Mattapoisett Coastal Resilience

Protecting Mattapoisett's Potable Water and Sewer Infrastructure in the Face of Climate Change: Assessing Risk and Identifying Solutions

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Prepared By:

Lisa McStay, *RPS ASA*

Kelly Knee, *RPS ASA*

Deborah Crowley, *RPS ASA*

Nathan Vinhateiro, *RPS ASA*

Jeremy Fontenault, *RPS ASA*

Kevin Flood, *Fuss & O'Neill*

Douglas Brisee, *Fuss & O'Neill*

Project Managers:

Lisa McStay, *RPS ASA*

Kevin Flood, *Fuss & O'Neill*

RPS ASA
55 Village Square Drive
South Kingstown, RI 02879

Fuss & O'Neill
146 Hartford Road
Manchester, CT 06040

Location	End Client	Internal Reviewer(s)
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Executive Summary

Increasing storm intensity and flooding caused by sea level rise has put the Town of Mattapoisett's potable water and wastewater infrastructure at risk. The Town, situated along the coast of Buzzards Bay, is particularly vulnerable to the impacts of hurricanes. A 2013 study projected the expansion of Mattapoisett's 1-percent annual chance storm floodplain resulting from 1, 2, and 4-foot sea level rise (SLR) scenarios and showed that the expanded floodplain extended north of I-195, to inland areas that, historically, have not experienced flooding. This preliminary analysis of vulnerability to storm surge clearly demonstrated the need to further quantify climate change impacts and to implement adaptation efforts to help insure resilience in the face of increasingly severe storms and flooding.

This report focuses on the quantification of risk and the development of risk reduction actions for critical infrastructure components within the Town including the Eel Pond sewer main, the Eel Pond sewer pump station, and the No. 2 and 3 pump stations.

Using storm surge and wave actions models, as well as a shoreline change assessment, these infrastructure components were assessed and visualized under a suite of sea level rise and hurricane conditions, and recommended adaptation actions were developed to increase resilience in the face of climate change. The adaptation actions were developed both for the infrastructure components listed above and for four additional facilities identified by the town over the course of the project.

The model results were also used to develop a quantitative risk analysis using FEMA's HAZUS model to estimate the damages to infrastructure from the modeled storm surge scenarios. Town officials and residents can leverage the results of this study to identify other vulnerable properties of interest.

The primary goals of the project were:

- To improve understanding of the vulnerabilities of public infrastructure (particularly potable water and wastewater) to current flood zones, hurricane storm surge, and future sea level rise for both local officials and residents.
- To identify and prioritize risk reduction strategies to guide the town in implementing future changes in infrastructure maintenance and to mitigate the short and long-term vulnerabilities of critical facilities.

The vulnerability analysis generally suggests that the Town's critical wastewater and potable water infrastructure would be only minimally impacted for Category 1 storms. Additionally, Category 2 storms with 0 and 1 ft of SLR also minimally impact the town facilities (with the exception of the Eel Pond sewer force main). However, Category 2 storms with 2 and 4 ft of SLR, as well as all Category 3 and 4 storms, are predicted to substantially impact critical infrastructure. Increasing sea level rise (SLR) exacerbates the geographic extent of inundation and inundation depths, as well as the projected damages from the storms.

Based on the modeling analysis and typical standards for the design of wastewater infrastructure, the project team recommends that the Town plan for at least a Category 2 storm occurring at current Mean Higher High Water (MHHW) and that the Town take actions to evaluate and protect water quality infrastructure against damage at those predicted water levels. Site-specific adaptation actions were developed that the Town could undertake at each individual site to protect infrastructure from structural damages, ensure functionality during storm events, and to be prepared for SLR. Examples of these projects recommended at the sewer and water pump stations include adding on-site generators, checking structures for buoyancy and relocating if necessary, adding barrier walls, and flood-proofing doors, windows, electrical systems and air intakes. The potential construction costs for the recommended projects at the various sites range from $60,000 - $1,467,000; however, that does not include costs for the necessary additional planning, modeling, permitting, and requisite engineering design that would be necessary. Recommended adaptation actions at the Eel Pond

Through this study the Town of Mattapoisett has taken important steps towards understanding and evaluating the potential impacts and vulnerabilities to climate change. This report provides information essential for planning and prioritizing climate change adaptation actions and identifying issues requiring additional study.

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1 Introduction

The Town of Mattapoisett, situated along the coast of Buzzards Bay in the Commonwealth of Massachusetts, is particularly vulnerable to flooding caused by hurricanes currently and is predicted to be even more so with the impacts of climate change associated with flooding. In 2013, Costa et al. (2013) projected the expansion of Mattapoisett's 1-percent annual chance storm floodplain (commonly known as the 100 year floodplain) resulting from 1, 2, and 4-foot sea level rise (SLR) scenarios. The results of which showed the floodplain increasing from 20.7% to 28.9% of the town's total land area under the most extreme 4 foot scenario. This expanded floodplain showed inundation extending north of I-195, to inland areas that, historically, have not experienced flooding. The expanded floodplain contains eighteen public properties, including schools and the Town Hall as well as public utilities infrastructure (Costa et al. 2013). This preliminary analysis of vulnerability to storm surge clearly demonstrated the need to further quantify climate change impacts and to implement adaptation efforts to help insure resilience in the face of increasingly severe storms and flooding.

This project, funded by the 2015 Coastal Community Resilience Grant Program, quantifies impacts from coastal hazards including storm surge, SLR, waves, and shoreline change at critical infrastructure sites around Mattapoisett. In addition, this report provides recommendations and budgets for flood proofing the impacted infrastructures.

Initially, the following four critical infrastructure features identified in the town as being particularly vulnerable:

1. Eel Pond Sewer Pump Station
2. Eel Pond Sewage Force Main
3. Number 2 Water Pump Station
4. Number 3 Water Pump Station

Throughout the course of the project, the Town also requested engineering recommendations for four additional sites:

1. Number 4 Water Pump Station
2. Water Treatment Facility
3. Water Distribution System Crossing US Highway 6 adjacent to River Road
4. Sewage Crossing over the Mattapoisett River along Phoenix River Trail

All sites considered in this study are shown on Figure 1-1.

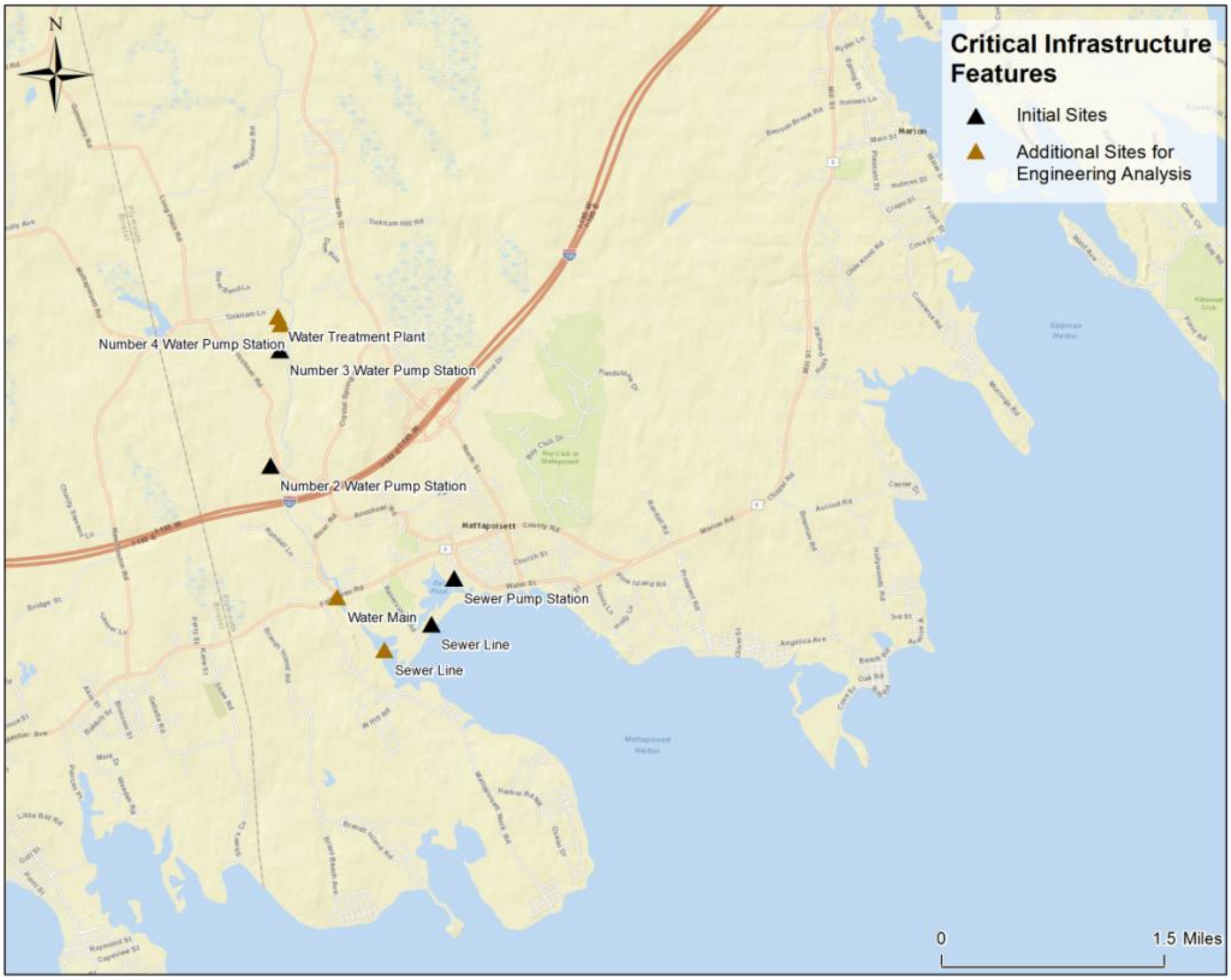

Figure 1-1: Vulnerable Critical Infrastructure Facilities and Features. The four infrastructure components initially identified are indicated by black symbols. The four additional sites selected for the engineering analysis are shown with brown symbols.

1.1 Storm Surge & Inundation Modeling

The project approach builds on previous investments by the State to better understand the vulnerabilities of Buzzards Bay communities in the face of climate change. Previous studies include *The Projected Expansion of the Floodplain with Sea Level Rise in Mattapoisett, Massachusetts*, prepared by Buzzards Bay NEP, and the *Climate Change Vulnerability Assessment for Water Quality Infrastructure for New Bedford, Fairhaven and Acushnet* prepared by SeaPlan, RPS ASA, and Fuss & O'Neill. The modeling approaches utilized in this project make use of the SLOSH and HAZUS models, which are used for planning purposes on both national and local scales and can be implemented easily and at a low cost by interested municipalities.

The project team used a methodology which leveraged storm surge data modeled from the previous project in Buzzards Bay. Waves were modeled at the Eel Pond Sewer Pump station using FEMA's Wave Height Analysis for Flood Insurance Studies (WHAFIS) software. Finally, a shoreline change assessment was conducted for the Eel Pond inlet using the USGS Digital Shoreline Analysis System (DSAS). A detailed description of modeling methodologies is provided in Section 2.

2 Methodology

2.1 Inundation Modeling

Inundation modeling was undertaken to characterize flooding for a range of storm conditions and sea-level rise scenarios. Model scenarios considered storms ranging in intensity from Category 1 to Category 4 hurricanes. In addition, scenarios with an enhanced Category 4 storm ("4+") were also considered to provide a theoretical extreme. However, the Category 4+ storm scenarios are purely hypothetical and should be interpreted as such. In fact, no hurricane exceeding Category 3 intensity has made landfall in New England in recorded history, and therefore even Category 4 storm conditions are unlikely to be seen in this region.

2.1.1 Storm Surge Modeling

The Sea, Lake, and Overland Surges from Hurricanes (SLOSH) model (Jelesnianski et al. 1992) was used to forecast hypothetical storm surges for Buzzards Bay under current and future conditions. SLOSH was developed by the NOAA/National Weather Service Meteorological Development Laboratory and is used operationally by NOAA's National Hurricane Center. SLOSH includes a surface wind model and can simulate overtopping of barrier systems, levees, and roads, flow through barrier gaps, and inland inundation.

The three main components of the SLOSH model inputs are the SLOSH basin, which provides the computational grid for the project area, a matrix of hurricane parameters, and the base water level. The hurricane parameters include the pressure deficit, the radius of maximum winds, landfall location, forward speed, and track direction. The model uses the hurricane parameters for each model scenario to calculate water in each model grid cell at each model time step.

The Providence/Boston (PV2) basin was used for modeling in this project. The grid, with a center point is between Providence and Boston, is used operationally by NOAA's National Hurricane Center. The highest resolution grid cell is located in the Narragansett Bay and is 0.2 NM. Of the basins available for the region, the PV2 basin is the most recent and offers the highest resolution. The vertical elevations in the PV2 basin are relative to NAVD88. The SLOSH PV2 grid domain is shown below in Figure 2-1.

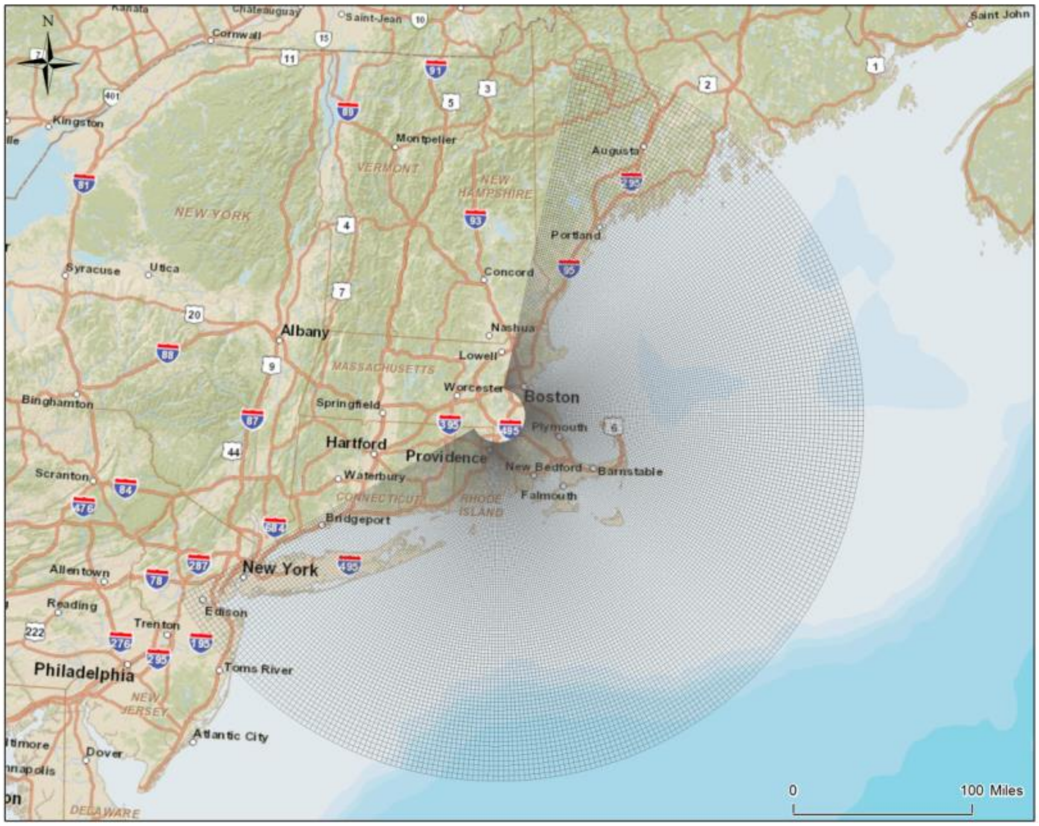

Figure 2-1: SLOSH Providence-Boston PV2 grid

The matrix of hurricane parameters modeled in SLOSH was developed from the catalog of storms used by NOAA to generate the composite Maximum Envelope of Water (MEOW) and Maximum of MEOWs (MOM) products produced for each hurricane category for the PV2 model basin. To account for the uncertain impacts of climate change on hurricane intensity, the ranges of pressure deficit, forward speed, and radius of maximum winds were all expanded. In addition, a sensitivity analysis was completed to determine the parameters that were most influential in causing high water levels in the area of interest. The analysis revealed that the largest pressure deficit (i.e., storm category) and fastest forward speeds lead to the largest storm surges. The sensitivity analysis also showed that additional sampling of the radius of maximum winds and the track direction should be included in the matrix.

