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

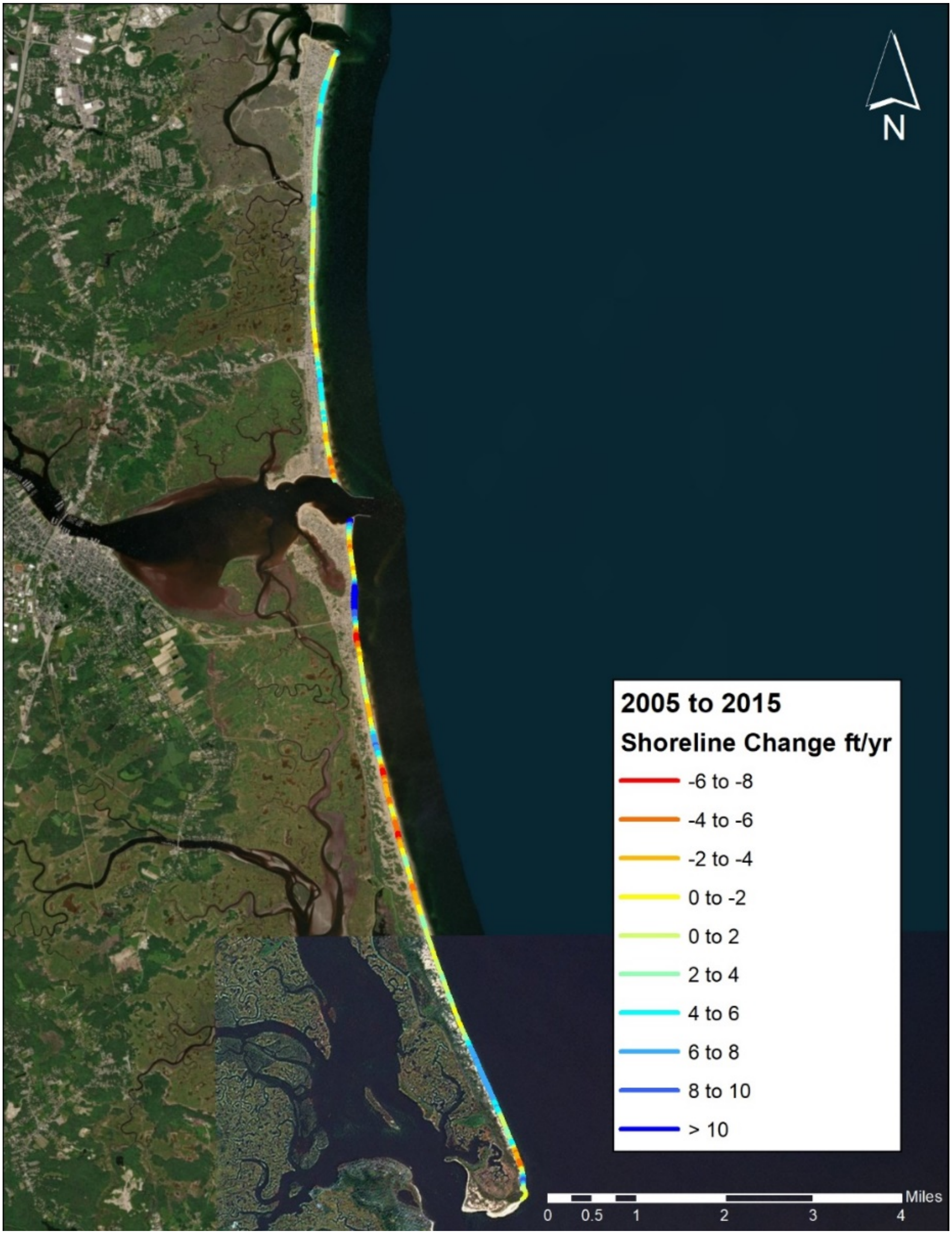

Figure 4.9 Shoreline change along Salisbury Beach and Plum Island between 2005 and 2015. Note that the accretion just north of center island can be attributed to the beach nourishment placed in 2010.

Formulation of Transport Calculations

The sediment transport equation employed for the longshore analyses is based on the work of USACE (1984). In general, the longshore sediment transport rate is proportional to the longshore wave energy flux at the breaker line, which is dependent on wave height and direction. Since the transport equation was calibrated in sediment-rich environments, it typically over-predicts sediment transport rates. However, it provides a useful technique for comparing erosion/accretion trends along the shoreline of interest. In the method described by USACE, the volumetric longshore transport, Q, past a point on a shoreline is computed using the relationship:

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

where I is the immersed weight longshore sediment transport rate, s is the specific gravity of the sediment, a' is the void ratio of the sediment, and ρ is the density of seawater. For this study, immersed weight longshore sediment transport, I, was computed using a method based on the so-called “CERC formula”,

$$I = KP_{ls}$$

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

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

where H_{sb} is the significant wave height at breaking, γ is the coefficient for the inception of wave breaking ($\gamma = H_b/h_b$), and α_b is the breaking wave angle. A value of K=0.39 is designated for use with significant wave heights (as output from SWAN).

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

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

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

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

where the coefficient K^* = 0.0013. The value of transport potential derived using this method represents the maximum possible at a particular location, given a rich sediment supply, and no structures (e.g., seawalls and groins) to modify the movement of sediment along the shoreline. From the

Using these empirical expressions of sediment transport potential, a computer code was developed which computed sediment transport potential along the Salisbury and Plum Island shoreline. Values of sediment transport are computed at evenly spaced grid cells, with positions that correspond to alongshore grid cells of the wave transformation model grid. For this application, transport potential calculations were performed using a 16.4-foot (5 m spacing) grid spacing, which corresponds to the grid spacing of the fine wave grid. The May 2005 shoreline, derived from satellite imagery was used as the input shoreline. The shorelines on either side of the inlet were modeled independently of each other with Figure 4.10 and Figure 4.11 showing transport potential for Salisbury and Plum Island respectively.

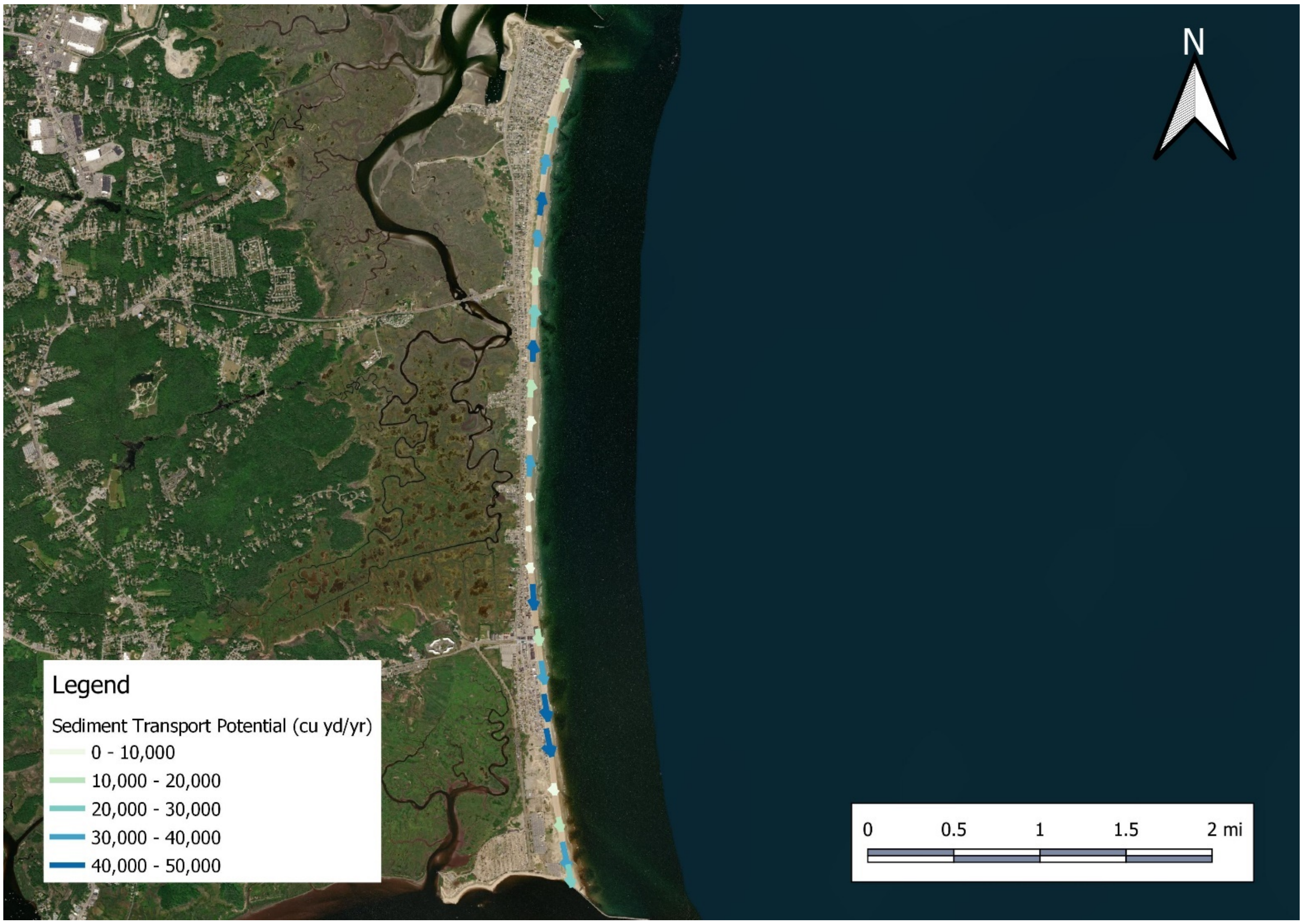

Figure 4.10 Computed average net sediment transport potential along the Salisbury Beach shoreline. Arrows indicate the direction of transport, while the color and size of the arrows corresponds to transport magnitude.

Figure 4.11 Computed average net sediment transport potential along the Plum Island shoreline. Arrows indicate the direction of transport, while the color and size of the arrows corresponds to transport magnitude.

Grain Size Distribution

Input into the sediment transport potential calculations include sediment grain size. A 0.68 mm representative grain size was determined for based on samples collected and analyzed by University of Massachusetts Amherst study (Woodruff et al., 2016).

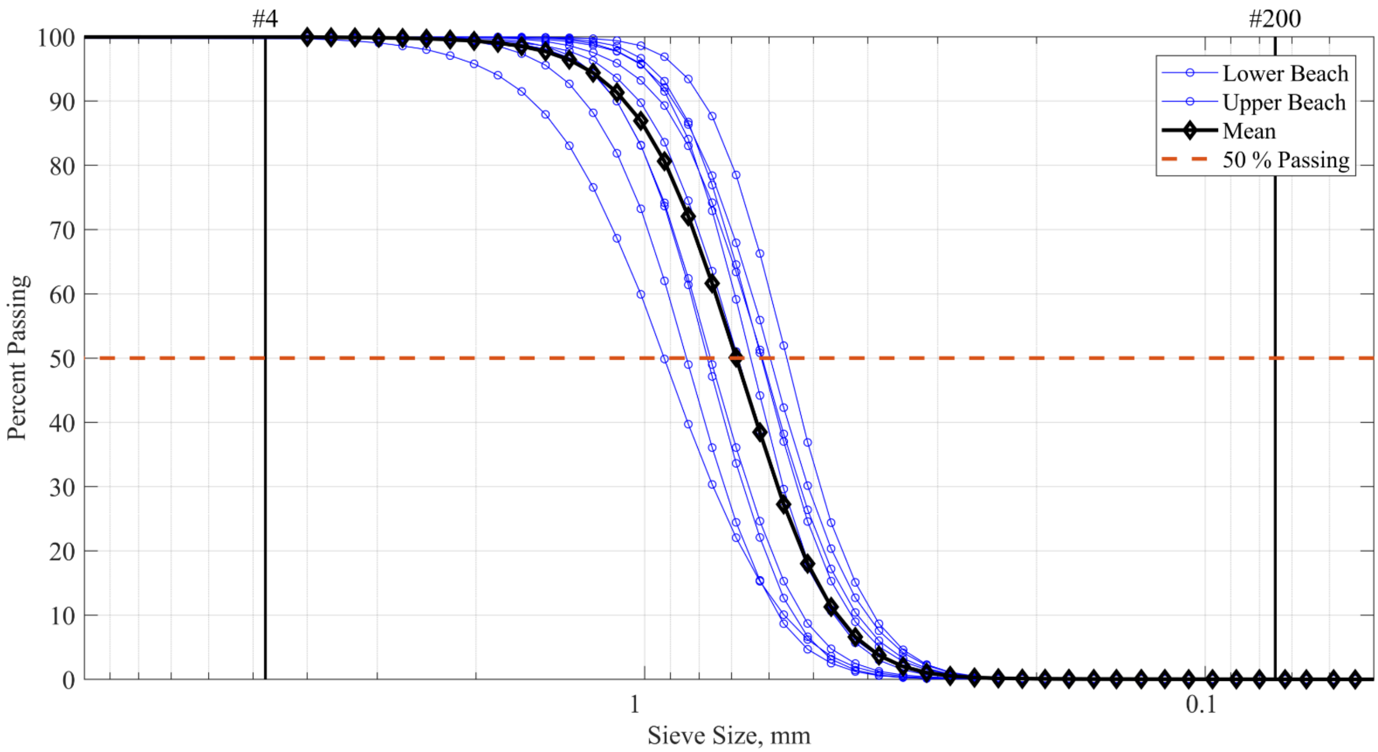

Figure 4.12 University of Massachusetts Amherst grain size data used to select a *d50* value of 0.68 mm for the one-line model.

