, permanent benchmarks and/or markers will be installed along the seawall for the purpose of future beach measurements. The elevations along the cross-sections will be plotted to monitor changes in the berm position. The transects can also be used to determine performance of the nourishment within the original design template. In addition, differential-GPS surveys will be performed using a backpack unit along the observed high water line. These surveys of the high water line will be performed immediately following nourishment and will take place on the same schedule as the cross-shore transects discussed above.

Monitoring reports will be prepared annually for the first five years after completion of the project, and then potentially at a less frequent rate depending on the project performance. These reports will include a summary of all data collected, information regarding the wave climate and storm activity, volume change over time, and an evaluation of shoreline change. In this manner, the performance of the beach fill can be evaluated relative to design predictions. The monitoring information will provide useful data needed to assess future re-nourishment requirements of the West Rodney French Boulevard shoreline. In addition, monitoring of any potential impact associated with nourishment will be included as part of the data collection and analysis effort.

Engineered beach nourishment projects have a limited design life; therefore, project planning includes the anticipation of future maintenance for beach fill. Results from the beach nourishment monitoring reports will be utilized to assess when re-nourishment should be considered. In general, overall beach volume can be considered the basis for “trigger conditions” for planning and construction actions. In addition, “hot spot” erosion (significant erosion that occurs over a short stretch of beach likely resulting from the influence of coastal engineering structures) needs to be considered, since loss of significant beach volume or width could limit the storm protection level provided by the nourishment. A proposed “trigger condition” includes if the beach width, defined as distance from the seawall to the observed high water line (based on the annual differential GPS surveys), decreases to 30% of the design width in any of the groin compartments. Should this occur,, actions will be taken to re-nourish the “hot spot” out to its original design width.

The project construction has several environmental implications that need monitoring to ensure reduction of adverse impacts. A plan for the monitoring of eelgrass will be necessary to determine the extent of any migration of nourished sediment into the habitat. Annual diver surveys in late May will map the extents of eelgrass habitat for the first two years following project completion. A report will be prepared following the completion of the survey. Shellfish that exist in the project template will be collected prior to construction. Following placement of the nourishment, shellfish populations that were identified in the shellfish survey may be reseeded to promote population growth. Additional monitoring of any bird nesting, particularly plovers, will also follow project completion. Further discussion of environmental impacts is included in section 5.