Table 2-1: Final matrix of hurricane parameters used as SLOSH model inputs. Values that are bolded were added to the matrix based on the sensitivity analysis or feedback from stakeholders from the New Bedford, Acushnet, and Fairhaven project.

Parameter	Values	# Variations
Landfall Location	Evenly Spaced along the shoreline	**12**
Pressure Deficit (ΔP)	20, 40, 60, 80, **90** mb	**5**
Radius of Maximum Winds (R)	**20**, 30, **40**, 45, **50, 55** NM	**6**
Forward Speed (T)	20, 30, 40, 50, 60, **70** mph	**6**
Track Direction (θ)	N, NNE, NNW, NW, **NtW, NWtW, NtE**	**7**
Matrix Total Cases		**15,120 per water level** **60,480 total**

The base water level input to SLOSH is typically defined as a tidal elevation. For the purpose of this study, four base water levels were used: current conditions and three SLR scenarios. A tidal elevation alone, and in combination with 1, 2, and 4 ft of SLR, were used for each set of hurricane parameters. According to the National Climate Assessment, global sea levels are projected to rise anywhere from 1 to 4 feet by 2100 and SLR in the Northeast US is expected to exceed the global average (Walsh et al. 2014). The current conditions used in SLOSH modeling were defined as mean higher high water (MHHW). MHHW is the average of the higher high water of each tidal day and thus represents areas that are, on average, wet one per day. Although there are two NOAA CO-OPS stations in the project area, the New Bedford sub-ordinate station was selected as it is referenced to NAVD88. However, this station is linked to the Newport, RI station and therefore the Newport data was transformed to data at the New Bedford station using a 1.05 multiplicative relationship calculated by NOAA using a series of simultaneous relationships between the two stations. The four water levels used to initialize SLOSH in this study are provided in Table 2-2.

Table 2-2: Base water level inputs used in the SLOSH model. The tidal elevation was defined as MHHW for each water level scenario. Location names and associated numbers refer to the tide predictions stations.

Tidal Elevation	Newport, RI (8452660)	New Bedford, MA (8447584) Offset = Newport*1.05	New Bedford, MA (8447584) with SLR		
	FT Relative to NAVD88		1 ft	2 ft	4 ft
MHHW	1.81	1.9005	2.9005	3.9005	5.9005

A total of 60,480 storm tide grids were created in running all combinations of hurricane parameters presented in Table 2-1 under the four base water level scenarios, given in Table 2-2. To summarize the model output, the results were aggregated by hurricane category (Categories 1 – 4 and extreme 4, based on pressure deficit parameters of 20, 40, 60, 80, and 90 mb, respectively) and base water level scenarios (0, 1, 2, 4 ft SLR) to create 20 summary grid based on the MOM approach used by NOAA. The MOM approach uses the maximum storm tide for each grid cell from a group of model results to represent a worst case snapshot. Thus, the resulting grid does not represent the storm tide from one particular event; rather it provides a worst-case water level elevation for each location (grid cell). The results of this aggregation approach are 20 summary grids showing the worst case water level in each grid cell for all combinations of base water levels and hurricane category.

It is important to note that the SLOSH modeling conducted for this project does not account for several potentially significant effects, including the presence of waves, flooding due to precipitation, and riverine flow. A separate wave modeling study (Section 2.3) was conducted for the infrastructure component most proximal to the coastline to characterize the effect of waves.

2.1.2 Depth Grid Processing

A Digital Elevation Model (DEM) was created for use in this project that covered the entire extent of the Town of Mattapoisett. Existing ground surface elevations were derived from the following three data sources available publically on the MassGIS website (MassGIS 2015):

- 2011 USGS Northeast LiDAR (Massachusetts State Plane Feet)
- 2006 Plymouth County 4 ft LiDAR (Massachusetts State Plane Feet)
- 2013 NED 1/3 arc-second LiDAR (NAD83)

A section of the Plymouth LiDAR dataset had to be edited to remove bad data artifacts. The NED data was extracted for the missing region in Mattapoisett using a buffer of approximately 150 ft buffer. The three datasets were then merged in ArcGIS using the Mosaic Blend tool in the Spatial Analyst extension.

The 20 summary grids created from the SLOSH outputs were post-processed in ArcGIS to create depth grids. Using the 20 summary grids, center points of each grid cell were calculated, and then interpolated onto a higher resolution grid (corresponding to the LiDAR DEM) using an inverse-distance-weighted method. The LiDAR DEM elevations were then subtracted from the higher resolution grid of water surfaces to create depth grids. Thus, the depths represent the water surface elevation minus the ground elevation. This concept is shown schematically in Figure 2-2 (without wave action).

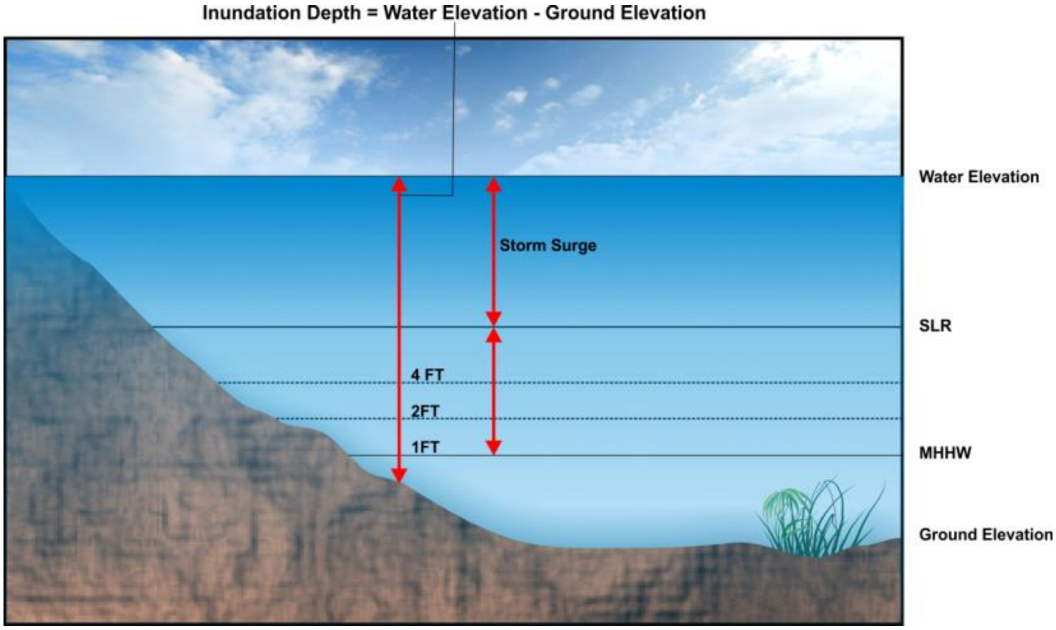

Figure 2-2: Illustration of "inundation depths"

The depths were then assessed using a series of ArcGIS-based scripts which account for spatial variation inherent in storm surge output and which remove areas of hydraulically disconnected flooding.

2.2 Risk Visualizations

Two-dimensional inundation maps were created for the entire Town of Mattapoisett using the inundation depth grids described in Section 2.1.2. These maps show both the spatial extent and depth of flooding (in ft) for each storm category and SLR scenario (20 total). Overall, inundation is limited geographically to the town's boundaries. The inundation maps for all 20 scenarios are provided in Appendix A.

Three-dimensional site-specific visualizations were created for the three critical facilities identified in Section 1.1. RPS ASA used field surveys provided for the Engineering Analysis (Section 2.5) and contracted with Growe Geospatial for the construction of the three-dimensional buildings.

Growe Geospatial created photo-realistic buildings using Trimble SketchUp, which were then imported into ArcGlobe. Water surface layers corresponding to the maximum flooding at the site were created, clipped to the inundation extent for each scenario were created, and overlain on the building in ArcGlobe. Images for all sites and scenarios are provided in Appendix A.

2.3 Wave Modeling

Due to its proximity to the coastline, wave modeling was conducted for the Sewer Pump station along Eel Pond. The purpose of this modeling was to determine wind wave effects coincident with the maximum storm surge for each of the 20 scenarios defined above.

The WHAFIS (Wave Height Analysis for Flood Insurance Studies) model was used for this study. WHAFIS 4.0 is a DOS-based, one-dimensional, overland wave propagation model. It is used by FEMA to predict wave heights associated with hurricane storm surge for delineation of coastal high-hazard zones for Flood Insurance Studies (FEMA, 1988).

User-defined transects specify topographic, vegetative, and cultural features such as obstructions. WHAFIS uses the transect information, in combination with information including wind speed, wave setup, and incident wave height and period to calculate wave heights, wave periods, wave crest elevations, flood insurance risk designations, and flood zone boundaries along each transect. If incident wave conditions are not specified, WHAFIS computes these values using input wind speeds by applying either fetch or depth limited methodologies. WHAFIS calculated incident wave conditions do not include the effects of wave refraction, wave diffraction, or dissipation due to bottom friction (FEMA, 2005).

The 10 meter, 10-minute sustained wind speed required for the wind wave effects analysis is calculated from the 10 meter, 1-minute average wind speed output from SLOSH. The wind speed adjustment methodology outlined in the U.S. ACE Coastal Engineering Manual, Part II, Chapter 2 Meteorology and Wave Climate (U.S.ACE, 2008) was used to convert between the 1-minute and 10-minute wind speed. The incident wave height and period were assumed based on guidance presented in this manual pertaining to local seas generated by various meteorological phenomena (Table II-2-2 in the USACE CEM). The transect data incorporated setup by assuming that the surf zone was coincident with the shoreline and as such assigned the breaking wave depth to the water depth at the shoreline. The maximum setup ($\mathrm{n_{max}}$) was calculated using:

$$\mathrm{n_{max}} \cong 0.232 * \mathrm{H_b}$$

Where $\mathrm{H_b}$ is the breaking wave depth. This approximation is based on the Guidelines and Specifications for Flood Hazard Mapping Partners (FEMA, 2005) and was applied uniformly across the landward transect points.

The output provided from the WHAFIS modeling includes both the controlling wave height and crest elevation above the input still water elevation (SWEL). The controlling wave heights were calculated in the model and represent the highest 1 percent of waves during the modeled conditions. The controlling wave heights were transformed to the significant wave height using based on the relationship that controlling wave height equals 1.6 times the significant wave height (U.S.ACE, 2008). The controlling wave height is limited in the WHAFIS model to 78 percent of the local SWEL and the model assumes that 70 percent of the controlling wave height lies above the SWEL (FEMA, 1988). The concept of inundation depths including wave effects (from the controlling wave height) is shown in Figure 2-3.

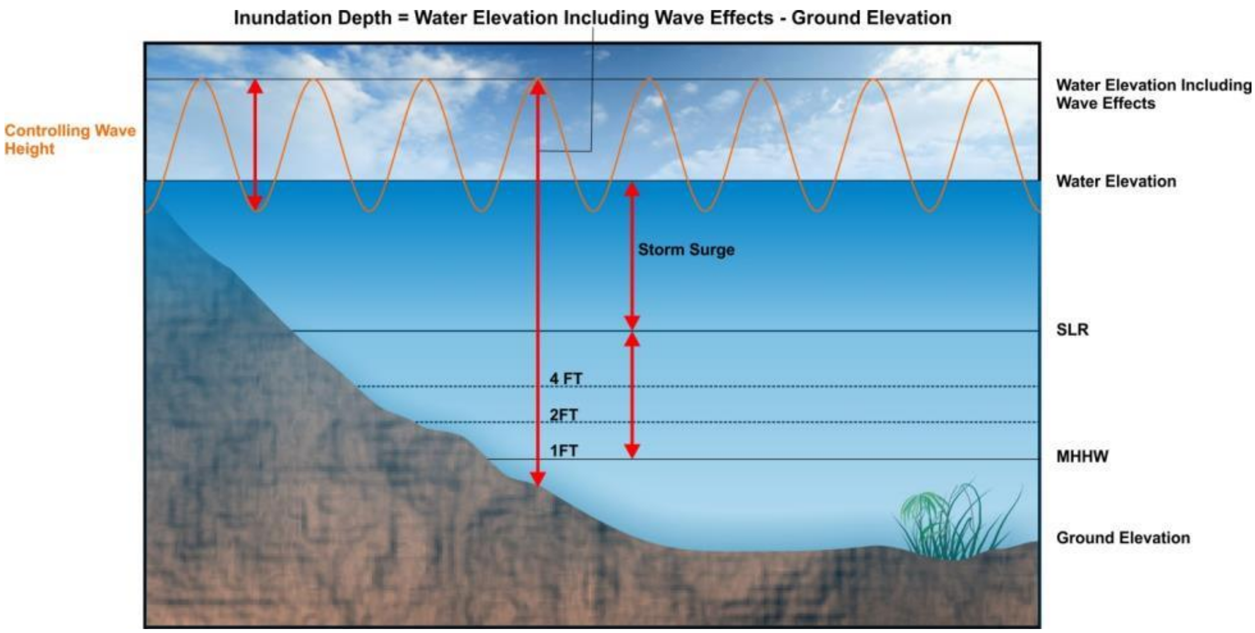

Figure 2-3: Illustration of "inundation depths" including wave effects.

2.4 Shoreline Change Assessment

Mattapoisett, a town with an extensive ocean-facing coastline, has been exposed to chronic beach erosion throughout time. The western inlet that connects Eel Pond to Mattapoisett Harbor initially formed in the 1960's after a coastal storm breached the beach and salt marsh. Since that time the position of the inlet (called the West Channel) has migrated south and west in response to storms and the prevailing alongshore transport.

The potential for future shoreline change is of concern for the Town of Mattapoisett due to the presence of a twelve-inch diameter sewage force main, installed in 1977, which traverses the barrier beach and crosses the channel. While the submerged portion of the pipe is encased in concrete, the inlet has continued to migrate toward the shallower, uncased portion of the sewer line since its installation. The locations of the encased/uncased portions of the sewer line were obtained from the town. These locations were provided to the project team by the Town via the Buzzards Bay National Estuary Program (BBNEP) and should be considered approximate.

A diagram showing the approximate locations of the encased and uncased portions of the sewer line and the terminology used to describe the banks in subsequent text is provided below in Figure 2-4.