One-Line Shoreline Modeling

Using this expression of sediment transport potential, a computer model was developed which simulates the conditions along actual shorelines, where coastal engineering structures impact actual sediment transport rates. The goal of the shoreline change modeling is first to predict measured shoreline change and longshore sediment transport rates, and subsequently use the model to evaluate beach management alternatives for both Salisbury and Plum Island.

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

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

where Q is sediment transport at a particular shoreline transect, x is alongshore width of a computational cell, y is the cross-shore position of the shoreline, t is time, q is a source term, D_B is the berm elevation of the beach, and D_C is the depth of closure. Values of sediment transport are computed at evenly spaced grid cells, with positions that correspond to alongshore grid cells of the wave transformation model grid.

Groins and seawalls, which act to hinder sediment transport and prevent shoreline erosion, can be included in the model simulation.

The one-dimensional model grid developed for Salisbury extends along the same 3.7-mile-long shoreline segment used to compute transport potential, and uses the same 16.4-foot (5 meter) grid spacing. Required input parameters for the shoreline model are the depth of closure and beach berm height, which together define the active beach profile, meaning the littoral area where wave induced sediment transport is the predominant transport mechanism. The depth of closure is an estimation of the seaward limit of the beach profile. By definition, areas where no depth changes occur are located beyond the depth of closure. For this study, the depth of closure was estimated using the method of Hallermeier (Dean and Dalrymple, 2001). Although sand motion can occur at bottom depths that are greater than the depth of closure (e.g., during storms), the net flux of sediment is not great enough to cause changes in the beach profile. The depth of closure is about half the depth for incipient sediment motion (Hallermeier, 1978). The depth of closure (h_c) can be computed using the relationship developed by Birkemeier (1985),

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

which uses the significant wave height and period that is expected to be exceed only for 12 hours each year, H_e and T_e. A useful approximation to this is given by $h_c = 1.57H_e$, where H_e is computed as $H_e = \overline{H} + 5.6\sigma_H$, and $\overline{H}$ and $\sigma_H$are the mean wave height and standard deviation of the wave record, respectively. Using a 35-year wave hindcast from WIS station 63045, H_e is computed to be 14.6 feet (4.5 meters), which results in a depth of closure of 22.9 feet (7 meters). Therefore, the depth of incipient sand motion is 45.8 feet (14 meters).

Similar to the computation of sediment transport potential, output from the wave modeling analysis is used to drive the shoreline evolution model. A time series of wave conditions was created using the Atlantic WIS hindcast (Station 63045) so that the 27 wave cases Table 4.1 representing mean annual conditions occurring from different compass sectors could be used in a time dependent simulation of shoreline movements. At each model time step (10 minutes) during the course of the seven-year model calibration period, a wave case from the 27 modeled cases was selected based on each separate wave record from the WIS hindcast. For hourly periods where waves were not propagating onshore from any of the ten compass sectors of Table 4.1, no waves were applied to the model shoreline for that time step.

Coastal engineering structures along the modeled shoreline segment are included in this model. Three groins are included in the lower grid. The groins act to impound sand, and are included in the model by introducing a permeability factor that reduces the transport rate across the grid cell where each groin exists. Permeability ranges between 0.0 and 1.0, where 0.0 would be a completely impermeable block to transport and 1.0 represents a structure that has no sand holding capacity (e.g., completely unraveled or filled to bypassing). For the groins at the north end of Plum Island, the permeability factors were set at 0.6, 0.75, and 0.85 (north to south) to represent the ability of each groin structure to by-pass sediment. If at any point during the simulation the shoreline accretes past the tip of the groin, the permeability is set to 1 and sand is allowed to move across the structure uninhibited. There are some revetment structures located along Plum Island to protect homes. The revetment acts to limit the shoreward movement of the shoreline as it moves during the course of the simulation. If the shoreline at any grid cell erodes to the point where it comes into contact with the revetment, the shoreline is not allowed to move farther shoreward. Unlimited accretion is allowed in front of the revetment.

Model performance was calibrated over a 10-year simulation between 2005 and 2015, for both the upper (Salisbury; Figure 4.13) and lower (Plum Island; Figure 4.14) grid shorelines. The model input shoreline was digitized from the 2005 aerial set. Wave cases were generated using the WIS record, which has nearly complete coverage of this time period, less a few months in early 2015. The computed shoreline at the end of the 10-year simulation was compared to the shoreline digitized using the 2015 aerial orthophoto set. Calibration was achieved by applying a background erosion rate in order to minimize the RMS error. A background erosion rate of 1.7 feet per year was applied to the Salisbury model and a rate of 2.3 feet per year to the Plum Island model. The error of the final calibration run was 11.7 feet and 10.4 feet for Salisbury and Plum Island respectively, which are comparable to the uncertainty associated with the aerial photo analysis (14.1 feet; Figure 4.9). The calibrated one-line model was then used in Section 5 for analysis of alternatives.

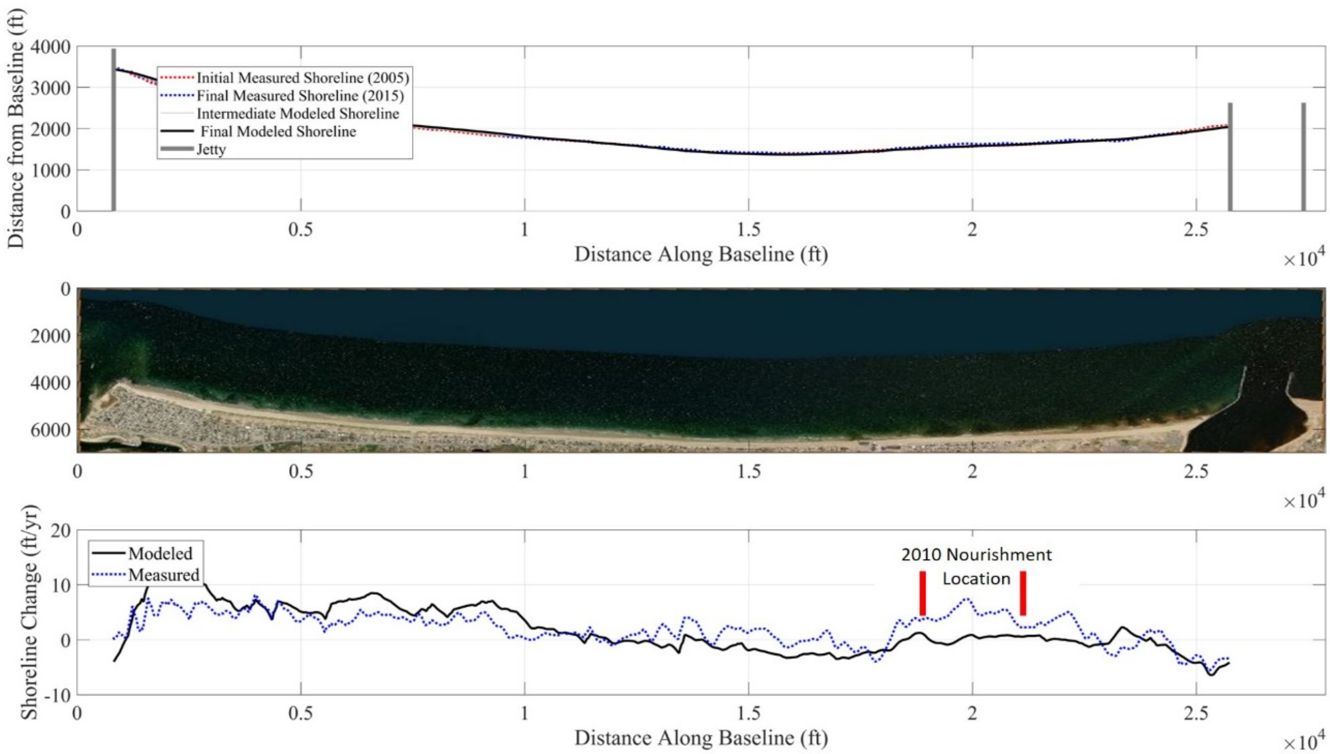

Figure 4.13 Comparison of modeled and measured shorelines for Salisbury Beach for the shoreline model calibration period between 2005 and 2015. The calculated RMS error for the sandy segment of the shoreline is 11.7 feet, and the R^2 correlation coefficient is 0.94.

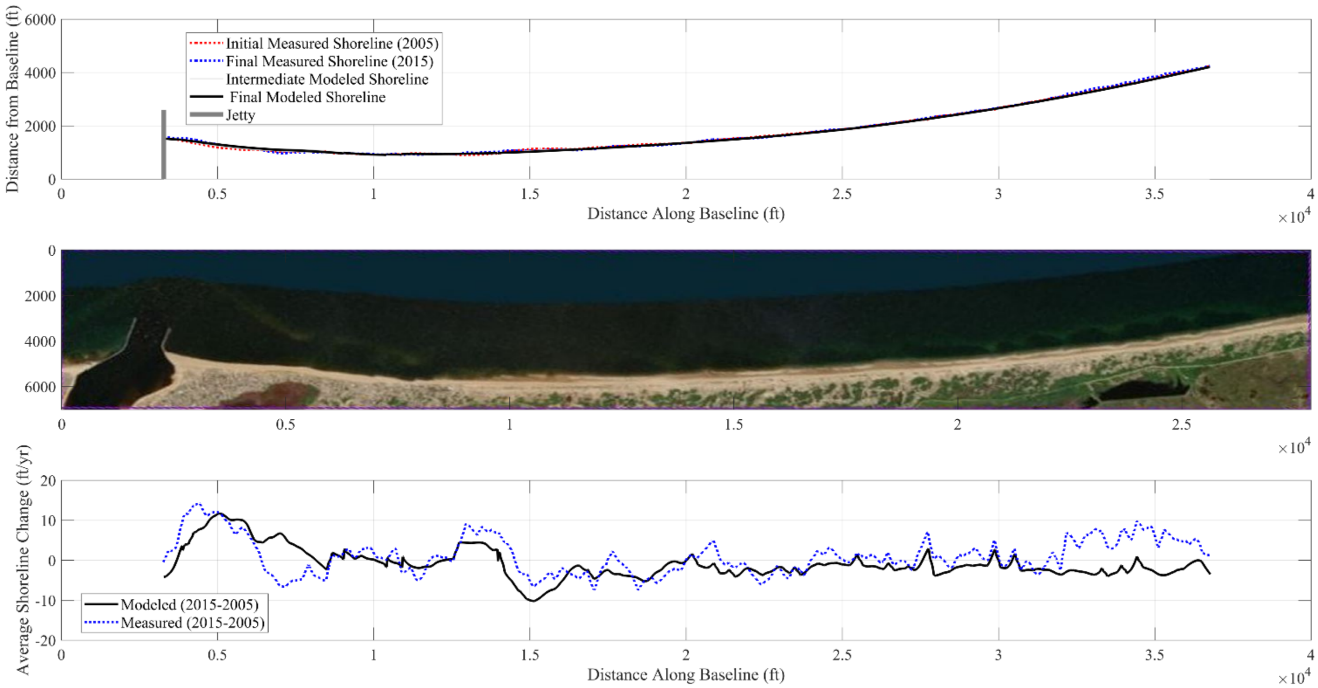

Figure 4.14 Comparison of modeled and measured Plum Island shorelines for the shoreline model calibration period between 2005 and 2015. The calculated RMS error for the sandy segment of the shoreline is 10.4 feet, and the R^2 correlation coefficient is 0.92.