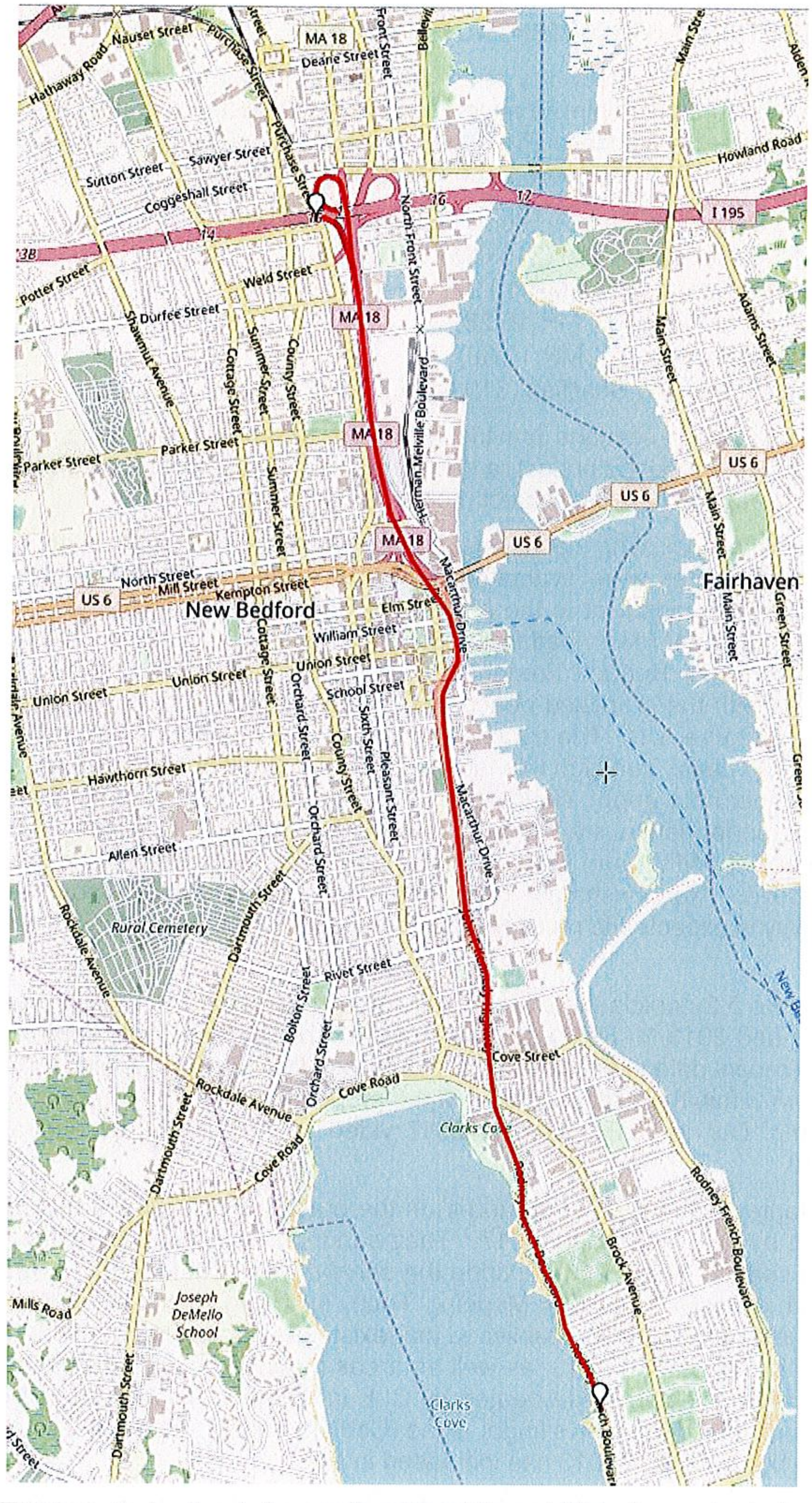


Figure 4.30 Truck route for the delivery of sediment from I-95 to the project shoreline along West Rodney French Boulevard.

5.0 POTENTIAL ENVIRONMENTAL EFFECTS OF PREFERRED DESIGN

The proposed project has been designed and will be constructed using the best available measures to minimize adverse impacts to coastal resource areas as defined by the Massachusetts Wetlands Protection Act (WPA). The Proposed project is located within and/or abutting the following coastal resource areas:

- Land Subject to Coastal Storm Flowage (310 CMR 10.04)
- Land Under the Ocean (310 CMR 10.25)
- Coastal Beach (310 CMR 10.27)
- Coastal Dune (310 CMR 10.28)
- Coastal Bank (310 CMR 10.30)
- Land Containing Shellfish (310 CMR 10.34)

The following sections provide definitions of coastal resource areas that will be affected by the proposed project, a description of the proposed work to occur within each resource area, and how the Project meets performance standards.

To assess existing conditions and potential project impacts to resource areas, biological resources were evaluated within the project 'footprint', as well as within the nearshore region adjacent to the project. Stantec was contracted to provide a shellfish habitat assessment (described in Section 5.6, below) and eelgrass survey within Clarks Cove along West Rodney French Boulevard, as support for permitting requirements associated with the proposed beach renourishment project from Hazelwood Park to the Town Pier (Figure 5.1). The survey was performed on a super-tide, full moon extending over two field days. The high tide facilitated diving conditions in the nearshore. Wind was 5-10 miles per hour (mph), cloudy skies with light rain. Stantec found that this project will meet the performance standards in the Commonwealth of Massachusetts Wetlands Protection Act (WPA) and will not significantly, adversely impact shellfish habitat within Clarks Cove. Renourishment may temporarily affect shellfish populations; however, productivity is expected to recover within one year. The full report is provided in Appendix D.

Figure 5.2 depicts the extents of eelgrass present in the project area at the time of the 2001 and 2013 MADEP eelgrass surveys. These extents are approximate and were the most recent data available through the MassGIS OLIVER mapping tool. Eelgrass beds are shown within the middle section of the proposed project (Figure 5.2) and south of the boat ramp. During CLE's January 2017 video survey, additional eelgrass habitat was identified.

Stantec was asked to ground-truth the presence of eelgrass observed in video as collected by CLE in January 2017. They conducted a diver-assisted survey during the growing season (May 2017). During the survey, historical MADEP eelgrass maps were reviewed (MassGIS OLIVER Mapping Tool), including maps available from 2001 and 2013. Field observations established that existing eelgrass was present beyond the area mapped by MADMF in 2013, as well as areas mapped by CLE (Figure 5.3); however, a large section of eelgrass delineated by CLE in January 2017 was observed to be dead and occupied by dead man's finger algae (Codium fragile). Regardless, the eelgrass area mapped by MADEP in 2013 has increased in size.

A further survey was performed to delineate the extents of the coarse-grained material that exists along the upper part of the intertidal beach (see Figure 2.5 and Figure 2.8 for examples). A visual inspection indicated that much of this material consists of angular stones, brick, and concrete. Based on the plans related to historic improvements to the seawall in 1978 (Figure 2.4), it is apparent that the likely source of the coarse-grained material on the beach is from dismantling of the previous seawall which consisted of cobble-sized angular stone held together with mortar. Therefore, the extent of this material on the project plans (Appendix B) is described as "Anthropogenic Stone, Brick, etc." Based on the survey, the total beach area covered by this material is approximately 11,200 square feet, where a majority of the material appears to be placed as protection for the steel Combined Sewer Overflow (CSO) pipes that extend onto the beach.

Figure 5.1 Proposed Project Area illustrating sand nourishment (tan shading) and T-head structures (breakwater sections shown in gray).

Figure 5.2 MADEP Eelgrass Areas (retrieved data June 2017).