Figure 2-4: Diagram of the sewer line along the Eel Pond Inlet. Data showing locations of the force main and encasement was provided by BBNEP.

In order to help provide guidance for the engineering recommendations regarding the encasement of the sewer main, the potential changes in the configuration of the inlet mouth and adjacent barrier beach were evaluated. The project team used the USGS Digital Shoreline Analysis System (DSAS; Thieler et al., 2009) and historic erosion/accretion rates to forecast shoreline position envelopes near the West Channel to time horizons of 25, 50, and 100 years into the future (from 2015). The shoreline change envelope provides the polygon extent of possible shoreline locations, produced by combining all of the predicted shorelines calculated for a particular time horizon.

As this method relies on “snapshots” of shoreline positions derived from historic imagery, and because processes responsible for shoreline change occur on many different timescales (e.g. episodic storms, long-term changes), results should only be considered as first-order estimates of future erosion/accretion. Actual shoreline changes may differ due to storm frequency, wave climate, rate of sea level changes, changes in sediment supply, and coastal engineering. Thus, further analysis should be completed to fully and adequately characterize the risk to the Town's infrastructure.

DSAS is a freely-available GIS-based computer software program that calculates rate-of-change statistics from a time series of multiple shorelines. DSAS works by generating orthogonal transects at user-defined intervals and calculates the rate-of-change based on either end point rate (EPR), linear regression (LRR), or weighted linear regression (WLR) methods. The analysis completed for the Eel Pond Inlet included the following steps:

- Historic shorelines (digitized by the USGS and the Buzzards Bay National Estuary Program [BBNEP]) were imported to ArcGIS. Shorelines digitized from imagery in 1978, 1991, 1994, 1997, 2001, 2003, 2009, 2010, 2012, and 2014 were included in the analysis.
- All vector shorelines were merged into a single feature class and attributed with a date and uncertainty field. Datasets that did not include documented uncertainties were assigned a value of 5.5 m, which corresponds to the average uncertainty in the position of the high water line derived from air photos by Hapke et al., (2010).
- DSAS was used to generate orthogonal transects at 1 m spacing alongshore. Transects were cast from an inland baseline along either side of the West Channel (Figure 2-5).

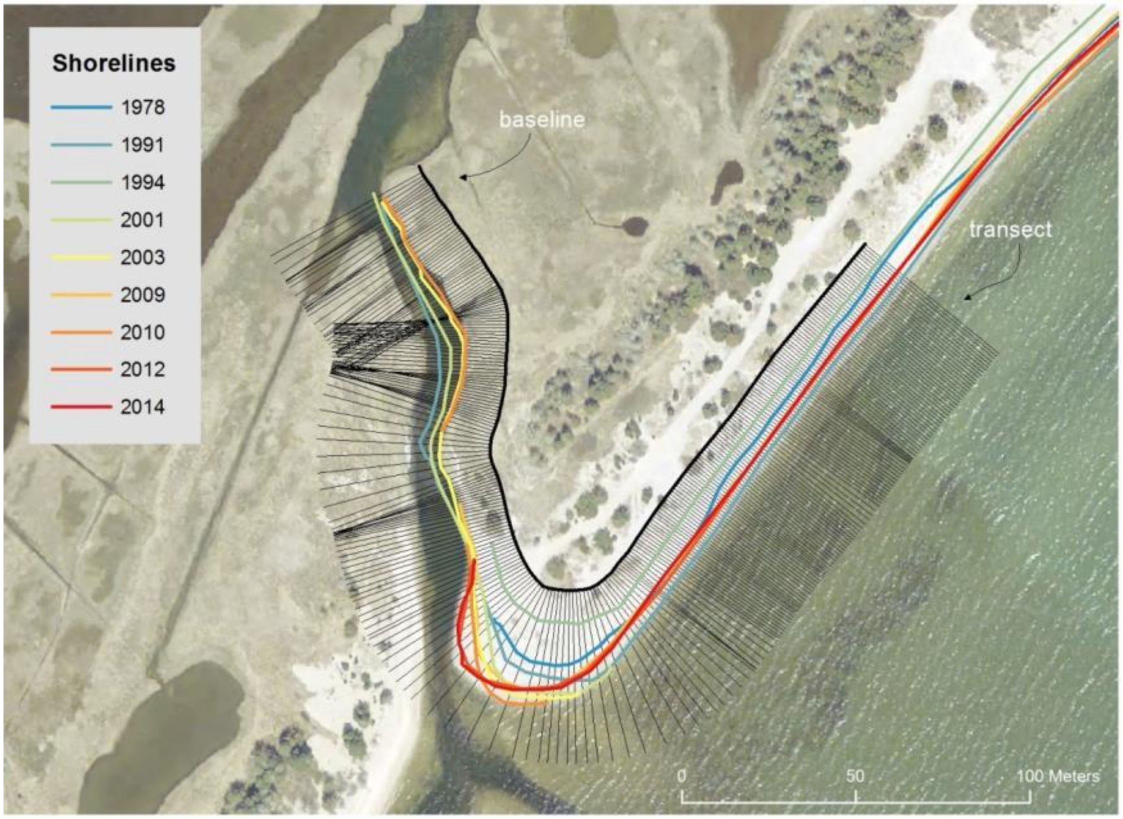

Figure 2-5: Vector shorelines, baseline, and transects used by DSAS to compute shoreline changes on the north bank of the West Channel.

- Rates of erosion/accretion at each transect were calculated using DSAS. Rates of shoreline change were calculated using the end point rate (EPR), linear regression (LRR), and weighted linear regression (WLR) methods (Figure 2-6).

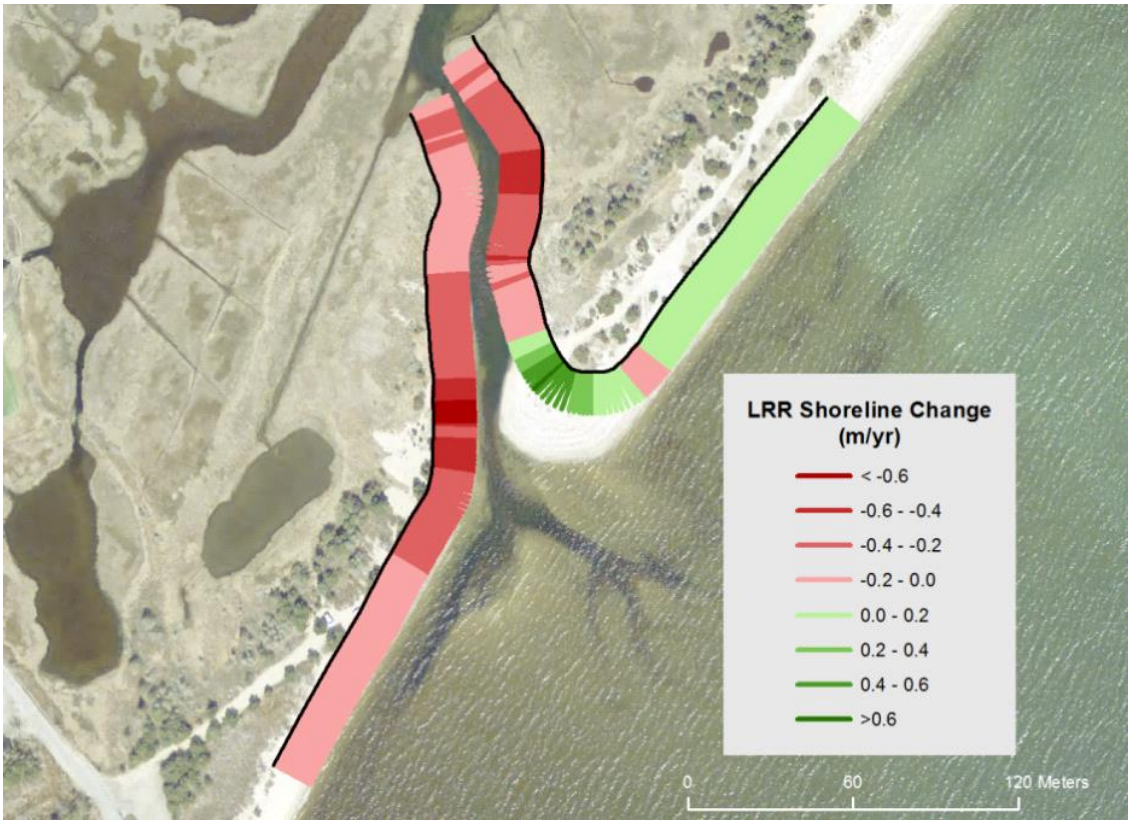

Figure 2-6: Example showing LRR shoreline change rates computed at each transect using DSAS. Green shows zones of shoreline accretion; red indicates erosion.

- Future shoreline positions were then forecast at each transect at 25, 50, and 100 years from 2015 based on the various rates. A distance weighting function was used for shoreline positions between vertices. New polyline features were developed for each time horizon (3), and rate method (3), (9 total layers).
- A polygon represented a total envelope of change was produced for each time horizon based on the range of calculated rates (EPR, LRR, WLR) at each transect.

2.5 Quantitative Risk Analysis

As part of this study, a quantitative risk analysis was performed for the Town of Mattapoisett under the suite of storm conditions defined in Section 2.1. This analysis utilized the GIS-based Federal Emergency Management Agency (FEMA) Hazus-MH Flood model, which includes a nationally applicable methodology for estimating losses from natural hazards. Thus, the model can be effortlessly applied to any local municipality in the county. However, it is important to note that because the predefined database is aggregated on a national level using Census information, the data is coarse and may be out of date. The output of the model illustrates extents of high risk areas and summarizes populations, as well as assets at risk.

Hazus was applied as part of the quantitative risk analysis to each of the twenty storm scenarios individually using the following methodology:

1. The inundation hazards were defined as the grids of inundation depths produced as part of the inundation modeling for all combinations of storm and SLR scenarios. These grids were exported from ArcGIS in the native ESRI GRID format for use in the model and imported into the Hazus program.
2. The Hazus program includes a single, fully integrated set of functions, default inventory, and reporting functions that were used for this analysis. This default database, which includes general building stock and essential facilities, was used to define the inventory of assets to include in the assessment.
3. Damages were determined by intersecting inundation with assets and losses that were estimated using default functions predefined within the program.

2.6 Engineering Analysis

Engineering evaluations of eight critical water and wastewater facilities in the Town were also performed by Fuss and O'Neill. The facilities evaluated included (see Figure 1-1 for location map):

1. Eel Pond Sewer Pump Station
2. Eel Pond Sewage Force Main
3. Number 2 Water Pump Station
4. Number 3 Water Pump Station
5. Number 4 Water Pump Station
6. Water Treatment Facility
7. Water Distribution System Crossing US Highway 6 adjacent to River Road
8. Sewage Crossing over the Mattapoisett River along Phoenix River Trail

The evaluation included site visits to each location and collection of elevations at the facilities for the exterior components, building, and critical equipment. From this information floor plans were developed for the different facilities, manufacturer's information for critical equipment was collected, deficiencies or concerns for those items that could be damaged or lost during a flood event were identified, and recommendations to improve the resiliency of each facility in the event of various flood events were provided.

The results from the coastal hazard analysis (storm surge and wave action modeling) were used to compare existing conditions to the flooding results for the various storm conditions. The purpose of these analyses was to determine the height of floodwater above the ground elevation of each facility or water body in the case of crossings so that potential options for mitigation/adaptation could be evaluated.

The evaluation of each facility utilized Category 2 and Category 3 storms as the basis for developing adaptation/mitigations measures and budgetary opinions of cost. These storms were identified as representative as they are the most severe storms experienced historically and more severe storms would result in inundation levels that are so significant that they cannot be cost effectively protected against. A preliminary list of recommendations for improvements at each facility is presented in the results section below.

The shoreline change analysis of the existing wastewater force main near the West Channel in the vicinity of Eel Pond (Section 3.4) was used to determine the erosion and migration of the shoreline and how this affects the existing force main.

3 Results

3.1 Inundation Modeling

The results of the inundation modeling showed that inundation at each of the facilities increased with increasing storm intensity. In general, the impacts from Category 1 storms with any of the modeled SLR scenarios are minimal and do not cause inundation that impacts any of the three facilities assessed. Additionally, the impacts from Category 2 storms are likely to be minimal throughout the majority of the town, with the exception of the locations along the coastline (e.g., the Eel Pond Sewer Pump Station). As sea level rise approaches 4 ft, the impacts associated with a Category 2 storm, increase substantially. Predicted inundation depths calculated for the three critical facilities (excludes the Eel Pond Sewer crossing) are provided in Table 3-1.

Table 3-1: Inundation depths (rounded to the nearest foot) at the three critical infrastructure facilities, without the addition of wave action. Empty cells (grayed out) represent scenarios that did not yield any inundation at the site.

	Depth of Inundation (ft)																			
Storm Category	No SLR					1 ft SLR					2 ft SLR					4 ft SLR				
	1	2	3	4	4+	1	2	3	4	4+	1	2	3	4	4+	1	2	3	4	4+
Eel Pond Sewer Pump Station		<1	6	10	13		1	7	11	14		2	7	12	15		4	9	14	17
No. 2 Pump Station			4	11	13			5	12	14		1	7	12	15		3	9	14	16
No. 3 Pump Station			2	7	10			3	9	11			4	9	12		<1	6	11	13

Overall, the analysis indicates that the storm factors that lead to the highest water levels in the Mattapoisett region were:

1. Landfall in either Eastern Connecticut or Rhode Island,
2. An angle of approach between 168 and 180° from North (storms headed towards the NtW to N directions),
3. A radius of Maximum Winds of 40 to 50 NM, and
4. A high forward speed (60 or 70 mph).

3.2 Risk Visualizations

Both two-dimensional town-wide maps and three-dimensional site specific visualizations for the three critical facilities (excludes the Eel Pond Sewer crossing) were created as part of this project.

The maps show the locations and the extent (i.e., depth) for each combination of storm category and SLR scenario (20 total). Inundation depths are indicated using a color scale, with the maximum value in each map varying in accordance with the maximum inundation depth within the town for the specified scenario. Critical infrastructure, including the four wastewater and potable water sites, is identified on each map. Maps for all scenarios, both with and without SLR, are provided in Appendix A. Three-dimensional visualizations for the three facilities (excludes the Eel Pond Sewer Crossing) and are also provided in Appendix A. Overall, inundation was limited to the town boundaries.

3.3 Wave Modeling

Wave modeling was conducted for the Eel Pond Sewer Pump Station only. As stated in Section 2.3, the WHAFIS model was used to predict controlling wave heights associated with the hurricane storm surge, Controlling wave heights (also known as 1% wave heights) represent the highest 1 percent of waves during given conditions. The controlling wave heights, as well as corresponding inundation depths, were calculated for each storm category and are presented below in Table 3-2. The resulting inundation depths were subsequently used to develop engineering recommendations for the Eel Pond Sewer Pump Station.

Table 3-2: WHAFIS-calculated Controlling Wave Heights (1%) and corresponding Inundation Depths (including wave effects) for the Eel Pond Sewer Pump Station under a suite of storm categories modeled for Buzzards Bay . Empty cells (grayed out) represent scenarios that did not yield any inundation at the site.