4.3 CMS-FLOW Hydrodynamic Model

The CMS numerical modeling package that was used for this study was developed by USACE Coastal Hydraulics Laboratory (CHL). The present hydrodynamic analysis of the inlet was undertaken to evaluate the flow in and around the inlet, in addition to associated sediment transport patterns. Potential alternatives would be evaluated for adjusting dredging or structures in the vicinity to determine solutions to prevent further erosion of the shoreline. The package includes separate CMS-Wave and CMS-Flow models that can be run independently or together to simulate sediment transport and morphology change that results from the combination of waves and hydrodynamic currents. CMS is an integral part of the Surface Water Modeling System (SMS) software application (graphic user interface). SMS is used to create model grids and specify all required model inputs and runtime parameters.

Simulations of hydrodynamics and sediment transport in the CMS system are computed using CMS-Flow. The latest releases of CMS and SMS allow a particular type of irregular Cartesian grid, called a “telescoping mesh”, to make more efficient refinements of the grid in locations of interest within the model domain without forcing small cell sizes in areas where it is not required or desired. Each level of mesh refinement is achieved by dividing a cell edge into two equal segments that become the edges of two new cells with a quarter of the area of the larger cell. As a result, no grid cell edges in a mesh are ever connected to more than two adjacent cells. This allows a very rapid change in mesh cell size over short distances.

Hydrodynamic features included in CMS-Flow include stable wetting and drying, with the possibility of ponding, wind forcing, spatially varying bottom friction, inclusion tide gates, and multiple methods for designating hydrodynamic boundary conditions. CMS-Flow also can model salinity transport. Boundary conditions can be specified by using a time series of water level elevations from a source such as a NOAA tide gage or some other source of tide data. Alternately, boundary conditions can be can be extracted from a larger hydrodynamic simulation that the CMS domain is nested within.

4.3.1 CMS-Flow Model Grid

The development of the hydrodynamic model for the Merrimack River Inlet proceeded first with the creation of the model grid. The grid specifies the special extent of the model domain, and includes the model topography/bathymetry and is used to designate other spatially varying model parameters, such as friction coefficients. The model grid is shown in (Figure 4.15). The grid is made up of 165,042 total cells, with 160,265 active computational cells. Cell dimensions range between 6.56 feet (2 meters) and 419.95 feet (128 meters) for the entire mesh, including non-active upland areas.

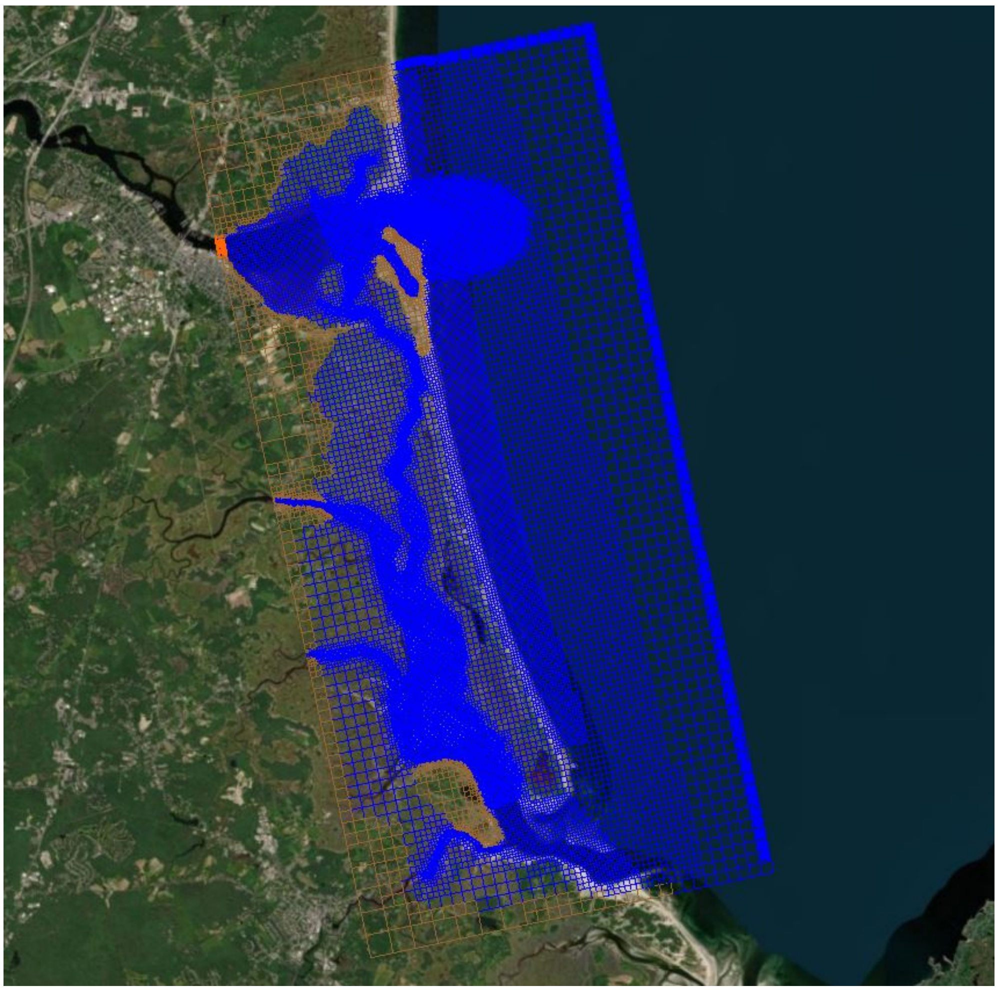

Figure 4.15 Extents of the CMS Flow grid for the inlet. The telescoping mesh allows for coarser grid cells in regions of minimal interest and for finer detail in regions of high interest in and around the inlet

The larger CMS Flow grid was developed for the entire marsh system to generate the boundary condition behind Plum Island at the turnpike bridge (Figure 4.16). A flow boundary condition was applied at the entrance to the Merrimack River, which was generated using an RMA-2 (USACE) model developed for MassDOT to examine the Whittier Bridge (Figure 4.17). An offshore boundary condition was applied along open ocean cells with the Fort Point Station in New Hampshire. The offshore boundary data were supplied from a 2018 USACE study in the region (see Appendix C). Two nested grids were employed to analyze flow patterns in different regions of interest. The telescoping mesh feature of CMS allows the user to increase the resolution of areas of interest. Two smaller CMS Flow grids were developed to analyze regions of interest. The inlet model has 165,042 cells (Figure 4.18) and the bypass bar model has 412,539 cells (Figure 4.19).

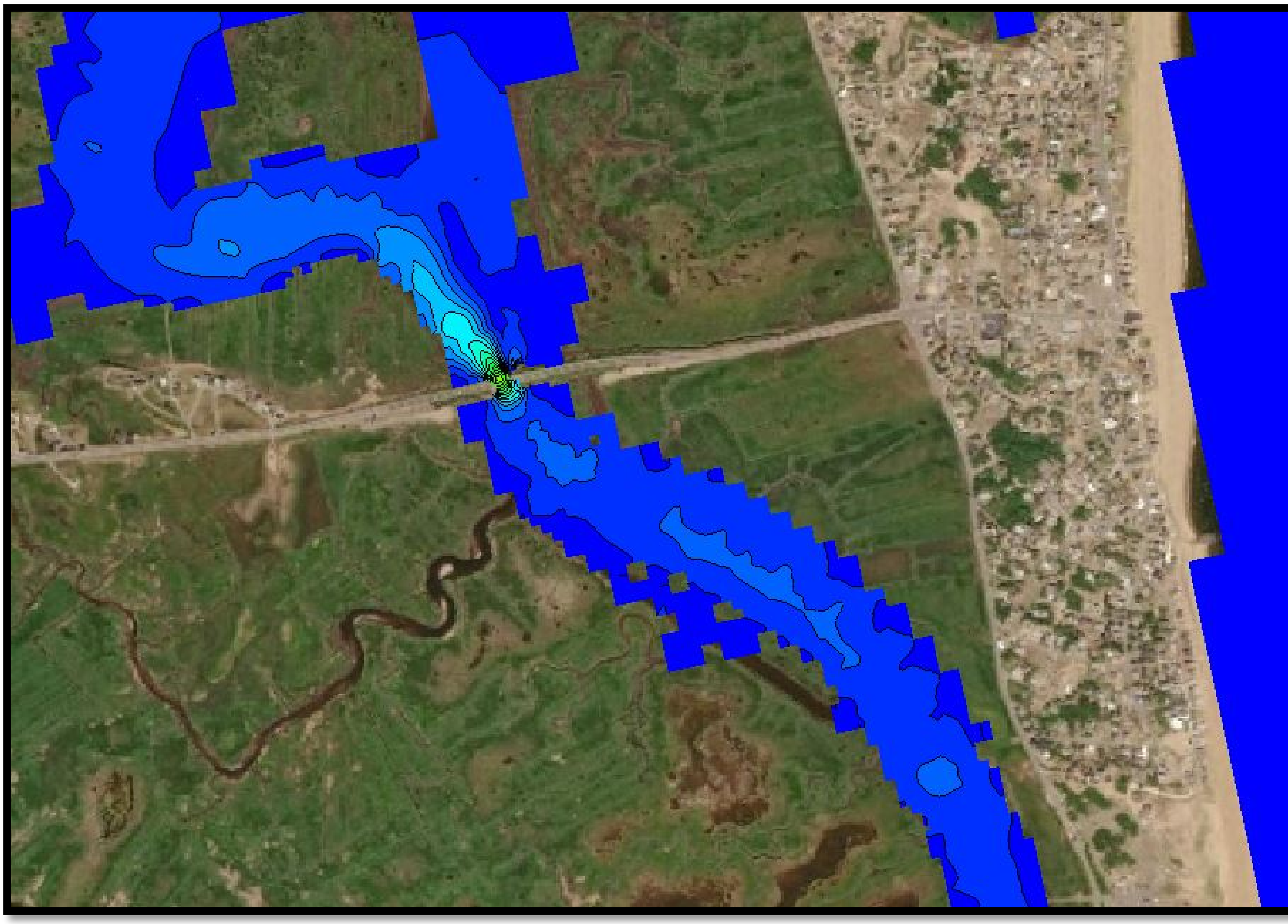

Figure 4.16 Plum Island turnpike bridge location of boundary condition.

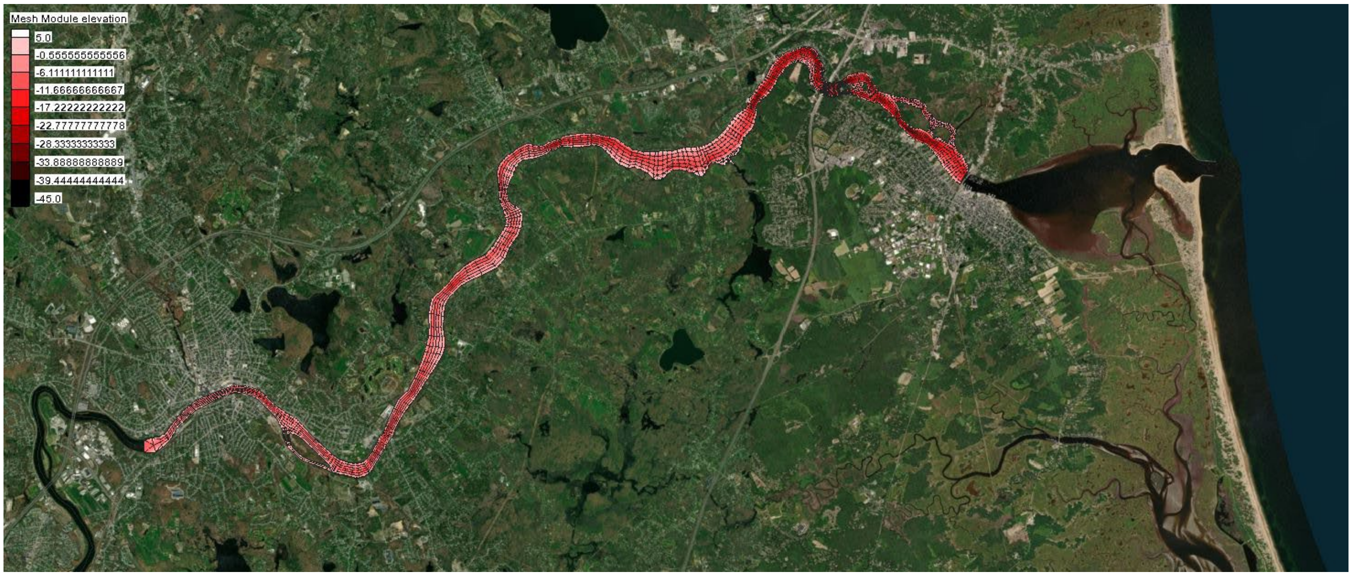

Figure 4.17 Calibrated RMA-2 model grid for the Merrimack River.