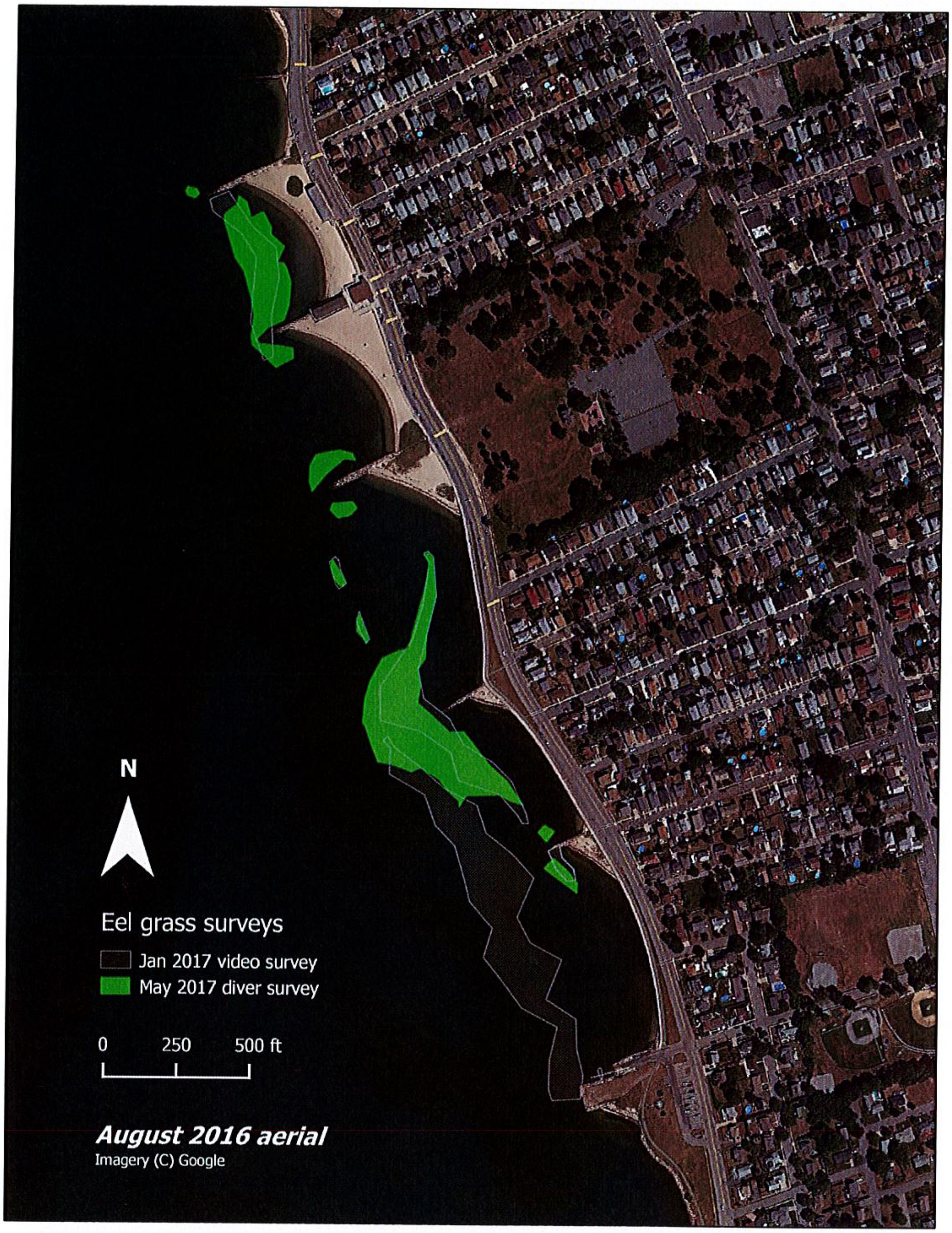


Figure 5.3 Stantec Eelgrass Assessment Results May 25, 2017 with CLE Eelgrass Video Assessment Results (January 2017).

5.1 Land Subject to Coastal Storm Flowage

Pursuant to 310 CMR 10.04, Land Subject to Coastal Storm Flowage (LSCSF) means "land subject to any inundation caused by coastal storms up to and including that caused by the 100-year storm, surge of record or storm of record, whichever is greater". The areas mapped by the Federal Emergency Management Agency (FEMA) on community Flood Insurance Rate Maps (FIRM) as the 100-year flood plain within the coastal zone are included within LSCSF. LSCSF may be significant to the interests of storm damage prevention, flood control, pollution prevention, and wildlife habitat. LSCSF in this area contains other jurisdictional resource areas which are important for storm damage prevention and flood control.

The current flood insurance rate map (FIRM) for this area, depicted as Figure 5.4, indicates that the 100-year storm encompasses the entire Project Area. According to FEMA and the National Flood Insurance Program, any building located in an A or V zone is considered to be in a Special Flood Hazard Area, and is lower than the Base Flood Elevation V zones are the most hazardous of the Special Flood Hazard Areas. There are currently no performance standards for work in LSCSF. The proposed berm nourishment will affect approximately 266,315 ft^2 of LSCSF. The proposed project is not anticipated to alter the existing drainage patterns of the site and will enhance the storm damage prevention capacity of the site.