Controlling Wave Heights and Corresponding Inundation Depths (ft)																				
Storm Category	**No SLR**					**1 ft SLR**					**2 ft SLR**					**4 ft SLR**				
	1	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**
Inundation Depth (no wave effects)		<1	6	10	13		1	7	11	14		2	7	12	15		4	9	14	17
Controlling Wave Height (ft)		2	7	11	14		3	8	12	15		4	8	13	15		5	10	14	17
Inundation Depth[1]		4	14	22	28		5	15	24	29		7	16	25	31		11	19	29	34

[1] Includes wave effects – 70% of the controlling wave height is superimposed on top of the existing inundation depths plus wave effects, as defined by the definition of controlling wave height in Section 2.3.

3.4 Shoreline Change Assessment

Imagery collected between 1978 and 2014 was initially qualitatively analyzed for general shoreline change trends. This collection of imagery showed a general drift in position of the West Channel toward the southwest (eroding on the southern bank and accreting on the northern bank). The shorelines extracted from the imagery along the north bank are shown in Figure 3-1.

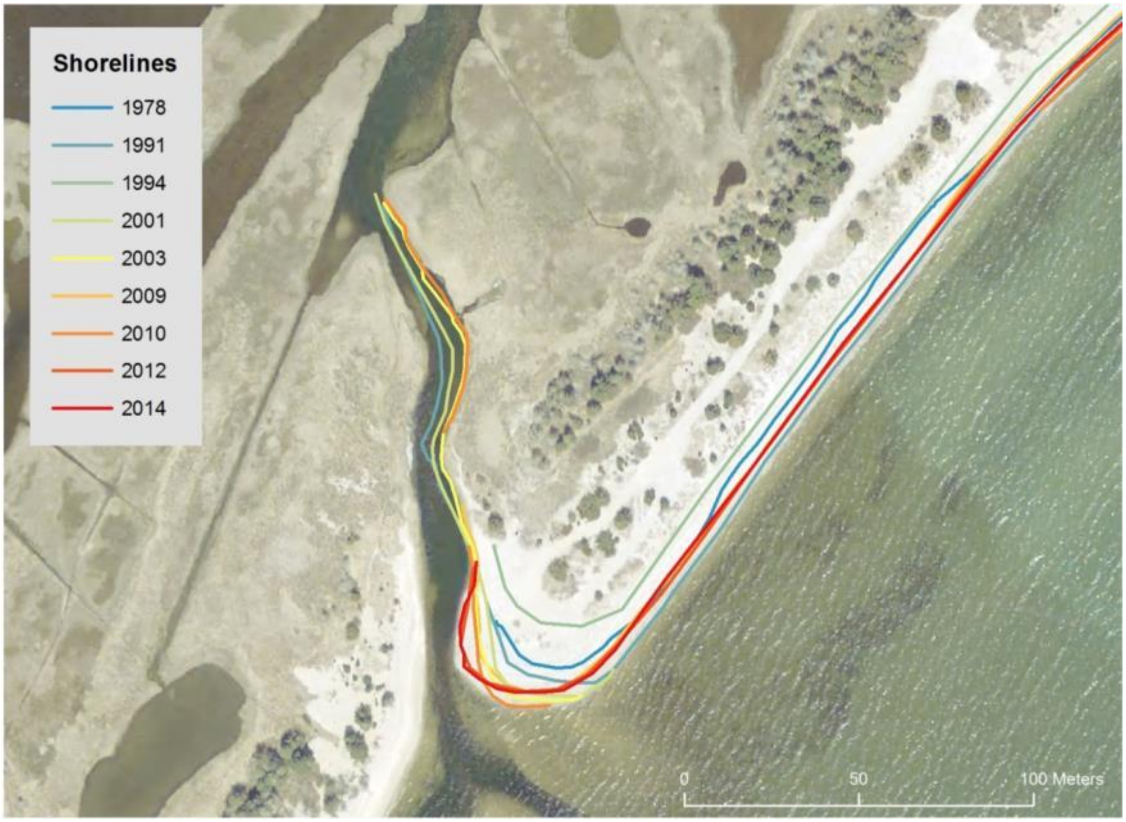

Figure 3-1: Shorelines extracted from imagery from 1978 to 2012 showing general drift in the position of the West Channel towards the southwest.

As stated in Section 2.4, future shoreline positions at time horizons of 25, 50, and 100 years (from 2015) were forecast based on the various rates described (EPR,LRR,WLR). An example showing the projected shoreline at the 25 year time-horizon using the LRR rate of change method is provided in Figure 3-2.

Figure 3-2: Projected shoreline configuration at the 25-year time horizon using LRR rates of change. The position of the sewer force main (including extent of armoring) is based on plans (data provided BBNEP).

Polygons were then created representing a total envelope of change for each time horizon. The total envelope of shoreline change shows the distance between the furthest and closest shoreline to the baseline, thus representing the total change in shoreline movement for the specific time horizon. Figure 3-3, Figure 3-4, and Figure 3-5 show the polygons representing the total envelope of change at the 25, 50, and 100 year time horizons.

Figure 3-3: Envelope of potential shoreline positions at the 25-year time horizon based on the range of calculated shoreline change rates.

Figure 3-4: Envelope of potential shoreline positions at the 50-year time horizon based on the range of calculated shoreline change rates.

Figure 3-5: Envelope of potential shoreline positions at the 100-year time horizon based on the range of calculated shoreline change rates.

In general, the results of the shoreline change assessment showed that the rates of shoreline change in the area of the Eel Pond are highest at the inlet mouth. The maximum rate of shoreline erosion is -0.66 m/yr (along the south bank) and maximum rate of accretion is +0.67 m/yr (along the north bank). DSAS transects that intersect with the sewer main indicate shoreline movement at an approximate rate of 0.5 m/yr (along the north bank) and -0.3 m/yr (along the south bank) in the vicinity of the pipe.

As discussed above, results of the shoreline change assessment should be considered only as first-order estimates of future erosion and accretion. The configuration of the future shoreline may differ due to storm activity, wave climate, the rate of sea level rise, sediment supply, and presence of coastal structures. Potential changes from a short-term, catastrophic event may not be fully captured in this assessment because (i) few major storms have impacted the site during the period of observation, and (ii) the full beach morphology is not considered in when assessing shoreline changes using GIS vector data. It is important to note that the beach and frontal dunes on either side of the West Channel have exceedingly low volume, and as a result, the sand reservoir is likely to become depleted by wave action (or overwash completely) during a moderate to severe storm, exposing a much longer section of the sewer line. Therefore, additional analyses should be completed to adequately characterize the risk to the sewer force main.

To that end, the Federal Emergency Management Agency (FEMA) has developed criteria for evaluating a dune – referred to as the "540-rule" – to determine if it is considered an effective barrier to storm surges and associated wave action during the base flood event (100-year storm). The FEMA 540-rule definition states: "primary frontal dunes will not be considered as effective barriers to base flood storm surges and associated wave action where the cross-sectional area of the primary frontal dune, as measured perpendicular to the shoreline and above the 100-year stillwater flood elevation and seaward of the dune crest, is equal to, or less than, 540 square feet" (see Figure 3-6). Several communities have further revised this guidance to establish a "1100-rule" suggesting that frontal dunes will only be effective barriers for 100-year storms events if the sand reservoir is 1,100 square feet or greater.

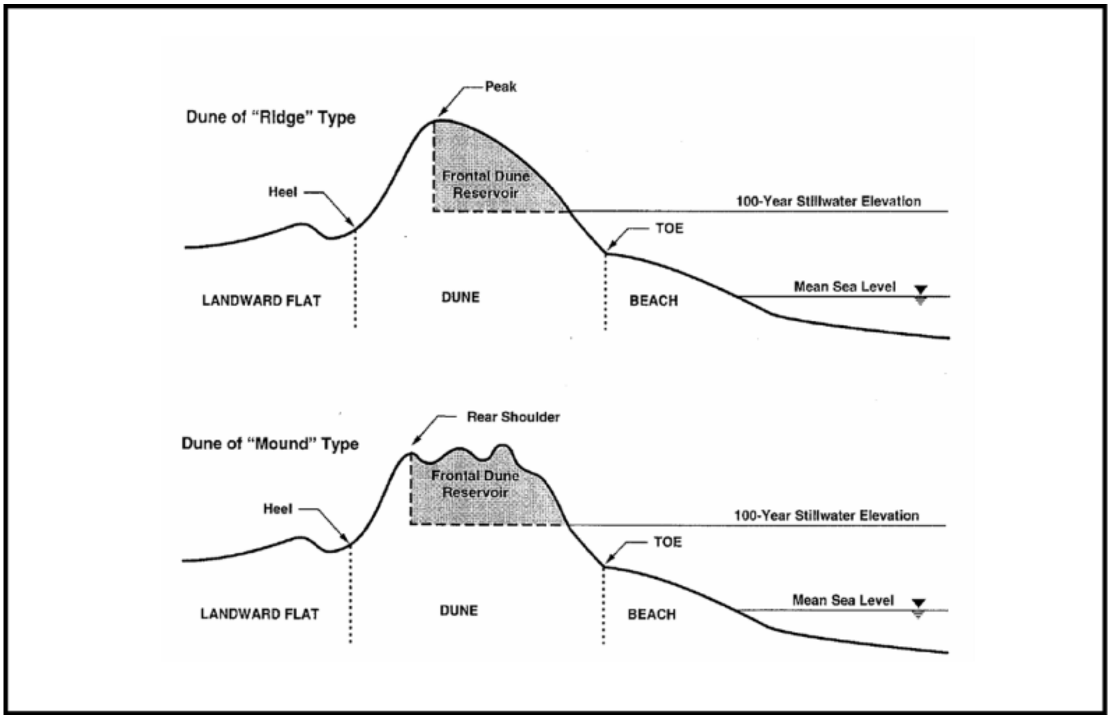

Figure 3-6: Schematic showing the factors considered when determining dune failure potential and subsequent flood zone mapping (FEMA, 2003).

The stillwater flood elevation (SWEL) at FEMA transect 195, which crosses the beach approximately 1,000 ft south of the inlet, was compared to cross-shore profiles on either side of the inlet (Figure 3-7). Ground elevations referenced to NAVD88 were derived from the digital elevation model developed for this project. The 1-percent annual chance stillwater elevation at this location as estimated by FEMA is 12.8 ft NAVD88 (Plymouth County FIS). As shown in Figure 3-8, this elevation exceeds the beach profiles in this area indicating that no portion of the beach should be considered an effective barrier to storm surge. The dune peak is, in fact, several feet below the SWEL, and accordingly, susceptible to total inundation in the event of a large storm. In such an event the beach and dune would likely erode to a point where some portion of the sewer main (encased or otherwise) becomes exposed.

Figure 3-7: Location of cross-shore profiles on either side of the inlet and FEMA transect used to estimate SWEL.

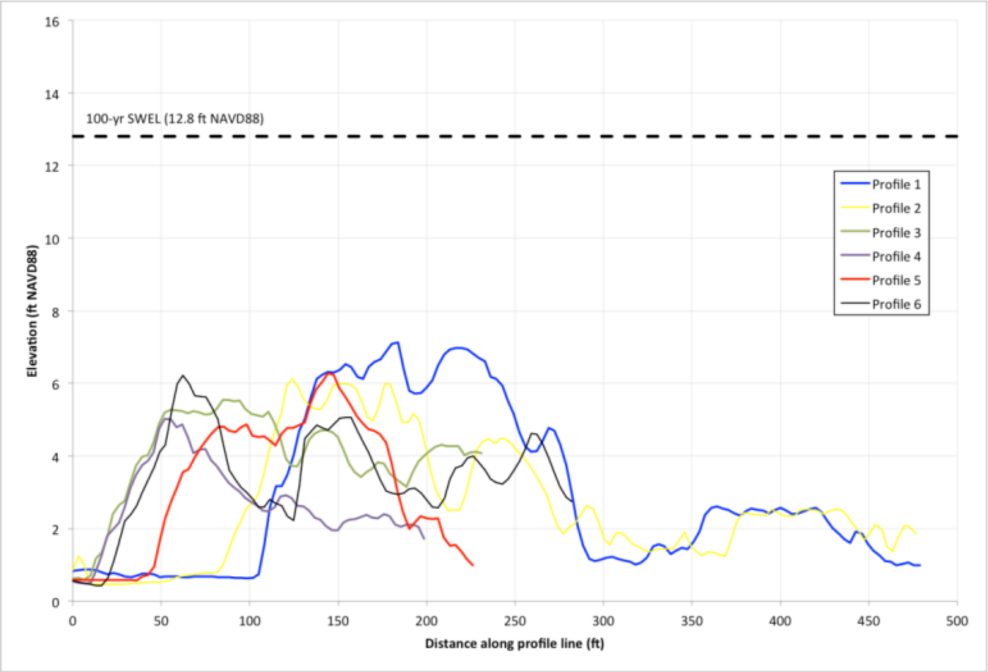

Figure 3-8: Beach profile elevations compared to SWEL at FEMA transect 195. The 1-percent annual chance stillwater elevation is several feet above the topography on either side of the Eel Pond inlet.

3.5 Quantitative Risk Analysis

The quantitative risk analysis was performed for each storm scenario individually using the Hazus-MH flood model. Each of the individual model runs resulted in large volumes of data including tables, summary reports, and spatial data. The output including damage to essential facilities and total substantial damages to buildings are summarized in the Table 3-3 and Table 3-4, respectively. Figures depicting the total economic loss, as well as the substantial damage, based on the census blocks for each of the twenty storm scenarios are provided in Appendix B.

Substantial damage to buildings can include both direct building loss and business interruption. The direct building losses are estimated costs to either repair or replace the caused damage. Business interruption losses are associated with the inability to run a business due to the damage sustained during a flood event.

As stated earlier, it is important to note that because the predefined database is aggregated on a national level using Census information, the data is coarse and may be out of date. Thus, it is recommended that the results should be treated as conservative estimates and the results should be used only to compare the scale of damages between the different storm scenarios.

Table 3-3: Damage to essential facilities (loss of use) and building-related economic loss for each of the 20 storm scenarios.

Scenarios	Damage to Essential Facilities – Loss of Use Number of Buildings			Building-related Economic Loss (Millions of Dollars)
	Fire Stations	Police Stations	Schools	
Category 1 with 0 ft SLR	0	0	0	34.2
Category 2 with 0 ft SLR	0	0	1	122.05
Category 3 with 0 ft SLR	0	0	2	232.38
Category 4 with 0 ft SLR	0	0	2	352.48
Category 4+ with 0 ft SLR	0	0	2	420.13
Category 1 with 1 ft SLR	0	0	1	47.87
Category 2 with 1 ft SLR	0	0	1	135.04
Category 3 with 1 ft SLR	0	0	2	253.87
Category 4 with 1 ft SLR	0	0	2	388.08
Category 4+ with 1 ft SLR	0	0	2	439.00
Category 1 with 2 ft SLR	0	0	1	58.94
Category 2 with 2 ft SLR	0	0	1	152.41
Category 3 with 2 ft SLR	0	0	2	274.25
Category 4 with 2 ft SLR	0	0	2	390.91
Category 4+ with 2 ft SLR	0	0	2	462.19
Category 1 with 4 ft SLR	0	0	1	98.52
Category 2 with 4 ft SLR	0	0	2	191.58
Category 3 with 4 ft SLR	0	0	2	318.02
Category 4 with 4 ft SLR	0	0	2	434.44
Category 4+ with 4 ft SLR	1	0	2	502.49

Table 3-4: Total number of substantially damaged buildings for each of the 20 storm scenarios.