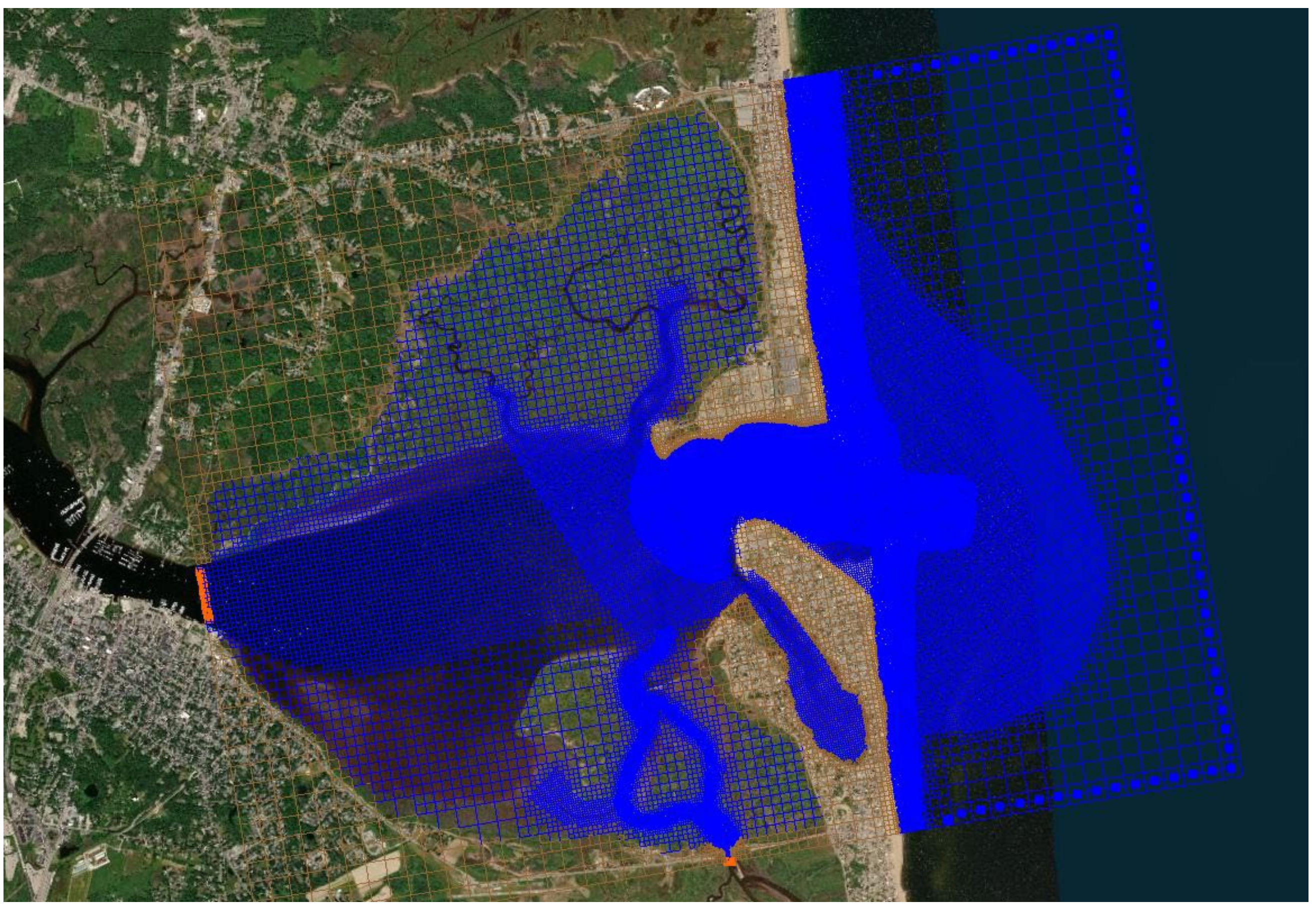

Figure 4.18 Extents of the CMS Flow grid for the inlet. The telescoping mesh allows for coarser grid cells in regions of minimal interest and for finer detail in regions of high interest in and around the inlet

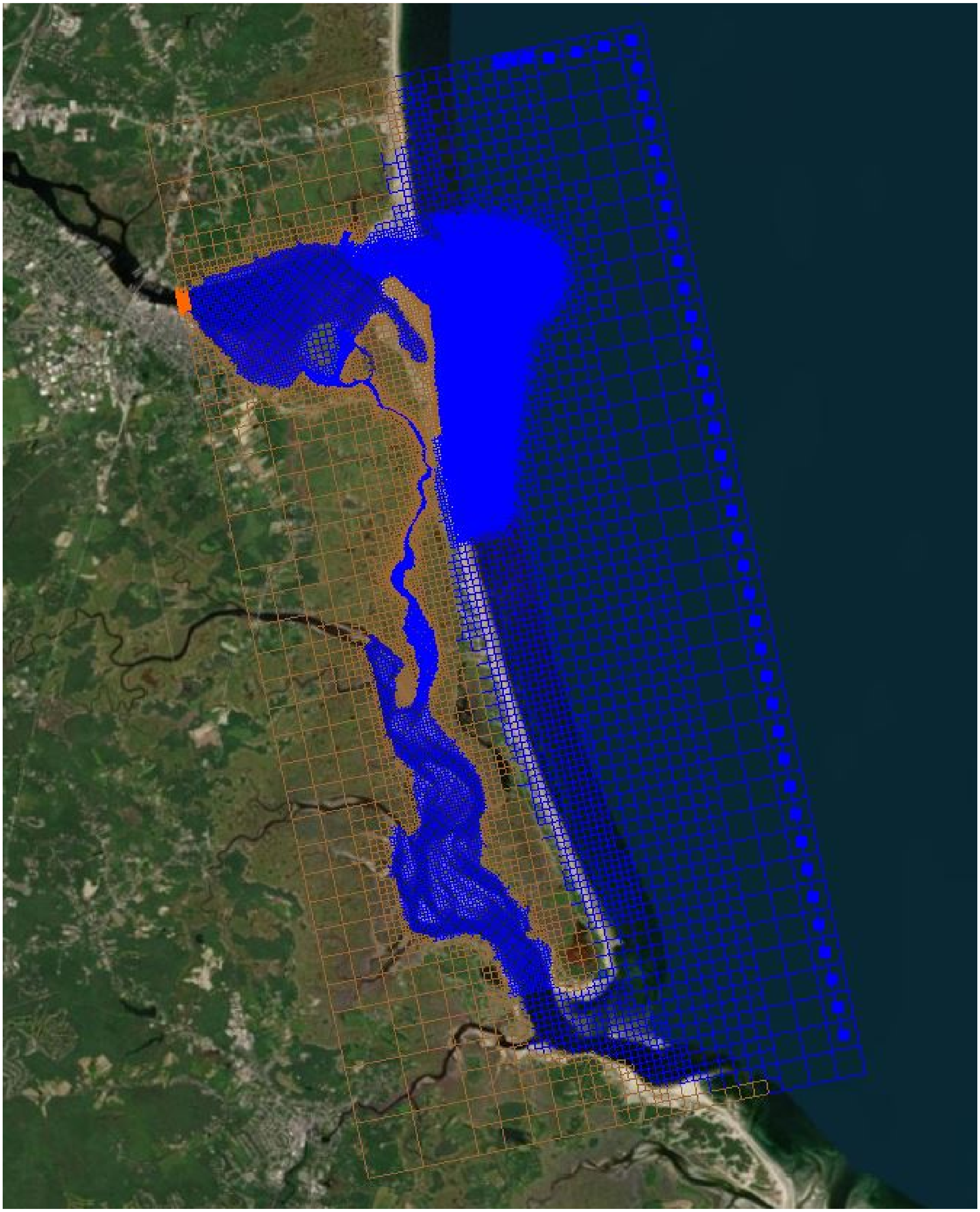

Figure 4.19 Extents of the CMS Flow grid for the bypass bar. The telescoping mesh was focused on the 3 km of bypass bar to the south of the southern jetty.

As described above, the combination of tidal currents and waves within the entrance channel to the Merrimack River requires a time-dependent analysis of coastal processes to quantify sediment transport patterns along the shorelines immediately west of the two jetties. This time-dependent approach will incorporate typical tides and freshwater inflow through several tidal cycles combined with time-varying wave conditions as input conditions to drive sediment transport. The modeling tool for this analysis is CMS, which Applied Coastal has utilized for sediment budget analyses at both Petit Bois Pass (Alabama) and Bournes Pond Inlet (Falmouth, Massachusetts).

Due to the computational requirements of this type of modeling, 'typical' conditions were evaluated to quantify sediment transport pathways. These typical conditions were based on an approximate one-month long simulation (January 2006; see section 4.3.2). To model a full Winter season, a morphologic acceleration factor was employed to extend the simulation beyond the one-month period. Calibration data for the model was based on both historical shoaling patterns, as well as hydrodynamic data available from a modeling effort performed for the Merrimack River. Similar to the open ocean shoreline analysis, the modeling analysis within the Merrimack River entrance can provide quantitative information regarding sediment transport pathways that helped form the basis for the regional sediment budget.

4.3.2 CMS WAVE

CMS-Wave is a finite difference spectral wave model. The wave action balance equation is the basis for model formulation, similar to SWAN. Model formulation includes wave diffraction terms, though the model must be calibrated to ensure that diffraction is correctly applied in areas where this process is an important contributor to wave propagation (such as behind breakwaters and jetties). Other physical processes that are included in CMS-Wave are wind-wave growth, wave energy dissipation by breaking and white capping, wave shoaling and refraction, and reflection. As presently implemented in SMS, model grids for CMS-Wave can be either regular Cartesian grids (like with STWAVE) made up of square cells with equal edge dimensions, or irregular Cartesian grids made up of rectangular elements. The irregular mesh capability allows a method of model mesh refinement in areas of interest without requiring the same small cell size be used for the entire grid.

Wave data used for each CMS-Wave model run can come from most any source of wave data, including buoy measurements or wave hindcasts. In addition to spectral data, parameterized wave data can be used to develop input spectra data required by CMS-Wave. WIS hindcast data provide 35-year-long records of waves at many stations along the coastline of the United States (Tracey, 2002, see Section 4.1.1), but only wave parameters (e.g., height, period, and direction) and not complete 2D spectra. These data can be imported into SMS and used to create spectra for a CMS-Wave simulation. Within SMS, the user must select the type of frequency spectrum (e.g., JONSWAP or Bretschneider) and the directional spreading function to use for the calculation of a two-dimensional spectrum.

The ability to nest wave simulations into grids with larger-scale simulations is also a feature of CMS-Wave. This is another method that can be employed to efficiently refine a grid mesh only in areas that are considered most important for the analysis. A grid nested within a larger mesh receives spatially-varying open boundary wave spectra along the full length of the offshore open boundary of the grid, extracted at locations that correspond to the cells of the nested grid.

A single month of wave conditions was selected from the 35-year WIS record at station 63045 for CMS Wave runs. It was determined that January 2006 met the criteria for best fit month. Normalized wave energy was the parameter utilized to identify the month. First, the wave energy was plotted for each compass sector of the 35-year period (Figure 4.20). The average was taken for each compass sector. To select a best fit year, the root mean square deviation (RSME) of each year relative to the average of each year were assessed. The year with the lowest RSME was 2006. Due to the computational requirement to run a full year of a simulation, a month was chosen as a duration to simulate. A similar procedure was followed for selecting a month by comparing the RSME of each month to the average conditions. January of 2006 was selected as the month as it met the criteria of being average, but also included some characteristic storm events to drive transport. Once the models were calibrated and provided boundary conditions, several preliminary management alternatives were evaluated.

