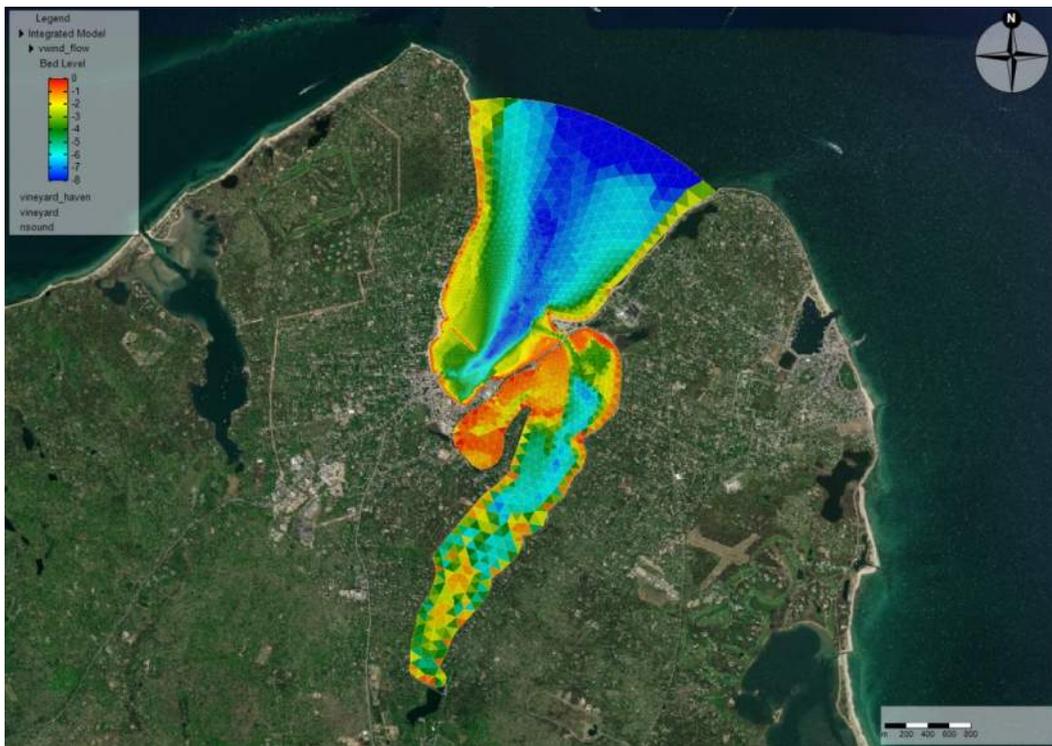


Attachment I

Sediment Transport Study

Sediment Transport Analysis for Packer & Tisbury Marine Terminal Facility, Vineyard Haven Harbor, MA



April, 2020

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1.0 INTRODUCTION

Analyses of tidal circulation and sediment transport were performed to quantify any potential alterations to the coastal environment associated with proposed improvements at the Packer & Tisbury Marine Terminal Facility (Packer Facility) in Vineyard Haven Harbor. Applied Coastal utilized a combined hydrodynamic, wave, and sediment transport model to serve as the basis for evaluating the influence of design alternatives on tidal circulation, accretion, and erosion in the project area. As described in the Environmental Notification Form (ENF) previously submitted, a series of structural improvements including a wave fence, bulkhead, and solid fill pier. As shown in Figure 1.1.

The modeling effort has been tailored to address known environmental regulatory concerns of the project. Specifically, the proposed modeling focuses on providing a detailed analysis of alterations to tidal circulation caused by the pile array and other structural elements (e.g. new bulkheads and wave fence), as well as changes to tidal and wave-induced sediment movement potentially caused by the proposed alterations. For this reason, it is important to developing appropriate modeling tools that can simulate the coastal processes of combined tidal currents and wave dynamics, as well as accurate incorporation of coastal engineering structures.

Understanding that the quality of numerical model results is directly dependent on appropriate data sources, the modeling effort also focused on developing the most appropriate data sources. Site-specific bathymetry and geotechnical data were available from Foth Infrastructure & Environment, LLC (Foth). Due to the need for larger-scale analysis of both regional tides, winds, and waves, additional data sources were derived from available U.S. Army Corps of Engineers (USACE), National Ocean and Atmospheric Administration (NOAA), and in-house Applied Coastal sources. These data sources provided the necessary information to parameterize the numerical modeling suite to ensure the results were scientifically-defensible.

Given the availability of data sets that can be utilized to parameterize the coastal processes modeling effort, the Delft3D modeling suite was selected to provide detailed circulation and sediment transport information, including simulation of ‘annualized’ morphologic change (accretion and erosion patterns). This model was originally developed at Delft University in the Netherlands, where the software is continuously improved and developed with advanced techniques as a result of the research work. For this project, the model was utilized in a two-dimensional, depth-averaged manner, as no vertical stratification exists in Vineyard Haven Harbor (Howes, *et al.*, 2010). For the modeling associated with alterations at the Packer Facility, the evaluation consists of the following main tasks:

- Evaluation of available data sets including the full range of input data requirements for the Delft3D modeling suite: bathymetry/topography, *in situ* sediment characteristics, regional tide, wind, and wave data, and the details of the proposed engineering modifications to the Packer Facility.
- Development of a two-dimensional (2D) depth-averaged hydrodynamic, wave, and sediment transport model for the existing conditions, based on state-of-the-art

Delft3D modeling suite, where the model was developed in a manner that allows prediction of annualized morphologic change.

- Modification of the existing conditions model to incorporate proposed engineering modifications to the Packer Facility and run simulations of these modified conditions to determine annualized morphologic change for each alternative. These model results were utilized to compare the various alternatives to the existing conditions.
- Modification of the existing conditions model to incorporate proposed engineering modifications to the Packer Facility and run simulations of these modified conditions to determine alterations to circulation patterns for each alternative. Specifically, dispersion of a conservative tracer within the area of proposed engineering modifications was simulated to illustrate changes in tidal circulation and mixing. These model results were utilized to compare the various alternatives to the existing conditions.

The numerical modeling tools developed have been used to quantify changes to circulation and/or sediment transport pathways resulting from proposed alterations to the Packer Facility. The results are critical for quantifying these potential changes to local coastal processes, which is a necessary component to support the environmental permitting process.

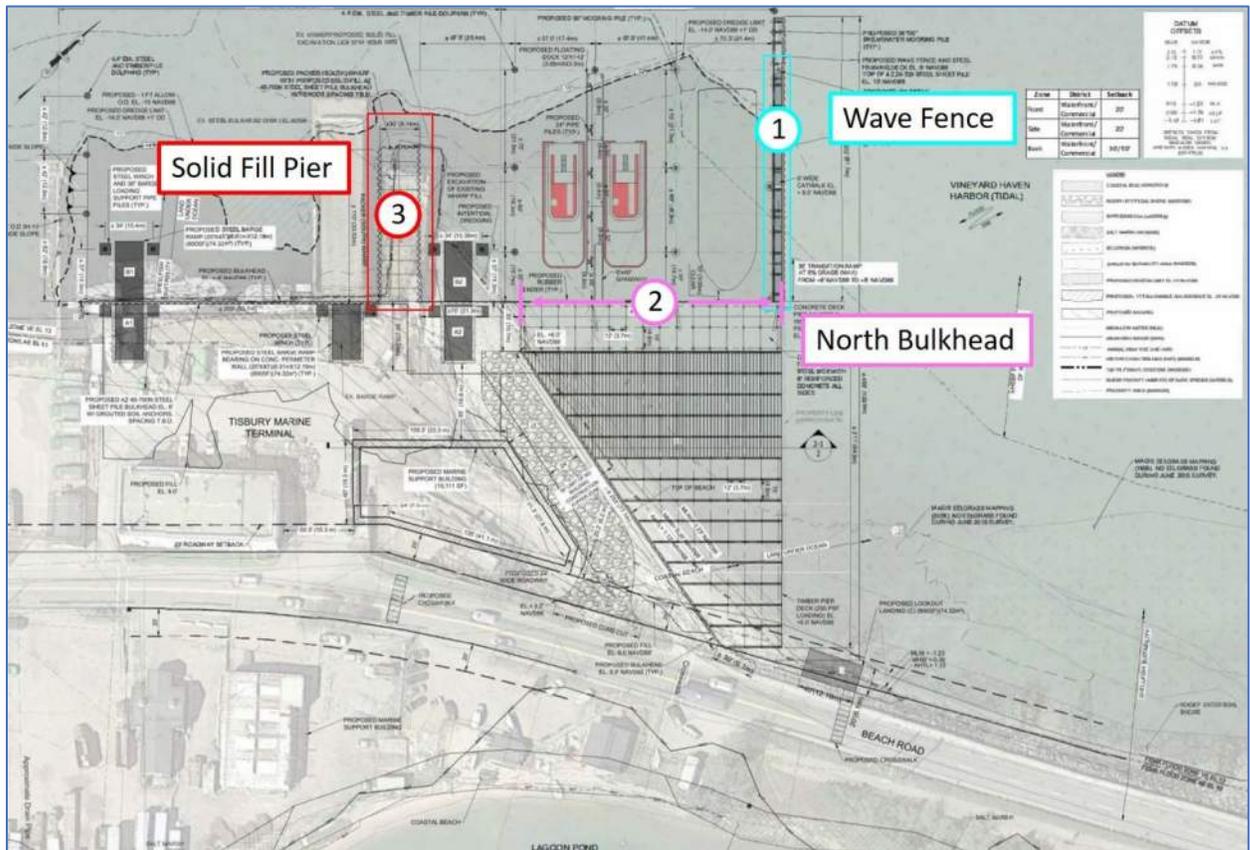


Figure 1.1 Initial plans for the Tisbury Marine Terminal facility provided by Foth.

1.1 Coastal Processes Model Development

For Vineyard Haven Harbor, sediment transport is primarily driven by wave action, as the tidal currents are relatively modest due to both the small tide range and the unconstrained nature of the harbor/estuary entrance. Due to the relatively minor freshwater inflow (Howes, *et al.*, 2010) and the generally shallow nature of the embayment area (i.e. waters less than 30 feet deep), the flow within Vineyard Haven Harbor is not vertically stratified. Therefore, use of a two-dimensional (2D) hydrodynamic model of the system provides appropriate detail of the littoral processes in the region of the Packer Facility, where engineering improvements are planned.

The Delft3D (Deltares) software was employed for this study. Delft3D is a state-of-the-art integrated modelling suite that can simulate both the two-dimensional and three-dimensional hydrodynamics of riverine and tidal systems, as well as morphology change, wave propagation, and water quality. Recent improvements to the models offered by Deltares include use of unstructured meshes (made up of combinations of triangular and quadrilateral elements). Due to the vertically well-mixed conditions associated with Vineyard Haven Harbor, as described above, a two-dimensional depth-averaged hydrodynamic analyses is most appropriate for assessing tidal flow within the system, with no loss in accuracy relative to the tidal circulation and sediment transport processes that are the focus of this assessment.

The development of the two-dimensional hydrodynamics, waves, and sediment transport model of Vineyard Harbor provided a tool to assess detailed coastal processes analyses of the existing conditions, including quantification of sediment migration in the vicinity of the Packer Facility. The model was developed utilizing wave conditions that can propagate into Vineyard Haven Harbor over a year-long period as the basis for annualized sediment transport conditions. Typical tidal conditions were superimposed onto the long-term wave record to ensure the minor influence of tidally-driven sediment transport also is considered as part of the modeling effort. Based on site observations, it appears that the tidal currents within the inner harbor region are relatively minor and that sediment transport processes of *in situ* material is dominated by waves, especially during easterly storm conditions.

1.2 Alternatives Evaluation

In addition to providing information regarding existing conditions, the modeling tools also provide the ability to evaluate various engineering alternatives. For this case, the model grid will be modified to include the bulkhead, solid fill pier, pile-supported platform, and wave fence depicted on the conceptual project plans provided by Foth. Specifically, the following alternatives were evaluated:

1. *Scenario 1*: Existing conditions
2. *Scenario 2*: Conceptual design as presented in the Proposed Site Plan dated November 26, 2019 that extends the existing Packer Facility basin to the east at a facility depth of -18.4 feet NAVD. This proposed design includes the following four (4) elements:

- a. Solid fill pier reconfigured with a slightly reduced footprint
 - b. New bulkhead extending ~190 feet to the east to from a contiguous shoreline with the existing Packer Marine Facility bulkhead. The elevation of this bulkhead is +6 feet NAVD.
 - c. Incorporation of a solid wave fence extending from the eastern end of the new bulkhead for a distance of ~212 feet to the north
 - d. Incorporation of a pile-supported platform south of the new bulkhead, where the 1-ft diameter wood piles are spaced on a 15-ft grid. The landward edge of the platform will consist of a wall feature that is landward of the active sediment transport zone; therefore, there is no interaction of this wall with wave action.
3. *Scenario 3*: Implementation of *Scenario 2*, with a revised bulkhead elevation that lowers the top height of the bulkhead to an elevation that is 2 feet above the existing grade to the adjacent area south of the bulkhead. This potentially could improve tidal circulation by allowing tidal flow to circulate over the structure.
 4. *Scenario 4*: Implementation of *Scenario 3*, with the wave fence designed in a manner that allows a gap along the seafloor to potentially improve tidal circulation. The bottom of the wave fence for this simulation was 3 feet above the seafloor, with a transition from -18.4 feet NAVD to the existing grade at a 1:3 (v:h) slope from the Wave Fence extending in an easterly direction.
 5. *Scenario 5*: Implementation of *Scenario 2*, with a revised bulkhead elevation that lowers the top height of the bulkhead to an elevation that -5 feet NAVD for a distance of 40 feet south of the wave fence end and -4 feet NAVD for another 40 feet south of that. This bulkhead lowering creates an 80 foot “window” opening of the bulkhead to allow flow to pass. This potentially could improve tidal circulation by allowing tidal flow to circulate over the structure, while still reducing infilling of the boat basin.

For each of the alternative configurations above, the modeling was conducted for the same input parameters utilized for the existing conditions run (i.e. simulation of annual transport rates and typical tidal circulation conditions). In this manner, the effect of the various proposed structure configurations could be compared in a quantitative manner to determine relative changes to sediment transport rates, erosion/accretion, and tidal circulation patterns.

1.3 Historical Facility Development

To better evaluate the sediment transport pathways in the vicinity of the Packer Facility, understanding development of the commercial port facility is useful for determining the influence of natural versus anthropogenic influences. Vineyard Haven Harbor historically established itself as a commercial harbor along the route between the major ports of New York City and Boston. Prior to the establishment of the Cape Cod Canal, ships traveling between the two major ports sailed through Vineyard and Nantucket Sounds, and around Cape Cod. Vineyard Haven Harbor served as a critical port of refuge, especially since the shoals, as well as the tidal currents and storm wave activity, made the Cape and Islands area frequently treacherous to navigate. For example, between 1865 and 1915, some two thousand ships were wrecked, and more than seven hundred lives

lost, between Gay Head at the gateway to Vineyard Sound and Provincetown on the tip of the Cape (Dunlop, 2010).

The naturally deep thoroughfare down the major length of the protected harbor (Figure 1.2), running northeast to southwest, provided the basis for a commercial port in Vineyard Haven Harbor, where in the late 1800s there were lofts to repair sails; a shipbuilding company to repair hulls and rigging; and a marine hospital (Dunlop, 2010). Vineyard Haven (called Holmes Hole until 1871) was home to the Holmes Hole Marine Railway starting in the 1840s that constructed numerous schooners along the barrier beach system separating the Harbor from Lagoon Pond (see 1890 chart in Figure 1.3). Subsequent to the marine railway development, thousands of feet of harbor shoreline were commandeered as a construction site for barges and high-speed rearmament/personnel boats for World War II.

The series of oblique views of the Harbor from 1856, 1880, and 1890 are shown in Figure 1.4, Figure 1.5, Figure 1.6, respectively. These sketches illustrate the gradual development of the port facilities along the low-lying section of the barrier beach system. As illustrated in Figure 1.4, it is clear that after construction of the pier facility that was developed to reach the naturally deep water in Vineyard Haven Harbor, access to the pier was along the inter-tidal beach and no formal road was developed along the barrier beach. Subsequent time periods indicate that the development of causeways across Lagoon Pond and roadway construction led to man-made stabilization of the barrier beach system. These long-term stabilization efforts prevented the low-lying barrier beach some naturally migrating landward as a result of gradual sea-level rise over the past 100+ years or to periodic storm events.

As depicted in Figures 1.2 and 1.7, very little bathymetric change occurred between the mid-1800s and 1948 along the area immediately north of the barrier beach system, as the shallow shoal system adjacent to the shoreline continued to persist. As shown in Figure 1.2, the initial pier construction from the barrier beach system was located at the closest proximity to the naturally deep Harbor channel. Overall, it appears that prior to WWII, the water depths adjacent to the barrier beach were generally natural. However, development in the WWII time-period formalized the vertical structures along the harbor-facing portion of the port that was part of the historical marine railway.

Figure 1.8 provides photographic evidence of the historical low-lying nature of the barrier beach in 1900 relative to the port facilities constructed along the barrier beach. In addition, Figure 1.7 visually indicates the narrow beach system to the east of the port area, where a series of groins were established to mitigate erosion. Over the subsequent decades, State Road and adjacent upland development has been stabilized to ensure access between Vineyard Haven and Eastville. These stabilization efforts have worked to prevent landward migration of the barrier beach system; however, they also have prevented natural erosion processes from occurring. Therefore, the sediment supply along the beach system has decreased over time and sediment availability along the shoreline between the Packer Facility and the State Road bridge to Eastville is minimal.



Figure 1.2 Section of the 1847 U.S. Coast Survey navigation chart for Holmes Hole (Vineyard Haven Harbor) illustrating the natural deep channel along the length of the Harbor, as well as early pier development in the vicinity of the Packer Facility.

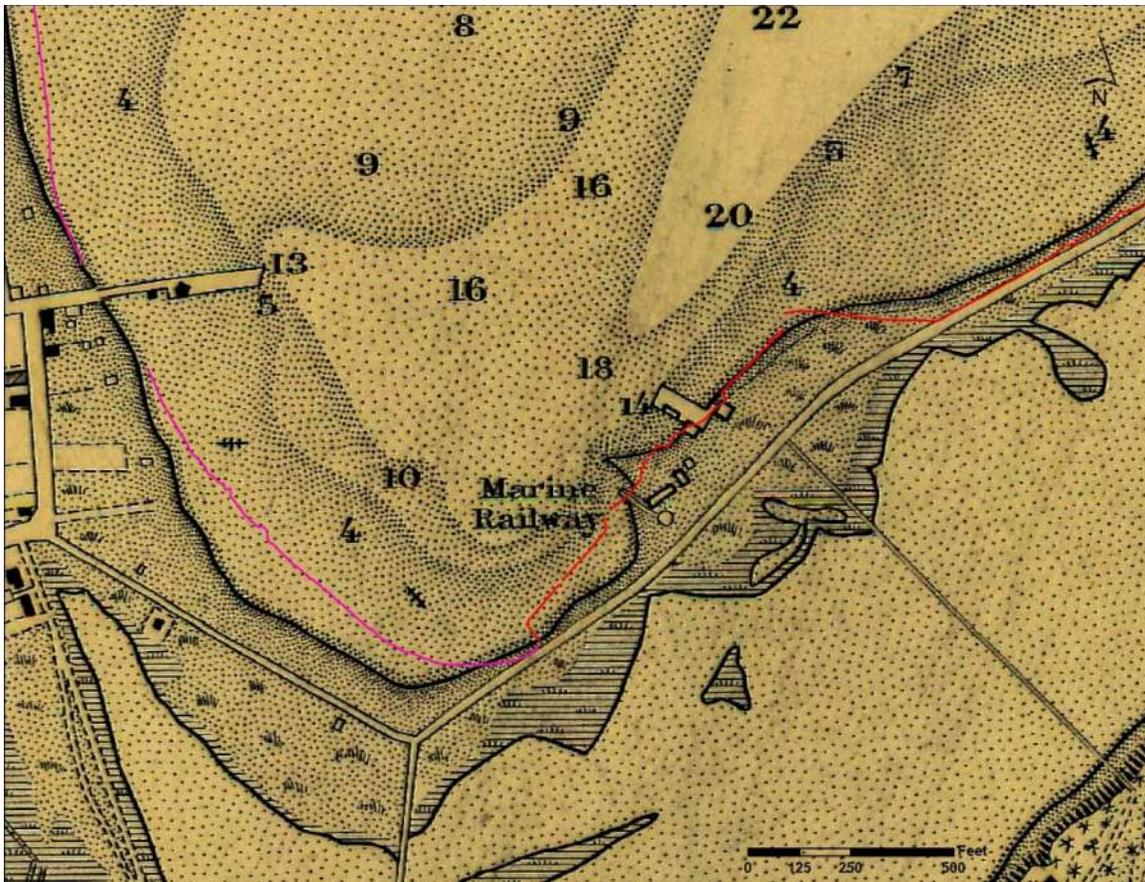


Figure 1.3 Section of the 1890 U.S. Coast Survey navigation chart for Vineyard Haven Harbor illustrating the marine railway development in the vicinity of the Packer Facility, as well as creation of a roadway along the length of the barrier beach system and causeways across the marsh system.