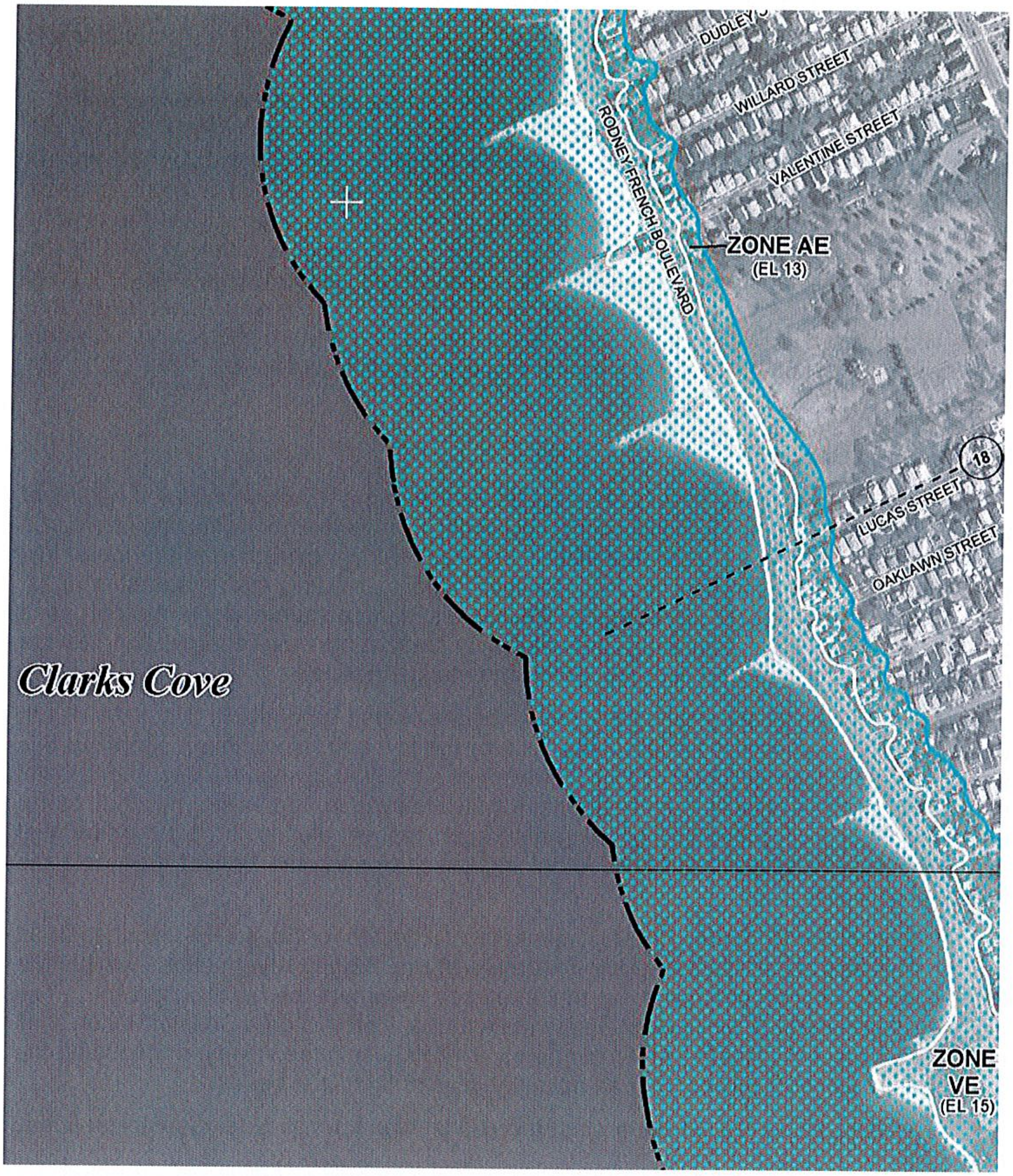


Figure 5.4 FEMA flood insurance rate map. Regions in turquoise are subject to inundation by the 1% annual chancel flood, as determined by FEMA (https://msc.fema.gov/portal/).

5.2 Land Under the Ocean

Land Under the Ocean (LUO) is defined at 310 CMR 10.25(2) as "land extending from the mean low water line seaward to the boundary of the municipality's jurisdiction and includes land under estuaries". This resource area is presumed significant to provide feeding areas, spawning and nursery grounds and shelter for coastal organisms, to reduce storm damage and flooding by diminishing and buffering the high energy effects of storms, provide a source of sediment for seasonal rebuilding of coastal beaches and dunes, and to provide important food for birds and invertebrates.

The proposed beach nourishment template will convert 119,301 square feet (2.7 acres) from LUO to Coastal Beach. Additionally, the T-head groins will fill 24,271 square feet of LUO. However, removal of three existing groin sections (shown on the Project Plans in Appendix B) will create 24,270 square feet of new LUO. Therefore, impacts associated with coverage of LUO resources is offset by creation of LUO in the historic 'footprint' of the three groin structures. In the groin removal areas, it is anticipated that the surficial layer of armor stone will be retained to serve as mitigation for the coarse-grained material (shown as "Anthropogenic Stone, Brick, etc." on the plans in Appendix B) that exists within the intertidal zone of the Coastal Beach. In total, there is approximately 11,200 square feet of this man-derived material within the project area.