Scenarios	Total Number of Buildings Substantially Damaged
Category 1 with 0 ft SLR	0
Category 2 with 0 ft SLR	0
Category 3 with 0 ft SLR	12
Category 4 with 0 ft SLR	180
Category 4+ with 0 ft SLR	400
Category 1 with 1 ft SLR	0
Category 2 with 1 ft SLR	0
Category 3 with 1 ft SLR	19
Category 4 with 1 ft SLR	281
Category 4+ with 1 ft SLR	501
Category 1 with 2 ft SLR	0
Category 2 with 2 ft SLR	0
Category 3 with 2 ft SLR	28
Category 4 with 2 ft SLR	291
Category 4+ with 2 ft SLR	561
Category 1 with 4 ft SLR	0
Category 2 with 4 ft SLR	0
Category 3 with 4 ft SLR	101
Category 4 with 4 ft SLR	455
Category 4+ with 4 ft SLR	724

3.6 Engineering Analysis

3.6.1 Existing Conditions

Existing conditions for each site were gathered for the engineering analysis to be conducted. Floor plans of each facility are provided in Appendix C.

3.6.1.1 Eel Pond Wastewater Pump Station

The Eel Pond Wastewater Pump Station is located on Goodspeed Island Road (a dirt road) just south of Depot Street on the northeast end of Eel Pond (See Figure 1-1). The facility is more than 35 years of age and is equipped with most of its original equipment excluding the pumps and their controls. It is surrounded by a chain link perimeter fence and is bounded by Eel Pond to the West and a tidal cove on the east and south of the facility. An exterior photo of the pump station is provided in Figure 3-9. The floor plan of the facility is provided in Figure C-1 in Appendix C.

Figure 3-9: Eel Pond Wastewater Pump Station

The facility is split into two sides 1) the dry side is where the electrical controls, piping, emergency power, materials, file cabinets and pumps are located, and 2) a wet side that includes the wetwell where wastewater flow enters the station and is shredded by a comminutor located on the lower level

The dry side contains all electrical equipment, emergency power equipment, control equipment, file cabinets, and chemicals located on the main level.

The intermediate level of the dry side has the discharge piping (8-inch ductile iron pipe that increases in size to 12-inches) that conveys the wastewater from the facility (Figure 3-10). These levels also have shelving units that store PVC piping, fittings, ductile iron gaskets, sampling equipment, wiring and miscellaneous materials.

Figure 3-10: Interior areas on the dry side of the Pump Station

The lowest level houses three 45 HP Flygt dry pit submersible pumps that extract water from the wetwell and lift the wastewater to the discharge piping on the intermediate level through 8-inch ductile iron discharge piping. The power cable for the pumps is not installed in conduit. There is also a sump pump in this area that discharges back to the wetwell.

The facility contains duct work that extends from the roof of the facility to the main level, as well as the lower levels to provide clean air. Heat is produced by a boiler at the lowest level of the dry side of the facility and is conveyed to the upper levels through aluminum ductwork.

Figure 3-11: Comminutor in Wet Area

The wet side of the facility has a main level that includes electrical lighting and receptacles which are explosion proof. There is also ductwork to provide fresh air to the intermediate level of the wet side where the comminutor is installed in a channel (Figure 3-11). The channel has instrumentation that measures the flow entering the facility. The intermediate area is above the wetwell where the raw wastewater enters the station.

This facility pumps the wastewater through a force main that travels down Goodspeed Island Road to the West Channel along the southwest corner of Eel Pond and continues through the Phoenix Rail Trail which crosses the Mattapoisett River, prior to discharge to the collection system on Mattapoisett Neck Road. The facility also has an emergency generator that utilizes diesel fuel that is stored in an above-ground 500-gallon steel storage tank on the south side of the facility.

Issues Noted during Site Investigation

The following deficiencies and/or concerns related to flooding and limited to the analysis completed herein are listed below and shown in Figure 3-12 and Figure 3-13:

- File cabinets, chemical for odor control, tools, and other equipment are stored directly on the finished floor. (2, 4)
- Electrical switchgear has instrumentation in the low buckets of the panel

Figure 3-12: Issues noted during site visit at the Eel Pond Sewer Pump Station

- Storage shed located on the western face of the pump station building is susceptible to flooding. (7)
- Bioxide storage tank located on the northern side of the pump station is filled with chemical and is no longer being used.(9)
- Ventilation and heating duct work penetrates the floor on the main level and lower levels. (8)
- Emergency generator sits on a housekeeping pad and has steel "I" beams for dunnage to support the engine. The engine is only approximately 12-inches above the finished floor elevation.
- Lower levels below the main floor contain PVC pipe fittings, sampling equipment, gaskets and miscellaneous materials that are stored on the floor and on temporary shelving. (1)
- Lowest level contains three (3) 45HP, 60 Hz, 1775 rpm, 460 volt Flygt pumps with electrical wires that are not enclosed in conduit. (6)
- Lower level contains an expensive composite flow sampler. (3)
- Wet well intermediate level contains a Franklin Miller 3hp, 60 Hz, 230/460V comminutor utilized for shredding incoming raw wastewater.
- The fuel tank located on the southern exterior side of the pump station building is mounted to the concrete pad and is only 2 ft above the existing grade of the area. (5)

Figure 3-13: Issues noted during site visit at the Eel Pond Sewer Pump Station

3.6.1.2 Eel Pond Wastewater Force Main

The force main exiting the Eel Pond Pumping Station runs for 9,600 linear feet by heading south-southwest along Goodspeed Island Road before entering the barrier beach near the West Channel (Figure 3-14). This force main continues westward and includes another crossing over the Mattapoisett River by utilizing the Phoenix Rail Trail before continuing cross country and going past Mattapoisett Neck

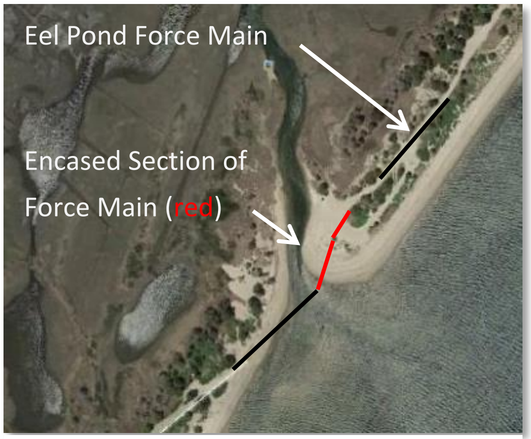

Figure 3-14: View of West Channel & Aerial of force main layout

Road and Brandt Island Road to discharge to a gravity sewer on Shaw Road in Fairhaven.

The force main layout is provided in Figure 3-14. This layout provided by the Town shows the approximate location of the main and the portion of the existing force main that was encased during installation to protect against the tidal influences of water entering this cove over the tidal cycle.

As the years have progressed, the encased portion of the force main has become covered as the shoreline has migrated toward the southwest due to changes in tidal influences and storm events that have affected this coastline.

3.6.1.3 No. 2 Pump Station

Figure 3-15: No. 2 Pump Station Wellhouse

The Acushnet Road No. 2 Pump Station Wellhouse and Tubular Wellfield are located just north of Interstate 195 in the northwestern section of Mattapoisett (See Figure 1-1). The wellhouse and wellfield are located just west of the Mattapoisett River, which travels along the eastern edge of the property. An exterior photo of the wellhouse is shown in (Figure 3-15). The facility includes at least 26 shallow tubular wells located around the wellhouse on the property. These wells are interconnected and groundwater is extracted from the wells using a vacuum tank that feeds the wellhouse's lower level, which acts as a clearwell. A booster pump rated for approximately 250 gpm (gallons per minute) is located on the main level of the facility. This pump is used to pump water from the lower level into the system. The site plan and floor plan are shown in in Figure C-2 and Figure C-3, respectively, in Appendix C to this document.

Figure 3-16: Issues Noted during Site Visit at the No. 2 Pump Station

Issues Noted during Site Investigation

The following deficiencies and/or concerns related to flooding and limited to the analysis completed herein are listed below and shown in Figure 3-16:

- An air compressor located only 6 inches above facility floor (1)
- An electrical conduit located on facility floor.
- Facility unit heater rests on the floor. (2)

- A desk and file cabinets containing data and information related to the existing system at the facility rests on the floor. (2)
- An electrical service cabinet is located only 6 inches above the facility floor. An electrical meter located only 3 ft above facility floor. (4)
- A Sulzer, 250 gpm, 180 ft total dynamic head (TDH) booster pump is located only 6"± above the facility floor.
- Alarm service wires are not protected and located only 4 ft above the facility floor. (8)
- Four (4) vents located outside of the building for lower level need to be covered.(3)
- There are 8 windows at the facility that are only 3 ft above the finished floor. (3)
- Propane tanks are located on existing grade outside of the building. (5)
- Floor penetrations to lower level are not watertight.
- Valves located inside are inoperable. (6)
- Tubular Wells are close to existing grade and not watertight (7)

3.6.1.4 No. 3 Pump Station (Wellhouse)

The wellhouse for the No. 3 Pump Station (Figure 3-17) is located at the end of Hereford Hill Road, south of Wellhouse No. 4 and the Water Treatment Facility in the northwestern section of Mattapoisett (see Figure 1-1). The facility is just west of the Mattapoisett River and the well is within the wellhouse. The site has a well maintained surrounding grass area and the exterior of the pump station has a propane tank for heating and a hydrant for blowoff of the transmission main.

Figure 3-17: Well No. 3 Wellhouse Facility

The well is shallow (67 ft deep) and the pump is rated for 780 gpm. The water is pumped from the wellhouse to the water treatment facility before being sent out to the distribution system. Emergency power is provided by a generator that is housed at the Water Treatment Facility to the north. A floor plan of the facility is included in Figure C-4 in Appendix C.

Issues Noted during Site Investigation

The following deficiencies and/or concerns related to flooding and limited to the analysis completed herein are listed below and shown in Figure 3-18:

- Vertical turbine pump located only 6 inches above the facility finished floor elevation. (1)

- Force main piping on the discharge end of the vertical turbine pump is located approximately 1 ft above the facility finished floor elevation. (2)
- Storage units and cabinet files rest directly on the finished floor. (3)
- Switchgear is located on a housekeeping pad approximately 6 inches above the finished floor elevation. (4)
- Chrysler motor on the housekeeping pad is no longer being used. (5)
- Propane tank is located on existing grade outside of the building. (8)
- Electrical feed to the main cabinet is only approximately 3 ft above the finished floor elevation.
- Conduit located on the north wall is only 6 inches above the finished floor elevation.
- Receptacles within the facility are only 2 ft above the floor elevation and are not GFIs. (7)
- Variable Frequency Drive (VFD) for the existing vertical turbine pump is located approximate 16 inches above the finished floor elevation. (6)

Figure 3-18: Issues Noted during Site Visit at the No. 3 Pump Station

3.6.1.5 No. 4 Pump Station (Wellhouse)

The No. 4 Pump Station wellhouse (Figure 3-19) is located north of Hereford Hill Road and to the south and east of the Water Treatment Facility in the northwestern section of Mattapoisett (Figure 1-1). The facility can be accessed from the Water Treatment Plant and Hereford Hill Road through gates that are closed after hours. The facility houses the well. The exterior of the pump station has a propane tank for heating and a hydrant for blowoff of the transmission main.

Figure 3-19: No. 4 Pump Station Wellhouse

The well is shallow (76 ft deep) and the pump is rated for 750 gpm. The water is pumped from the wellhouse to the water treatment facility before being sent out to the distribution system. Emergency power is provided by the emergency generator that is housed at the Water Treatment Facility to the north. A floor plan of the facility is included in Figure C-5.

Issues Noted during Site Investigation

The following deficiencies and/or concerns were noted during the site visits completed at this facility and shown by Figure 3-20:

- Vertical turbine pump located only 6 inches above the facility finished floor elevation. (2)
- Discharge piping is located approximately 1 ft above the facility finished floor. (3)
- Storage units and cabinet files rest directly on the finished floor. (4)
- Electrical switchgear on housekeeping pad only approximately 6 inches above the finished floor. (1)
- Alarm panel is located approximately 2 ft above the finished floor elevation. (5)
- Propane tank located on existing grade outside of the building. (6)
- Electrical feed to the main cabinet is only approximately 3 ft above the finished floor elevation. (7)
- Conduit located on the north wall is only 6 inches above the finished floor elevation.
- Receptacles within the facility are only 2 ft above the finished floor elevation and are not GFIs. (5)
- Vents in Front of building only 16 inches above finished floor.

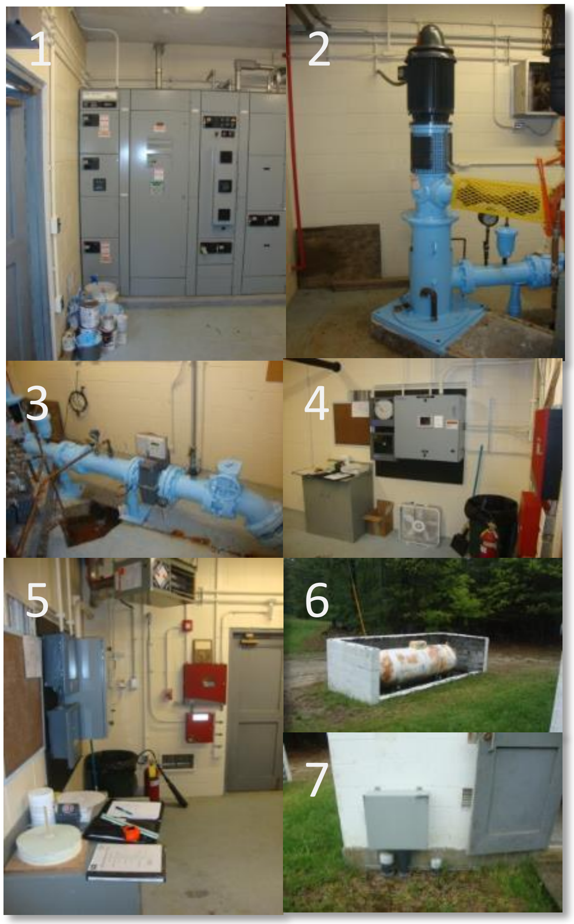

Figure 3-20: Issues Noted during Site Visit (No. 4 Pump Station).

3.6.1.6 Water Treatment Facility

The Mattapoisett River Valley Water District's shared Filtration Facility (Figure 3-21) is located on Tinkham Lane in the northwestern section of Mattapoisett (see Figure 1-1). The facility is used to treat groundwater supplies that include the No. 3 and 4 Water Pump Stations in Mattapoisett's water system, in addition to wells owned by the Towns of Fairhaven and Marion. The Treatment Plant is a two story facility that was completed in 2008 and has the capacity to treat up to six (6) million gallons per day. The facility provides water to Mattapoisett, Fairhaven, Marion and a small area of Rochester.

Figure 3-21: Mattapoisett River Valley Water District Water Treatment Facility

The treatment system utilizes ozone for oxidation of minerals and this is followed by Ultrafiltration, a series of membranes that the water passes through, to eliminate the water's contaminants. Upon completion, the water is chlorinated to provide a residual for the distribution system.

Ancillary components of the overall facility that surround the main building include the main transformer that provides power to the facility, oxygen storage and ozone generation equipment, a propane storage tank, and a generator building that provides emergency power to the treatment facility, as well as the wellhouses for the No. 3 and 4 Pump Stations. This is shown on the Site Plan provided in Figure C-6 in Appendix C.