Figure 1.4 Artist's rendering of Holmes Hole (Vineyard Haven Harbor) in 1856, where the pier facility along the barrier beach separating the harbor from Lagoon Pond is depicted in the foreground. Note, no roadway existed along the barrier beach at this time and goods were transferred to town along the sandy beach.

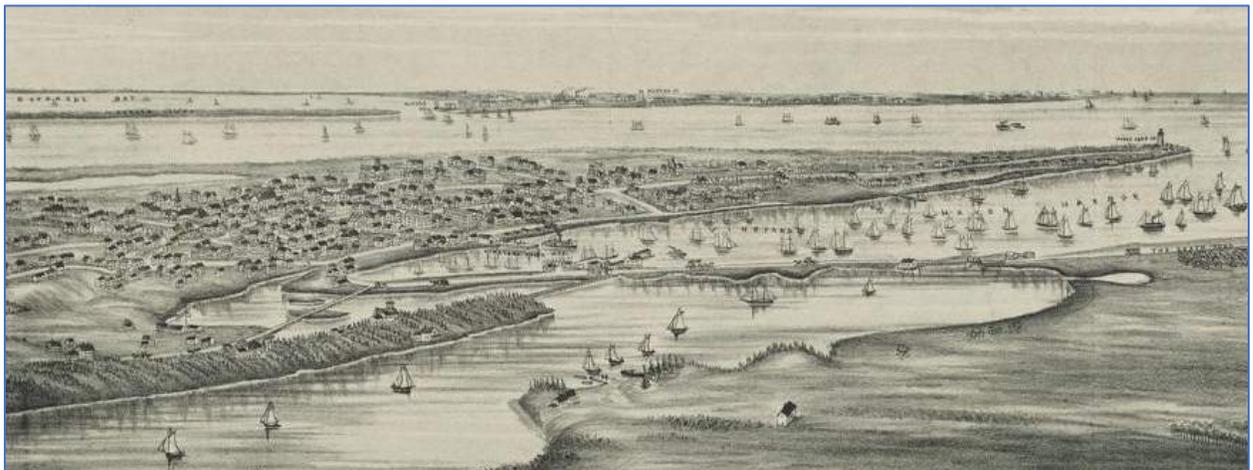


Figure 1.5 Birdseye view of Vineyard Haven Harbor in 1880 (from the south) illustrating the causeways constructed from the mainland to the barrier beach, as well as the roadway along the beach.



Figure 1.6 Birdseye view of Vineyard Haven Harbor in 1890 (from the west) illustrating the causeways constructed from the mainland to the barrier beach, as well as the roadway and pier facility along the beach.



Figure 1.7 Post-World War II aerial photograph from 1948 illustrating the shipbuilding facilities along the barrier beach system, the series of groins to the east of the present-day Packer Facility, and the breakwater protecting the main harbor basin. Based on the photograph, it appears that the water immediately adjacent to the upland had not been dredged.



Figure 1.8 Photograph from 1900 showing the Steamship Pier in the foreground. The low-lying barrier beach system and pier facility along the barrier beach can be seen in the upper part of the photograph.

2.0 COASTAL PROCESSES ANALYSIS

The Delft3D (Deltares) software was employed for this hydrodynamic and sediment transport evaluation in the vicinity of the Packer Facility. As indicated previously, the vertically well-mixed conditions associated with Vineyard Haven Harbor indicated that a two-dimensional depth-averaged hydrodynamic analyses was most appropriate for assessing tidal flow within the system, with no loss in accuracy relative to the tidal circulation and sediment transport processes that are the focus of this assessment.

To develop appropriate conditions at the project site, a series of larger scale tidal and wave generation models were needed to adequately parameterize the near-field Vineyard Haven Harbor conditions. In this case, the overall tidal conditions were developed from an U.S. Army Corps of Engineers ADCIRC model of the North Atlantic Ocean. In addition, local wave conditions are driven from both wind energy in Vineyard/Nantucket Sounds, and to a lesser extent wave energy from the open Atlantic Ocean. The Delft3D model was utilized to generate wave conditions at the entrance to Vineyard Haven Harbor. In this manner a finer-scale grid of Vineyard Haven Harbor could be separated out from the larger-scale grid to allow more reasonable computation time for the series of structural alternatives evaluated. A more detailed description of the modeling approach is provided in the following sections.

As a general approach to the sediment transport evaluation, typical tidal conditions were superimposed onto the long-term wave record to ensure the minor influence of tidally-driven sediment transport also is considered as part of the modeling effort. Based on site observations, it appears that the tidal currents within the inner harbor region are relatively minor and that sediment transport processes of *in situ* sandy material is dominated by waves, especially during easterly storm conditions. To provide a conservative analysis, it was assumed that water circulation dynamics in the harbor were driven strictly by tidal currents and wave mixing was negligible.

2.1 Grid Development

2.1.1 Nantucket/Vineyard Sounds Flow Grid and Bathymetry

The flexible mesh flow grid consists of primarily triangular elements that encompass all of Vineyard and Nantucket Sounds (Figure 2.1). There were 33,938 edge elements and 11,488 nodes used in the flow grid. Each node represents a position where hydrodynamic and sediment transport calculations were made. Topography/bathymetry data collected from a variety of sources that will be discussed in Section 2.2 were then interpolated to the mesh grid cell centers (Figure 2.2). A closeup of the flow grid (Figure 2.3) and bathymetry (Figure 2.4) are included for the project area. This region contains a higher density of elements and nodes to improve accuracy of numerical computations.



Figure 2.1 The flexible mesh flow grid, which consists of 33,938 edge elements and 11,488 nodes.

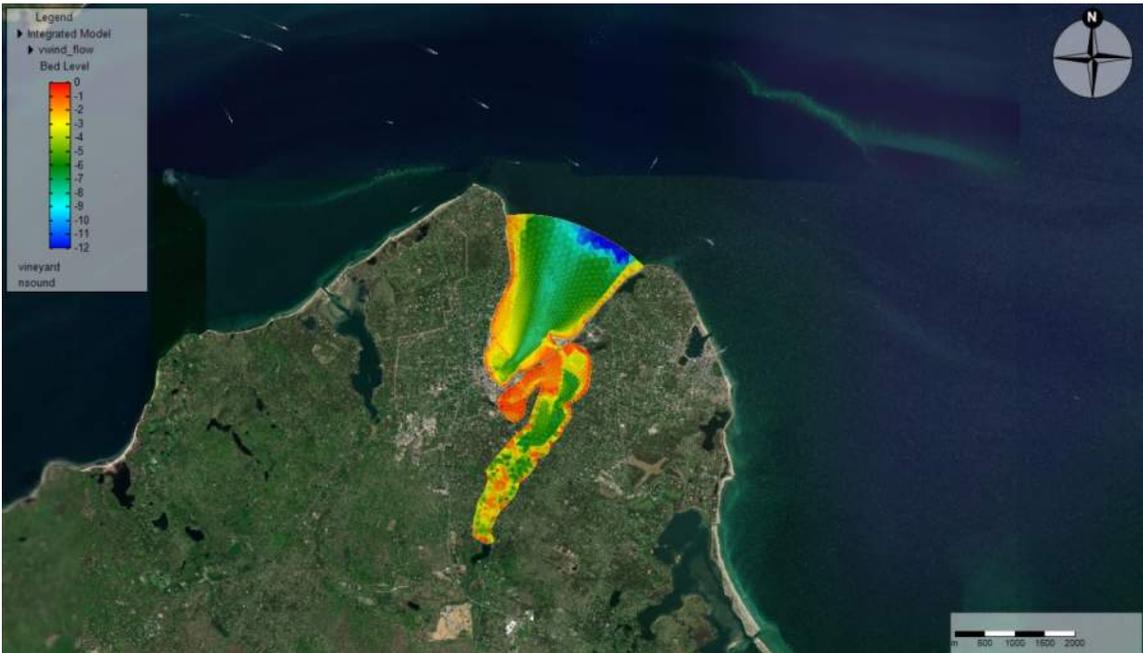


Figure 2.2 Snapshot of the interpolated flow grid bathymetry and topography. Warmer colors (orange and red) represent higher elevations and cooler colors (blue and green) represent lower elevations relative to the NAVD88 datum.

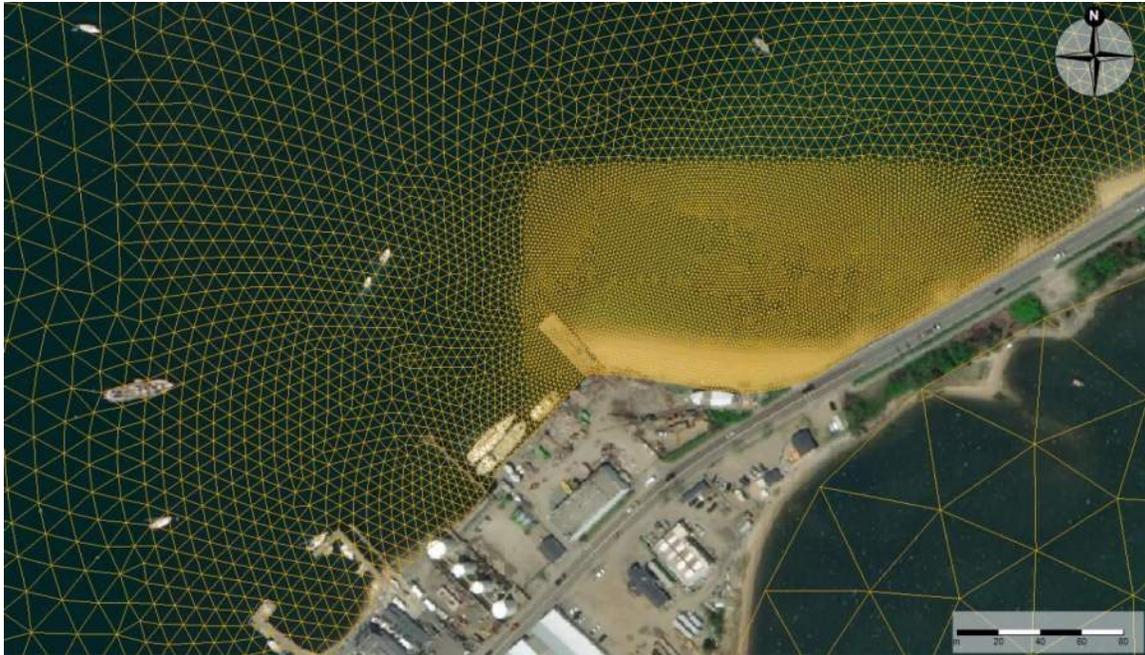


Figure 2.3 Closeup image of the flow grid flexible mesh. The project area is represented by additional mesh elements and nodes to increase the resolution and accuracy of the model results.

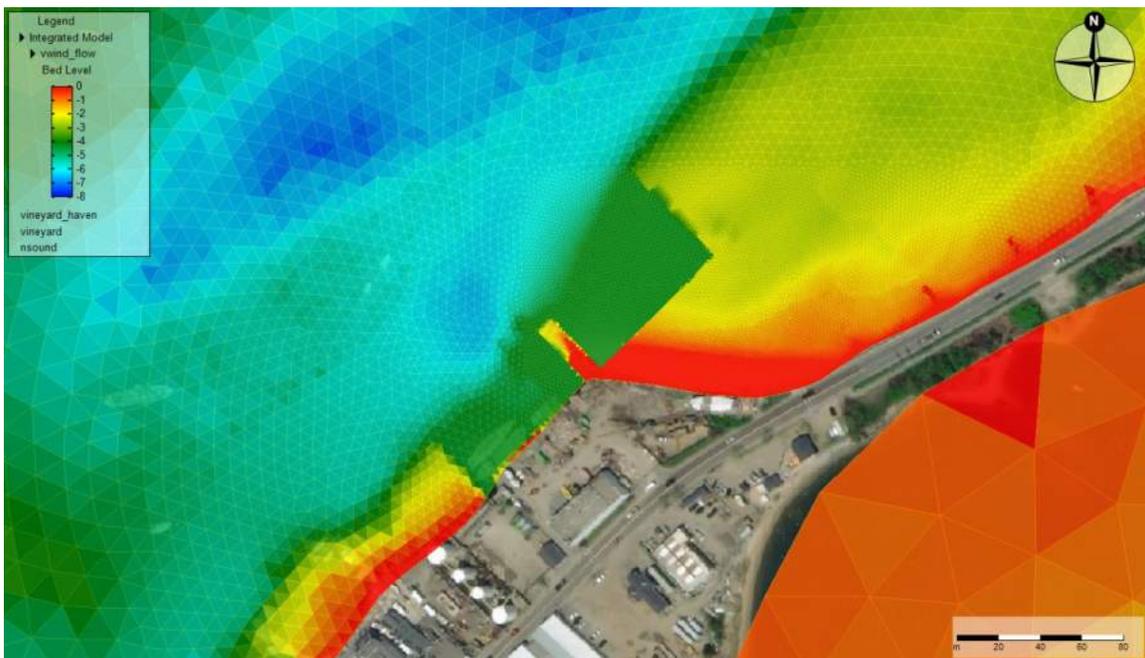


Figure 2.4 Closeup of the project area interpolated flow grid bathymetry. Warmer colors represent higher elevations and cooler colors represent lower elevations relative to the NAVD88 datum. The approximate proposed dredged outline at a depth of 18.4 feet NAVD88 (4.267 m) is shaded in green.

2.1.2 Wave Grid and Bathymetry

The structured wave grid consists of rectangular cells, and encapsulates all of Nantucket Sound from the end of the Elizabeth Islands to the west and Monomoy and Nantucket to the east (Figure 2.5). There were 107,500 wave grid cells included in the coarse wave grid. Each node represents a position where wave transformation calculations were made. This coarse wave grid was run with purely wind generated waves. See Section 2.5 for further discussion. These wave results were then output at the edge of a finer resolved wave grid at the edge of Vineyard Haven Harbor (Figure 2.6). The same topography/bathymetry data set that was used for the flow grid was used to interpolate to both the coarse (Figure 2.7) and Vineyard Haven Harbor fine (Figure 2.8) wave grids . A closeup of the Vineyard Haven Harbor wave grid is included in Figure 2.9. This region contains a higher density of elements and nodes to improve accuracy of numerical computations in the project area.

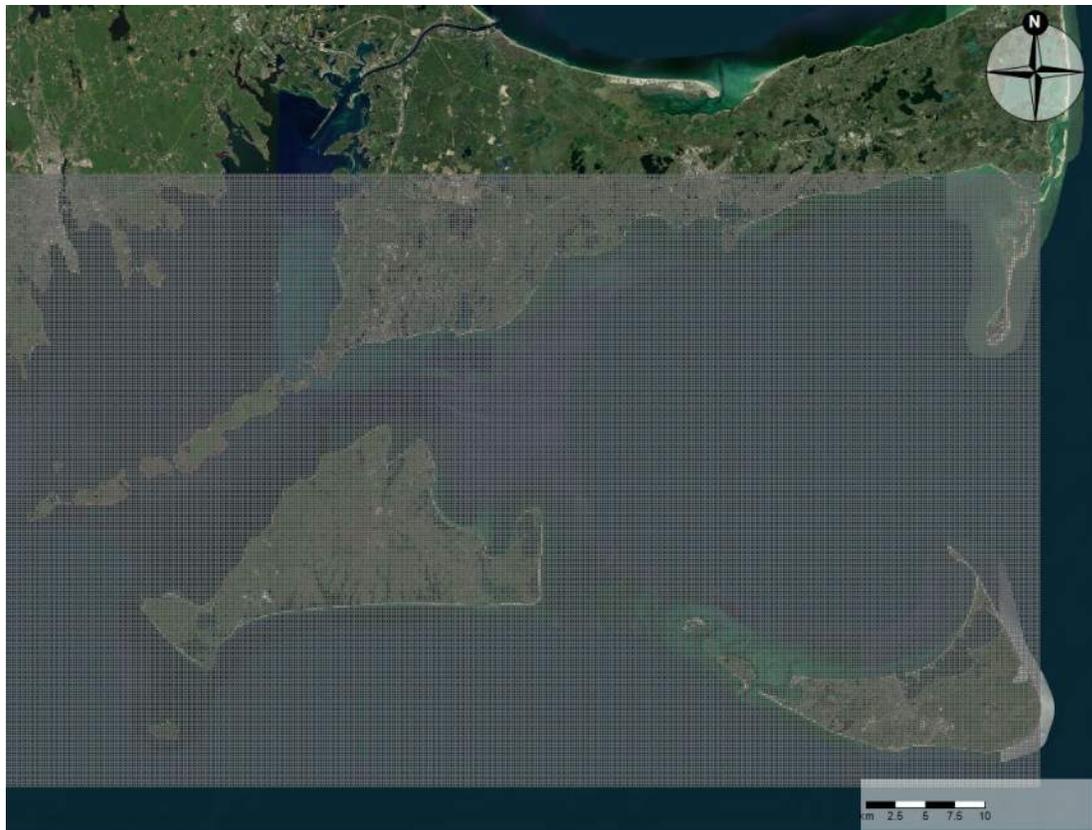


Figure 2.5 Nantucket Sound coarse wave grid.



Figure 2.6 Fine resolution wave grid for Vineyard Haven Harbor.

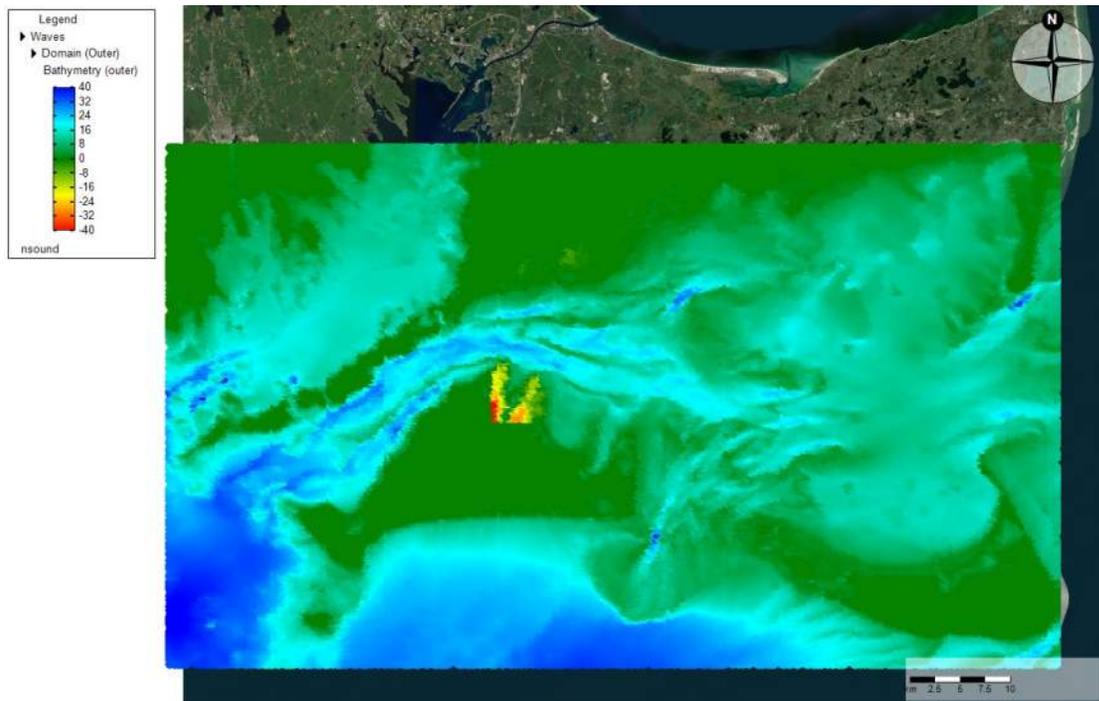


Figure 2.7 Snapshot of the interpolated point data for the coarse wave grid.