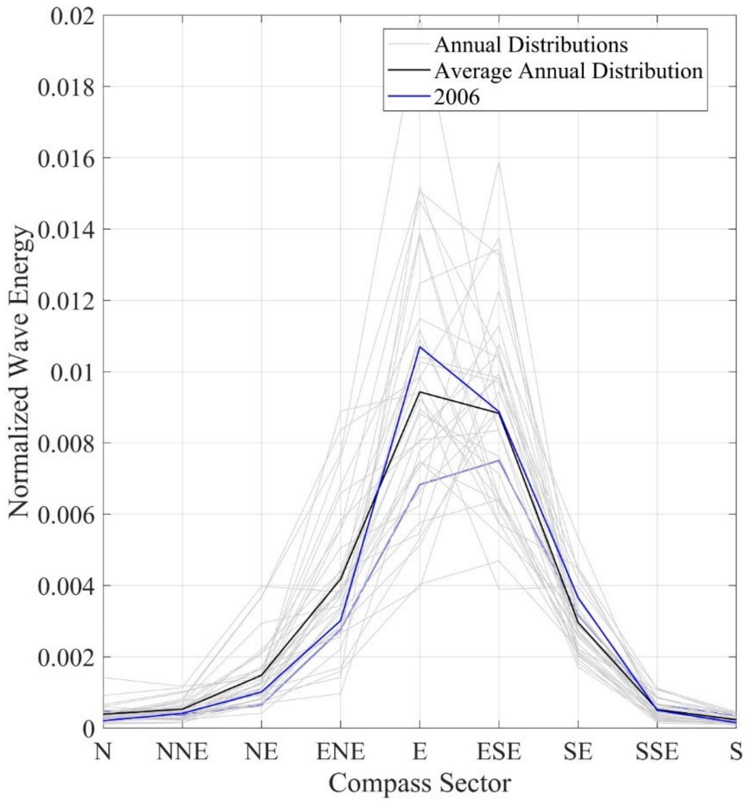

Figure 4.20 Normalized wave energy for each year of the 35-year WIS record.

5.0 POTENTIAL SHORELINE ADAPTATION ALTERNATIVES

Applied Coastal used analysis of existing conditions, historical records, and modeling to assess potential shoreline management alternatives for each section of shoreline (Figure 5.1). The two-phased modeling approach detailed above, which incorporates different tools to assess the ocean-facing and interior Merrimack River shorelines, provides the most defensible approach for comparing shoreline stabilization methods for each region.

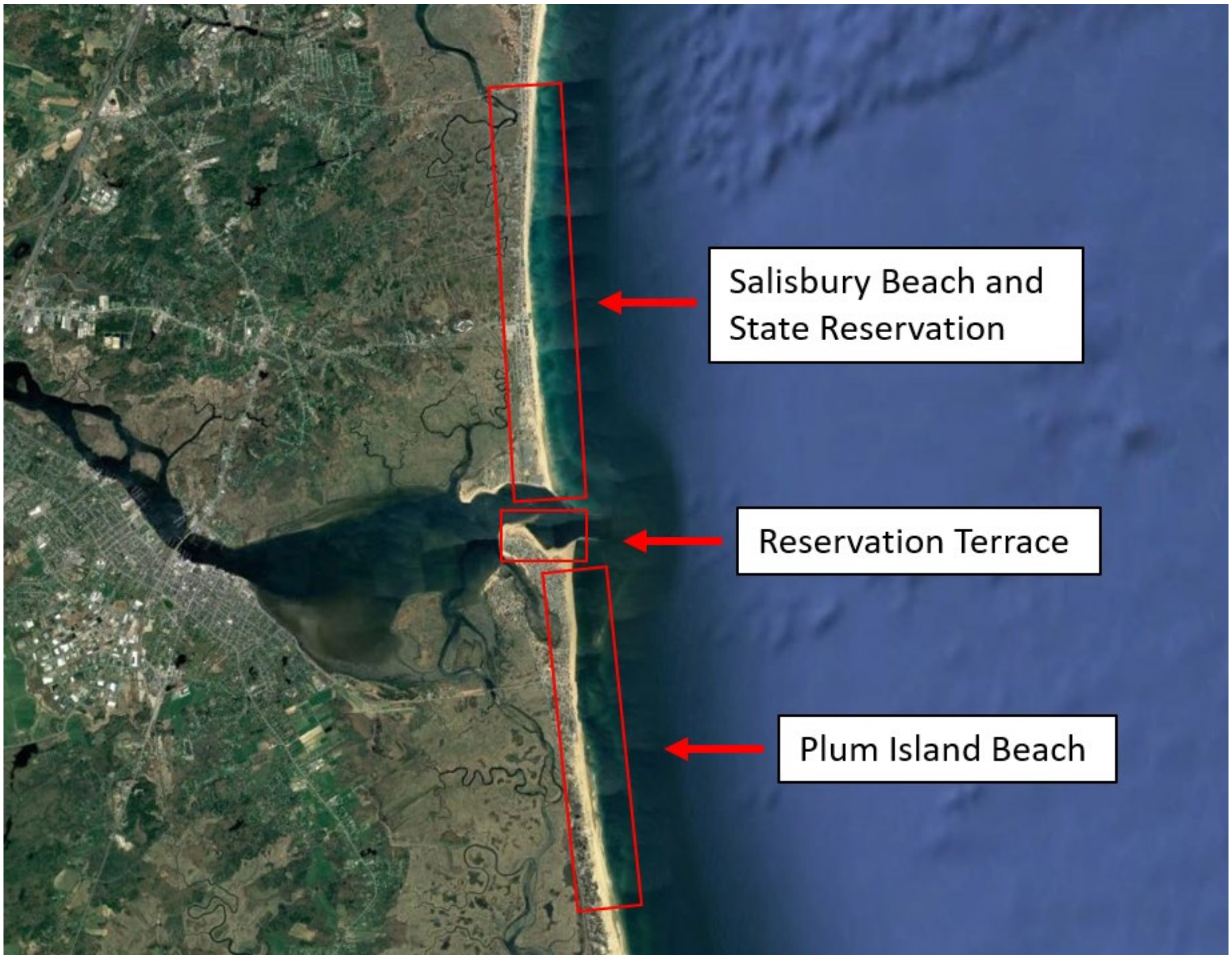

Figure 5.1 Aerial photo identifying each of the regions that were analyzed in the study. Each region has its own associated management challenges and recommended solutions.

5.1 Salisbury Beach and State Reservation

Salisbury Beach, located north of the Merrimack River inlet, maintains a moderately stable shoreline with net annual littoral drift in a southerly direction. Evidence for the net southerly direction include deposition along the north side of the north jetty and powerful northeast storm wind and waves. Short term analysis of shoreline change indicates that there are periods of localized erosion and accretion, likely tied to changes in the position and orientation of the sandbar located several hundred feet offshore. Although the lower part of the profile remains relatively stable, recent storm events (e.g., March 2018 northeasters) eroded much of the upper beach as waves and runup reached properties along Salisbury Beach. Lack of consistent sediment supply and storm events have resulted in net erosion of the shoreline.

There have been some attempts to curb net erosion of the shoreline with smaller nourishment projects. Other than a 2010 nourishment of 40,000 cubic yards, recent nourishments (in the past 20 years) have been small (< 10,000 cubic yards). Although the nourishments proved beneficial by either extending the shoreline seaward or built up dunes for additional storm surge protection, the volumes have not been sufficient to mitigate net erosion of the shoreline. Applied Coastal believes a substantial nourishment provided to the shoreline with periodic replenishment would be an effective management solution for Salisbury.

5.1.1 Salisbury Beach Nourishment

Beach nourishment refers to an engineered beach that is designed to withstand storm conditions including the effects of storm surge and wave action. A nourishment at Salisbury Beach would add sediment seaward of the existing beach profile to absorb and dissipate wave energy, thereby increasing protection to infrastructure and property currently threatened by overtopping and storm damage. Addition of this volume of beach compatible sediment is designed to last several years, where the design life is dependent on the local sediment transport dynamics and berm overtopping potential. Once nourishment material is in place, coastal processes will rework the nourishment material to create an equilibrated beach profile. The ongoing sediment transport will migrate the nourishment material both cross-shore and alongshore.

The sediment transport potential modeling results indicate that transport rates reach a maximum south-directed magnitude just south of the Music Hall. Overall, the transport rates do not vary substantially, so a nourishment project would be most effective along shoreline stretches that have seen historical damage based on repetitive loss data. Building on the insights provided by the sediment transport potential analysis, the shoreline model was used to simulate different nourishment templates, to investigate how the increasing berm widths and sediment volumes would evolve with time along the coast. Each nourishment was designed to raise the berm elevation to 11 feet NAVD88 and extend the existing shoreline seaward widths ranging from 25 to 75 feet (Figure 5.2). Dimensions of the nourishment were selected based on the likely availability of resources based on cost and typical dredging volumes. The footprints of the recommended nourishment alternatives were selected based on the location of repetitive loss claims (Figure 3.2). Figure 5.3 shows the approximate extents of shoreline recommended for nourishment, which range in length from 5,000 to 9,000 feet.

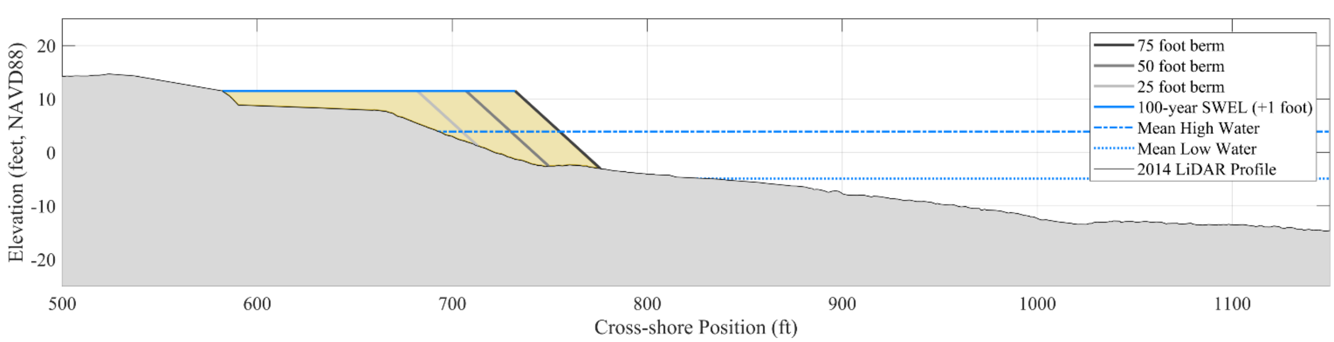

Figure 5.2 Profile view of each nourishment alternative (75, 50, and 25-foot berms). Each berm is designed to an elevation of 11 feet NAVD88, or one foot above the FEMA 100 year still water elevation. Properties along the shoreline sit at an elevation at or above15 feet NAVD88.

Figure 5.3 Approximate extents of shoreline considered for nourishment along the Salisbury shoreline. The general footprint of the recommended nourishment location varied between 5,000 and 9,000 feet, depending on the availability of nourishment material and funding.

Nourishments are typically designed for a lifespan, at which point management decisions can be made about perusing future replenishments of the beach for shore protection, recreation, and preservation of natural resources. Model runs for each fill scenario (Table 5.1) were executed for a simulated 15-year period. One metric used to measure a beach nourishment's performance is based on the percentage of fill remaining, compared to the un-nourished beach. Typically, a value of 30% of the original volume is used to determine the point at which a replenishment is necessary. The percent fill remaining is computed as the volume of sand remaining (V, at a particular point in time $t=n$) within the original project limits, divided by the total volume of the nourishment at time zero: The state

$$\%_{remaining} = \frac{V_{t=n}}{V_{t=0}} \times 100 \, .$$

Though sand from the original nourishment through time disperses up- and down-drift from the nourishment template, benefiting the beach outside of the original project limits, it is not counted in the estimation of fill performance. Based on results of the shoreline modeling, the nourishments are able to maintain 30% of their fill for 15 to 20 years. A comparison of fill performance for the four scenarios is included in Figure 5.4, showing percent fill remaining after each modeled year. It is seen that the 50 ft wide tapered berm nourishment outperform the 25 ft berm nourishment with the same total length. This is because the erosion rates for both fill designs are essentially the same. Since the wider design has more shoreline to erode, it will logically last longer than a design with less width. Table 5.1 compares the dimensions and relative performances of each alternative. The modeled position of the Scenario 4 berm immediately after construction and 15 years into the future relative to the 2005 and 2015 shorelines is shown in Figure 5.5.