Issues Noted during Site Investigation

The following deficiencies and/or concerns were noted during the site visits completed at this facility and are shown in Figure 3-22:

- Propane tanks sit directly on existing grade (1)
- Transformer sits directly on existing grade (2)
- Ozone unit and oxygen tanks are installed approximately 1 ft above existing grade (3)
- Generator building sits on the existing grade (4)
- Entrances to the water treatment facility are at existing grade (5)
- Exhaust and ventilation louvers are install approximately 16 inches above existing grade (5)
- Equipment such as a ladder is stored on the existing grade on the southern face of the building (6)

Figure 3-22: Issues Noted during Site Visit Water Treatment Facility

3.6.1.7 Water Distribution System Crossing

Figure 3-23: Water Distribution System Crossing on Route 6 (Fairhaven Rd)

An existing water transmission main (Figure 3-23) that is insulated and strapped to supports adjacent to a bridge over the Mattapoisett River on US Highway 6 (Fairhaven Road) directly across from River Road is another critical facility that is being evaluated (Figure 1-1). This main serves as the only feed into the Town of Fairhaven and would result in a loss of supply if damaged or washed out during a flood event.

Issues Noted during Site Investigation

The following deficiencies and/or concerns were noted during the site visits completed at this facility (Figure 3-24):

Figure 3-24: Issues Noted during Site Visit at the Water Distribution Crossing

- Hydrant and valve located at east side of crossing appears worn and the valve box partially covered (1)
- Steel beam providing support over box culvert is rusted and paint has delaminated (2)
- Minor soil scour is evident on the ground supporting pipe (3)
- The condition of the blow off valve could not be determined during visit (4)

3.6.1.8 Sewer Crossing along Phoenix Rail Trail over the Mattapoisett River

Figure 3-25: Phoenix Rail Trail Crossing

The force main that runs along the Phoenix Rail Trail (Figure 3-25) is a continuation of the force main that exits the Eel Pond Pumping Station and runs south-southwest along Goodspeed Island Road and crosses the West Channel. After crossing the West Channel, the force main continues and runs within the Phoenix Rail Trail. It crosses over the Mattapoisett River by being strapped under a wooden foot bridge that associated with the trail. The force main continues in the southwesterly direction cross country and crosses Mattapoisett Neck Road and Brandt Island Road before discharging to a gravity sewer on Shaw Road in Fairhaven (Figure 1-1). A diagram of the site is provided in Figure 3-25.

Issues Noted during Site Investigation

The following deficiencies and/or concerns were noted during the site visit completed at this facility (Figure 3-26):

- Limited access to the sewer force main for routine inspection (1)
- Manhole for blowoff is not fully accessible (2)
- Insulation for piping has not been inspected (3)
- The force main is exposed just prior to crossing of foot bridge (4)

Figure 3-26: Issues Noted during Site Visit at the Phoenix Rail Trail Sewer Crossing

3.6.2 Analysis and Recommendations

Using the results from the coastal hazard analysis presented in Section 3.1, inundation depths at each facility were used to investigate the vulnerabilities of each site and develop engineering recommendations.

3.6.2.1 Eel Pond Pump Station

The Eel Pond Pump Station lies within the 100 year floodplain Zone VE which has an elevation of approximately 18 ft (NAVD 88). This zone is a coastal flood zone with wave action hazards. The floodplain elevation is approximately 4 ft above the main floor of the pump station. The projected inundation depths for this facility above the ground elevation (13.48 ft) just outside the facility are provided in Table 3-5 (note that these values are rounded to the nearest foot). The inundation levels quickly increase as the severity of the storm and the amount of SLR increases. There is no appreciable flooding for a Category 1 storm, but the level of inundation climbs during a Category 2 storm as the projected SLR rates increase. Category 3 storms begin with approximately 6 ft of inundation, which increases to approximately 9 ft with SLR of 4 ft. Category 4 storms are projected to produce inundations greater than 10 ft for all SLR scenarios, with a maximum of approximately 17 ft for a Category 4+ storm and SLR of 4 ft.

Table 3-5: Inundation depths (rounded to the nearest foot) at the Eel Pond Pump Station, without the addition of wave action. Empty cells (grayed out) represent scenarios that did not yield any inundation at the site.

Storm Category	Depth of Inundation (ft)																			
	No SLR					1 ft SLR					2 ft SLR					4 ft SLR				
	1	2	3	4	4+	1	2	3	4	4+	1	2	3	4	4+	1	2	3	4	4+
Eel Pond Sewer Pump Station		<1	6	10	13		1	7	11	14		2	7	12	15		4	9	14	17

3.6.2.1.1 Options for Flooding Protection – Eel Pond Pump Station

The inundation levels provided above were compared to the existing layout of the facility. Based on the inundation levels above, several mitigation options are available for the protection of critical equipment against flooding, wave action and Sea Level Rise.

The most common adaptation measure involves providing floodproof doors and hatches, raising critical equipment and electrical/controls above flood elevations, relocating important files, materials, equipment and tools stored at the facility to an upland location and fortifying the structure to limit floodwaters from entering the facility.

For this facility, Table 3-6 was developed to list the critical components that will need to be addressed. The measures recommended are for Category 2 storms with 2 ft of SLR. This storm was used since it is one of the most severe storms that have occurred in the past. More intense storms will result in inundation levels that are so significant that they could not be protected against in a cost effective way.

Table 3-6: Critical Components and Adaptation Measures – Category 2 storm with 2 ft of SLR

Critical Infrastructure	Elev. (ft)	Water Surface Elevation (ft)	Inundation above Item (ft)[2]	Proposed Remedial Action
Ground Elevation	13.48	15.68	2.20	--
Door Threshold	13.92	15.68	1.76	Floodproof Doors
Electrical Switchgear Buckets	13.92	15.68	1.76	Floodproof Doors
Generator	14.91	15.68	0.77	Floodproof Doors
Fuel System for Gen Set	15.50	15.68	0.18	Floodproof Doors
Pump Control Panels	14.02	15.68	1.66	Floodproof Doors

[2] Inundation (excludes ground elevation)

The Town currently stores file cabinets, chemical for odor control, tools, PVC pipe fittings, sampling equipment, gaskets and miscellaneous materials either on the main floor or below the main floor. It is recommended that this material be relocated and stored at the Water Treatment Facility as a precaution in case there is flooding and the remedial actions proposed fail.

While the power supply for the pumps is fed through cables that extend from the main Motor Control Center down to the head of the pumps, and these would be protected with the floodproofing measures described above, it is recommended that they be placed within conduit that would further protect the wiring.

For a Category 3 storm (the other severe storm that has been seen in this area in the past) with 2 ft of SLR, there would be over 7 ft of inundation from the ground elevation at the site. Floodproofing of the doors would protect against water getting into the facility, but if this fails, even raising the critical components listed above by 4 ft would not protect the components against flooding. The level of water would begin to submerge the equipment.

A more conservative protection of the facility would include the installation of a barrier wall along the current perimeter of the chain link fence line. The height of the wall is proposed at 9 ft. This would protect against inundation associated with a Category 3 storm with no SLR, however wave action could cause the flooding to overtop the barrier wall. A Category 3 storm with an SLR of 1 ft or more would overtop the wall due to the expected wave action and allow water to get into the facility.

Table 3-7: Critical Components and Adaptation Measures – Category 3 Storm with No SLR.

Critical Infrastructure	Elev. (ft)	Water Surface Elevation (ft)	Inundation above Item (ft) [3]	Proposed Remedial Action
Ground Elevation	13.48	19.31	5.83	--
Door Threshold	13.92	19.31	5.39	Barrier Wall
Electrical Switchgear Buckets	13.92	19.31	5.39	Barrier Wall
Generator	14.91	19.31	4.40	Barrier Wall
Fuel System for Gen Set	15.50	19.31	3.81	Barrier Wall

[3] Inundation (excludes ground elevation)

A layout of the barrier wall and its proposed details including a 12 foot wide entrance that would be sealed with aluminum stop logs is included in Figure C-7 in Appendix C to this document.

Finally, a comprehensive option (not included in the opinion of costs developed) for protecting against floods is a portable facility that can be moved if a storm is expected. The wetwell for a facility would remain in place, but a portable facility housed within a climate controlled trailer would be equipped with the required electrical switchgear, variable frequency drives (VFD's), SCADA controls system, and important alarms. The connections to the wetwell and pump station are still below grade and are specified for being able to withstand saltwater. An example of this type of arrangement is presented in Figure 3-27. The costs for a facility such as this would range from $1.5 to $3.0 Million.

Figure 3-27: Sea Girt Pump Station Mobile Enclosure
South Monmouth Regional Sewerage Authority
Wall Township, New Jersey
Thanks to Mike Ruppel, Executive Director, SMRSA
Rina Dalal. T and M Associates

Budgetary Opinion of Cost - Eel Pond Wastewater Pump Station

Flood protection improvements proposed at the Eel Pond Pumping Station include furnishing and installing flood proof doors, relocating files, materials, equipment and tools stored at the facility to an upland location and fortifying the structure when protecting against a Category 2 storm with 2 ft of SLR. The costs provided are for materials and installation, and are based on quotations from equipment manufacturers, costs included in RS Means and previous construction projects completed. The costs also include Division 1 costs (this includes permits, bonds, and contractor overhead/profit), engineering legal and administrative costs (15% of construction costs) and a 25% contingency. See Table 3-8 for a summary of these costs.

Table 3-8: Summary of Costs for Eel Pond Pumping Station- Category 2 Storm

Item	Budgetary Opinion of Cost
Clean Materials Stored in PS and Relocate	$5,000
Furnish /Install Floodproof Door	$18,000
Furnish /Install Floodproof Door	$38,000
Relocate Storage Shed	$5,000
Place Electrical Wiring in Conduit for Pumps	$5,000
Subtotal	**$71,000**
Division 1 Costs (21%)	$15,017
TOTAL Construction Costs	$86,017
Engineering Legal and Administration (15%)	$12,902
Contingency (25%)	$21,504
TOTAL COST (Rounded to nearest 1000)	**$120,000**

The costs for the barrier wall include the costs for the materials and installation, and are based on quotations from equipment manufacturers, costs included in RS Means and previous construction projects completed. The costs also include Division 1 costs (this includes permits, bonds, and contractor overhead/profit), engineering legal and administrative costs (15% of construction costs) and a 25% contingency (Table 3-9).

Table 3-9: Summary of Costs for Eel Pond Pumping Station- Category 3 Storm

Item	Budgetary Opinion of Cost
Furnish/Install Flood Barrier Wall	$300,000
Clean Materials Stored in PS and Relocate	$5,000
Relocate Storage Shed	$5,000
Place Electrical Wiring in Conduit for Pumps	$5,000
Subtotal	**$315,000**
Division 1 Costs (21%)	$66,623
TOTAL Construction Costs	$381,623
Engineering Legal and Administration (15%)	$57,243
Contingency (25%)	$95,406
TOTAL COST (Rounded to nearest 1000)	**$534,000**

3.6.2.2 Eel Pond Wastewater Force Main

The Eel Pond Wastewater Force Main lies within the 100 year floodplain Zone VE that has an elevation of approximately 17 ft (NAVD 88). This zone is a coastal flood zone with wave action hazards. The flood elevation is approximately 7 ft above the existing ground elevation. The expected area of inundation begins at approximately 9 ft above the current ground elevation (10.50 ft NAVD88) with a Category 1 storm with no SLR and increases to over 30 ft when a Category 4+ storm with 4 ft of SLR is projected. The amount of inundation in this area and the expected scour based on DSAS evaluations completed by RPS ASA would cause the shoreline to continue to shift to the south and west (see Section 3.4 for an in-depth description of the projected shoreline change).

This shift would further expose the force main that is not encased, making it more vulnerable to damage. It is important to note that the projected migration is based on information available for the period of 1978 through 2014. There have been no significant storms on record at this location during this period. Thus, if more significant storms (as detailed in Table 3-10) occur, the erosion and migration of the shoreline near the inlet and surrounding this area could be more significant than shown here, resulting in more of the force main being exposed and vulnerable.

Table 3-10: Inundation depths (rounded to the nearest foot) at the Eel Pond Sewer Force Main, without the addition of wave action. Empty cells (grayed out) represent scenarios that did not yield any inundation at the site. The values listed are the level of inundation above the typical ground elevation in the area of the sewer force main, which is estimated to be 10.5 ft (NAVD88).

	Depth of Inundation (ft)																			
Storm Category	**No SLR**					**1 ft SLR**					**2 ft SLR**					**4 ft SLR**				
	1	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**
Sewer Force Main	9	14	19	29	27	12	15	20	25	28	12	16	21	26	28	13	18	22	27	30

3.6.2.2.1 Options for Flood Protection - Eel Pond Force Main Crossing

The encased portion of the force main potentially subjected to tidal influences has shifted as the shoreline has migrated and the effects of continued shoreline erosion have occurred. Based on the analysis completed by RPS ASA, the shoreline will continue to migrate to the southwest over the next 50 to 100 years, further exposing the existing force main. Shoreline projections 50 years into the future are shown in Figure 3-28.

To combat this migration, additional encasement of the existing force main is recommended to protect the portion that will now be under the channel as the shoreline continues to shift. Approximately 300 ft of additional force main will require encasement to cover the projected shift and protect the force main from the various flooding and tidal influences projected in this area. This is based on the available mapping provided by the Town showing approximately 8 feet of cover for this portion of the main and the amount of migration projected. Further analysis of the depth of cover and the potential for migration should be undertaken to determine if additional encasement or lowering of the force main is necessary.

Figure 3-28: Example showing the predicted 50 year shoreline migration and the portion of the sewer line that would need to be encased

Another option that was not considered within the scope of this project, but should be mentioned, is for the force main to be installed deeper underground by the process of directional drilling. This would eliminate the need for having to encase the existing main. This option would result in more of the force main needing to be replaced. There would still need to be cofferdamming, sheeting, and shoring at the sending and receiving pits. There would also be additional efforts needed when making the connections. The new piping would be installed and then there would need to be temporary bypass pumping while the connections from the existing force main to the newly installed piping could occur. A cost for this was not fully developed, but it would range from roughly 2.5 million to almost 3 million dollars for engineering and construction.

A final option that was also not considered in this scope, but should be mentioned, is the complete removal the force main from the barrier beach, which would require a different route for conveying flows. These options should be evaluated further to determine if the existing force main should be protected additionally as described above, if the main should be installed differently, or directed a different way since another crossing downstream of this crossing is also vulnerable.

Budgetary Opinion of Cost - Eel Pond Force Main Crossing

Flood protection improvements proposed at the Eel Pond force main crossing of the West Channel include installing cofferdamming to isolate the area where the force main is installed, excavating and protecting the excavation for forming around the force main, encasing an additional 300 linear ft of force main, backfilling the excavation, removing the protection and cofferdamming, and restoring the area. The costs provided are for materials and installation, and are based on quotations from equipment manufacturers, costs included in RS Means and previous construction projects completed. The costs also include Division 1 costs (this includes permits, bonds, and contractor overhead/profit), engineering legal and administrative costs (15% of construction costs) and a 25% contingency.