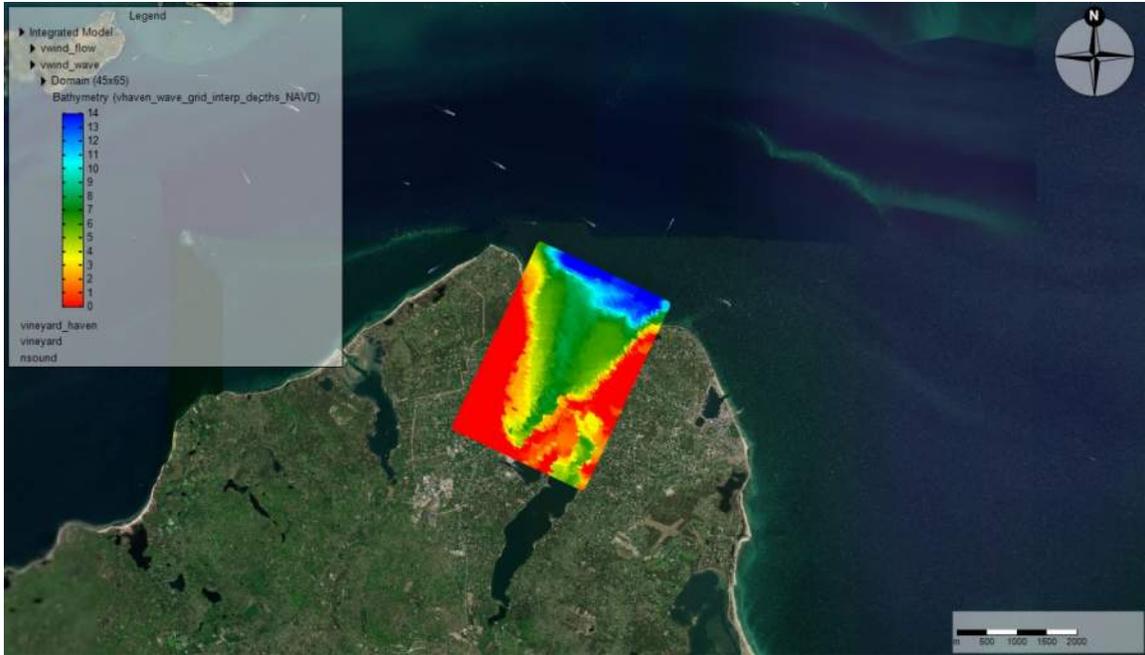


Figure 2.8 Snapshot of the interpolated point data for the fine wave grid. The Flow grid mesh is included for reference.

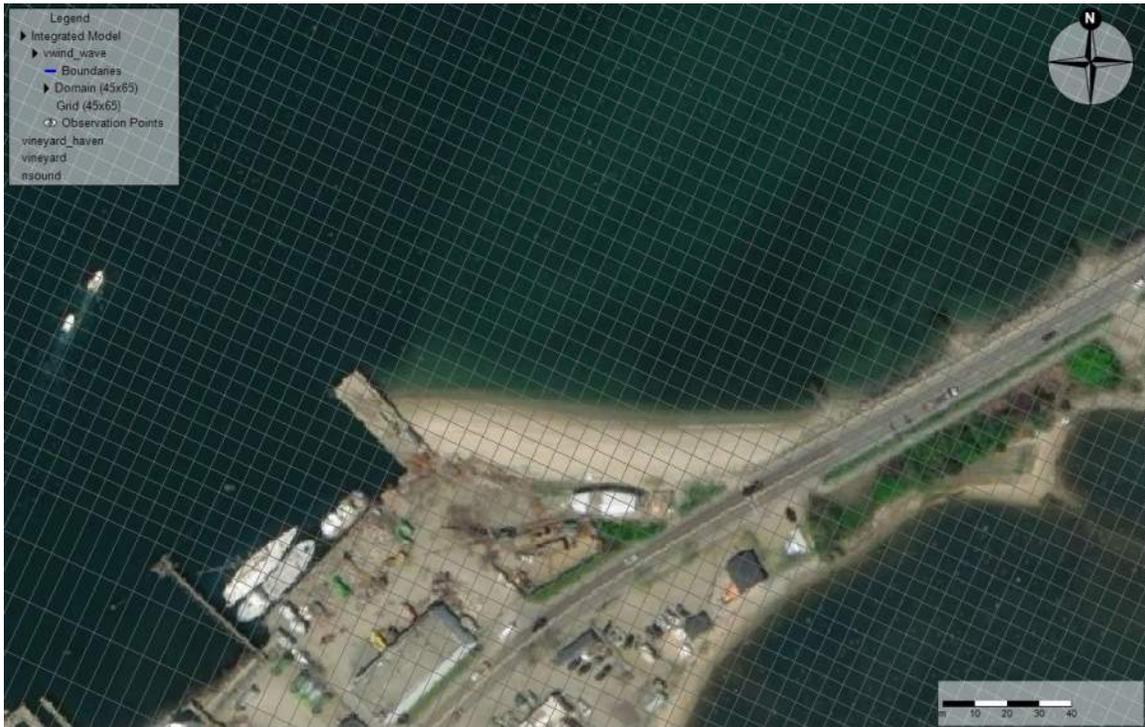


Figure 2.9 Closeup of the fine wave grid at the Tisbury Marine Terminal project area.

2.2 Survey Data

Light Detection and Ranging (LiDAR) survey data was used in addition to fathometer data to provide an accurate depiction of the bathymetry and topography in the project area. LiDAR data provided three-dimensional surfaces of topographic, as well as limited nearshore bathymetric, information that could be evaluated within appropriate mapping software. These LiDAR data were collected by the United States Army Corp of Engineers (USACE) as a post Superstorm Sandy 2013- 2014 dataset. Also included in the dataset were high resolution bathymetry data collected using a fathometer by Foth in the project area vicinity. The high-resolution bathymetry data within Vineyard Haven Harbor are included as a contour plot in Figure 2.10.



Figure 2.10 Bathymetry contours in Vineyard Haven Harbor.

2.3 *In-Situ* Sediment Characteristics

Sediment sampling data were utilized to evaluate the sediment composition in the project area. Specifically, the evaluation focused on determining representative sediment characteristics that would be input into the sediment transport model. Surface grain size data were available from a series of six (6) cores taken in the vicinity of the Packer Facility in 2019 (see Figure 2.11). Surficial subsamples of these cores were evaluated to determine the natural sediment composition adjacent to the Packer Facility. Based on a review of the grain size information from these cores, the majority of surficial samples contained significant anthropogenic material (e.g. a poorly sorted mixture of material ranging from gravel to silt), and were not representative of natural sediment characteristics

observed along the barrier beach system or in the nearshore area. The surficial sample taken from B-11, located the furthest offshore, had a composition most representative of natural sediment along the southern Vineyard Haven Harbor shoreline, where the median grain size (d_{50}) was determined to be 0.4 mm, as illustrated in Figure 2.12. Individual sample grain size distributions for each of the core samples evaluated are provided in Appendix A. The median grain size (d_{50}) of 0.4 millimeters (mm) was used for the characteristic grain size for the Delft3D model.

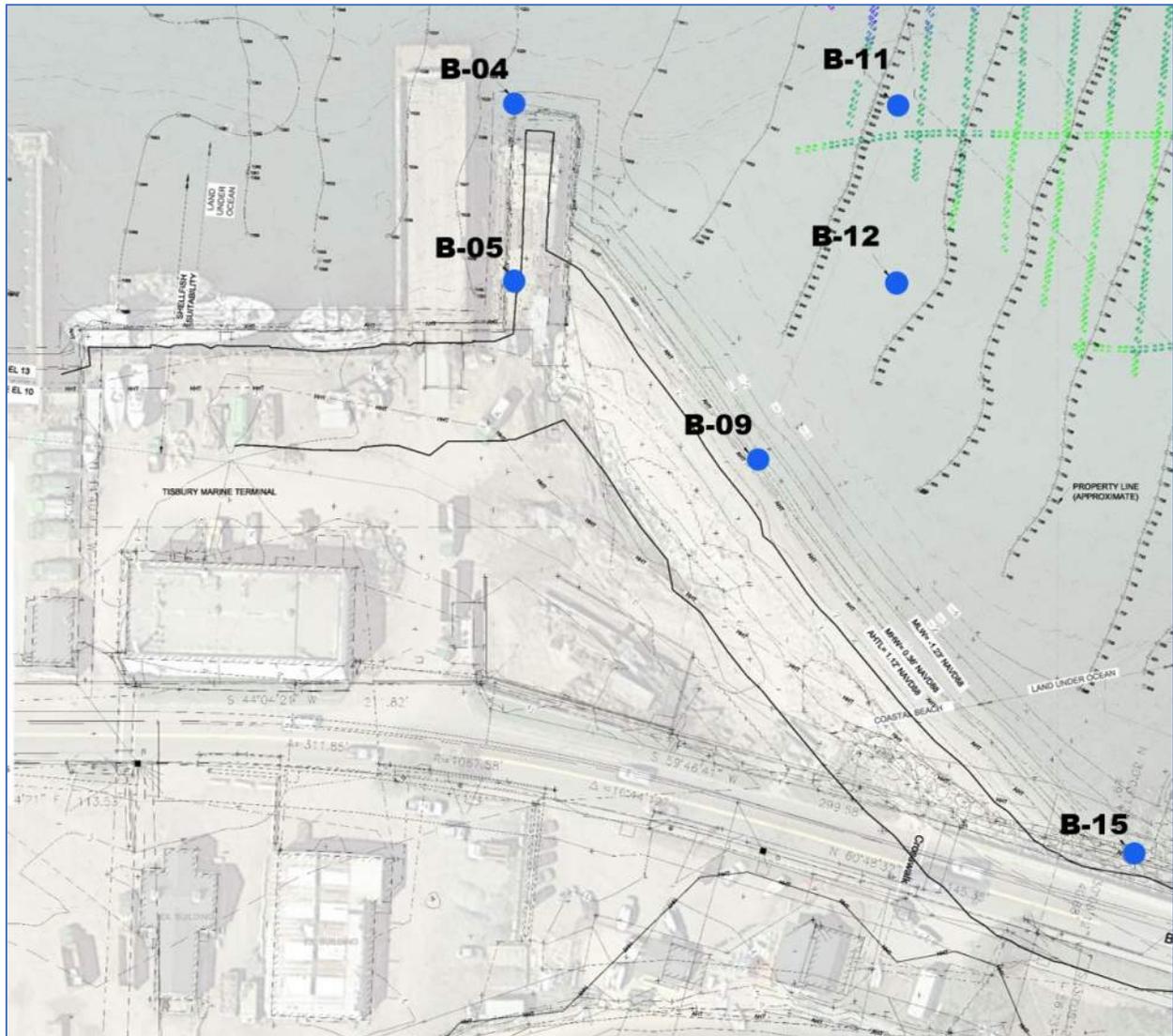


Figure 2.11 Sediment core locations for the surficial subsamples used in the sediment evaluation.

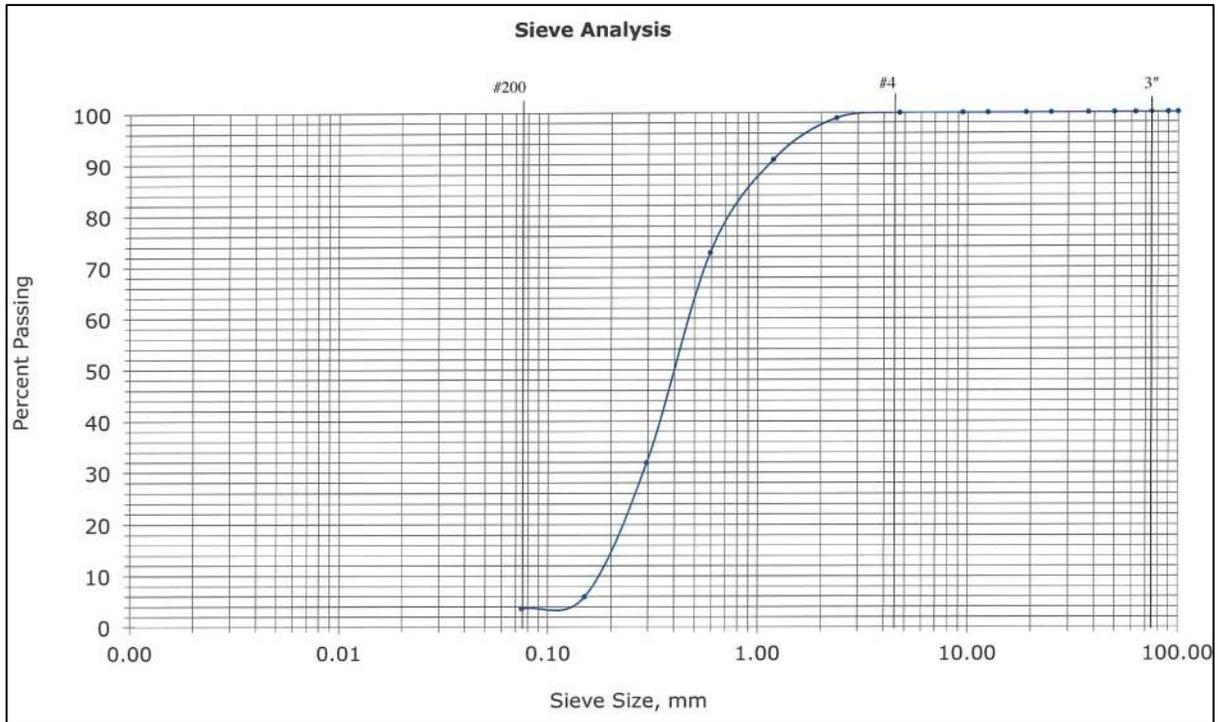


Figure 2.12 Sieve analysis results from the surficial subsample analyzed at location B-11.

2.4 Hydrodynamic Data

The goal of the hydrodynamic modeling was to accurately depict flow patterns, tidal water elevations, and current velocities within the project area. These computed model flows are used to evaluate tidal circulation characteristics to compare the impact of various structural alternatives. In addition, the current patterns also influence sediment transport in the presence of waves. To accurately parameterize the flow model, appropriate tidal forcing conditions are required, specifically time varying water elevations and flow across the open boundaries of the model. These boundary conditions were derived from the large-scale ADCIRC model (see below); however, it was important to ground-truth the ADCIRC model output with real measurements of tidal elevations local to the project.

Since the model was focused on astronomical tides only, the time period of the model simulation was not critical, rather a time period when synoptic tidal measurements were available in the proximity of Vineyard Haven was more important. The time period between May 6th, 2004 and June 15th, 2004 was selected, as Applied Coastal has tide data within Vineyard Haven Harbor and Lagoon Pond over this period from another unrelated study (see Howes, *et al*, 2010). A timeseries of water surface elevations was output at an observation point within the project area following completion of the flow simulation. The results plotted in Figure 2.13 with a mean range of 1.66 feet are consistent with observed tides in Vineyard Haven Harbor.

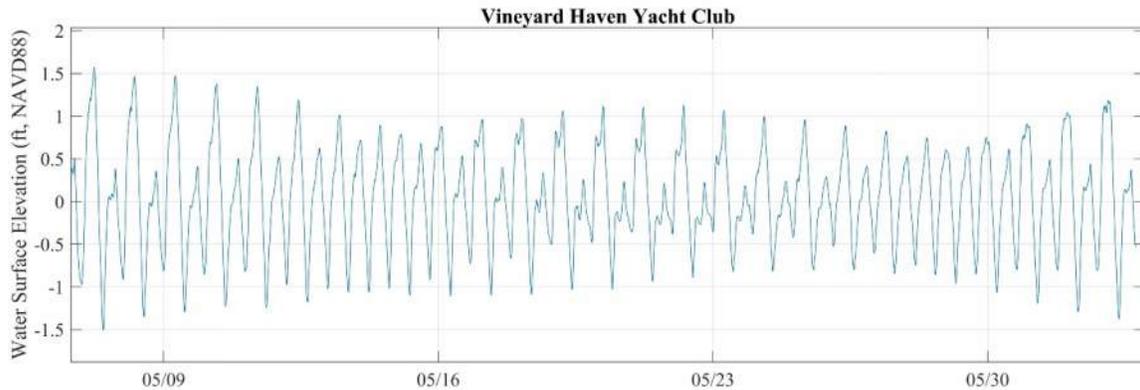


Figure 2.13 Simulated tide for the Vineyard Haven Harbor using Delft3D.

2.4.1 ADCIRC Hydrodynamic Model

Tidal boundary conditions used to drive the hydrodynamics of the D-Flow model were derived using the synoptic-scale (domains that span 1,000 km and more) circulation model ADCIRC. The region modeled with ADCIRC is centered on the Gulf of Maine and Nantucket Sound (Figure 2.14), and reaches northeast to the entrance of the Gulf of St. Laurence and southwest to include Chesapeake Bay. The southeast limit of the grid extends about 1,000 km southeast of Nantucket, well beyond the continental slope and into the abyssal plain of the North Atlantic Ocean.

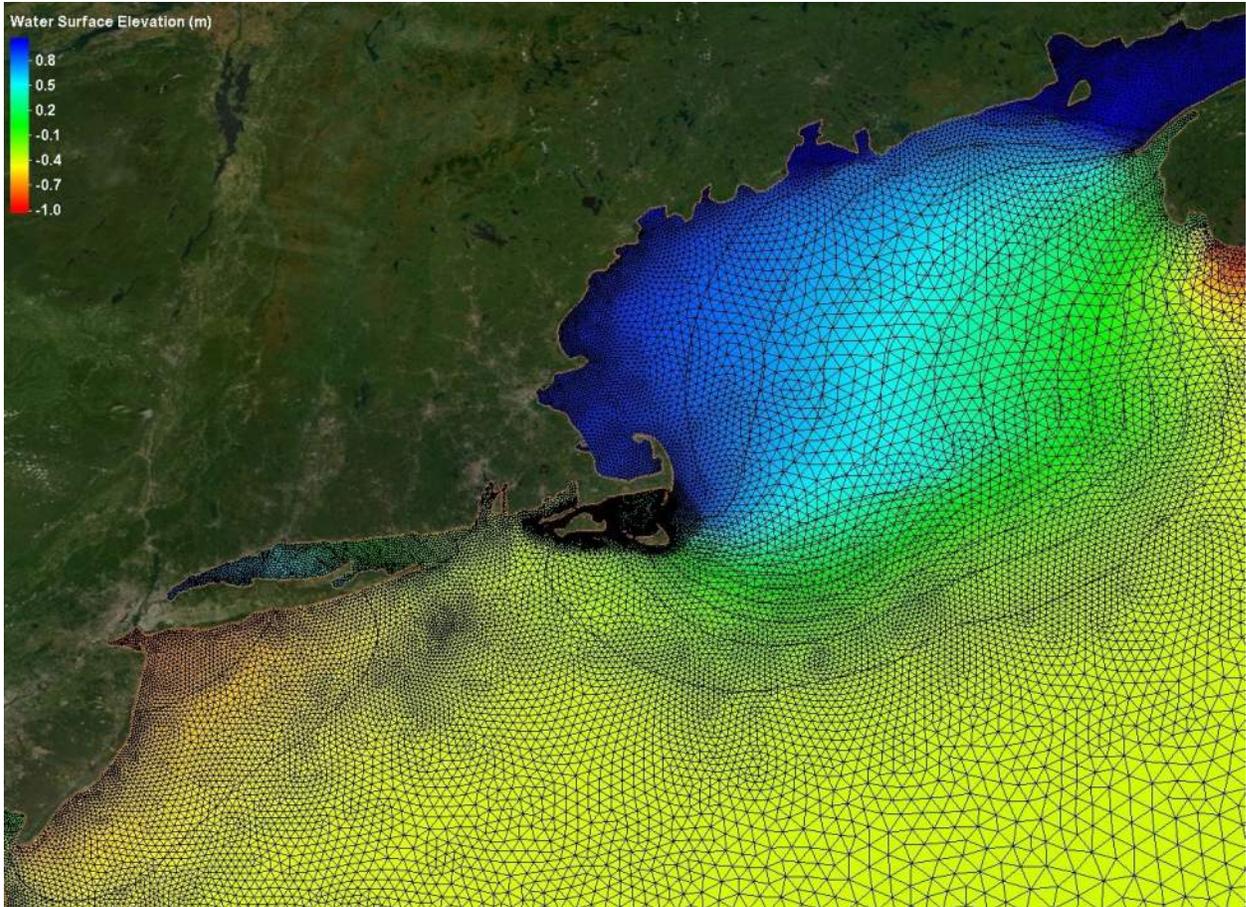


Figure 2.14 Detail of ADCIRC model grid used to generate tidal boundary conditions used to drive circulation in the D-Flow coastal processes model. This plot shows the triangular mesh of the model grid, color contours of water surface elevation (relative to MSL), and elevation isolines from a single model time step.