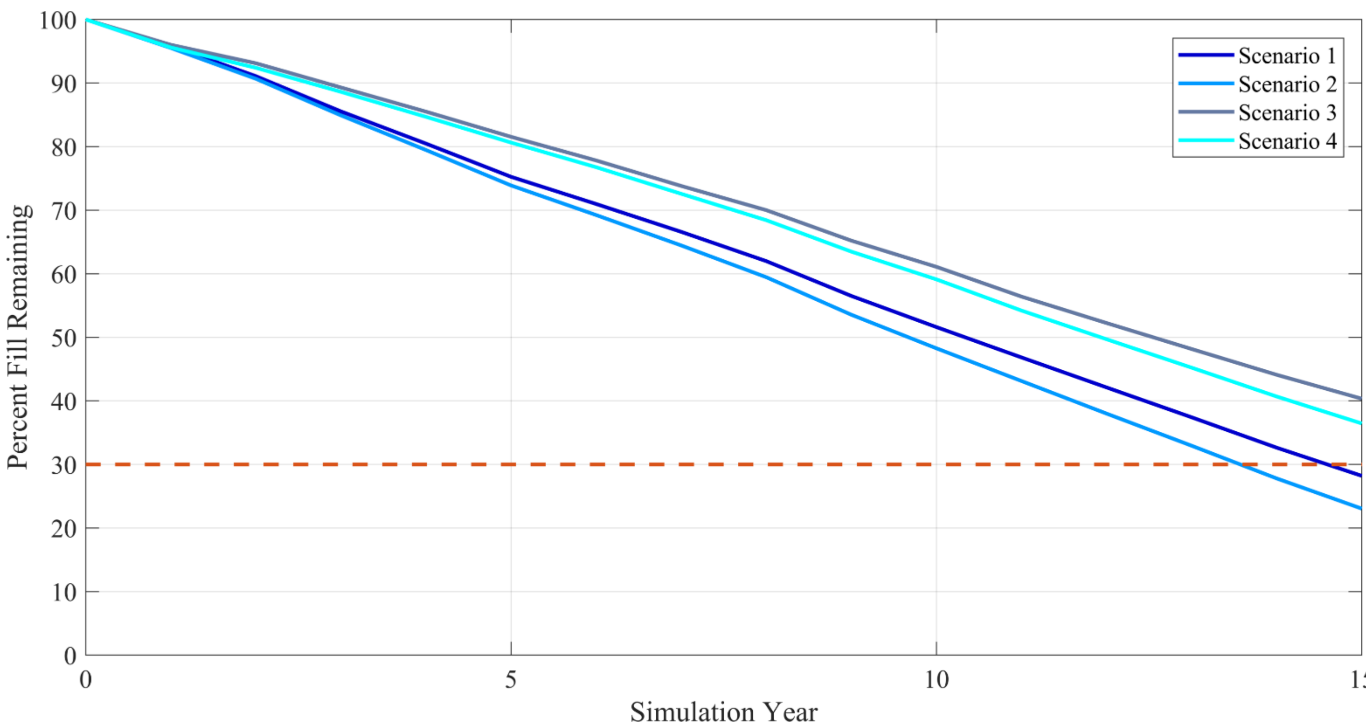

Figure 5.4. Performance comparison between nourishment scenarios for Salisbury Beach.

Table 5.1 Modeled beach fill scenarios for Salisbury Beach.

scenario	Volume (yd^3)	berm width (ft)	fill length (ft)	Year of 30 % Fill Remaining
1	125,000	50, tapered 25	9,000	14.6
2	100,000	25	9,000	13.5
3	150,000	75	5,000	17.4
4	110,000	50	7,000	16.7

Figure 5.5. Modeled position of the Scenario 4 berm after 15 years relative to the 2005 and 2015 shorelines.

Figure 5.6 compares 25-, 50- and 75-foot nourishment widths along two different cross-shore profiles. Erosion along the nourishment template is not uniform, so an additional metric to compliment percent fill remaining, is to the measurement of beach width changed relative to nourishment performance. An example metric would be a so-called trigger point, at which a portion of the shoreline erodes back to, indicating a need for a nourishment in that location. A potential trigger point could be a 15-foot buffer beyond the existing berm. At the Music Hall location, the nourishment retreats to this point after 2, 6, and 11 years respectively for the three nourishment widths. The concept of a trigger point can be important for resupplying sections of shoreline that have experienced a focusing or 'hot-spot' erosion during minor or moderate storm events, when a full-scale nourishment is not necessary for the continued protection of the shoreline. These 'hot-spots' fills can be supplied with an emergency sand supply that the next section will discuss in further detail.

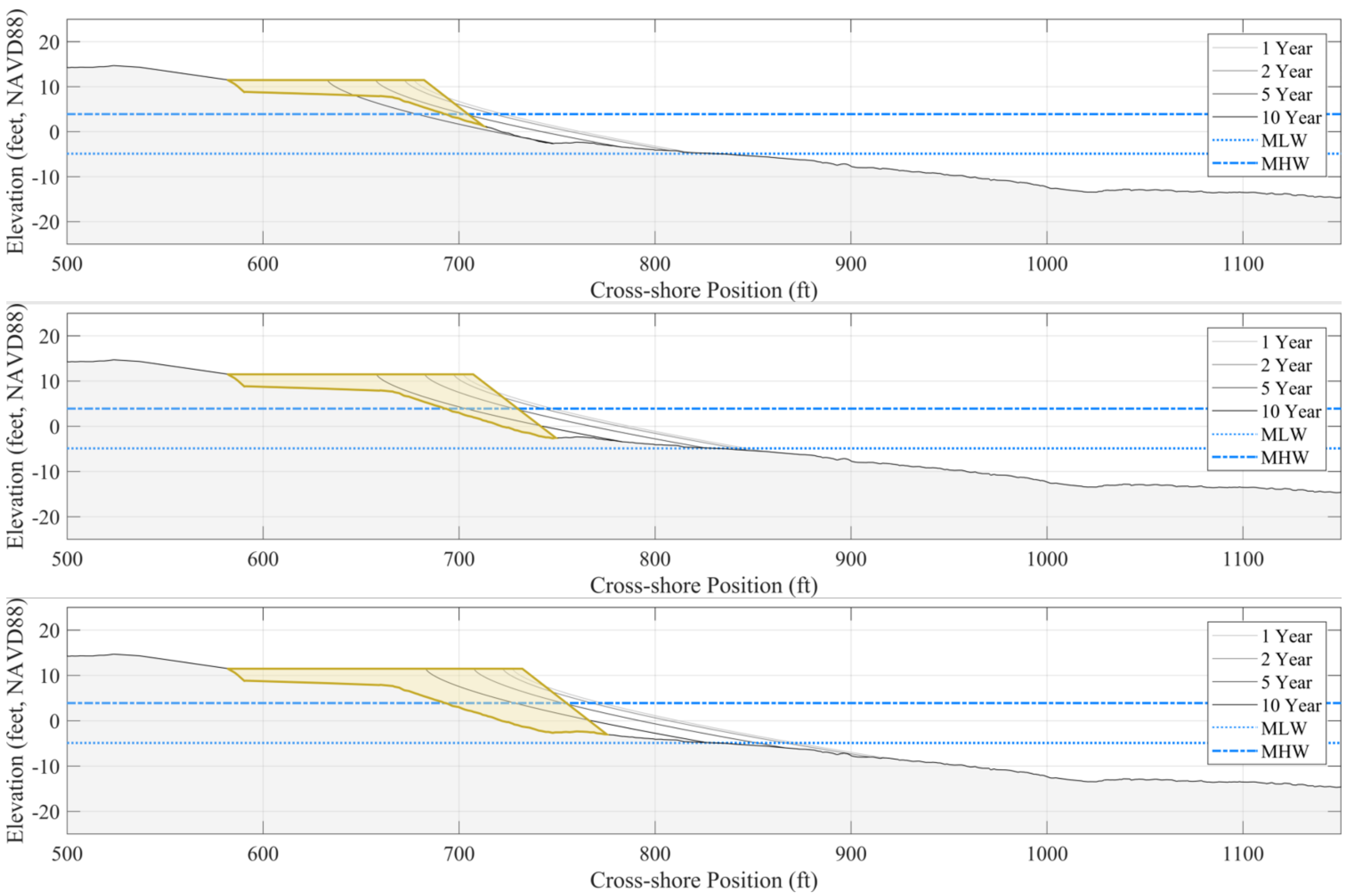

Figure 5.6. Comparison of 25 (top), 50 (middle), and 75 (bottom) foot berm shoreline positions along a transect (2014 LiDAR) crossing the Music Hall in Salisbury.

5.1.2 Emergency Sand Source (SSA)

In the 2008 Salisbury Beach Management Plan (BMP), it was determined that DCR would create and maintain a Sand Stockpile Area (SSA) approximately four acres in size behind the headquarters building at Salisbury Beach State Reservation and would maintain at minimum 30,000 cubic yards of material. Discussions by the project group determined that a volume of 30,000 cubic yards would be difficult to maintain for logistical reasons. A lesser volume determined by DCR based on historical volume requirements would simplify management and mobilization. This additional sand source would be available for deployment along the Salisbury coast following a storm event. This sand will be important for locations that would not receive nourishment under the recommended nourishment plan. For example, on the north side of the inlet along Salisbury Beach State Reservation, tidal currents scour the edge of the channel. Several hardened features along the shoreline, including Badgers Rock and two groin structures, deflect currents away from the shoreline. However, there is a stretch of beach at the inner end of the jetty that has been eroded back to the seawall. This section of shoreline might could use some material if the shoreline continues to recede.

5.2 Reservation Terrace

As mentioned in Section 3.3, Applied Coastal evaluated forces acting on the Reservation Terrace shoreline and determined strong flood tidal currents and waves occurring during a high storm tide would generate critical erosional forces. The shoreline is mostly sheltered from both ebb and flood currents running along the channel. However, greater tide ranges generate a gyre circulation pattern, which forms just to the west of the south jetty. The currents from this gyre have the velocity magnitude necessary to transport sediment suspended by waves away from the shoreline. The circulation forms during a flood tide and is amplified by greater tidal ranges (e.g., spring tides or storm events). At peak flow, modeled velocities are on the order of 2.5-3.0 ft/s along the south jetty (Figure 3.8).

Despite these dominant erosive forces along the shoreline, there are also several accretive forces that build the shoreline up. It is unlikely that the cross-shore directed transport results in all of the accretion occurring along Reservation Terrace, as some of the sand is ending up along the spit. A probable source of these periods of shoreline accretion is the bypassing of sediment over the jetty. To test this theory, model simulations using 2010 LiDAR elevations of the jetty (the jetty was slumping during this period) indicate flow across the jetty disturbs the gyre flow, as shown in Figure 3.7. The rehabilitation of the jetty in 2013, restored the design crest height, eliminating the bypassing of sediment over the jetty.

As mentioned above and Section 2, the jetties have been repaired on several occasions, notably in 1960's as well as more recently in 2013. To analyze the shorelines along Reservation Terrace, Applied Coastal used both historical imagery and LiDAR to identify approximate shoreline positions. Analysis of both LiDAR datasets and google aerial imagery indicate that the rehabilitation of the southern jetty in 2013 had implications on the shoreline along Reservation Terrace and northern end of Plum Island. A CMS Flow model was developed to better understand the characteristics of the gyre, in the hopes of reducing its impact on Reservation Terrace. Although a nourishment would provide the shore protection necessary for homes along Reservation Terrace, Applied Coastal believes a nourishment would be more effective if tidal currents and wave action are disrupted. Unless steps are taken to disrupt the erosive forces on the shoreline (i.e., structural improvements), the shoreline is likely to continue to erode at a rate of 30-70 feet per year, based on the LiDAR data available over the past 10 years. The natural unraveling or collapsing of the southern jetty following a storm event is another potential means of decreasing the erosion rate along the Reservation Terrace shoreline. However, the unraveling of the jetty would result in increased erosion at the east facing northern end of Plum Island, similar to pre-2013 conditions, Applied Coastal tested a number of alternatives to disrupt the flow of tidal currents and improve performance of a nourishment along the shoreline. The two structural modifications, in addition to nourishment options are discussed below.

5.2.1 Weir Jetty

An alternative to reduce flow velocities stemming from the gyre formation during flood tides would be to re-create pre-rehabilitation conditions along the jetty and allow flow to pass over sections in a controlled manner. The installation of a weir, or lowering of a section in the south jetty is proposed to reproduce these conditions and allow flow to pass over top of the lowered section to reduce gyre currents. The weir would mimic the jetty conditions prior to rehabilitation in 2013 and would likely be the least expensive alternative proposed.

Currents passing over the lowered jetty crest would partially block and reduce the velocities of the gyre to minimize the volume of sediment that would be transported from the shoreline. Different weir lengths were modeled in CMS to determine an optimal weir length. Results indicate that a length of 80 to 100 feet

(25 to 30 meters) achieves an effective reduction in currents, while minimizing the impact to the jetty. A weir length greater than 100 feet (30 meters) reduces currents at a position along Reservation Terrace by 45 - 50%. One important impact of the jetty weir to consider is that it would allow sediment to pass over top of the weir and into the inlet. The bypassing of sediment would likely help stabilize the Reservation Terrace shoreline, but could prove erosive to the northern end of Plum Island if designed poorly allowing substantial volumes of sediment to bypass the structure. Removed stone could be used as a jetty spur to better manage the passage of water and sand through the opening.