5.3 Coastal Beach

Pursuant to 310 CMR 10.27(2), Coastal Beach refers to unconsolidated sediment subject to wave, tidal, and coastal storm action which forms the gently sloping shore of a body of salt water and includes tidal flats. Coastal beaches extend from the mean low water line to the coastal bank or the seaward edge of existing man-made structures. Coastal beaches dissipate wave energy, serve as sediment source, serve the purposes of storm damage prevention and flood control by dissipating wave energy, and provide habitats for shellfish, marine fisheries, birds and marine mammals.

Based on the existing conditions shown on the plans (Appendix B), a coastal beach exists for the length of the Project Area. Due to the low natural sediment supply to this stretch of shoreline, the beach has lowered over time, and in many areas, the coastal beach elevation is below the mean high water elevation, providing minimal storm protection. It is anticipated that ongoing natural processes, along with relative sea-level rise, will cause complete loss of the remaining beach fronting the Project Area over the next few decades.

A total of approximately 114,000 square feet (2.6acres) of Coastal Beach, measured from the existing mean low water line to the toe of the existing revetment/seawall in the Project Area, will be enhanced by the proposed sediment berm nourishment. The performance standards for Coastal Beach state that any project on a Coastal Beach shall not have an adverse effect by increasing erosion, decreasing the volume or changing the form of any such coastal beach or an adjacent downdrift coastal beach.

The proposed project will protect the critical characteristics for Coastal Beaches (310 CMR 10.27(1)) as follows:

a. Volume (quantity of sediments) and form: *The proposed berm nourishment is not expected to impede the transport of beach sediments along the Project Area. The berm will provide an improved sediment supply.*
b. Ability to respond to wave action: *The proposed mixed-sediment berm will have a higher elevation and, compared to the existing beach, and have a greater ability to dissipate wave energy.*
c. Distribution of sediment grain size: *Sediment consistency (i.e., grain size) of the nourishment will be consistent with sediment from the beaches north of the project area, as well as adjacent to the existing groins in the project area. Due to anthropogenic alterations to the beach system along the areas where no high tide beach exists, the existing sediments should not be considered native.*

d. Water circulation: *The proposed beach nourishment will not affect water circulation.*
e. Water quality: *No impacts to water quality will be caused by the proposed mixed-sediment berm nourishment. Berm material will consist of clean sand and gravel, with less than 2% fines.*
f. Relief and elevation: *The proposed nourishment will raise the existing beach elevation to approximately +4.5 ft NAVD88 to reduce wave overtopping.*

The proposed project will meet the performance standards for Coastal Beach (310 CMR 10.27(3, 5, and 7)) as follows:

a. 310 CMR 10.27(3): *The proposed nourishment will not increase erosion, decrease the volume, or change the form of the existing beach. As designed, the project will increase the beach volume.*
b. 310 CMR 10.27(5): *The project consists of a nourishment of clean sediment of a grain size compatible with the native beach material located to the north of the project area. The project also consists of the construction of T-head groin sections.*
c. 310 CMR 10.27(7): *The project is not located within mapped habitat of rare vertebrate or invertebrate species.*

5.4 Coastal Dune

The Act defines Coastal Dune (310 CMR 10.28(2)) as "any natural hill, mound or ridge of sediment landward of a coastal beach deposited by wind action or storm overwash. Coastal dune also means sediment deposited by artificial means and serving the purpose of storm damage prevention or flood control." Although Coastal Dunes exist adjacent to the project, no work will be performed within the footprint of this resource area.

5.5 Coastal Bank

The Act defines Coastal Bank (310 CMR 10.30(2)) as "the seaward face or side of any elevated landform, other than a coastal dune, which lies at the landward edge of a coastal beach, land subject to tidal action, or wetland". The Coastal Bank is determined to be significant to storm damage prevention because it is a vertical buffer to storm waters. Therefore 310 CMR 10.30(7) applies: *Bulkheads, revetments, seawalls, groins, or other coastal engineering structures may be permitted on such a Coastal Bank except when such bank is significant to storm damage prevention or flood control because it supplies sediment to coastal beaches, coastal dunes, and barrier beaches.* The proposed beach nourishment will be placed in front of the existing seawall/revetments on West Beach and increase shore protection. As part of the project, spalling portions of the existing concrete seawall will be repaired; therefore, the project represents a slight improvement to the storm damage prevention aspects of Coastal Bank.