ADCIRC includes the application of tidal potential forcing of the entire modeled domain, a capability that is necessary to simulate tides in the region offshore of Nantucket and Vineyard Sounds. The interaction of macro-range tides (generally greater than 4 meters) generated in the Gulf of Maine and micro-scale tides (less than 2 meters) that occur in the New York Bight causes complex tidal circulation patterns in the areas offshore of Nantucket Sound, which leads to large variations in mean tide range in the Cape and Islands region (from greater than 3 feet at Cuttyhunk to nearly zero in the nearshore region southeast of Nantucket). The complex phasing of the tides in the region also causes swift tidal currents between Vineyard and Nantucket Sounds, in the vicinity of East Chop and West Chop. Tidal currents are of sufficient magnitude to regularly cause the formation of standing waves within the open water areas of Nantucket and Vineyard Sounds (e.g. the shoal system at Middle Ground).

For this application, tidal potential forcing was applied using seven tidal forcing constituents to generate tides within the model domain. These seven constituents represent different phenomena, such as the gravitational pull of the sun and moon, and

the orbit of the earth around the barycenter (center of mass) of the Earth-Moon system. The interaction of these constituents leads to the observed ocean tide. The modeled constituents include the K1 principal solar diurnal, M2 principal lunar semi-diurnal, which account for the majority of the observed tide in study area.

In addition to the tidal forcing applied to the interior of the model domain, a spatial-temporal varying water level boundary condition was applied to the full length of the offshore boundary of the ADCIRC grid. A time series of the astronomical tide was computed for each grid node along the open boundary of the model grid, using a global database of open ocean tidal constituent amplitude and phase. The boundary condition is specific to the modeled time period, starting on May 5, 2004 at 0000 hours.

Tide elevation time series were output from the ADCIRC run at a point corresponding to the flow boundary of the D-Flow model of Vineyard Haven Harbor. A comparison of constituent amplitudes calculated for the tide record measured in May/June of 2004 and those calculated for the time series of water level output from the ADCIRC model at the Vineyard Haven Harbor entrance (for the same time May/June 2004 time period) is presented in Table 2.1. The sum of errors between measured and modeled constituent amplitudes is less than 0.1 feet, which is comparable to the accuracy of the tide gauges used for this deployment, and represents an error of 5% compared to the maximum tide range. This demonstrates the ADCIRC model can be reliably used computed tides in the complex flow region of Nantucket Sound, and is therefore an appropriate tool for developing the long-term tidal boundary condition used for the D-Flow model of Vineyard Haven Harbor.

Table 2.1 Tidal constituents for Vineyard Haven Harbor, from the time period from May 14 through June 15, 2004, calculated for the measured record, and for ADCIRC model output at the Vineyard Haven Harbor entrance.							
	Amplitude (feet)						
	M2	M4	M6	S2	N2	K1	O1
Period (hours)	12.42	6.21	4.14	12.00	12.66	23.93	25.82
Measured tide	0.65	0.16	0.06	0.06	0.24	0.34	0.28
ADCIRC tide	0.63	0.18	0.10	0.06	0.18	0.27	0.28

2.5 Wave Modeling

The Delft 3D coastal processes model was run with wind and wave input derived from the Nantucket Sound NDBC buoy (44020), which has a record that spans the time period between 2009 and present (Jan 2020). A single year was selected from this record that would represent the most active wind-wave conditions for the fetches that approach the entrance to Vineyard Haven Harbor. In this manner, the annual sediment transport rates calculated represent a conservative or high value relative to typical annual sediment transport. In this case, 2018 was determined to be the most active year based on the measured wind speeds at the buoy for compass sectors with the longest fetches to the

entrance to Vineyard Haven Harbor (from north to east). Conditions during 2018 had the greatest cumulative wind-wave generating capacity for the selected sectors, and thus would have the greatest potential for wave-driven sediment transport at the study site. A plot of total relative wind-wave energy plotted for each year and compass sectors is presented in Figure 2.15. In this plot, the wind-wave energy for the year 2018 is plotted using the thick dot-dash line.

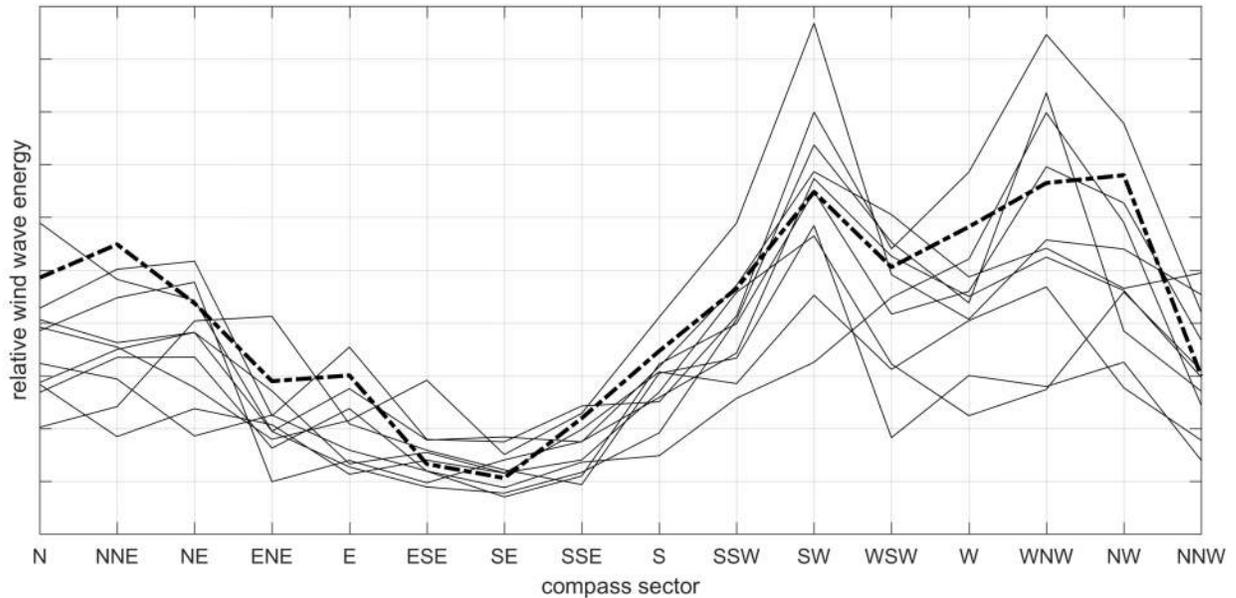


Figure 2.15 Plot of relative potential wind-wave energy (estimate as wind speed to the 1.5 power) by compass sector (x-axis) and year (plotted lines) from 2009 through 2019. The thick dot-dash line is the distribution for 2018, which is the most energetic for wave fetches from north to east, and has the greatest potential for sediment mobility at the Vineyard Have Harbor study site.

Winds from the buoy record for 2018 were applied to a regional scale D-Wave grid encompassing Nantucket Sound and the portion of Vineyard Sound with a line-of-sight fetch to Vineyard Haven Harbor. A time series of hourly wave conditions (Figure 2.16) was output from the Nantucket Sound grid at a location that corresponds to the northern open boundary of the fine wave grid used to propagate waves into Vineyard Haven Harbor.

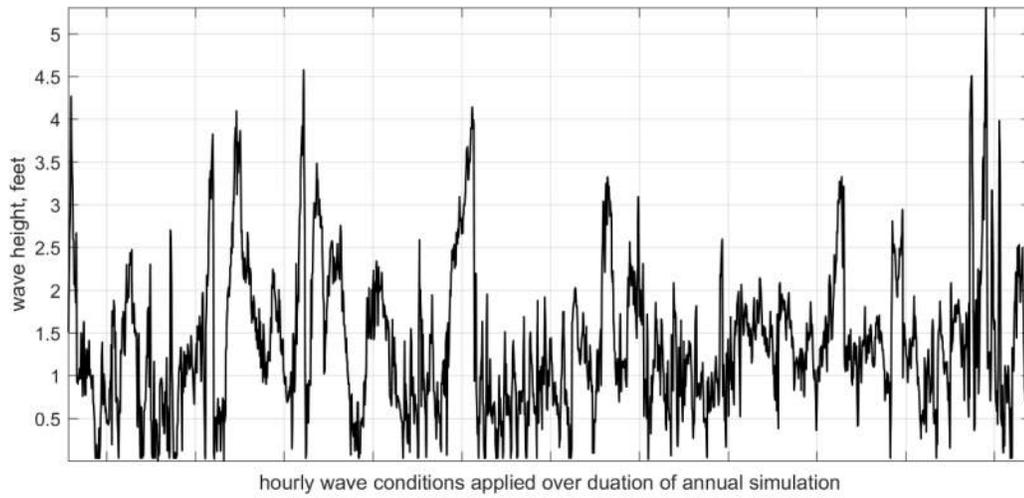


Figure 2.16 Time series of wave height applied to the Vineyard Haven Harbor wave grid open boundary.

3.0 ALTERNATIVES ANALYSIS RESULTS

Applied Coastal used analysis of existing conditions, historical records, and modeling to assess several structural alternatives for the Tisbury Marine Terminal. The Delft3D modeling approach detailed above, which incorporated different tools to assess forces along the project area, provides the most defensible approach for developing design. Schematically, the considered structural alternatives include a wave fence, bulkhead, and adjustments to the solid fill pier, which are generally shown in Figure 1.1. Each alternative was developed based on consultation with Foth. These alternatives were then evaluated under a detailed coastal processes analysis to determine appropriate design dimensions. The modeled dimensions are outlined in Table 3.1.

Table 3.1 Matrix of each structure alternative and the associated characteristics.

Scenario	Pile Platform	Wave Fence (Transmission)	Bulkhead (Elevation)	Solid Fill Pier
1. Existing Conditions	NA	NA	NA	Existing
2. Full Structures	Yes	0.0	6.0 Feet NAVD88	New Footprint
3. Bulkhead Elevation Reduction	Yes	0.0	2 Feet Above Seabed	New Footprint
4. Wave Fence Above Seabed	Yes	0.43	2 Feet Above Seabed	New Footprint
5. Bulkhead Window	Yes	0.0	-5 Feet and -4 Feet NAVD Across 80 Foot Window	New Footprint

3.1 Scenario 1: Existing Conditions

Prior to modeling the scenario alternatives, an existing conditions scenario was modeled to provide a baseline for comparison of potential impacts associated with each alternative design. The existing conditions used the combined topography/bathymetry dataset without the proposed dredging footprint. The only structure that was included in the project area was the existing solid fill pier; simulated as a “dry area” in the Delft3D model. The location of the structure relative to the project area and interpolated bathymetry for this scenario is included in

Figure 3.1.

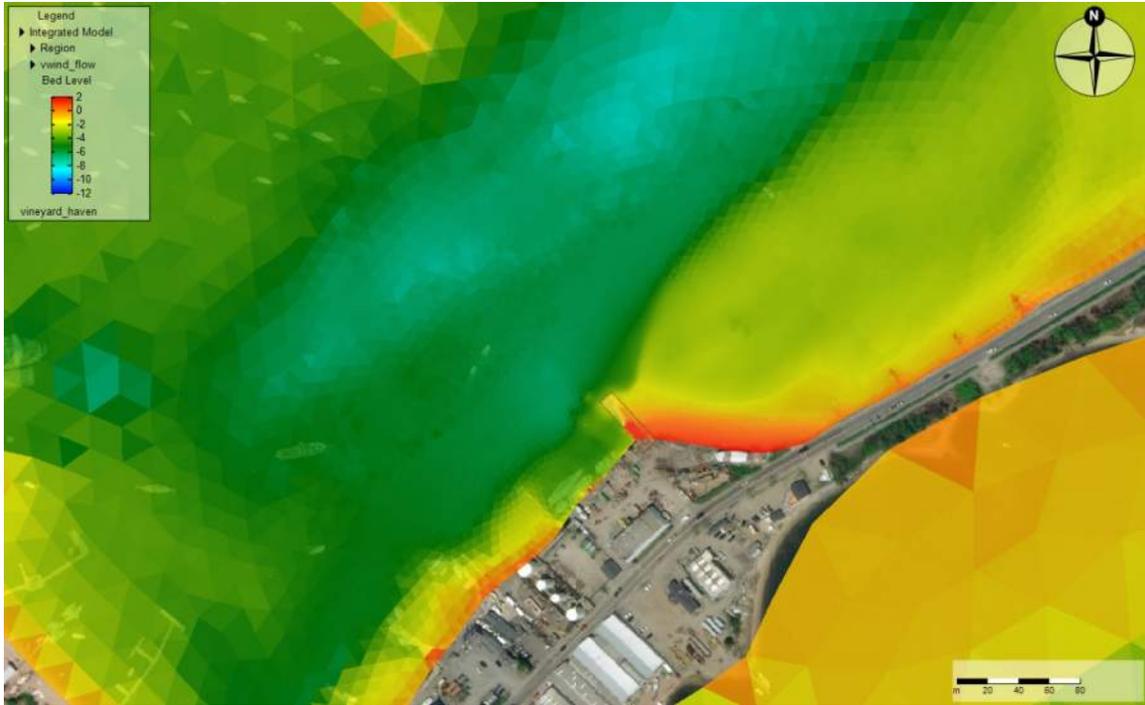


Figure 3.1 Interpolated bathymetry of the existing conditions scenario. The only structure included within the project area was the solid fill pier, included as a “dry area.”

3.1.1 Circulation

Overall, tidal circulation (i.e. the strength of tidal currents) in the project area is relatively weak, due to the microtidal conditions within the harbor system. Maximum modeled currents under existing conditions in the vicinity of the project area are around 8 cm/s or 0.25 ft/s. These existing currents are relatively small, and do not provide the necessary magnitude to mobilize *in situ* sediments. Therefore, any changes to flow within the project area associated with engineering improvements would not alter tidally-induced sediment transport patterns within the system. To illustrate the influence of tidal currents, plots of maximum flooding and ebbing tides within the project region were developed, where the timing of these conditions is provided on the tide plot shown in Figure 3.2. Maximum incoming or ‘flood’ tide conditions are shown in Figure 3.3 and maximum outgoing or ‘ebb’ tide conditions are shown in Figure 3.4.. Due to the orientation and geometry of the harbor and the shape of the tidal forcing curve within Vineyard/Nantucket Sound, ebb flow velocities are an order of magnitude less than that of flood conditions in the vicinity of the project area.

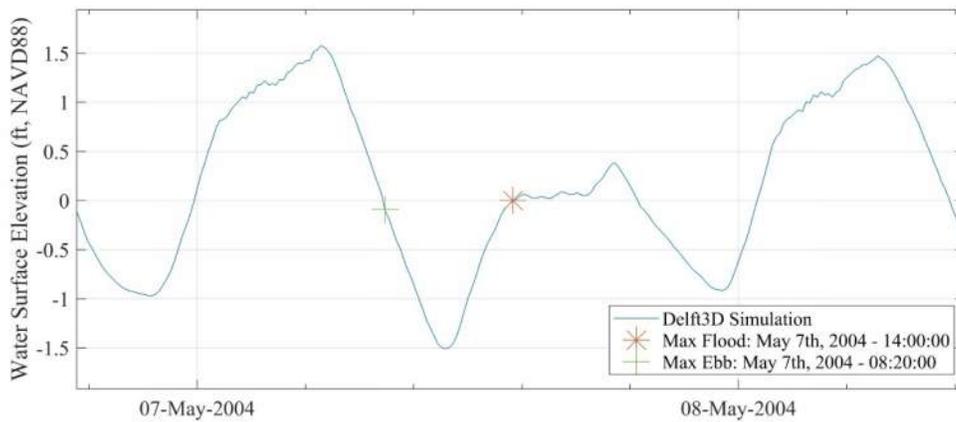


Figure 3.2 Modeled tide signal for a strong flood (May 7th, 2004 14:00:00) and ebb (May 7th, 2004 08:20:00) timestep for the simulation period, where the maximum flood tide conditions are shown in Figure 3.3 and the maximum ebb tide conditions are shown in Figure 3.4..

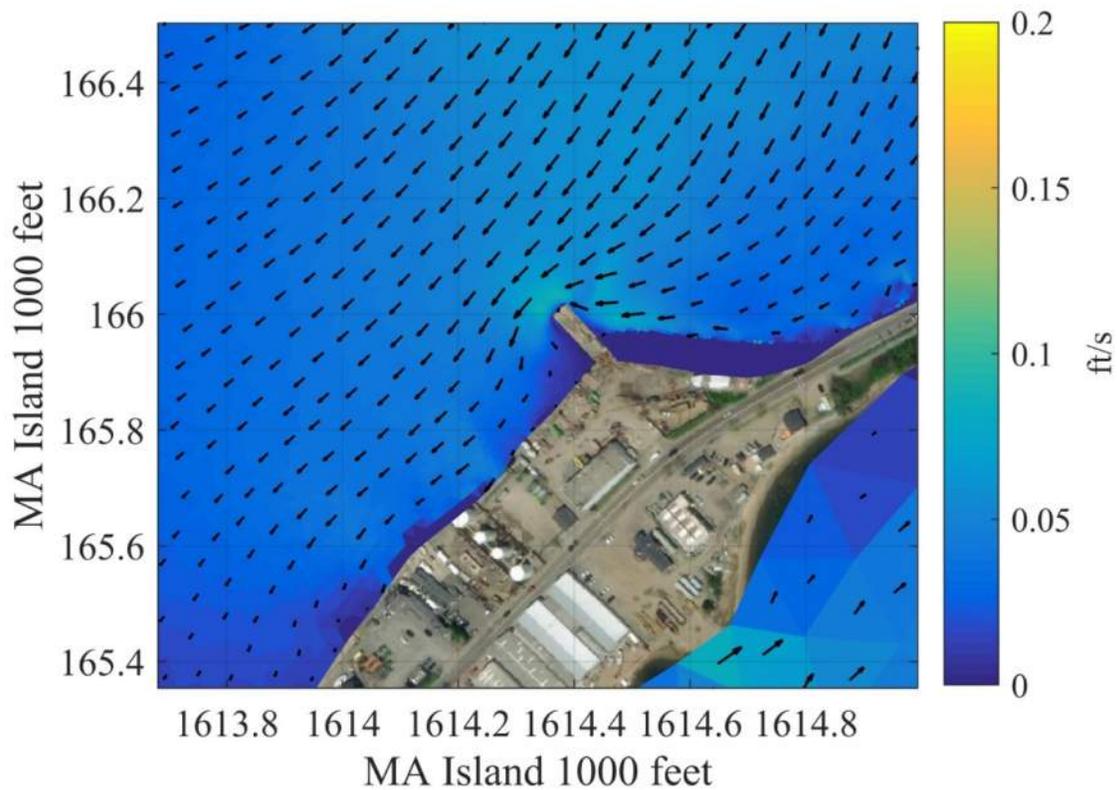


Figure 3.3 Flow velocity magnitude and direction during the May 7th, 2004 14:00:00 timestep under existing conditions (scenario 1). This timestep is during a period of maximum flood conditions with a modeled tide range of 3 feet.