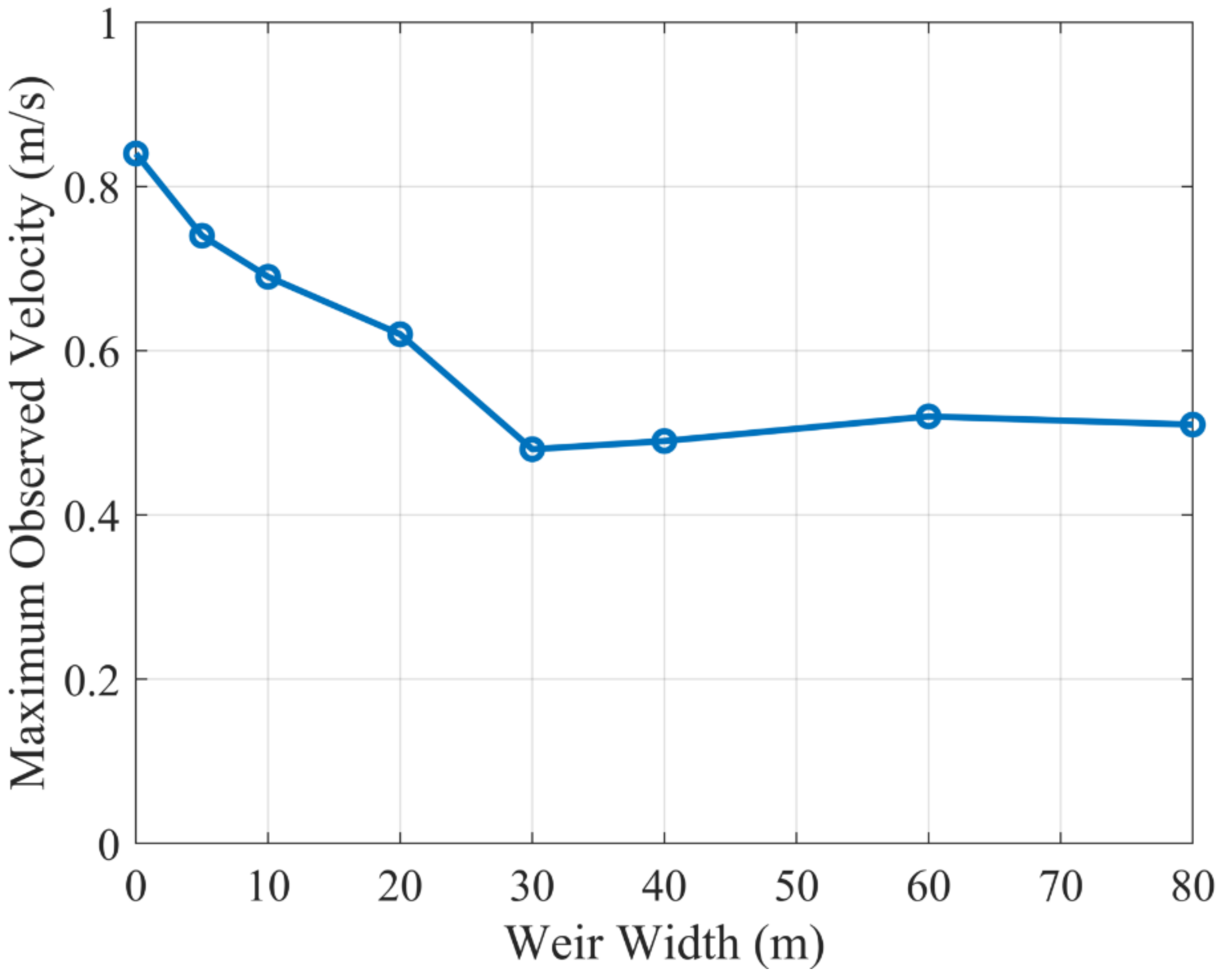

Figure 5.7 Relationship between weir length and maximum observed velocity at a position along the Reservation Terrace shoreline. Optimal reduction in currents are observed at a weir length of around 80 to 100 feet (25 to 30 meters).

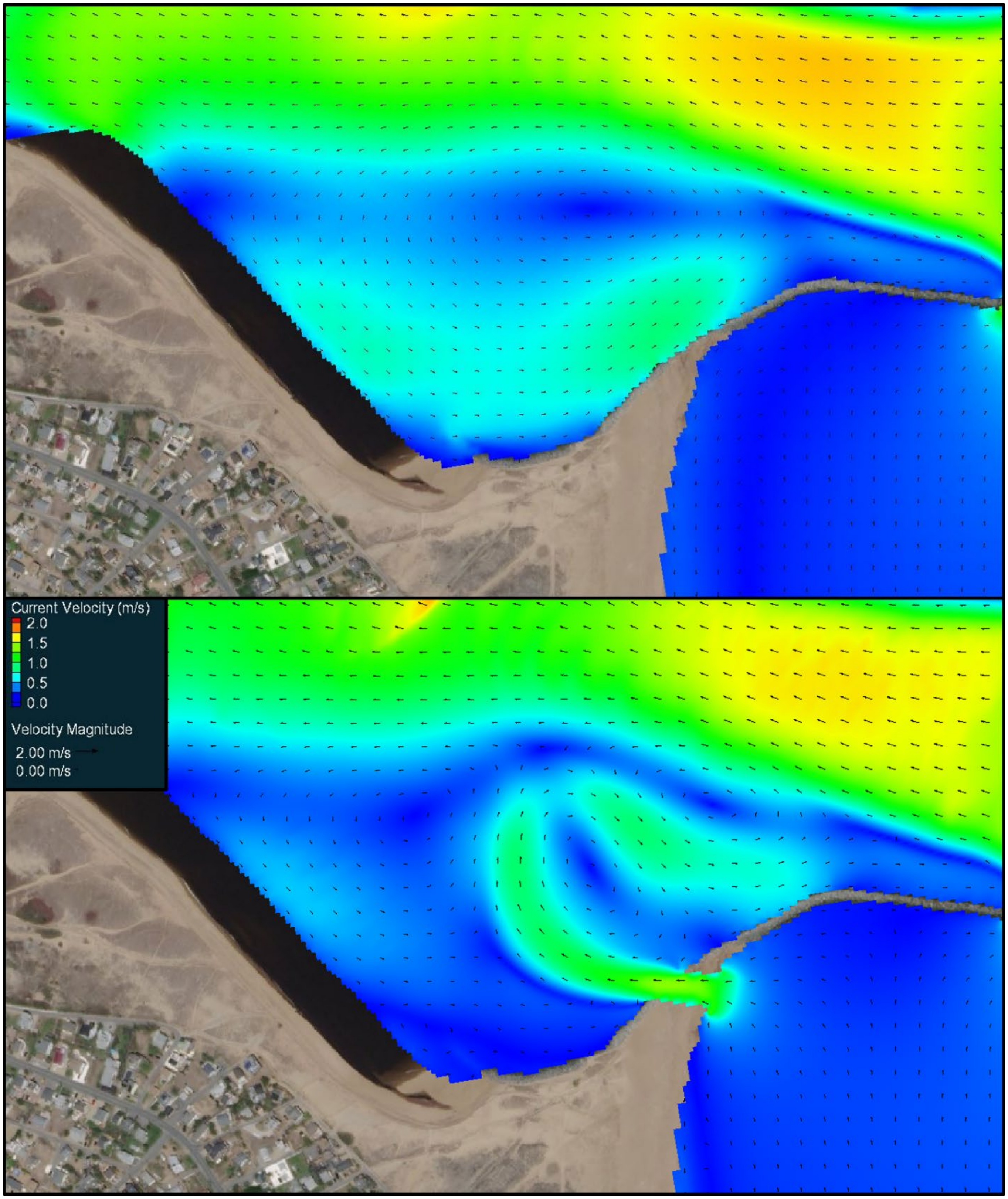

Figure 5.8 Current velocities at the same model time step for existing conditions (no sediment overtopping) (top) and 100 foot (30 meter) weir scenario (bottom). Velocities in the weir jetty scenario are reduced by half.

5.2.2 Offshore Breakwater

An alternative method to relocate the gyre away from the shoreline would be to extend the jetty back into the inlet. The extension would be divided into multiple breakwater sections that re-positions the current further into the inlet and reduce sediment suspension and transport away from the Reservation Terrace shoreline. Applied Coastal modeled the flow patterns around two breakwater sections of 10,000 square feet with a crest elevation of +7.5 feet NAVD as segmented extensions of the tail end of the south jetty (Scenario 1; Figure 5.9). A timber structure that runs from the center of the breakwater structure to the berm was included in the model. The connecting structure was included to prevent strong currents from developing in the gap between the structure and the shoreline. The timber section would also serve to reduce alongshore transport, and hold sediment in place along the Reservation Terrace shoreline.

Model results indicate successful relocation of the gyre further into the inlet with minimal (< 0.1 m/s) flow along the shoreline (Figure 5.10). This alternative would be expensive and difficult to permit, and further analysis into the downdrift effects would need to be measured. However, it would likely be the most effective measure to protect the Reservation Terrace shoreline against erosion.

A similar option of extending the jetty at its bend was considered (Scenario 2; Figure 5.9). Although Scenario 2 was successful in breaking up the gyre in model simulations, the positions of the jetty extensions would significantly increase the structure footprint relative to Scenario 1 due to the increased water depth at the Scenario 2 structure locations. Scenario 2 did not include a timber section connecting the breakwater structure to the shoreline as the distance between the Scenario 2 structure and the shoreline is three times that of Scenario 1. The timber structures also would not be effective at reducing currents in the alongshore direction due to the increased depths. Permitting of this option would be more difficult and therefore Scenario 2 was not pursued.

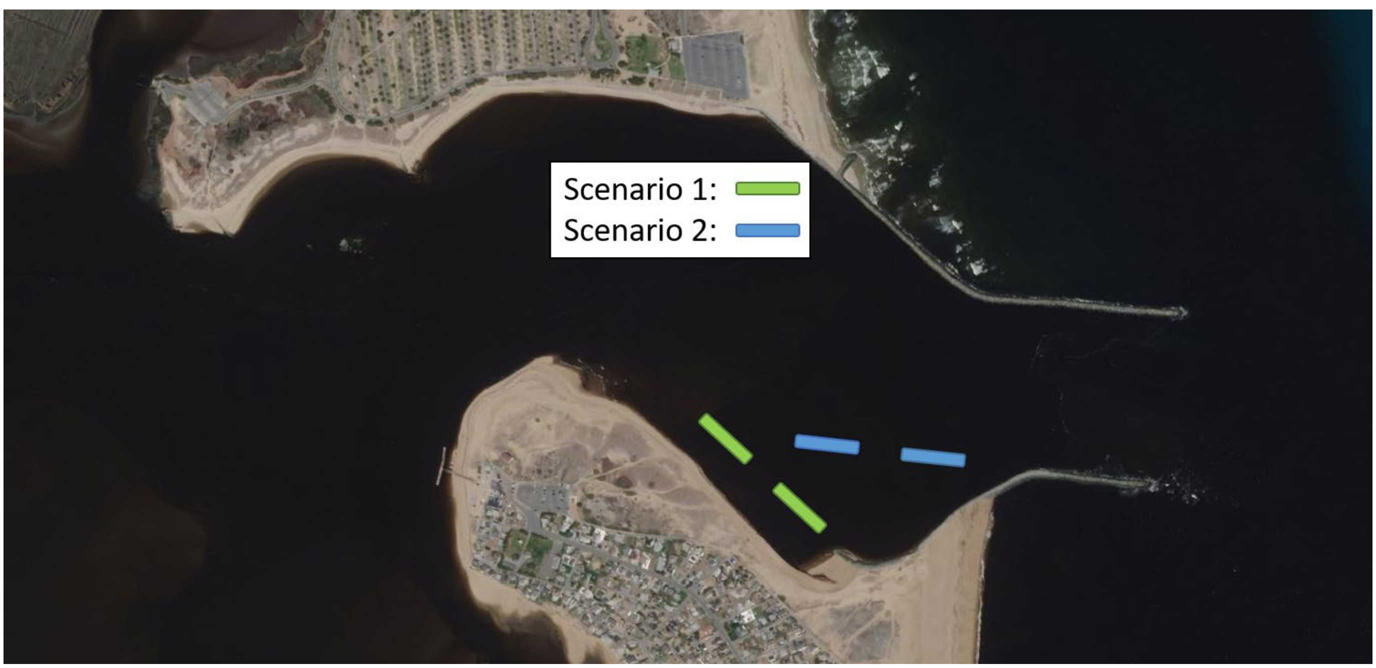

Figure 5.9 Scenarios considered for jetty additions inside the inlet. The first scenario (green) considered would extend the jetties from the end as a offshore breakwater sections. The second scenario (blue) would extend the jetty back from the dog-leg bend.