Table 3-11: Summary of Costs for Eel Pond Force Main Crossing

Item	Budgetary Opinion of Cost
Cofferdamming for Ocean and Inlet Area	$150,000
Excavation to Expose FM Piping, Deepen Piping	$125,000
Sheeting, Shoring, Dewatering, Protection around work	$150,000
Furnish /Install Concrete Encasement	$300,000
Backfill and Compaction	$90,000
Restoration	$50,000
Subtotal	**$865,000**
Division 1 Costs (21%)	$182,948
TOTAL Construction Costs	$1,047,948
Engineering Legal and Administration (15%)	$157,192
Contingency (25%)	261,987
TOTAL COST (Rounded to nearest 1000)	**$1,467,000**

3.6.2.3 No. 2 Pump Station

The Facility lies within Zone A of the 100 year floodplain, although no flood elevation has been determined for this area. The flooding projected is based on the flooding along the shoreline propagating inland following the river and crossing Route 195 through existing structures. As shown by the storm surge modeling for this location (summarized in Table 3-12), inundation levels quickly increase as the severity of the storm and the amount of SLR increases. There is no flooding for a Category 1 storm, and only slight flooding for a Category 2 storm as the SLR gets to 4 ft. Category 3 storms start with just above a 4 ft inundation level and this quickly increases to just above 9 ft with an SLR of 4 ft. Category 4 storms have inundations of over 10 ft for all SLR categories, topping out at just above 16 ft of inundation for a Category 4+ storm with SLR of 4 ft.

Table 3-12: Inundation depths (rounded to the nearest foot) at the No. 2 Pump Station. Empty cells (grayed out) represent scenarios that did not yield any inundation at the site.

Depth of Inundation (ft)																				
Storm Category	No SLR					1 ft SLR					2 ft SLR					4 ft SLR				
	1	2	3	4	4+	1	2	3	4	4+	1	2	3	4	4+	1	2	3	4	4+
No. 2 Pump Station			4	11	13			5	12	14		1	7	12	15		3	9	14	16

3.6.2.3.1 Options for Flooding Protection – No. 2 Pump Station (Wellhouse and Wellfield)

Table 3-13 and Table 3-14 provide the critical components and recommended action for the No 2. Pump Station location (Wellhouse and Tubular Wellfield). The tables list the critical components identified above and provides a remedial action based on the category of storm the base elevations are compared against. Again, Category 2 and Category 3 storms with 2 ft of SLR are used for the storms to protect against. More intense storms than this will result in inundation levels that are so significant that they could not be protected against.

Table 3-13: Critical Components and Adaptation Measures – Category 2 storm with 2 ft of SLR

Critical Infrastructure	Elev. (ft)	Water Surface Elevation (ft)	Inundation above Item (ft) [4]	Proposed Remedial Action
Ground Elevation	13.12	14.48	1.36	--
Door Threshold	14.37	14.48	0.11	Floodproof Doors
Electrical Switchgear	14.54	14.48	--	Floodproof Doors
Compressor	14.54	14.48	--	Floodproof Doors
Unit Heater	14.50	14.48	--	Floodproof Doors
Vents for Clearwell	13.70	14.48	0.78	Extend Vents and Cap
Floor Penetrations to Lower Level	14.50	14.48	--	Floodproof Hatch
Tubular Wells	varies	14.50	varies	Extend Well Caps
Propane Tanks	12.85	14.48	1.63	Place on Pad

[4] Inundation (excludes ground elevation)

It is recommended that materials such as the file cabinets, the desk and miscellaneous materials (all stored on the floor within the facility) be relocated to another facility upland of this site and protected against flooding as a precaution.

For a Category 3 Storm with 2 ft of SLR, the inundation depth increases to 6.7 ft above base elevations. Compared to the base elevations, some of the remedial actions proposed change due to the severity of the inundation. These are documented in Table 3-14.

Table 3-14: Critical Components and Adaptation Measures - Category 3 Storm with 2 ft SLR

Critical Infrastructure	Elev. (ft)	Water Surface Elevation (ft)	Inundation above Item (ft) [5]	Proposed Remedial Action
Ground Elevation	13.12	19.82	6.70	--
Door Threshold	14.37	19.82	5.45	Barrier Wall
Electrical Switchgear	14.54	19.82	5.28	Barrier Wall
Compressor	14.54	19.82	5.28	Barrier Wall
Unit Heater	14.50	19.82	5.32	Barrier Wall
Vents for Clearwell	13.70	19.82	6.12	Extend Vents and Cap
Facility Windows	16.12	19.82	3.70	Floodproof Windows
Floor Penetrations to Lower Level	14.50	19.82	5.32	Floodproof Hatch, Concrete repairs
Tubular Wells	varies	19.82	Varies	Extend Well Caps
Propane Tanks	12.85	19.82	6.97	Bury Tanks

[5] Inundation (excludes ground elevation)

While the inundation for this category storm is at 6.7 ft, the projected wave action for this category of storm will result in water levels that require protection for up to 9 ft of water.

A more conservative protection of the facility would be the installation of a barrier wall around the facility parallel to the building walls and placed in such a way as not to affect underground piping. The height of the wall is proposed at 9 ft. A layout of the barrier wall and its proposed details including a 10 foot wide entrance that would be sealed with aluminum stop logs is included in Figure C-8 in Appendix C.

Budgetary Opinion of Cost - No. 2 Pump Station (Wellhouse and Wellfield)

Flood protection improvements proposed at the Acushnet Road Wellhouse and Tubular Wellfield include furnishing and installing flood proof doors and hatches, raising critical equipment above flood elevations, relocating files, materials, equipment and tools stored at the facility to an upland location, and extending the clearwell vent piping and providing watertight caps. The costs provided are for materials and installation, and are based on quotations from equipment manufacturers, costs included in RS Means and previous construction projects completed. The costs also include Division 1 costs (this includes permits, bonds, and contractor overhead/profit), engineering legal and administrative costs (15% of construction costs) and a 25% contingency. See Table 3-15 for a summary of these costs.

Table 3-15: Summary of Costs for the No. 2 Pump House location (Wellhouse and Tubular Wellfield)

Item	Budgetary Opinion of Cost
Clean Materials Stored in PS and Relocate	$5,000
Furnish/Install Floodproof Door	$38,000
Extend Well caps for Tubular Wells	$5,200
Hatches for the Floor penetration to Lower Level	$30,000
Minor Concrete Floor Repairs	$25,000
Bury Propane Tanks	$10,000
Subtotal	**$113,200**
Division 1 Costs (21%)	$23,942
TOTAL Construction Costs	$137,142
Engineering Legal and Administration (15%)	$20,571
Contingency (25%)	$34,285
TOTAL COST (Rounded to nearest 1000)	**$192,000**

The more conservative protection of the facility would include the installation of a 9 ft flood barrier wall. The costs for the barrier wall include the costs for the materials and installation, and are based on quotations from equipment manufacturers, costs included in RS Means and previous construction projects completed. The costs also include Division 1 costs (this includes permits, bonds, and contractor overhead/profit), engineering legal and administrative costs (15% of construction costs) and a 25% contingency (Table 3-16).

Table 3-16: Summary of Costs for the No. 2 Pump House location (Wellhouse and Tubular Wellfield) – Category 3 Storm

Item	Budgetary Opinion of Cost
Furnish/Install Flood Barrier Wall	$360,000
Furnish/Install Floodproof Windows	$32,000
Hatches for the Floor penetration to Lower Level	$30,000
Extend Well caps for Tubular Wells	$5,200
Minor Concrete Floor Repairs	$25,000
Bury Propane Tanks	$10,000
Subtotal	**$462,200**
Division 1 Costs (21%)	$97,755
TOTAL Construction Costs	$559,955
Engineering Legal and Administration (15%)	$83,993
Contingency (25%)	$139,989
TOTAL COST (Rounded to nearest 1000)	**$784,000**

3.6.2.4 No. 3 Pump Station

The facility lies within Zone A of the 100 year floodplain although no flood elevation has been determined for this area. The facility is just west of the Mattapoisett River and has some wetlands and low lying areas just to north of the wellhouse. The flooding projected is based on the flooding along the shoreline propagating inland following the Mattapoisett River and crossing Route 195 through existing structures.

Inundation for this facility (rounded to the nearest foot) is summarized in Table 3-17. The inundation levels increase as the severity of the storm and the amount of SLR increases. There is no real flooding for Category 1 and Category 2 storms. Category 3 storms start with just below a 2 ft inundation level and this increases to just below 6 ft with an SLR of 4 ft. Category 4 storms have inundations over 7 ft for all SLR categories and this tops out at just above 13 ft of inundation for a Category 4+ storm with an SLR of 4 ft.

Table 3-17: Inundation depths (rounded to the nearest foot) at the No. 3 Pump Station. Empty cells (grayed out) represent scenarios that did not yield any inundation at the site.

	Depth of Inundation (ft)																			
Storm	No SLR					1 ft SLR					2 ft SLR					4 ft SLR				
Category	1	2	3	4	4+	1	2	3	4	4+	1	2	3	4	4+	1	2	3	4	4+
No. 3 Pump Station			2	7	10			3	9	11			4	9	12		<1	6	11	13

3.6.2.4.1 Options for Flooding Protection – No. 3 Pump Station (Wellhouse)

Table 3-18 and Table 3-19 list the critical components and their elevations identified above and provide remedial actions based on the category of storm the ground elevation is compared against. Category 2 and Category 3 storms with 2 ft of SLR are used as the representative storms to protect against.

Table 3-19 shows that no inundation is expected for a Category 2 storm and does not list any remedial actions.

Table 3-18: Critical Components and Adaptation Measures - Category 2 Storm with 2 ft SLR

Critical Infrastructure	Elev. (ft)	Water Surface Elevation (ft)	Inundation above Item (ft)	Proposed Remedial Action
Base Elevation	17.90	--	--	--
Vertical Turbine Pump	18.53	--	--	--
Discharge Piping	19.00	--	--	--
Propane Tank Outside of Wellhouse	--	--	--	--
Electrical Service Location	20.90	--	--	--
Conduit and Receptacles	19.00	--	--	--
Variable Frequency Drives	19.40	--	--	--

For a Category 3 Storm with 2 ft of SLR, the inundation depth increases to 3.8 ft above base elevations. When this is compared to the ground and door threshold elevations, remedial actions are proposed to alleviate the projected inundation. They include floodproofing doors and former wall penetrations, relocation of electrical items including the VFD to higher elevations, and burying the propane tank. These are documented below in Table 3-19.

Table 3-19: Critical Components and Adaptation Measures - Category 3 Storm with 2' SLR

Critical Infrastructure	Elev. (ft)	Water Surface Elevation (ft)	Inundation above Item (ft) [6]	Proposed Remedial Action
Base Elevation	17.90	21.70	3.80	--
Vertical Turbine Pump	18.53	21.70	3.17	Floodproof Doors
Discharge Piping	19.00	21.70	2.70	Floodproof Doors
Propane Tank Outside of Wellhouse	--	21.70	--	Bury Tank
Electrical Service Location	20.90	21.70	0.80	Raise
Conduit and Receptacles	19.00	21.70	2.70	Raise
Variable Frequency Drives	19.40	21.70	2.30	Raise

[6] Inundation (excludes ground elevation)

It is also recommended that materials such as the file cabinets and miscellaneous materials (all stored on the floor within the facility) be relocated to another facility as a precaution.

Budgetary Opinion of Cost - No. 3 Pump Station (Wellhouse)

Flood protection improvements proposed at the No. 3 Pump Station include furnishing and installing floodproof doors, raising critical equipment above flood elevations, relocating files, materials, equipment and tools stored at the facility to an upland location, fortifying the structure using a barrier wall 9 ft in height that would limit floodwaters from entering the facility for storms up to a Category 3 event with 4 ft of SLR, extending the well caps for the tubular wells, fortifying the wall openings and removing the old engine from the facility. The costs provided are for materials and installation, and are based on quotations from equipment manufacturers, costs included in RS Means and previous construction projects completed. The costs also include Division 1 costs (this includes permits, bonds, and contractor overhead/profit), engineering legal and administrative costs (15% of construction costs) and a 25% contingency.

Table 3-20 below provides a summary of the costs and Figure C-9 (Appendix C) provides the floor plan with the proposed conditions and recommendations.

Table 3-20: Summary of Costs for No. 3 Pump Station (Wellhouse)

Item	Budgetary Opinion of Cost
Clean Materials Stored in PS and Relocate	$5,000
Furnish/Install Floodproof Door	$38,000
Fortifying Wall Openings	$5,200
Raise Electrical Items	$25,000
Bury Propane Tank	$10,000
Subtotal	**$83,200**
Division 1 Costs (21%)	$17,597
TOTAL Construction Costs	$100,797
Engineering Legal and Administration (15%)	$15,120
Contingency (25%)	$25,199
TOTAL COST (Rounded to nearest 1000)	**$141,000**

3.6.2.5 No. 4 Pump Station

This facility lies just outside and to the west of Zone A of the 100 year floodplain area. Further, the facility is just west of the Mattapoisett River and just south of the Water Treatment Facility. The depths of inundation (rounded to the nearest foot) for this facility are summarized below in Table 3-21. There is no flooding for a Category 1 or Category 2 storm under all SLR scenarios considered. Category 3 storms do not yield any inundation with the exception of 4 ft SLR scenario, which results in less than 1 ft of inundation. Category 4 storms have inundation depths that start at approximately 2 ft for all SLR scenarios and top out at just over 9 ft for a Category 4+ storm with 4 ft of SLR.

Table 3-21: Inundation depths (rounded to the nearest foot) at the No. 4 Pump Station. Empty cells (grayed out) represent scenarios that did not yield any inundation at the site.

	Depth of Inundation (ft)																			
Storm Category	**No SLR**					**1 ft SLR**					**2 ft SLR**					**4 ft SLR**				
	1	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**
No. 4 Pump Station				2	4				3	5				3	6			<1	5	9

3.6.2.5.1 Options for Flooding Protection – No. 4 Pump Station (Wellhouse)

Due to the lack of inundation expected for Category 1, 2 and 3 storms (with the exception of a 4 ft SLR scenario), no remedial actions have been recommended at this site (Table 3-22).

Table 3-22: Critical Components and Adaptation Measures

Critical Infrastructure	Elev. (ft)	Water Surface Elevation (ft)	Inundation above Item (ft)	Proposed Remedial Action
Ground Elevation	23.22	--	--	--
Door Threshold	23.75	--	--	--
Vertical Turbine Pump	24.42	--	--	--
Discharge Piping	24.75	--	--	--
Propane Tank Outside of Wellhouse	22.09	--	--	--
Electrical Service Location	24.09	--	--	--
Conduit and Receptacles	25.75	--	--	--
Vents in Front of Building	25.15	--	--	--

Some conservative improvements that could be completed to address flooding from the more severe category storms (Category 4 and Category 4+) would include relocating data files and cabinets, floodproofing doors, sealing the floor opening that drains to the rear of the building, raising the wall openings used for ventilation and raising critical electrical equipment. These would only be necessary if a Category 4 storm was being prepared for. None are recommended for this facility at this time.