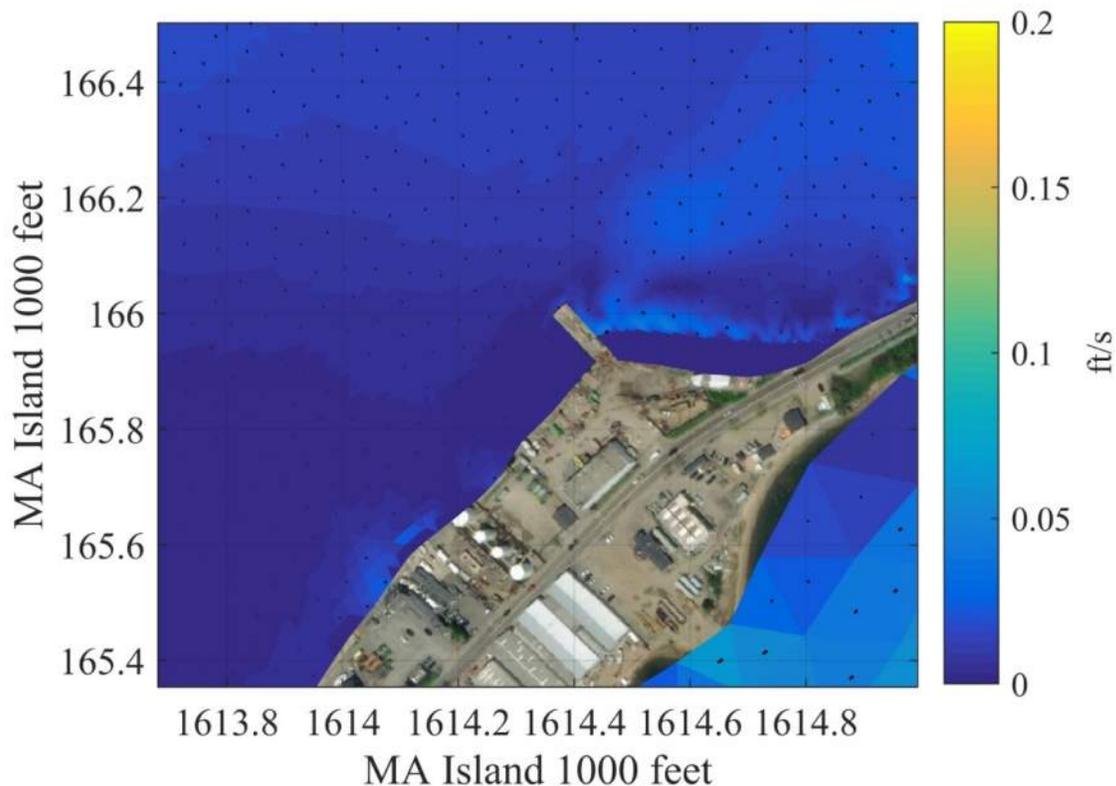


Figure 3.4 Flow velocity magnitude and direction during the May 7th, 2004 08:20:00 timestep under existing conditions (scenario 1). This timestep is during a period of maximum ebb conditions with a modeled tide range of 3 feet.

Changes to circulation patterns in the system were also evaluated by a conservative tracer study. Delft3D provides a salinity variable within the flow model, in which a salinity value can be prescribed spatially throughout the model domain. A value of 1 was prescribed to the area of the proposed boat basin expansion (see red area in Figure 3.5) and a value of 0 was included throughout the rest of the model domain (blue region). A diffusion coefficient of 0.00005 was selected based on recommended model parameters.

For existing conditions, most of the initial concentration within the project area disperses within the first 30 minutes (Figure 3.6), and nearly all the tracer disperses following 60 minutes (Figure 3.7). For comparison with other alternatives, tracer concentrations are also presented after 120 minutes (Figure 3.8). These dispersion results serve as the baseline conditions used to evaluate each structural scenario in their respective circulation impact section. Further, four (4) locations (A, B, C, and D in Figure 3.5) in the region of the proposed dredged basin were selected to provide a more detailed evaluation of tidal circulation for the various scenarios compared to the existing conditions, as described in more detail below.



Figure 3.5 Initial “salinity” value prescribed to the model. The red region was provided a value of 1 and the surrounding blue region contains a value of 0. The four observation points, A - D, were used to monitor the dispersion of the initial concentration.

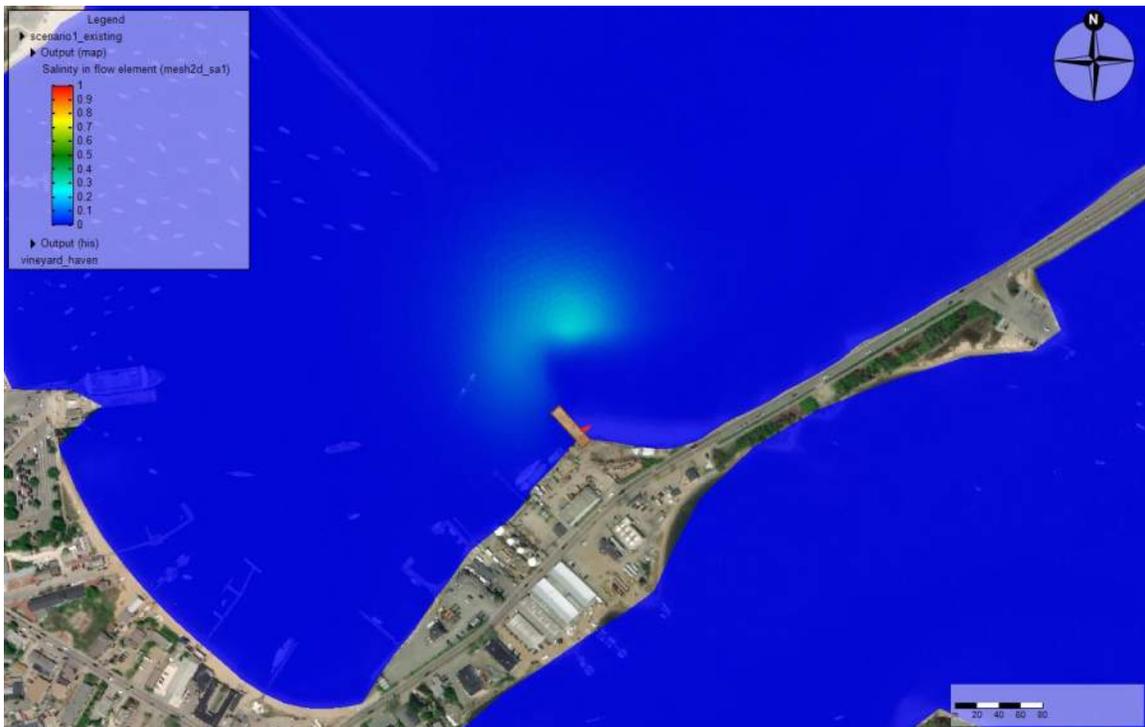


Figure 3.6 Conservative tracer results for the existing conditions scenario following a 60-minute period of tidal flow. The concentration has primarily left the project area and is mixing with the surrounding water.

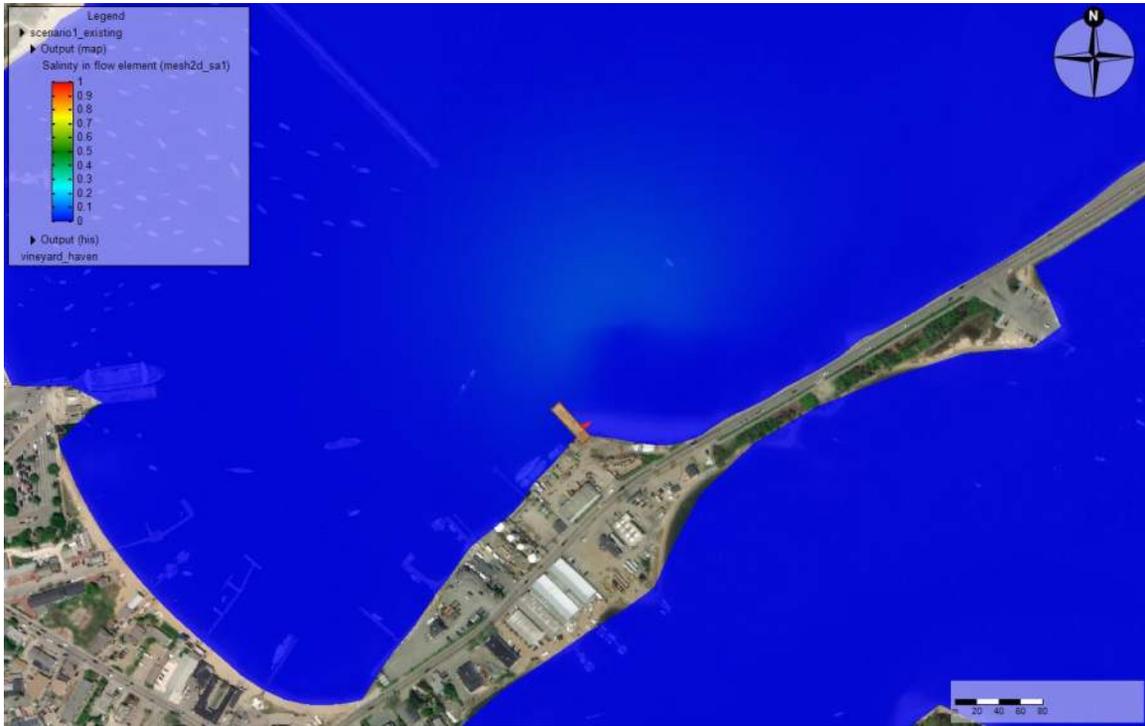


Figure 3.7 Conservative tracer results for the existing conditions scenario following a 120-minute period of tidal flow. The concentration has mostly dispersed at this point.

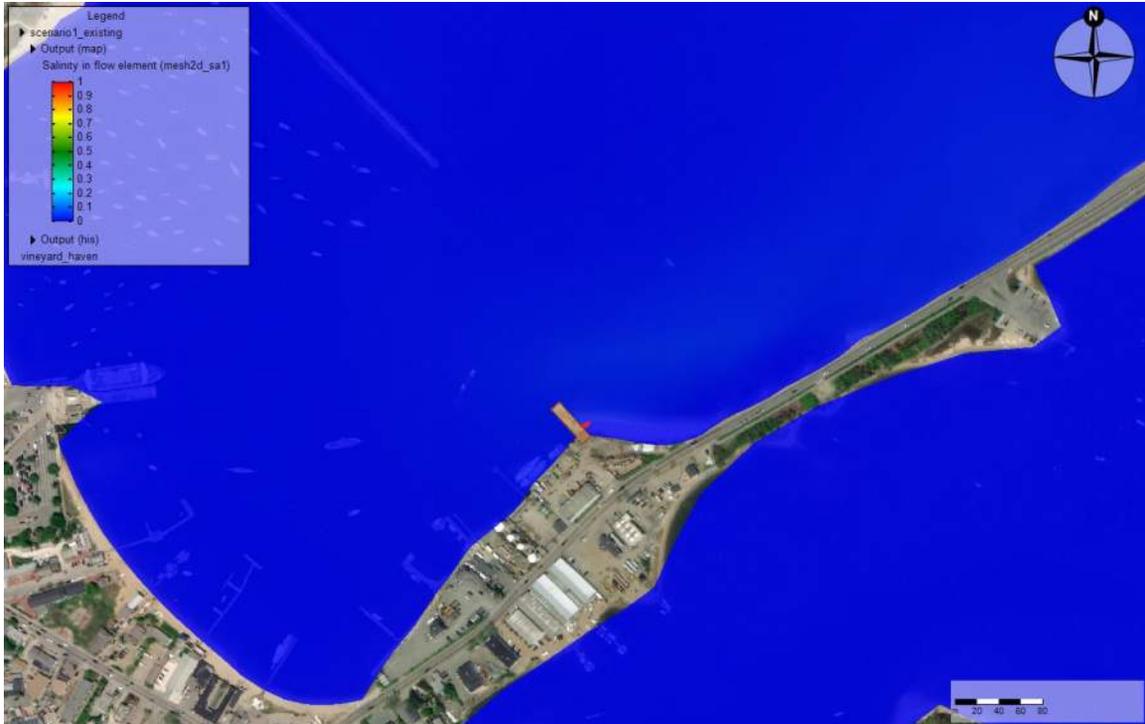


Figure 3.8 Conservative tracer results for the existing conditions scenario following a 360-minute period of tidal flow. The concentration has mostly dispersed at this point.

3.1.2 Wave Induced Sediment Transport

As described previously, time series of hourly wave conditions was output from the Nantucket Sound grid at a location that corresponds to the northern open boundary of the fine wave grid used to propagate waves into Vineyard Haven Harbor. The high-resolution wave and tidal conditions were simulated within Vineyard Haven Harbor as the driving forces responsible for sediment transport. Figure 3.9 provides the results of the sediment transport analysis run over a one-year time period, where the model computes morphologic change that occurs as a result of the combined tidal current and wave field. Due to the weak tidal currents and relatively mild wave climate within the confines of Vineyard Haven Harbor, sediment transport is limited to the shallow beach areas of the harbor. In this case, the dominant area of sediment movement is along the beach at the eastern extent of the existing Packer Facility. Sediment movement elsewhere in the project region is negligible.

Figure 3.9 illustrates that while sediment movement generally moves east-to-west along the beach (as shown by increased accretion along the area furthest to the west), the energetic wave climate in 2018 tended to reshape the beach profile, where erosion of the upper beach and accretion of the lower beach occurred.

In addition to morphology change, the sediment transport model also computed sediment flux (i.e. net sediment transport rates) along the beach. For this analysis, sediment flux was computed across three (3) shore-perpendicular transects along the beach, as shown in Figure 3.10. Sediment transport across each of these lines was computed for existing conditions and serves as the baseline for comparison of the various

structural alternatives. It should be noted that the mild wave conditions in the harbor prevent large volumes of sediment transport along the beach. In this case, the annual transport rates are generally below 10 cubic yards per year, which is several orders of magnitude lower than beaches that are exposed to either Nantucket Sound or open Atlantic Ocean waves.

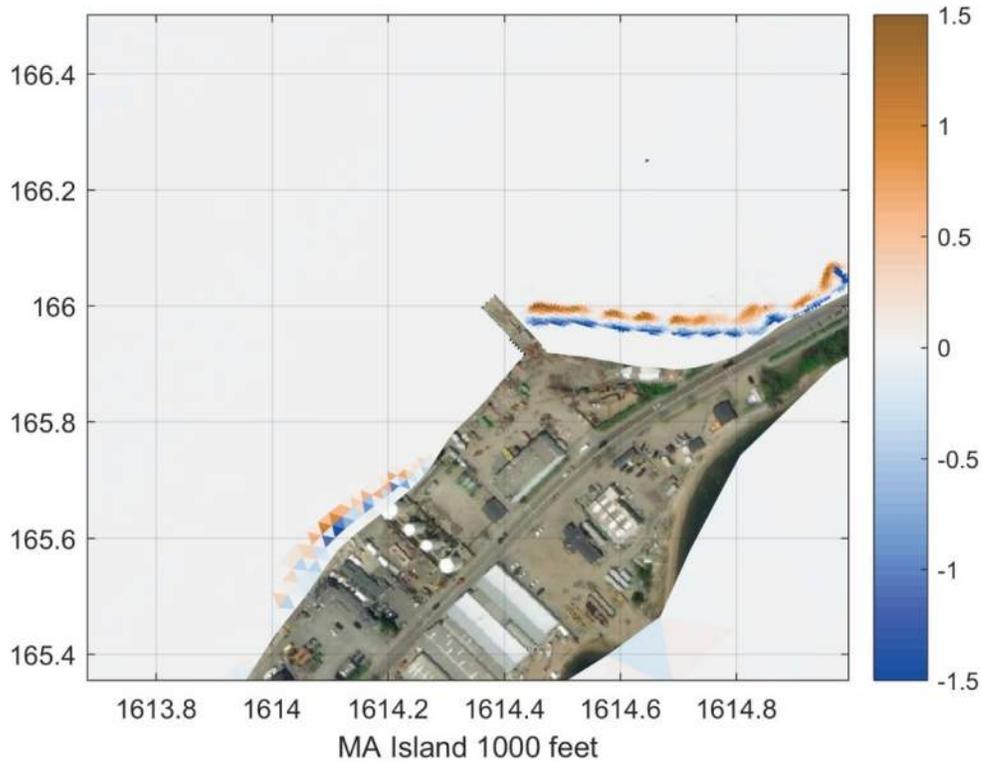


Figure 3.9 Plot of morphology change for the annual run of existing conditions, contours in feet. Blue shades indicate erosion and brown shades indicate accretion.



Figure 3.10 Map of study area with transect lines used to compute annualized cumulative sediment transport flux across cross-Sections 1, 2, and 3. The direction of the arrow on each cross-section indicate the direction of positive flux.

3.2 Scenario 2: Full Structures

Scenario 2 included all four proposed structures (pile supported platform, realigned solid fill pier, bulkhead extension to the east, and a solid wave fence), as depicted in the plans developed for the Environmental Notification Form submittal. Under this scenario, the wave fence would be constructed to the seafloor, with no gap on the bottom. This would effectively block any transmission of waves and sediment into the boat basin. The bulkhead would be designed to an elevation of 6.0 feet NAVD88, and extend from the base of the pier to the south end of the wave fence. Flow would therefore be unable to pass through the contiguous structure formed by the bulkhead and wave fence, thus disrupting existing circulation patterns. Finally, the existing solid fill pier would be reconstructed to modify the existing footprint. This scenario has the greatest overall impact on flow and waves out of all tested scenarios.

Some noticeable differences between flow under the existing conditions (Scenario 1) and Scenario 2 are increased velocities around the northwestern end of the wave fence (Figure 3.11) during a strong flooding tide. There is also an observed decrease in flow in the boat basin and around the pier array due to sheltering. During a maximum ebb tide, there is not much difference between Scenario 2 and existing conditions (Figure 3.12), as flows are minimal. It is worth noting that any noticeable difference is only observed during periods of greater than average tides, and that the velocities are insignificant in terms of their ability to mobilize and transport sediment.

A conservative tracer study was also utilized to provide a semi-quantitative assessment of changes to circulation patterns between scenarios. Due to the flow obstructions created by the combined bulkhead and wave fence structures depicted in Scenario 2, dispersion of the conservative tracer is inhibited. Comparison of the conservative tracer concentration at the four (4) locations depicted in Figure 3.5 within the region of the boat basin are shown in Figures 3.13 through 3.16, respectively. Based on these results, tidal circulation is inhibited by the structures, as it requires more than 12 hours for the tracer to disperse over a large portion of the basin, compared to less than 1 hour for existing conditions. Visually, this inhibited dispersion is illustrated in Figures 3.17 through 3.20, for times 0, 60 minutes, 120 minutes, and 360 minutes, respectively. Section 4 includes a further discussion of the tracer study between scenarios.

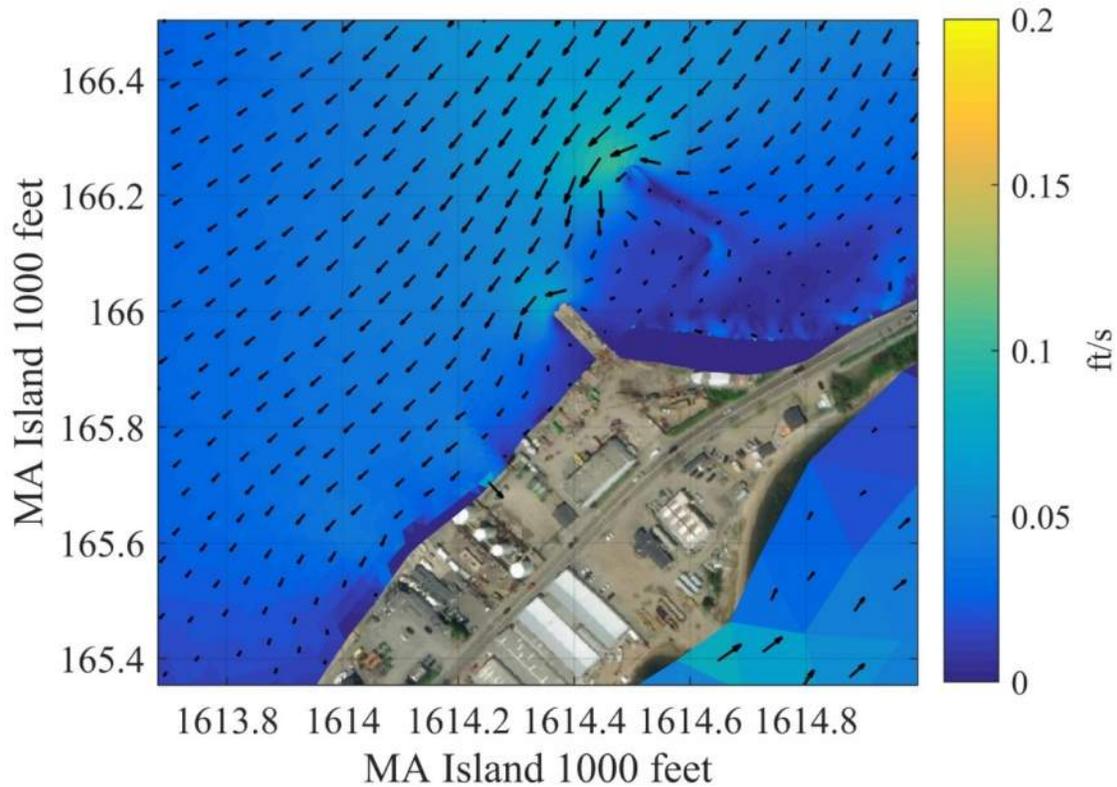


Figure 3.11 Flow velocity magnitude and direction during the May 7th, 2004 14:00:00 timestep (maximum flood tide) for Scenario 2 conditions.