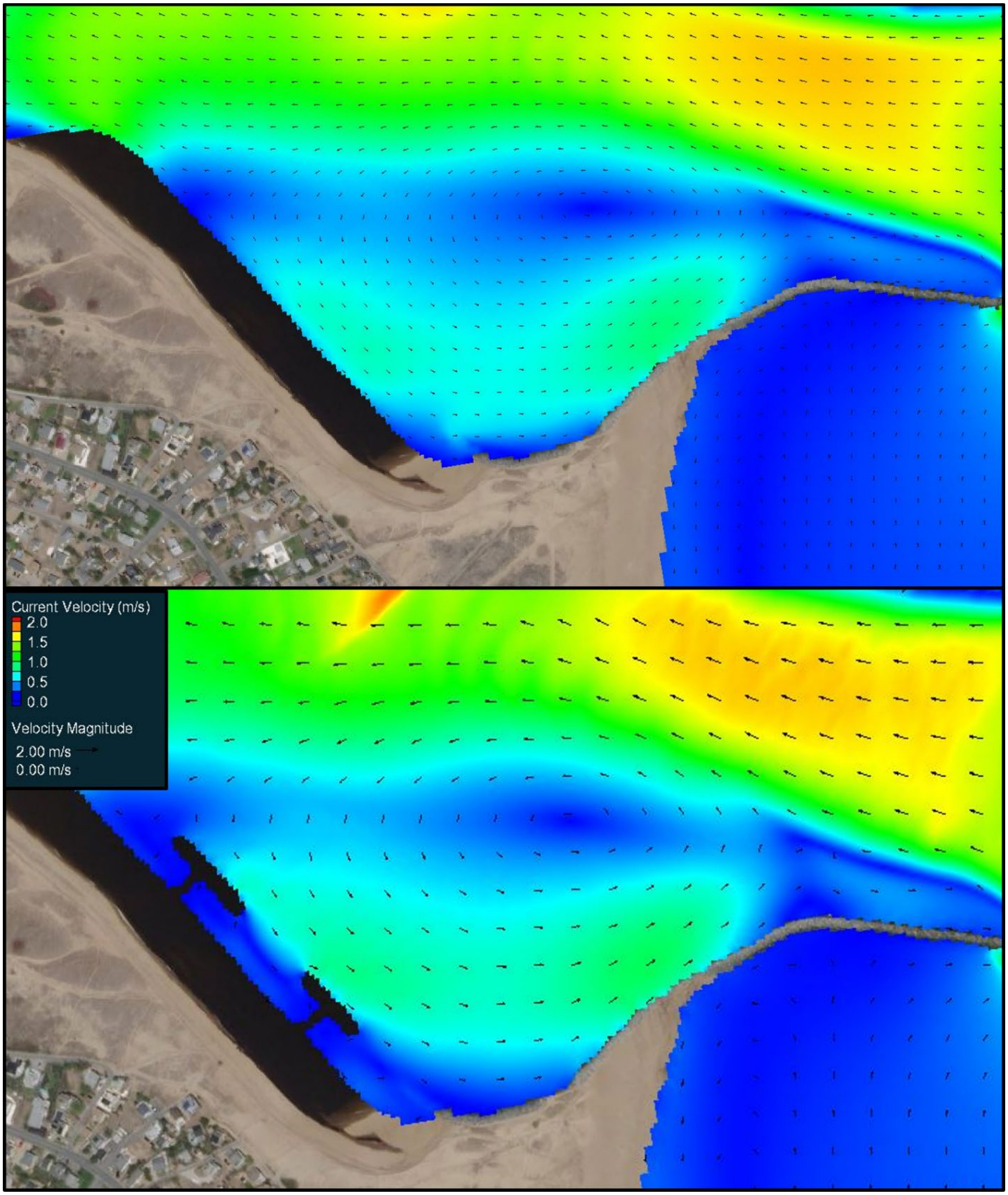

Figure 5.10 Model simulation results for Scenario 1 offshore breakwater addition to the south jetty. The structures effectively re-position the gyre further into the inlet.

5.2.3 Reservation Terrace Nourishment

A third alternative for Reservation Terrace would be provide a beach nourishment to extend the shoreline further into the inlet and provide a buffer for properties at the northern end of the island. The last time a significant nourishment was placed on the Reservation Terrace shoreline was in 1970 following the rapid erosion that succeeded USACE jetty rehabilitation. The nourishment was implemented as a sand dike to an elevation of 18.2 feet NAVD (Figure 5.11). Several smaller nourishments have been provided to the shoreline, but none have exceeded 1,000 cubic yards. A more substantial nourishment would be necessary to mitigate for the existing erosion rates that face the shoreline in recent years. The nourishment would create a 150-foot-wide berm at an elevation of 11 feet NAVD (one foot above the FEMA 100 year still water elevation). The estimated nourishment volume would be 48,100 and 69,000 cubic yards for a 650- and 950-foot nourishment respectively. It is anticipated that the existing bathymetry will also provide slight variability in the necessary volume to achieve the proposed nourishment geometry, as the estimates were based off 2015 LiDAR topography.

This alternative would only be a protection measure, and not a long-term solution to the erosion. However, it would provide the necessary protection, without the need to erect any permanent structures. A supplemental structure modification, as discussed in previous sections, would prolong the life of the nourishment and provide stability to the shoreline. Nourishment options and design should be considered following the reduction in currents and wave action along the Reservation Terrace shoreline. An alternative to providing nourishment material from external sources (e.g., offshore or upland), would be to transport material across the jetty in the form of bypassing or backpassing, depending on the condition of the jetty. In a sand tight scenario in which the jetty is not allowing sand to pass from the east facing beach to the Reservation Terrace shoreline, bypassing would transport sand south of the south jetty to the Reservation Terrace shoreline. A backpass would be utilized in an opposing situation in which the south jetty allows sand to flow freely, and would transport sand from Reservation Terrace to a location south of the south jetty. This plan is further outlined in section 6.1.

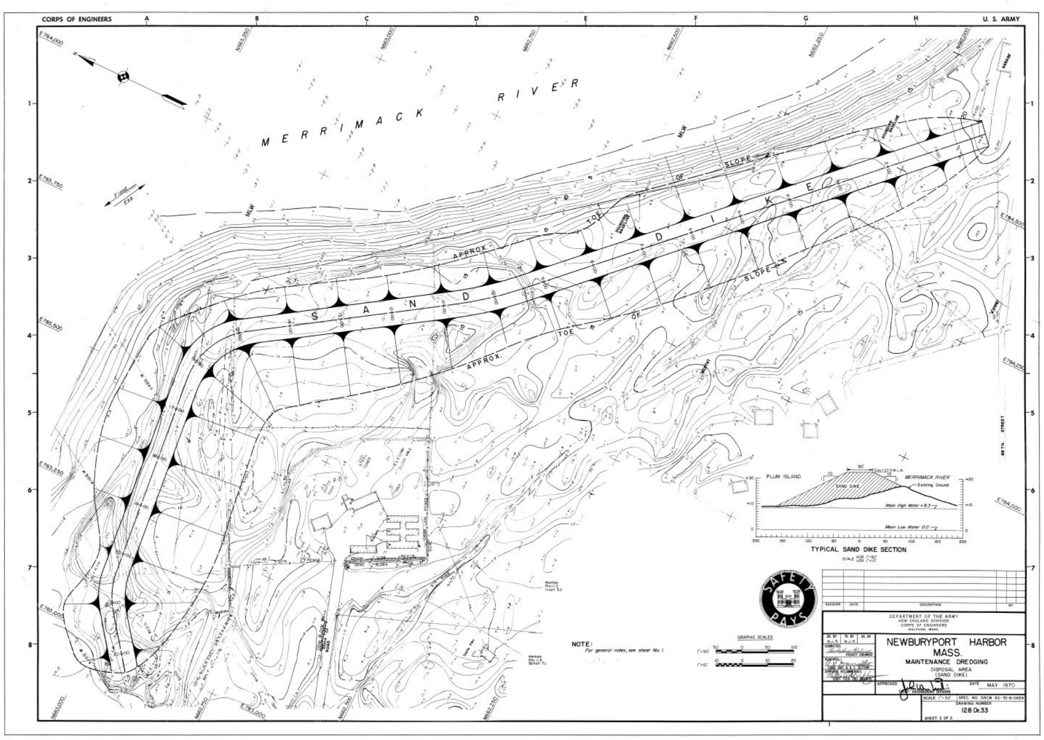

Figure 5.11 Final design plan for the 1970 Reservation Terrace sand dike. The berm elevation of the sand dike was 18.2 feet NAVD.

5.3 Plum Island

As mentioned in Section 1.1, the northern end of Plum Island has a net sediment transport pattern atypical to the north shore of Massachusetts. Northeast storms, while infrequent, dominate the sediment transport for most of the state, resulting in a net southerly directed transport for most of the region. However, the northern end of Plum Island exhibits a net northerly transport on the with transport rates between 50,000 and 100,000 cubic yards per year. In addition to the theories mentioned previously to explain the change in direction of transport and the erosion hotspot, Applied Coastal is considering currents generated behind the bar as an additional source of erosion, due to buildup of water during storm events and the associated release as the water level drops pulling sediment from the nearshore (Figure 5.12). Several alternatives were analyzed to determine a solution to the erosion hotspot located south of the center groin.

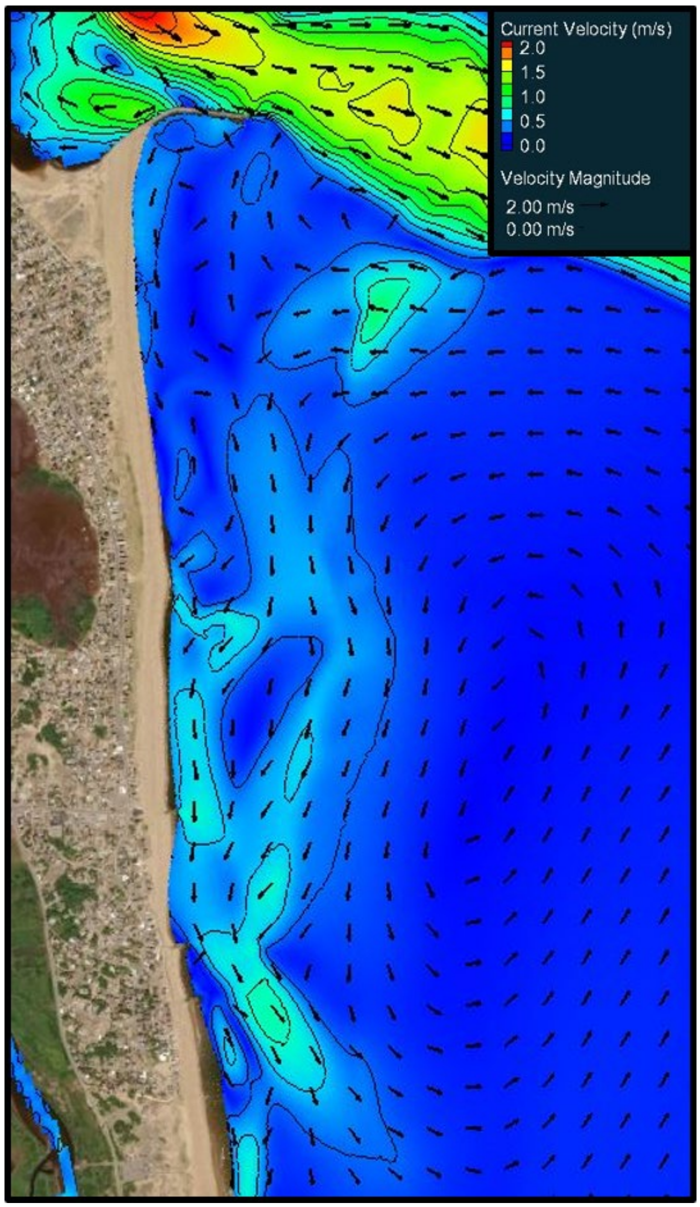

Figure 5.12 Example of the currents that are generated as water levels drop during a moderate storm event in which there is a buildup of water behind the ebb shoal.