Budgetary Opinion of Cost - No. 4 Pump Station (Wellhouse)

Potential long-term flood protection improvements proposed at the No. 4 Pump Station location are to protect the Wellhouse against severe Category 4 storms. The improvements include furnishing and installing floodproof doors, raising critical equipment a, relocating files, materials, equipment and tools stored at the facility to an upland location, sealing the floor opening that drains outside and relocating the wall openings. The costs provided are for materials and installation, and are based on quotations from equipment manufacturers, costs included in RS Means and previous construction projects completed. The costs also include Division 1 costs (this includes permits, bonds, and contractor overhead/profit), engineering legal and administrative costs (15% of construction costs), and a 25% contingency. Table 3-23 provides a summary of the potential long-term costs and Figure C-10 provides a floor plan with the proposed conditions and recommendations.

Table 3-23: Summary of Costs for No. 4 Pump Station Wellhouse Facility

Item	Budgetary Opinion of Cost
Clean Materials Stored in PS and Relocate	$5,000
Furnish/Install Floodproof Doors	$38,000
Raise Critical Electrical and Control Equipment	$25,000
Seal Floor Opening that drains outside	$4,000
Relocate Wall Openings	$10,000
Subtotal	**$82,000**
Division 1 Costs (21%)	$17,343
TOTAL Construction Costs	$99,343
Engineering Legal and Administration (15%)	$14,901
Contingency (25%)	$24,836
TOTAL COST (Rounded to nearest 1000)	**$139,000**

3.6.2.6 Water Treatment Plant Facility

This facility lies outside and to the west of the 100-year floodplain area, and just west of the Mattapoisett River (Figure 1-1). Table 3-24 provides a summary of the inundation depths (rounded to the nearest foot) the depth of inundation for this facility under various category storms. There is no flooding for a Category 1, Category 2 and Category 3 storms under all SLR scenarios viewed. Category 4 storms result in inundation up to approximately 3.5 ft as SLR increases. Category 4+ storms have inundations that start at just over 2 ft for no SLR and tops out at just above 6 ft of inundation for a Category 4+ storm with SLR of 4 ft.

Table 3-24: Inundation depths (rounded to the nearest foot) at the Water Treatment Facility. Empty cells (grayed out) represent scenarios that did not yield any inundation at the site.

	Depth of Inundation (ft)																			
Storm	**No SLR**					**1 ft SLR**					**2 ft SLR**					**4 ft SLR**				
Category	**1**	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**
Water Treatment Facility					2				<1	3				1	4				3	6

3.6.2.6.1 Options for Flooding Protection – Water Treatment Plant

As shown by Table 3-24 above, the only flooding occurs when the category of storm (4+) and SLR rise are significant. There is no inundation expected for Category 1, Category 2 and Category 3 storms with the exception of the 4 ft SLR scenario. Thus, no remedial actions have been recommended. This information is detailed in Table 3-25.

Table 3-25: Critical Components and Adaptation Measures

Critical Infrastructure	Elev. (ft)	Water Surface Elevation (ft)	Inundation above Item (ft)	Proposed Remedial Action
Ground Elevation	24.18	--	--	--
Door Threshold	24.43	--	--	--
Propane Tanks	23.18	--	--	--
Transformer	24.00	--	--	--
Ozone and Oxygen Equipment	23.49	--	--	--
Generator Building	22.86	--	--	--
Wall Openings	25.91	--	--	--
Vents/Pipe Openings	25.51	--	--	--

The most common mitigation steps for the type of flooding expected here would include sandbags and the coverage of wall penetrations at lower elevations of the building walls. The specific locations for the sandbags would include the following:

- Entrances to the treatment facility and Generator Building
- Critical equipment including the transformer, ozone generation equipment and propane tank
- Ventilation and wall penetrations on exterior of building
- Removal of any tools or equipment stored outside

Budgetary Opinion of Cost - Water Treatment Plant

Flood protection improvements proposed at the Mattapoisett River Valley Water District Water Treatment Facility is minimal in terms of capital costs. Since there is very little inundation expected at the site, with the exception of Category 4 and 4+ storms, the recommended measures provided are temporary and can be installed when a storm is expected. No costs have been developed for this facility.

3.6.2.7 Water Distribution System Crossing US Highway 6 adjacent to River Road

The water main crossing lies within Zone VE of the 100-year and Zone X of the 500-year floodplain areas with an approximate elevation of 17 ft (NAVD88). This area is listed as a coastal flood zone with wave action being its significant hazard.

Table 3-26 provides a summary of the inundation depths at the crossing under the various category storms. There is significant flooding around this main under all category storms. The inundation depths provided in the table include those calculated above the water surface elevation of the Mattapoisett River (assumed to be 2.84 ft NAVD88), as well as those calculated from the invert of the pipe (11.29 ft NAVD88). It should be noted that the inundation can be exacerbated by the tidal fluctuations the river sees in this area. The inundation depth ranges from just above 9 ft of the typical water elevation for a Category 1 storm with no SLR all the way to over 30 ft for a Category 4+ storm with 4 ft of SLR.

Important considerations regarding the Inundation depths above include the following:

- For a Category 1 storm with 2 ft of SLR, the roadway and the water main crossing will be covered with water.
- As the Category of storm increases and the SLR increases, the overtopping of the road and this entire area increases significantly.
- A Category 2 storm would result in more than 4 ft of water over the top of the water main and the road Category 3 and Category 4 storms result in more than 10 ft of water over the pipe and road

Table 3-26: Inundation depths (rounded to the nearest foot) at the Water Crossing along Rt. 6. Empty cells (grayed out) represent scenarios that did not yield any inundation at the site. Inundation depths are provided as depths above the water surface, followed by the inundation depths calculated above the invert of the pipe.

Depth of Inundation (ft)																				
Storm Category	No SLR					1 ft SLR					2 ft SLR					4 ft SLR				
	1	2	3	4	4+	1	2	3	4	4+	1	2	3	4	4+	1	2	3	4	4+
Water Distribution Crossing	9	15	19	24	27	11	15	20	25	27	11	16	21	26	29	13	18	23	27	30
		3	8	13	15		4	9	14	16		5	10	14	17	2	7	11	16	19

3.6.2.7.1 Options for Flooding Protection – Water Distribution System Crossing

Based on inundation depths predicted and the current layout of the water main crossing, only a few critical components require protection. Table 3-27 below provides additional details.

Table 3-27: Critical Components and Adaptation Measures – Category 3 with 2 ft SLR

Critical Infrastructure	Elev. (ft)	Water Surface Elevation (ft)	Inundation above Item (ft)	Proposed Remedial Action
Normal Water Elevation	2.84	22.84	20.84	--
Pipe Invert Elevation	11.29	22.84	11.55	
Hydrant and Valve	9.20	22.84	13.64	Replace
Strapping/Supports for Force Main	9.29	22.84	13.55	Inspect/Replace
Slopes for Pipe Supports	varies	22.84	varies	Fortify Slopes
Air Release Valve	12.89	22.84	9.95	Inspect/Replace

Budgetary Opinion of Cost - Water Distribution System Crossing

Flood Protection improvements proposed at the Water Distribution System Crossing on US Highway 6 (Fairhaven Road) include furnishing and installing a new hydrant and isolation valve on the eastern side of the water main, fortifying the slopes around the supports for the water main, routine inspections of the supports and replacement of the blow off valve on the western edge of the crossing. The costs provided are for materials and installation, and are based on quotations from equipment manufacturers, costs included in RS Means and previous construction projects completed. The costs also include Division 1 costs (this includes permits, bonds, and contractor overhead/profit), engineering legal and administrative costs (15% of construction costs), and a 25% contingency.

Table 3-28: Summary of Costs for Water Distribution System Crossing - US Highway 6

Item	Budgetary Opinion of Cost
Replace Hydrant and Valve	$5,000
Fortify slopes adjacent to supports for Water Main	$25,000
Inspection of support system (annual cost for inspection)	$2,500
Replace Blowoff valve	$3,000
Subtotal	**$35,500**
Division 1 Costs (21%)*	$7,508
TOTAL Construction Costs	$43,008
Engineering Legal and Administration (15%)	$6,451
Contingency (25%)	$10,752
TOTAL COST (Rounded to nearest 1000)	**$60,000**

3.6.2.8 Sewer Crossing along Phoenix Rail Trail over the Mattapoisett River

The force main lies within Zone VE of the 100-year floodplain (elevation of 20 ft NAVD 88). It also lies within a coastal barrier resource system (CBRS) area. The location is listed as a coastal flood zone with wave action being its significant hazard.

Table 3-29 shows the depth of inundation for this facility under various category storms. There is significant flooding around the force main under all category storms. The inundation depths provided in the table include those calculated above the water surface elevation of the Mattapoisett River (assumed to be 2.00 ft NAVD88). It is important to note that the invert elevation of the force main is approximately 12 ft NAVD88. When inundation depths shown in the table below exceed 10 ft, the water level is above the force main and when the inundation exceeds 12 ft, the inundation goes over the bridge as well.

The inundation depths range from just under 9 ft above the typical water elevation (just below invert of pipe) for a Category 1 storm with no SLR all the way to approximately 30 ft for a Category 4+ storm with 4 ft of SLR.

Table 3-29: Inundation depths[7] (rounded to the nearest foot) at the Phoenix Rail Trail Sewer Crossing. Empty cells (grayed out) represent scenarios that did not yield any inundation at the site.

Depth of Inundation (ft)																				
Storm Category	**No SLR**					**1 ft SLR**					**2 ft SLR**					**4 ft SLR**				
	1	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**	**1**	**2**	**3**	**4**	**4+**
Phoenix Rail Trail Sewer Crossing	9	14	19	23	26	9	14	20	25	27	10	15	20	25	28	13	17	22	27	30
		2	7	11	14		2	8	13	15		3	8	13	16	<1	5	10	14	18

Important considerations concerning the inundation depths summarized above include the following:

- Category 1 storm with 4 ft of SLR, the wooden bridge and the force main will be covered with approximately 6 inches of water
- As the storm category increases and the SLR increases, the overtopping of the bridge and this entire area increases.
- Category 2 storms under all SLR conditions will overtop the force main and bridge and depending on the wave action could begin to cause damage.
- Category 2 storms with 4 ft of SLR will result in approximately 5 ft of water over the top of the bridge and the force main.
- Category 3 storms with 4 ft of SLR and all Category 4 storms result in greater than 10 ft of water being over the pipe and bridge.

These are significant and the inundation will adversely affect not only the pipe but also the bridge abutments and all low lying areas surrounding this part of Town. Further, this type of inundation also could result in significant debris or vessels being washed into this area causing further damage to the abutments, bridge, and the force main being supported.

3.6.2.8.1 Options for Flooding Protection – Phoenix Rail Trail Sewer Crossing

Based on the inundation levels provided in Table 3-29 and the current layout of the force main crossing; there are numerous options for protecting this facility. The most important actions are:

- The blowoff manhole on the east side of the wooden bridge - The valve inside must be maintained to ensure operation and allow for air to be released if flow is lost to the force main and then started back up.
- Access to the abutments and to the underside of the bridge – Better access will allow for routine inspection of both the force main and the abutments to assess their condition.

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[7] Values provided are the inundation depths above the water surface, followed by the inundation depths calculated above invert of pipe

- The abutments for the wooden bridge - These are stacked stone embankments that are pinned at the top with earthen embankments behind them. During flooding the embankments will be exposed and could be subjected to significant wave action, scour and potential erosion. The abutments should be protected so that the force main will remain supported by the bridge.

A more comprehensive option for this section of force main is to relocate the force main underground and eliminate the force main being strapped under the bridge. This would negate many of the issues associated with the exposure of the force main and the bridge during severe events. By burying the main deeper and limiting its exposure, the main would be better protected during severe weather events. Another option would be to re-route this and the West Channel force main discussed earlier so that they would not be exposed. These options were not included in the budgetary estimate developed. More detailed analysis of these options should occur in future analysis of these areas.

Budgetary Opinion of Cost - Sewer Crossing along Phoenix Rail Trail over the Mattapoisett River

Flood Protection improvements proposed at the Sewer Crossing along Phoenix Rail Trail over the Mattapoisett River include providing access to the wooden foot bridge for inspection of the abutments and the sewer force main, inspecting the blowoff manhole and determining if it needs to be replaced, and budgeting for necessary improvements.

Table 3-30: Summary of Costs for Sewer Crossing along Phoenix Rail Trail - Mattapoisett River

Item	Budgetary Opinion of Cost
Providing Access to the Abutments and Force Main	$50,000
Inspection of blowoff manhole	$1,000
Inspection of abutments and force main (annually)	$5,000
Develop Budgets for Improvements Recommended	$10,000
Subtotal	**$66,000**
Division 1 Costs (21%)*	$13,959
TOTAL Construction Costs	$79,959
Engineering Legal and Administration (15%)	$11,994
Contingency (25%)	$19,990
TOTAL COST (Rounded to nearest 1000)	**$112,000**

4 Summary

4.1 Coastal Hazard Analysis

This coastal hazard analysis, conducted for the Town of Mattapoisett, showed that critical wastewater and potable water infrastructure are at risk under a variety of SLR and storm scenarios. Modeling of storm surge, conducted using NOAA's SLOSH model, showed that, while the impacts from Category 1 storms under SLR scenarios up to 4 ft are minimal and do not cause inundation that impacts any of the three facilities assessed, stronger storms did result in inundation at these facilities, with the extent (i.e., depth) of inundation increasing with increasing storm intensity. Impacts from Category 2 storms are largely limited to the coastline, with minimal impacts throughout the inland portions of the town. Maps showing the location and extent (i.e., depth) of inundation throughout the town were created for all scenarios, as were site-specific visualizations for each of the three facilities. These figures are provided in the appendices to this document.

Wave modeling was conducted for the Eel Pond Sewer Pump Station using FEMA's WHAFIS model. The results of this modeling were used as input to the Engineering Analysis.

Shoreline change at the inlet of the Eel Pond was assessed to inform the engineering recommendations for the Eel Pond Sewer Force Main, which runs across the inlet. The shoreline change assessment indicated that there is a general drift in position of the Eel Pond West Channel towards the southwest. The southern bank is eroding extensively and sediment is accreting along the northern bank. However, further analysis of the dunes along the banks showed that no portion of the beach should be considered effective barriers to storm surge. Dunes along each bank would likely erode in the event of a large storm and portions of the sewer main (encased or otherwise) have the potential to become exposed.

4.2 Engineering Analysis

Engineering analyses were conducted for the eight critical infrastructure sites throughout the Town. Due to the large volume of data resulting from the inundation modeling, recommendations were prioritized based on inundation depths for the facilities for two modeled inundation scenarios (Category 2 and Category 3 storms).

Budgetary opinions of cost were developed for the flooding protection details described at each Facility and detailed in Figures C-7, C-8, C-9, and C-10 provided in Appendix C. The costs are summarized for each facility in Table 4-1.

Table 4-1: Summary of Costs for Critical Facilities

Item	Budgetary Opinion of Cost
Eel Pond WW Pumping Station - Category 2	$120,000
Eel Pond WW Pumping Station - Category 3	$534,000
Eel Pond FM Crossing West Channel	$1,467,000
Acushnet Road Wellhouse and Tubular Wellfield - Category 2	$192,000
Acushnet Road Wellhouse and Tubular Wellfield - Category 3	$784,000
Well No. 3 Wellhouse Facility	$141,000
Well No. 4 Wellhouse Facility[8]	$139,000
Mattapoisett River Valley Water District Water Treatment Facility	$0
Water Distribution System Crossing - US Highway 6	$60,000
Sewer Crossing along Phoenix Rail Trail - Mattapoisett River	$112,000

[8] These are considered Long Term Costs. No Capital Costs are immediately recommended.

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