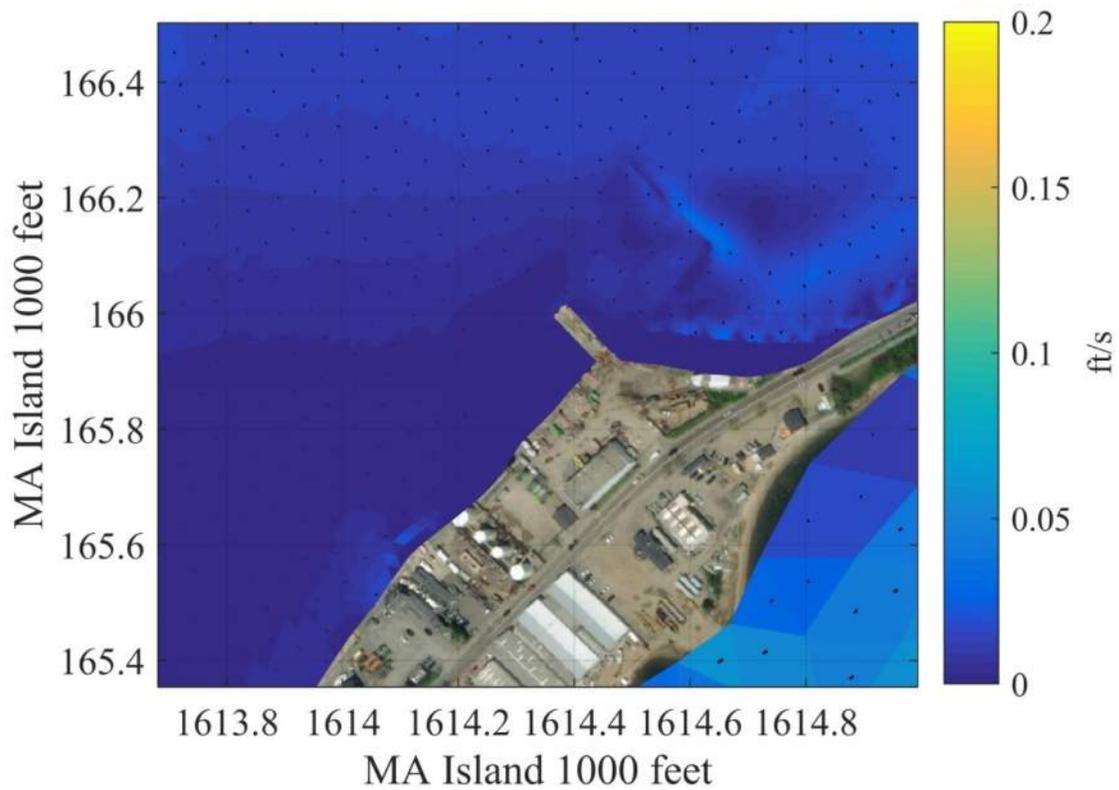


Figure 3.12 Flow velocity magnitude and direction during the May 7th, 2004 08:20:00 timestep (maximum ebb tide) for Scenario 2 conditions.

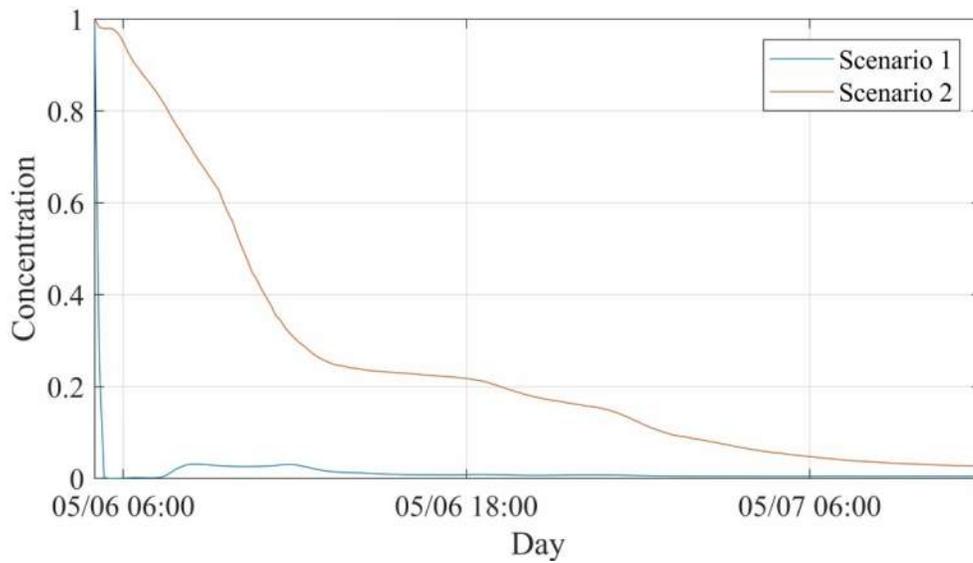


Figure 3.13 Conservative tracer time series results as observed from the included observation point A in the southeast corner of the boat basin (Figure 3.5).

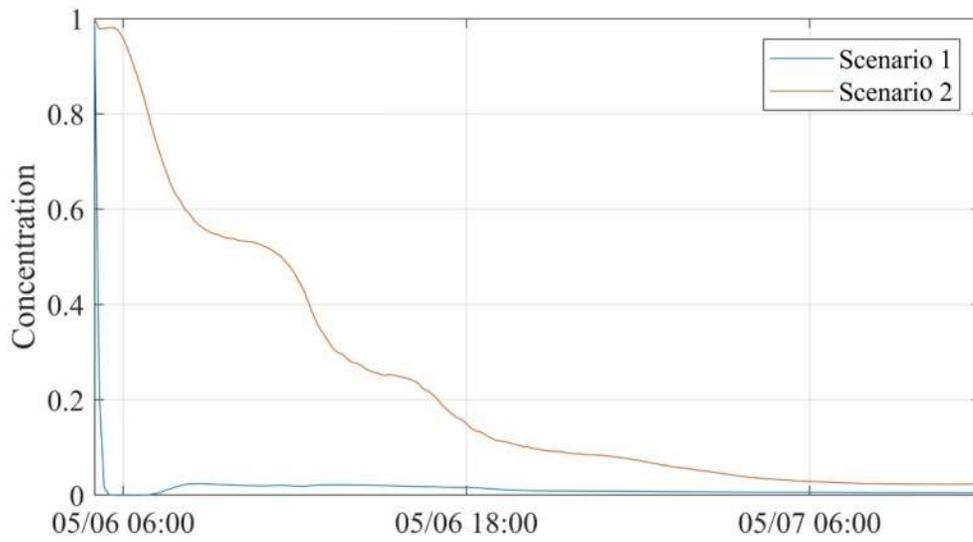


Figure 3.14 Conservative tracer time series results as observed from the included observation point B in the south edge of the boat basin (Figure 3.5).

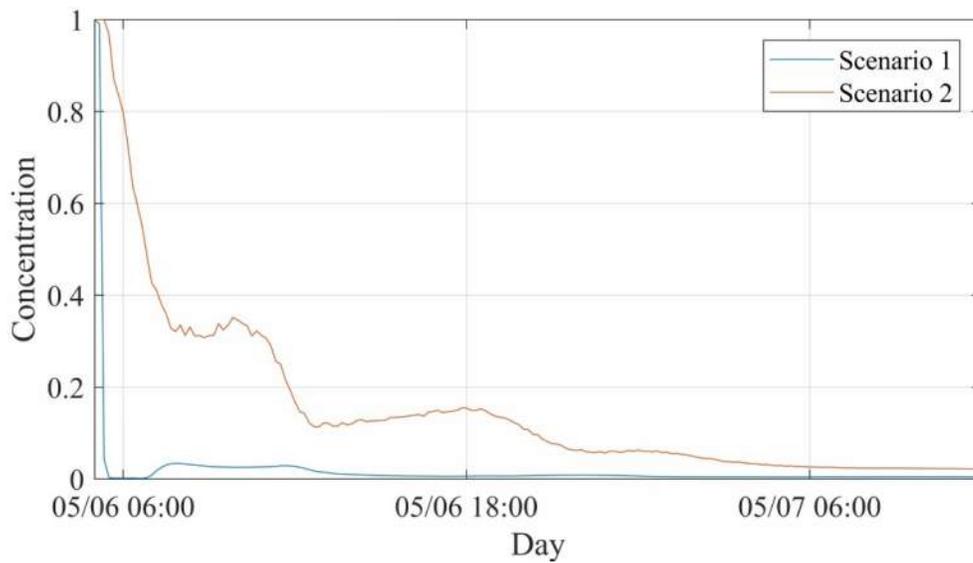


Figure 3.15 Conservative tracer time series results as observed from the included observation point C in the center of the boat basin (Figure 3.5).

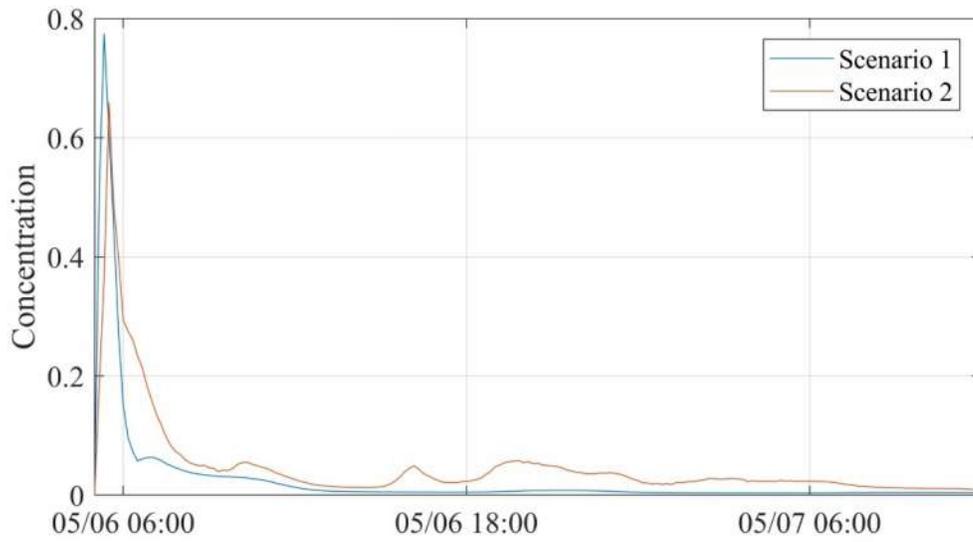


Figure 3.16 Conservative tracer time series results as observed from the included observation point D along the seaward edge of the boat basin (Figure 3.5).



Figure 3.17 Initial tracer value prescribed to the model with Scenario 2 structures included. The red region was provided a value of 1 and the surrounding blue region contains a value of 0.

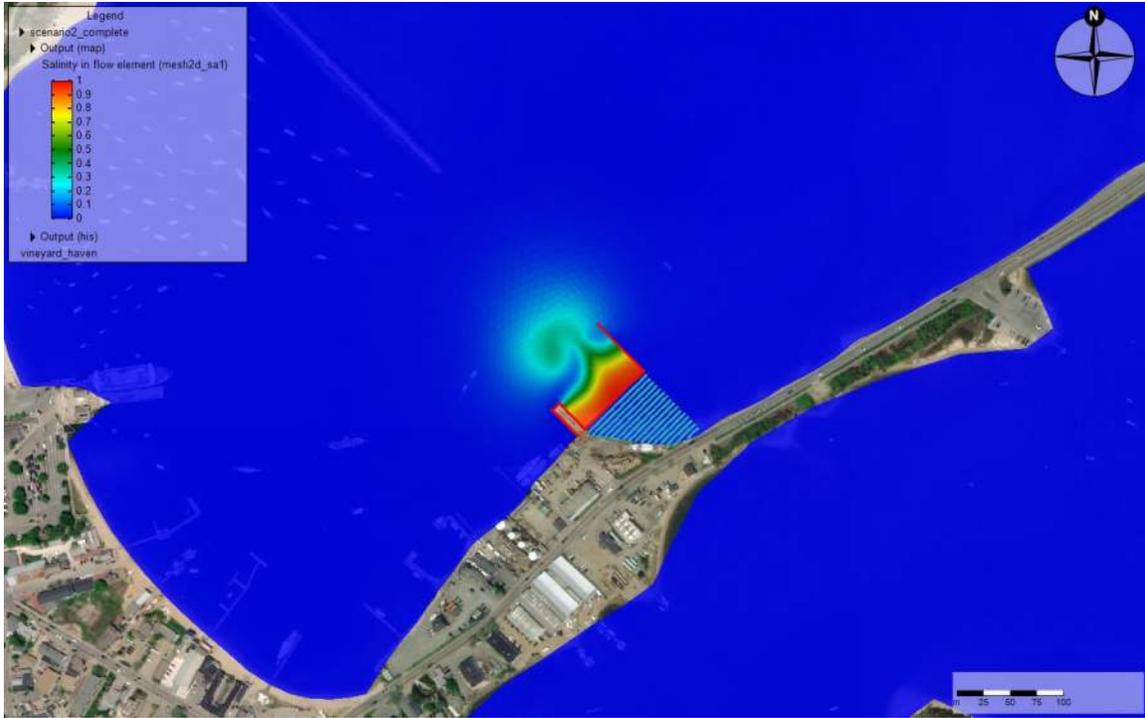


Figure 3.18 Conservative tracer results for the Scenario 2 following a 60-minute period of tidal flow. Most of the initial concentration still remains within the project area.

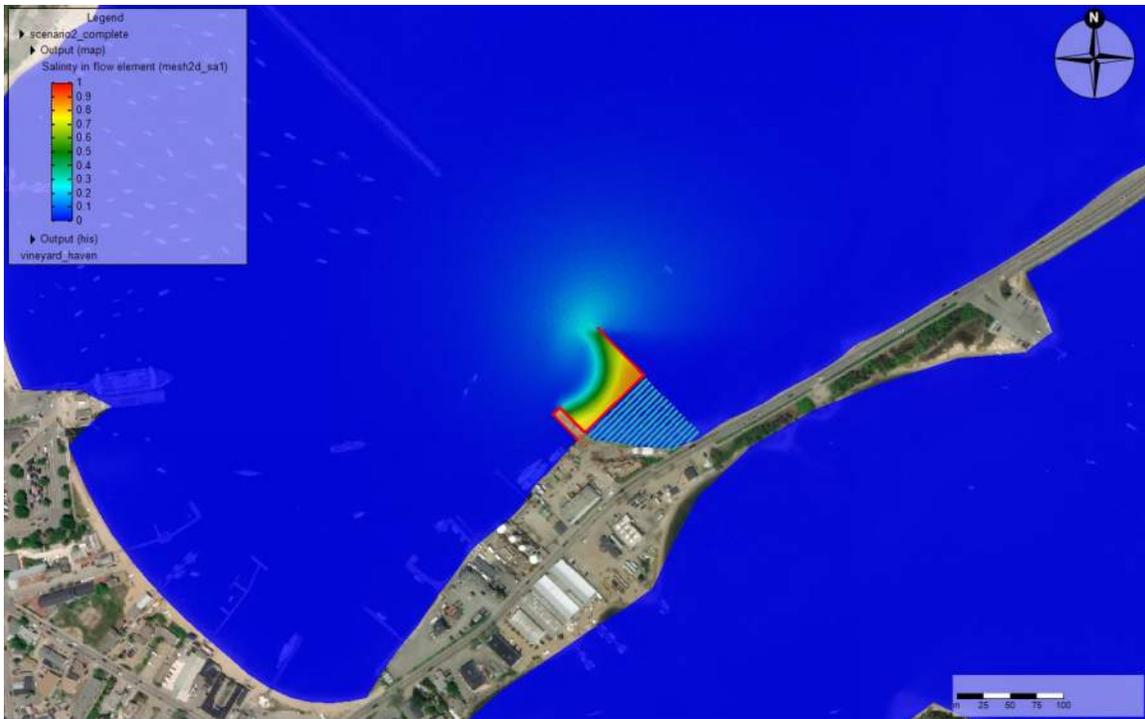


Figure 3.19 Conservative tracer results for the Scenario 2 following a 120-minute period of tidal flow.

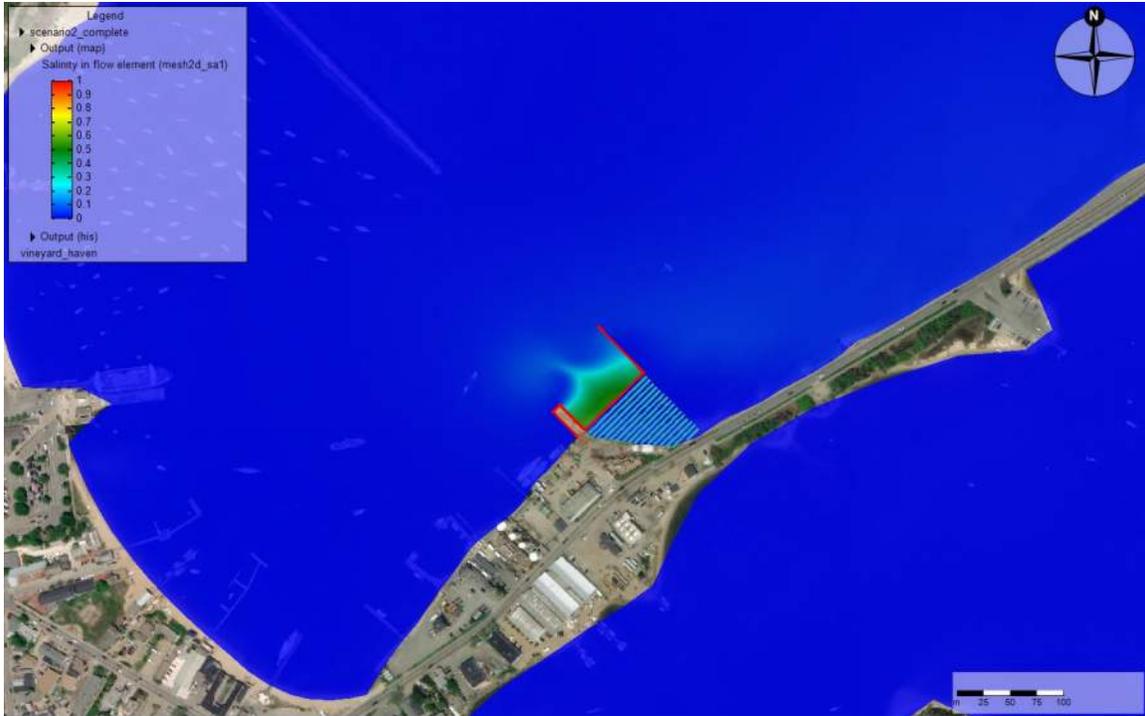


Figure 3.20 Conservative tracer results for Scenario 2 following a 360-minute period of tidal flow.

In addition to changes in circulation, wave-induced sediment transport was also evaluated for Scenario 2. Although hydrodynamic related sediment transport was included as well, the values alone were determined to be negligible based on a D-Flow model simulation without waves. Sediment transport results for Scenario 2 when compared with existing conditions for the modeled year indicates that there are small changes in sediment movement that are mostly due to wave sheltering by the new bulkhead. Figure 3.21 shows that no bathymetric change occurs within the dock facility. Southeast of the bulkhead, reduced wave heights in the area behind the bulkhead cause nearly zero change in the beach elevation within 50 feet of the wall. Beyond that point, the magnitude of change is the same as what occurs for existing conditions. This can be seen in the bathymetric change difference plot provide in Figure 3.22. In this plot, the surface of bathymetric change computed for Scenario 2 is subtracted from the surface of change computed for Scenario 1 existing conditions. It can be seen that the largest differences in this plot are southeast of the bulkhead, but these differences are small and strictly related to wave sheltering from the new structures that are a part of this scenario.