5.6 Land Containing Shellfish

Land Containing Shellfish is defined as "those resource areas likely to contain shellfish, to provide criteria for determining the significance of land containing shellfish, and to establish regulations for projects which will affect such land." Land Containing Shellfish can include Land under the Ocean, Tidal Flats, Rocky Intertidal Shores, Salt Marshes, and Land under Salt Ponds when any such land contains shellfish. Shellfish species included in this definition are: bay scallop (*Argopecten irradians*), blue mussel (*Mytilus edulis*), ocean quahog (*Arctica islandica*), oyster (*Crassostrea virginica*), quahog

(*Mercenaria merceneria*), razor clam (*Ensis directus*), sea clam (*Spisula solidissima*), sea scallop (*Placopecten magellanicus*), and soft shell clam (*Mya arenaria*). The project will impact approximately 257,500 square feet (5.9 acres) of Land Containing Shellfish within the Coastal Beach and LUO resource areas. The planned beach nourishment may temporarily affect shellfish populations; however, productivity is expected to recover within one year.

Due to the extensive size of the project area, shellfish sampling stations were located at 50-foot intervals along four transects set 50-feet apart (Figure 5.4). Shellfish stations at the proposed project sites were arranged in a grid pattern (Figure 5.5). The study area included sampling along transects, which extended from the mean low water line. Shellfish stations were evaluated for the presence, abundance, and type of shellfish within sampled substrate. Shellfish targeted for abundance calculations included quahogs, soft-shell clams, bay scallops, razor clams, and American oysters. Sediment characteristics were visually observed at each location. A total of one-hundred thirty-two (132) stations were surveyed for the presence and abundance of shellfish and sediment type by two Stantec divers. Ninety-two (92) quahogs, five (5) bay scallops, one (1) sea urchin were collected within the project study area.

The majority of the study area within Clarks Cove is shown to be suitable for quahogs. Additionally, there are small areas designated as suitable for bay scallops, razor clams and American oysters between Hazelwood Park and the boat ramp. Figure 5.6 depicts MADMF shellfish growing areas. This information was also acquired as geospatial data via the MassGIS OLIVER Online Mapping Tool. Clarks Cove is mapped as conditionally approved for shellfish growing and the City of New Bedford seeds the area with quahogs. This area can be open to City of New Bedford shellfishing subject to water and sediment quality. The area is conditionally approved for shellfish growing by MADMF and is considered suitable for quahogs, bay scallops, oysters, and razor clams within Clarks Cove.