The small magnitude of changes that result from the Scenario 2 configuration is also evident in the sediment transport rates determined for the beach for existing conditions and this scenario. Figure 3.23 and Figure 3.24 show a trace of cumulative sediment flux at cross-shore transect line placed both east of the project area and alongside the bulkhead, respectively (Figure 3.10 for locations of these transects). These sediment transport rates were computed for the duration of the annual simulation. The results show that net transport for existing conditions has a very small west-directed magnitude of 3.2 cubic yards per year. With the wave sheltering provided by the array of the structural

components that are a part of Scenario 2, the net transport magnitude is reduced to 1.3 cubic yards per year, and is directed now to the east. The small net transport flux of both scenarios indicates that alongshore transport is small even during active periods like the modeled year. Overall, the actual transport rates are almost negligible for both existing conditions and for the Full Structures alternative (Scenario 2)

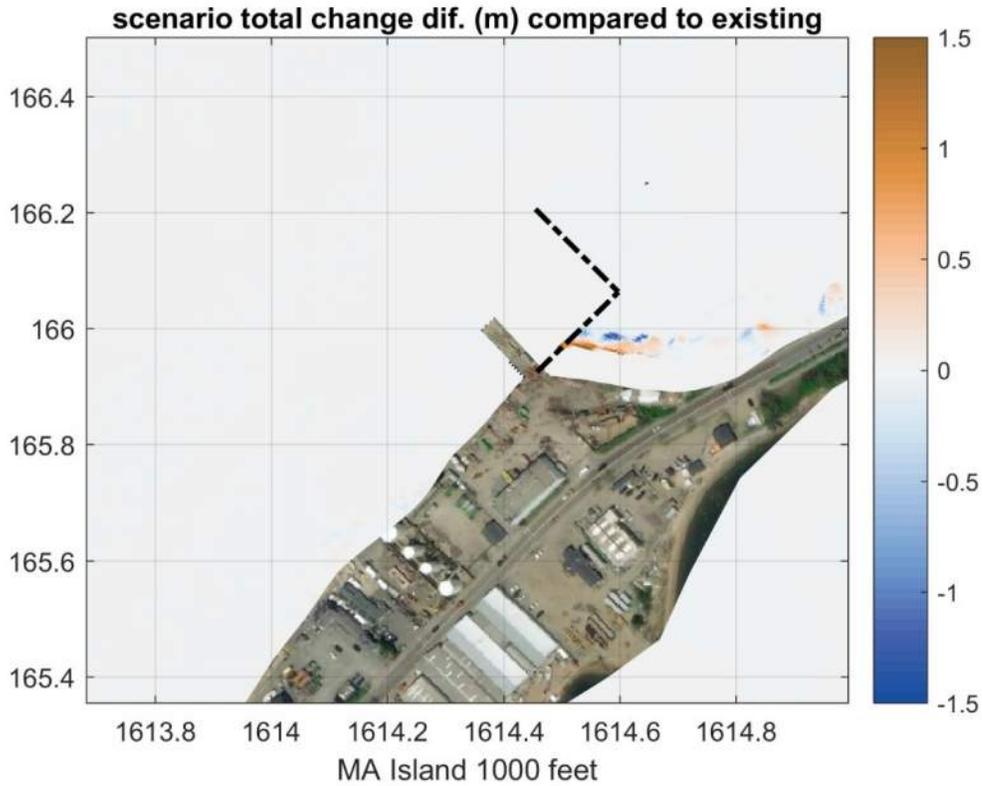


Figure 3.21 Plot of morphology change for the annual run of Scenario 2 conditions, contours in feet. Blue shades indicate erosion and brown shades indicate accretion. The location of the new wave fence and bulkhead are indicated using the black dot-dash line.

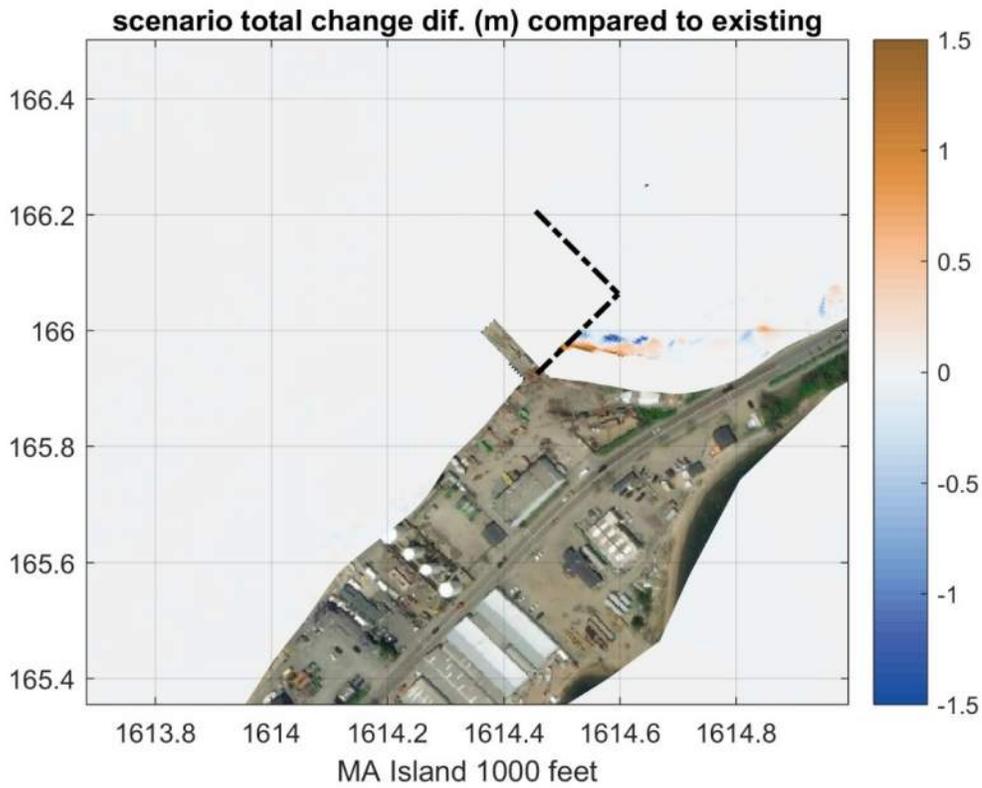


Figure 3.22 Color contour plot of difference in elevation change (in meters) between existing conditions and Scenario 2 plan. Brown shades indicate areas of less erosion in Scenario 2, while blue shades indicate areas of less accretion in Scenario 2. The location of the new wave fence and bulkhead are indicated using the black dot-dash line.

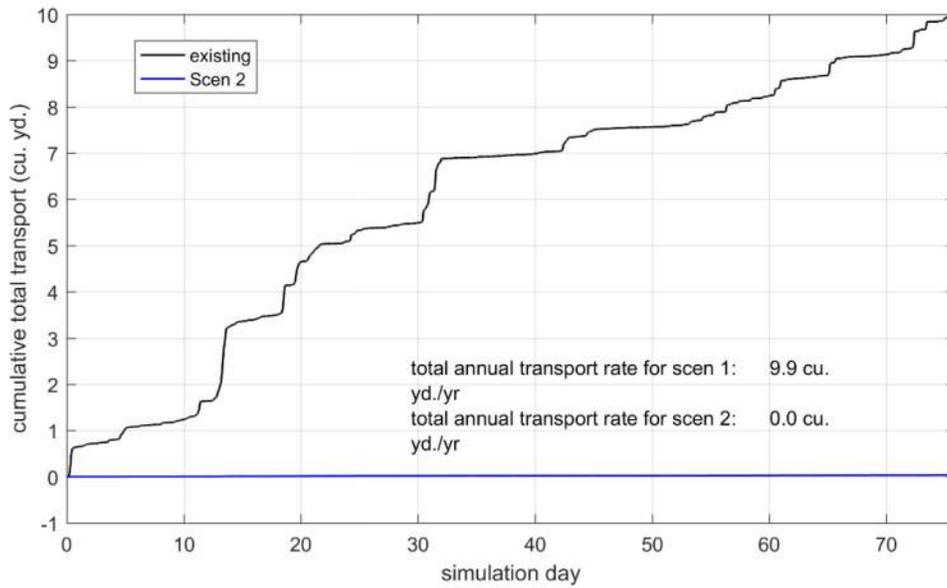


Figure 3.23 Cumulative sediment transport across the cross-shore transect line 1 indicated in Figure 3.10. Positive flux is directed to the southwest, while negative flux is directed to the northeast (toward Lagoon Pond inlet).

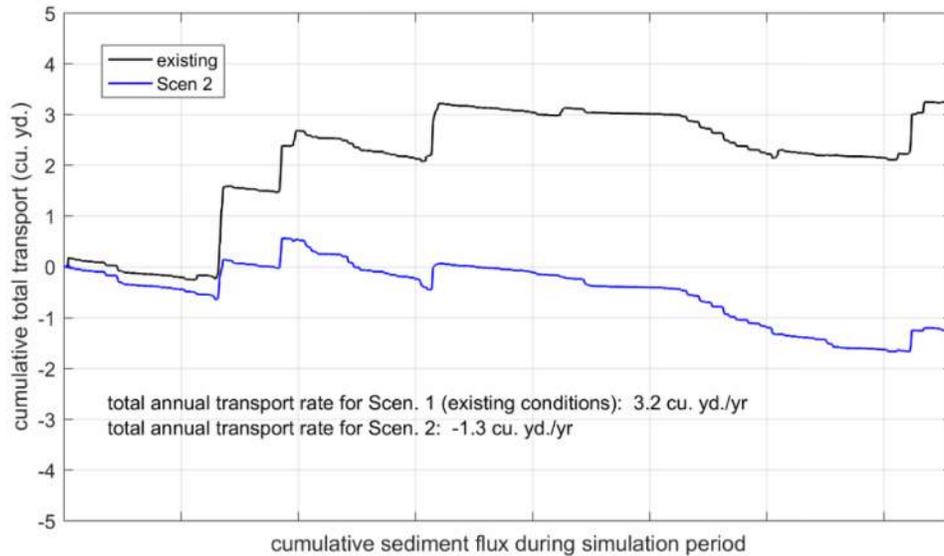


Figure 3.24 Cumulative sediment transport across the cross-shore transect line 3 indicated in Figure 3.10. Positive flux is directed towards the planned boat basin, while negative flux is directed to the northeast (toward Lagoon Pond inlet).

3.3 Scenario 3: Bulkhead Modification

Scenario 3 includes the complete solid wave fence and relocated solid fill pier, with a revised bulkhead elevation that lowers the top height of the bulkhead to an elevation that is 2 feet above the existing grade to the adjacent area south of the bulkhead. This scenario reduced circulation impacts in the wave basin when compared to Scenario 2, as flow was able to pass over the eastern side of the bulkhead.

Some noticeable differences between flow under the existing conditions (scenario 1) and scenario 3 are increased velocities around the northwestern and northeastern end of the wave fence (Figure 3.25) during a strong flooding tide. There is a slight modification in flow within the boat basin, however, lowering of the bulkhead improves flow on the western side of the basin. During a max ebb tide, there is not much difference between scenario 3 and existing conditions, as flows are minimal. It is worth noting that any noticeable difference is only observed during periods of greater than average tides, and that the velocities are insignificant in terms of their ability to mobilize and transport sediment.

Looking at the results of the conservative tracer study, there is a significant improvement in circulation when compared to scenario 2. Due to the lowering of the bulkhead flow is able to pass over the structure, flushing out the wave basin from the east. Comparison of the conservative tracer concentration at the four (4) locations depicted in Figure 3.5 within the region of the boat basin are shown in Figures 3.27 through 3.30, respectively. There is still a slight inhibition of circulation for this scenario behind the wave fence structure, as it takes a few hours to reduce the initial concentration (compared to less than 1 hour for existing conditions). Visually, this inhibited dispersion is illustrated in Figures 3.31 through 3.33, for times 0, 60 minutes, 120 minutes, and 360 minutes, respectively. Section 4 includes a further discussion of the tracer study between scenarios.

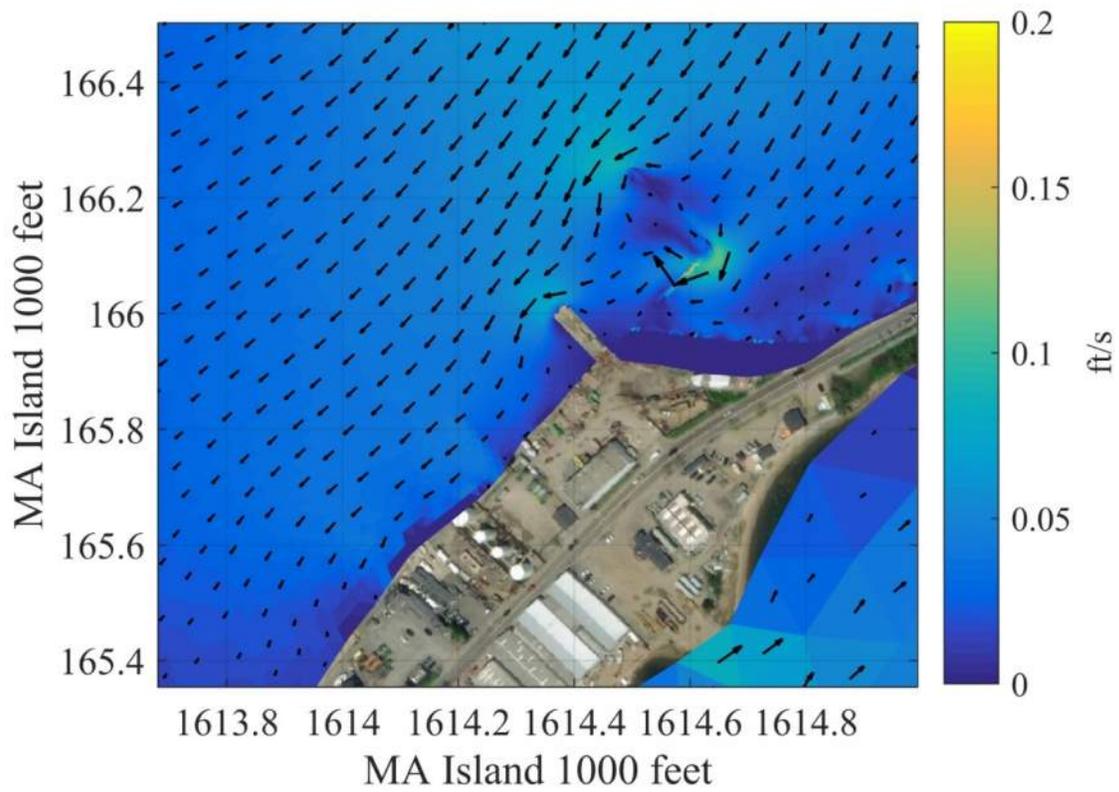


Figure 3.25 Flow velocity magnitude and direction during the May 7th, 2004 14:00:00 timestep (max flood tide) for scenario 3 conditions.

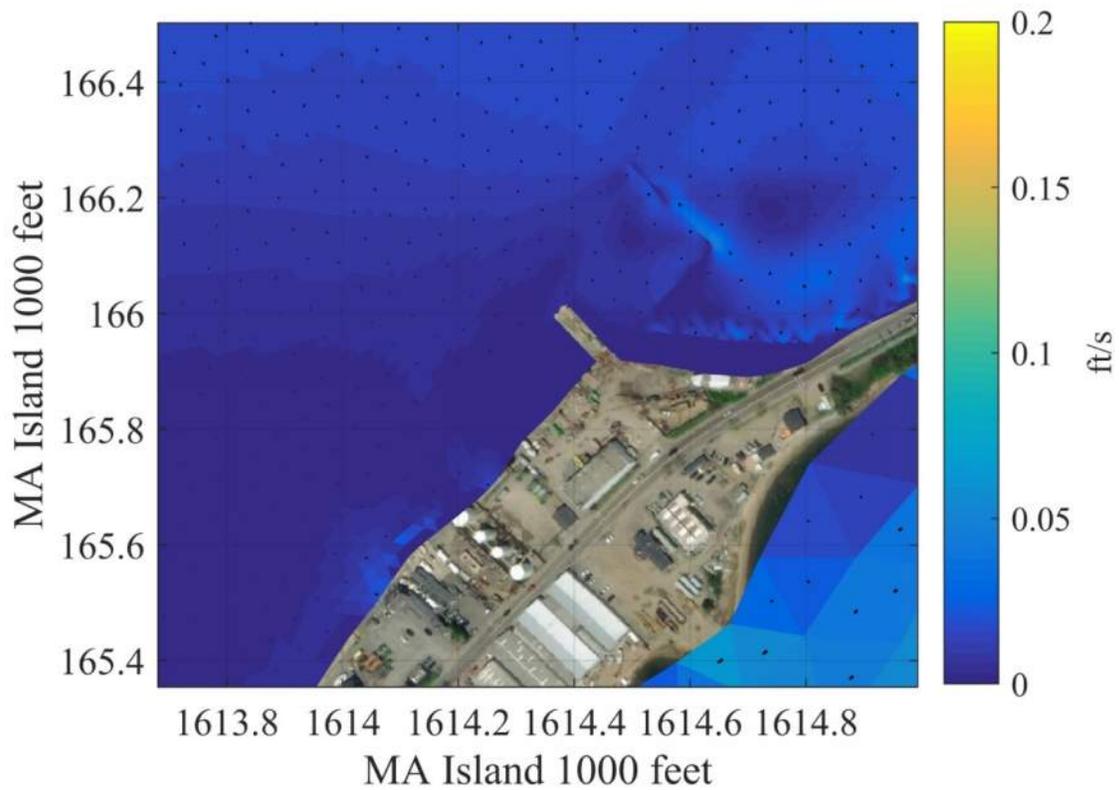


Figure 3.26 Flow velocity magnitude and direction during the May 7th, 2004 08:20:00 timestep (max ebb tide) for scenario 3 conditions.

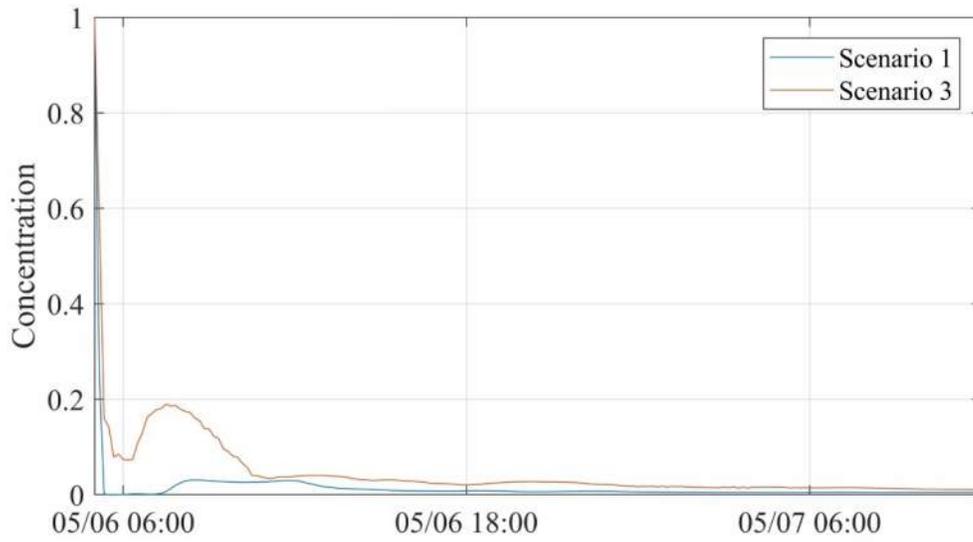


Figure 3.27 Conservative tracer time series results as observed from the included observation point A along the seaward edge of the boat basin (Figure 3.5).

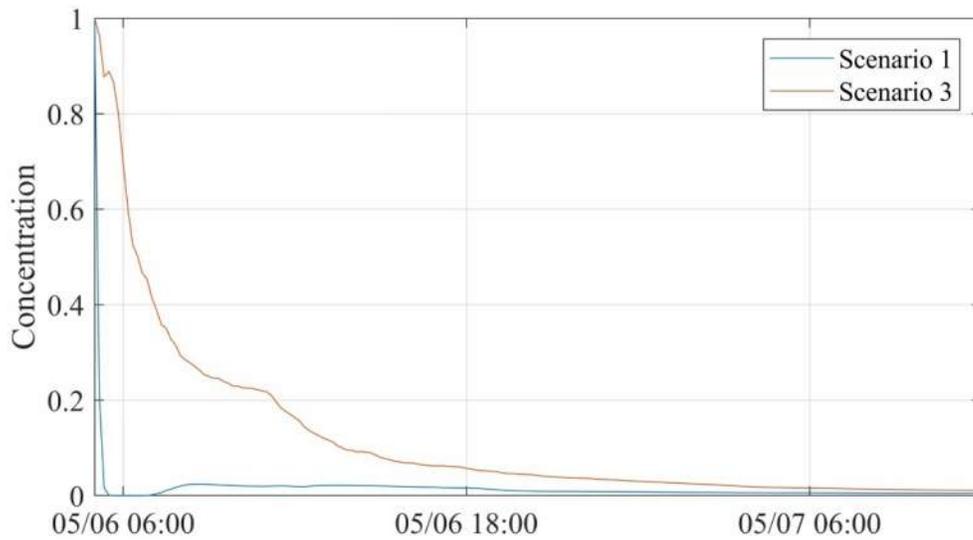


Figure 3.28 Conservative tracer time series results as observed from the included observation point B along the seaward edge of the boat basin (Figure 3.5).