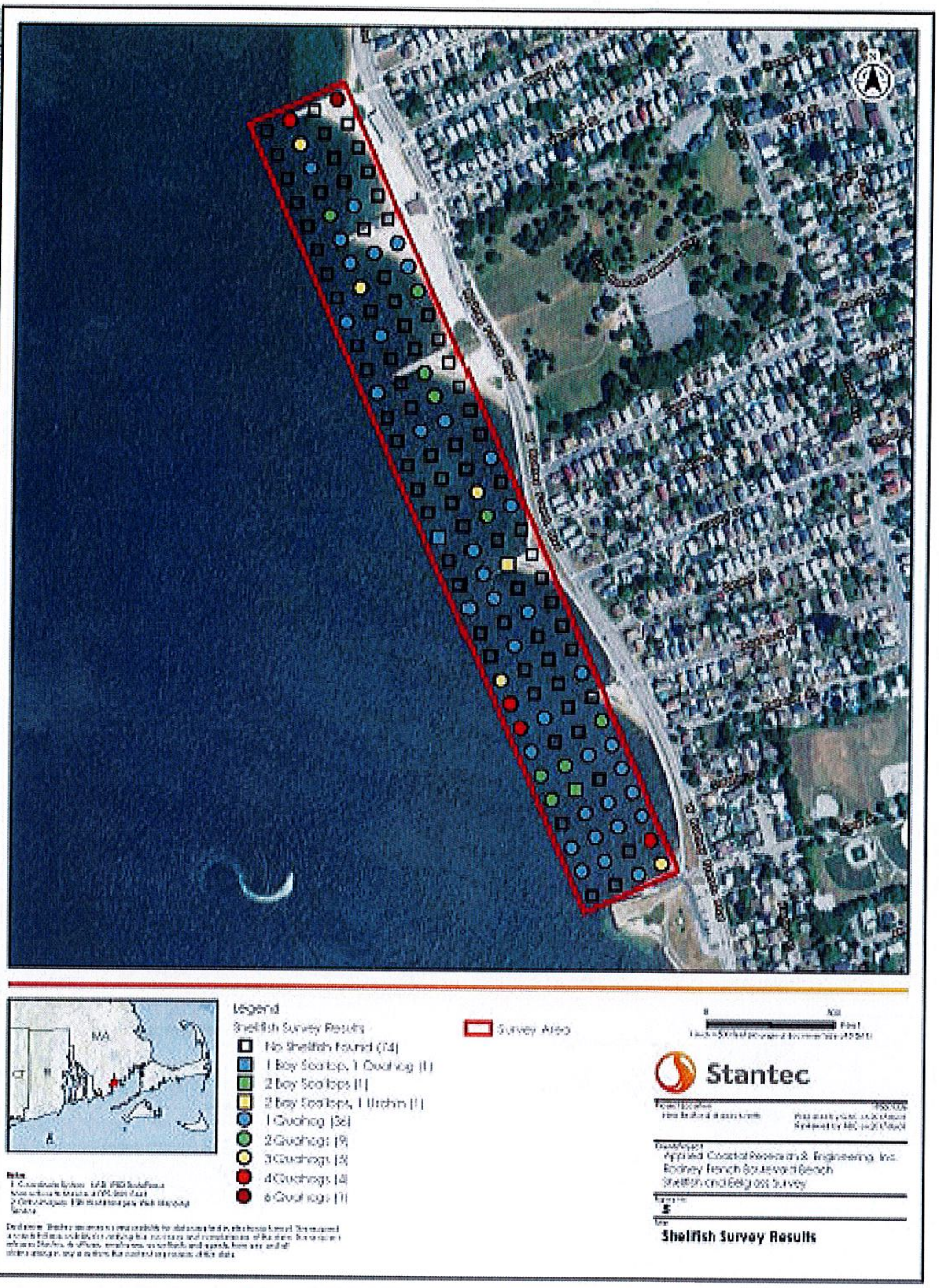


Figure 5.4 Stantec Shellfish Locations and Results, May 26, 2017.

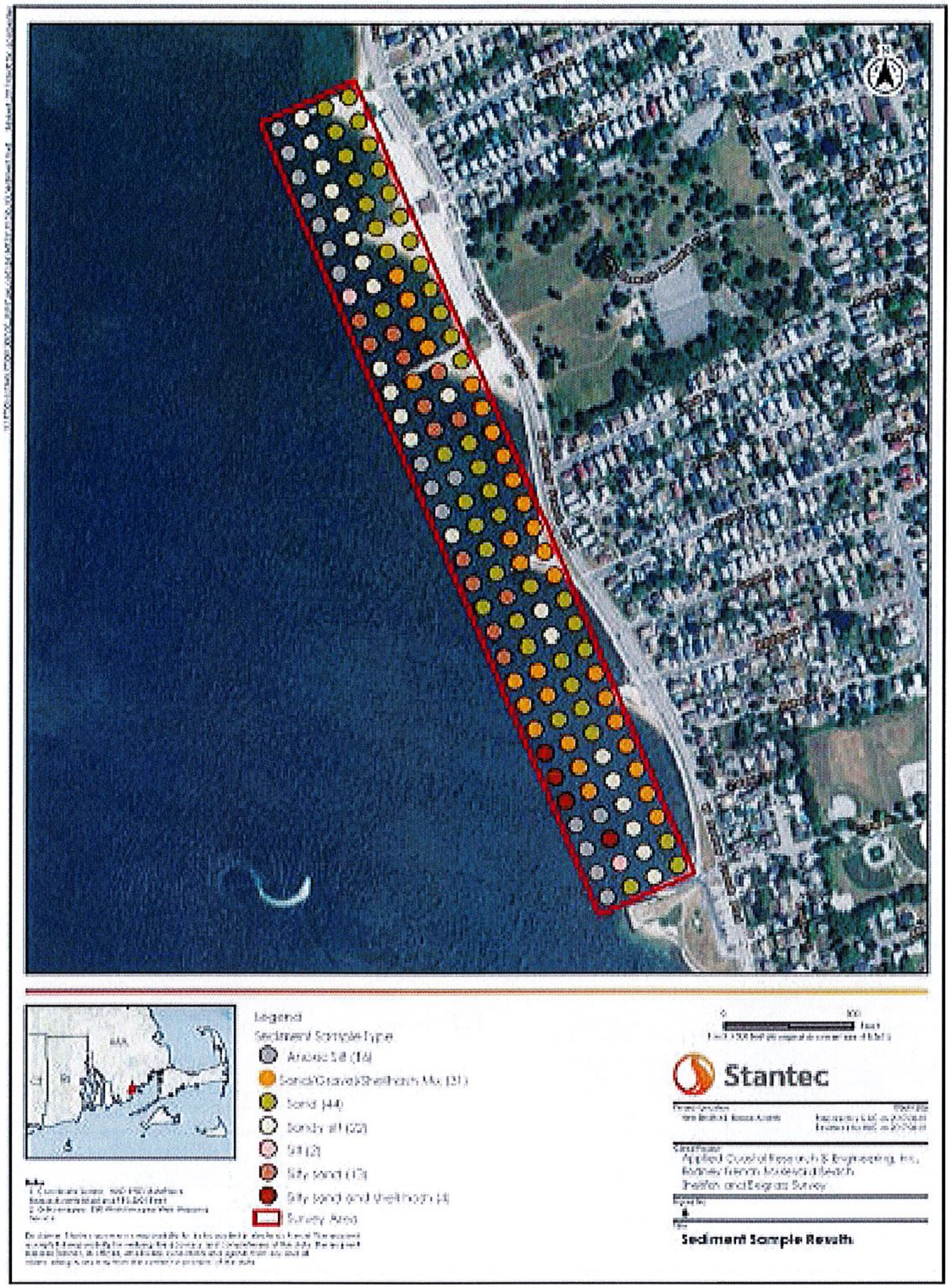


Figure 5.5 Stantec Visual Sediment Results, May 26, 2017.

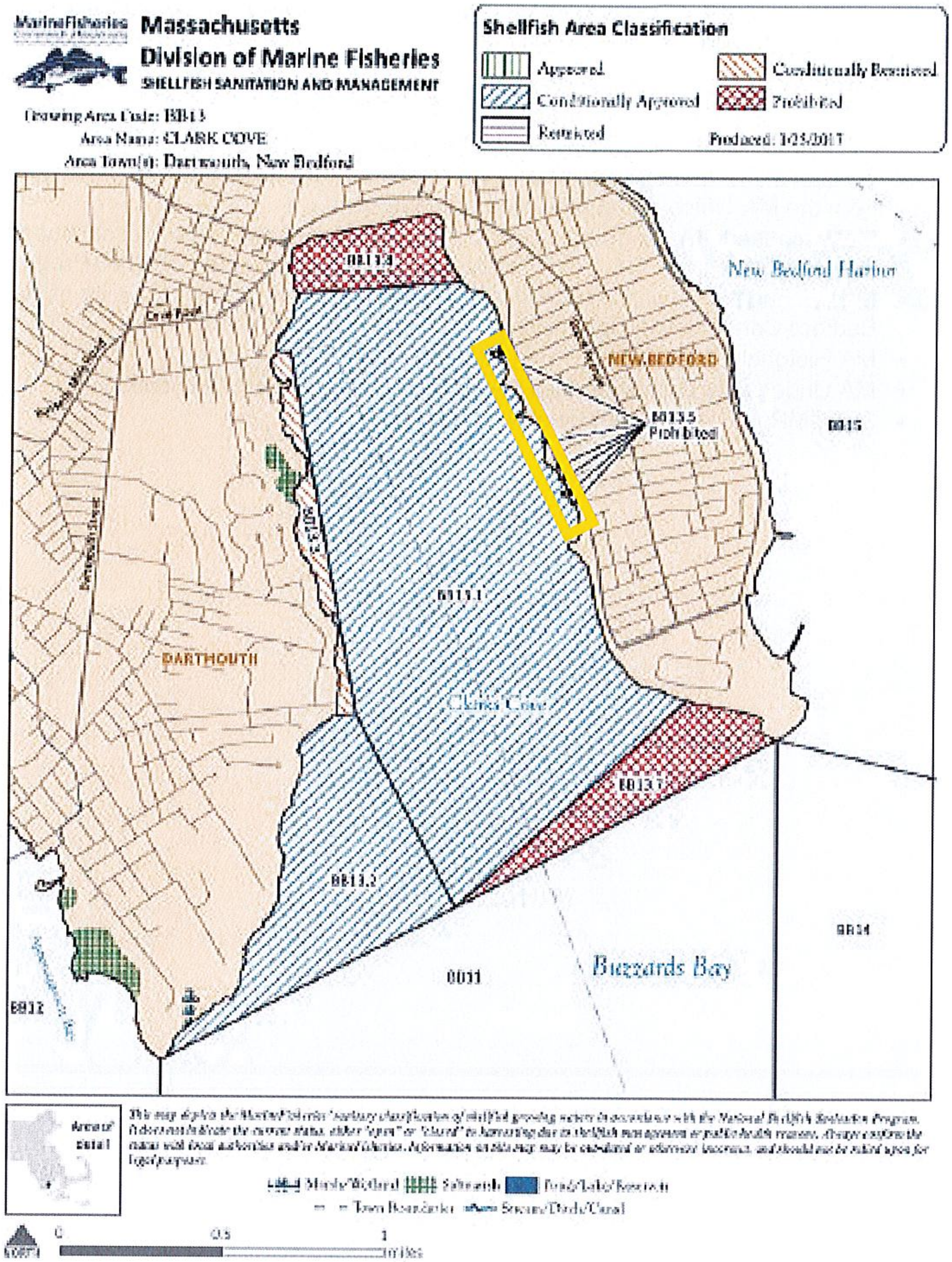


Figure 5.6 Shellfish growing areas in Clarks Cove, New Bedford, MA (MADMF, data retrieved June 2017). Yellow box represents surveyed area.

6.0 REGULATORY PERMITTING

The following federal, state, and local permits and reviews are anticipated to be required for the project:

- Federal Clean Water Act, Section 404 Permit – U.S. Army Corps of Engineers – Individual Permit, Pre-Construction Notification
- MGL Chapter 91 – Waterways Permit from Massachusetts DEP
- Coastal Zone Management Act – MA Coastal Zone Consistency Certification from the MA Office of Coastal Zone Management
- Massachusetts Endangered Species Act (MESA) Filing with the Massachusetts Division of Fisheries & Wildlife, Natural Heritage & Endangered Species Program
- Massachusetts Wetland Protection Act – Notice of Intent from the City of New Bedford Conservation Commission
- MA Historical Commission – Project Notification
- MA Underwater Archaeological Research Board – Project Notification
- 314 CMR9.00: 401 Water Quality Certification

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APPENDIX A – CLE INSPECTION REPORT

APPENDIX B – PERMIT-LEVEL PLANS

APPENDIX C – BRIGGS ENGINEERING GRAIN SIZE ANALYSIS

See attached plans.

APPENDIX D – STANTEC SHELLFISH AND EELGRASS ASSESSMENT REPORT