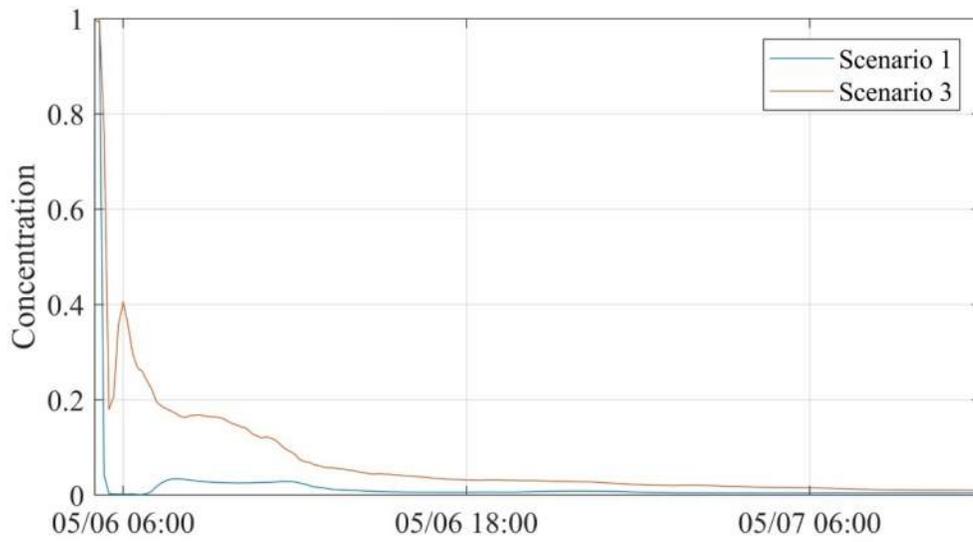


Figure 3.29 Conservative tracer time series results as observed from the included observation point C along the seaward edge of the boat basin (Figure 3.5).

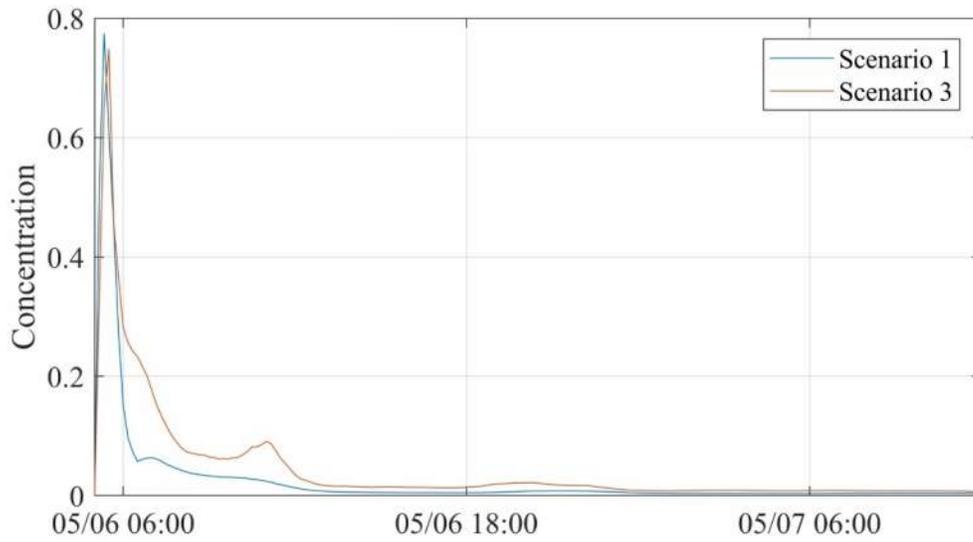


Figure 3.30 Conservative tracer time series results as observed from the included observation point D along the seaward edge of the boat basin (Figure 3.5).



Figure 3.31 Initial “salinity” value prescribed to the model with scenario 3 structures included. The red region was provided a value of 1 and the surrounding blue region contains a value of 0.

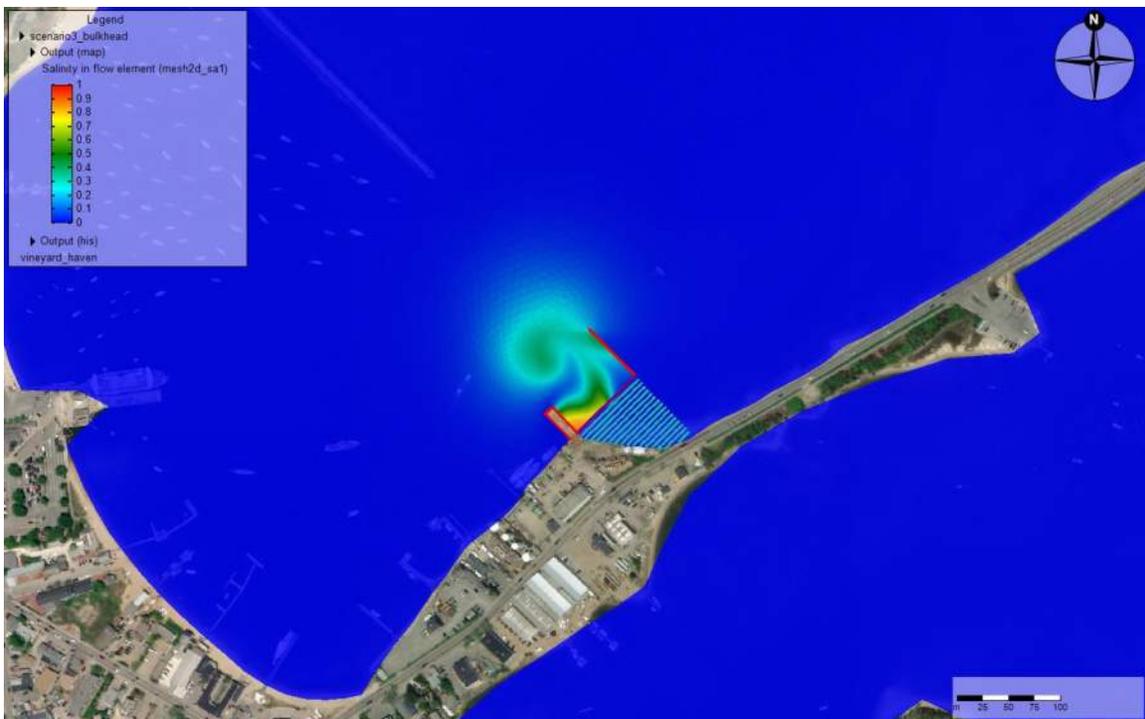


Figure 3.32 “Dye Test” results for the partial structure scenario following a 30-minute period of tidal flow.

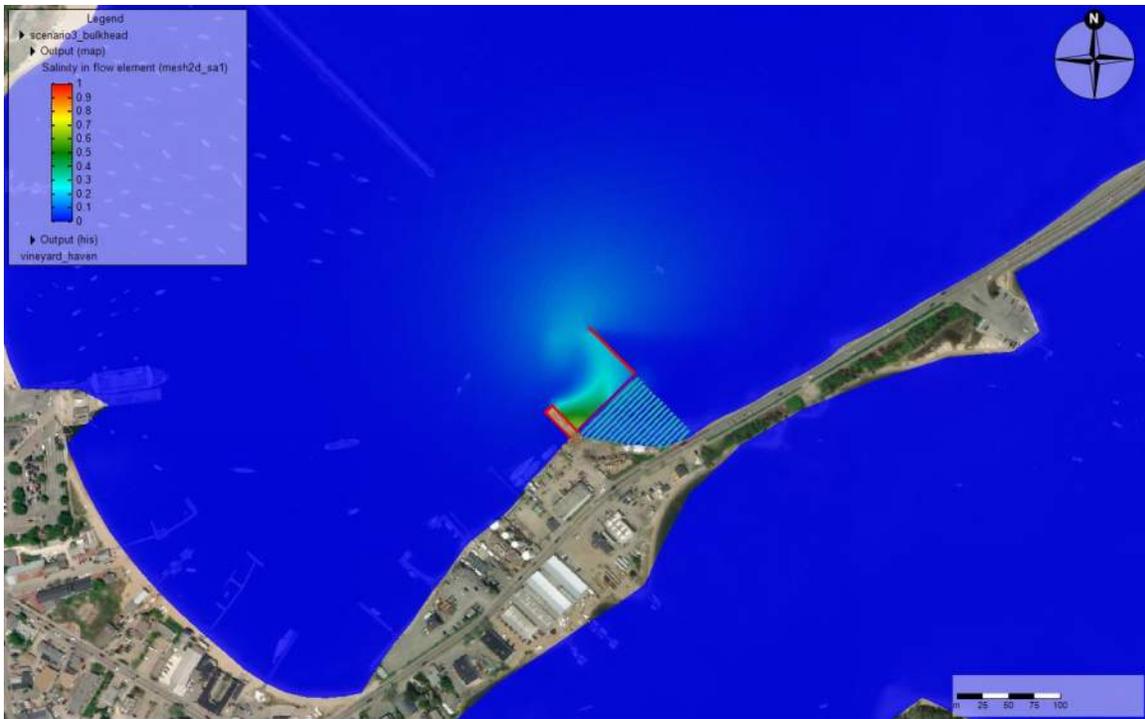


Figure 3.33 "Dye Test" results for the partial structure scenario following a 60-minute period of tidal flow.



Figure 3.34 "Dye Test" results for the partial structure scenario following a 360-minute period of tidal flow.

In addition to changes in circulation, wave-induced sediment transport was also evaluated for Scenario 3. Although hydrodynamic related sediment transport was included as well, the values alone were determined to be negligible based on a D-Flow model simulation without waves. Sediment transport results for Scenario 3 when compared with existing conditions for the modeled year indicates that there are small changes in sediment movement that are mostly due to wave sheltering by the new bulkhead. Figure 3.35 shows that no bathymetric change occurs within the dock facility. Southeast of the bulkhead, reduced wave heights in the area behind the bulkhead cause nearly zero change in the beach elevation within 50 feet of the wall. Beyond that point, the magnitude of change is the same as what occurs for existing conditions. This can be seen in the bathymetric change difference plot provide in Figure 3.36. In this plot, the surface of bathymetric change computed for Scenario 3 is subtracted from the surface of change computed for Scenario 1 existing conditions. It can be seen that the largest differences in this plot are southeast of the bulkhead, but these differences are small and strictly related to wave sheltering from the new structures that are a part of this scenario.

The small magnitude of changes that result from the Scenario 3 configuration is also evident in the sediment transport rates determined for the beach for existing conditions and this scenario. Figure 3.37 and Figure 3.38 show a trace of cumulative sediment flux at cross-shore transect line placed both east of the project area and alongside the bulkhead, respectively (Figure 3.10 for locations of these transects). These sediment transport rates were computed for the duration of the annual simulation. The results show that net transport for existing conditions has a very small west-directed magnitude of 3.2 cubic yards per year. With the wave sheltering provided by the array of the structural components that are a part of Scenario 3, the net transport magnitude is reduced to 0.3 cubic yards per year, and is directed now to the east. The small net transport flux of both scenarios indicates that alongshore transport is small even during active periods like the modeled year. Overall, the actual transport rates are almost negligible for both existing conditions and for the Bulkhead Modification alternative (Scenario 3)

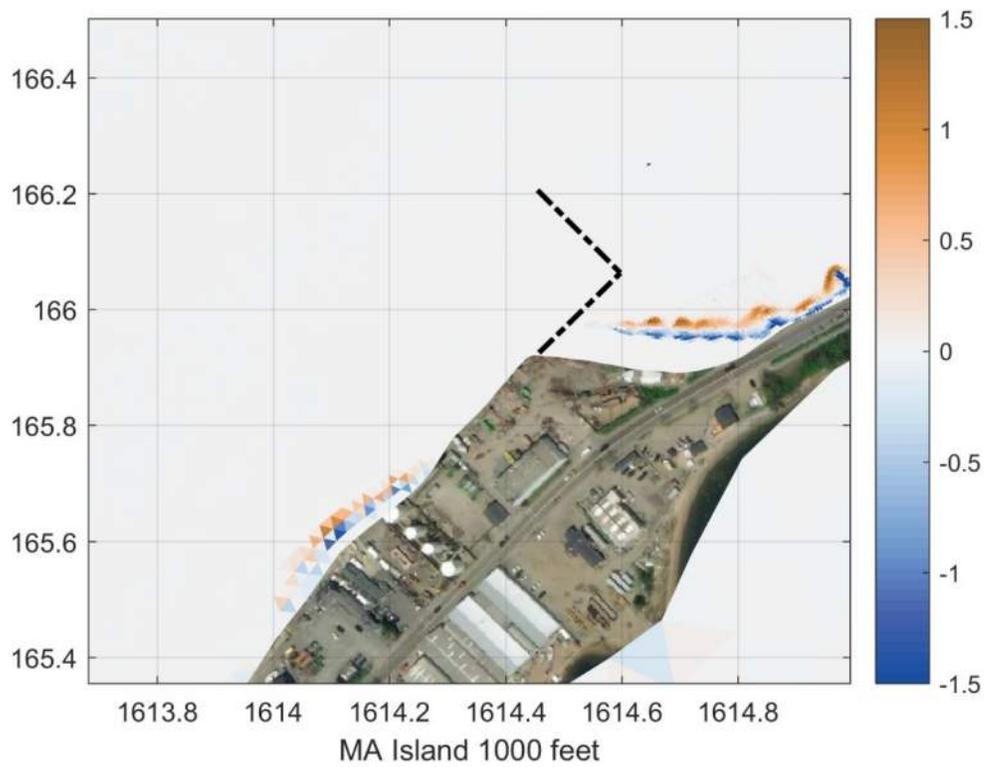


Figure 3.35 Plot of morphology change for the annual run of Scenario 3 conditions, contours in feet. Blue shades indicate erosion and brown shades indicate accretion. The location of the new wave fence and bulkhead are indicated using the black dot-dash line.

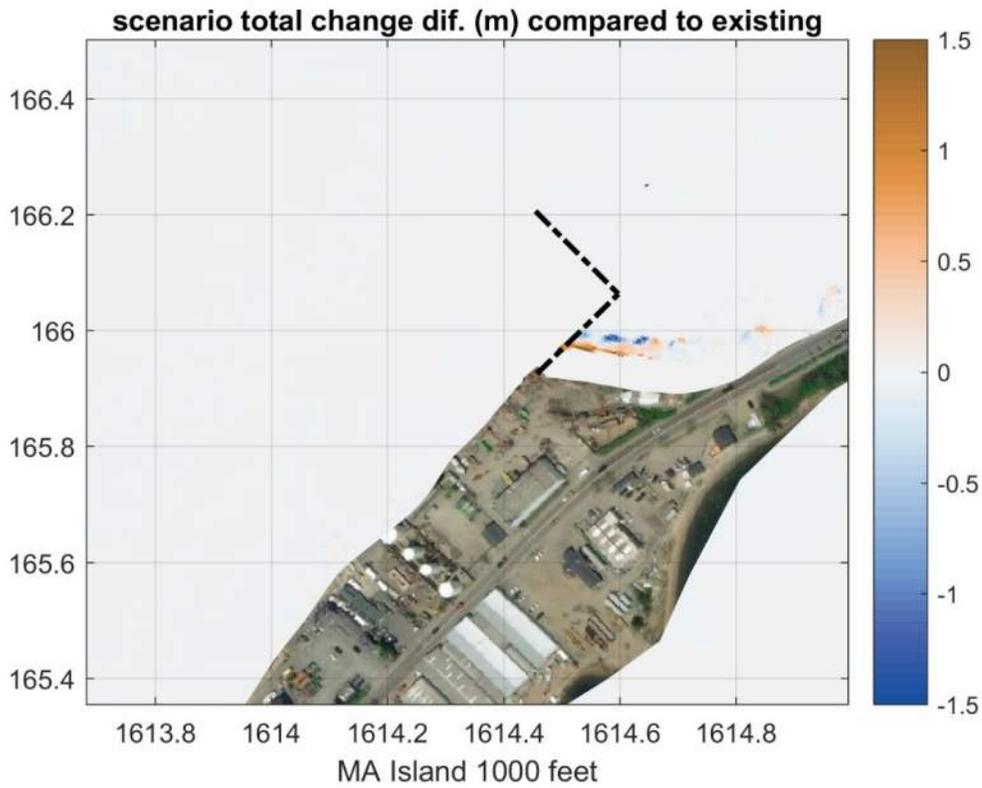


Figure 3.36 Color contour plot of difference in elevation change (in meters) between existing conditions and Scenario 3 plan. Brown shades indicate areas of less erosion in Scenario 3, while blue shades indicate areas of less accretion in Scenario 3. The location of the new wave fence and bulkhead are indicated using the black dot-dash line.

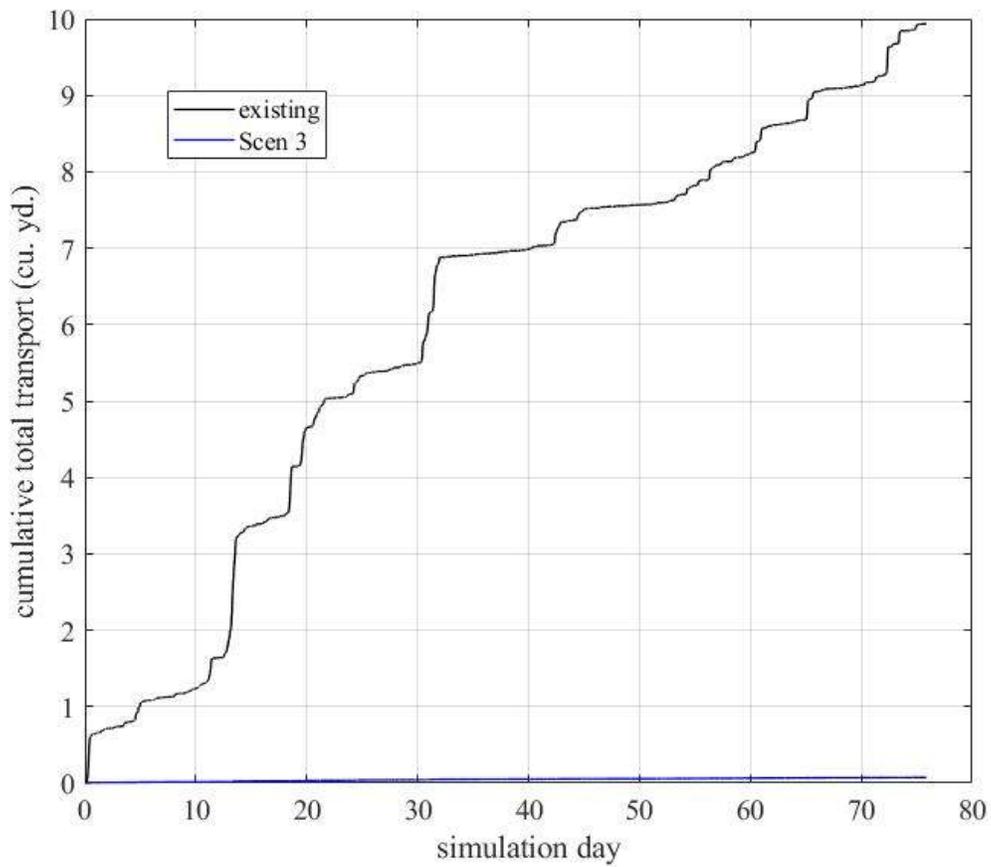


Figure 3.37 Cumulative sediment transport across the cross-shore transect line 1 indicated in Figure 3.10. Positive flux is directed to the southwest, while negative flux is directed to the northeast (toward Lagoon Pond inlet).

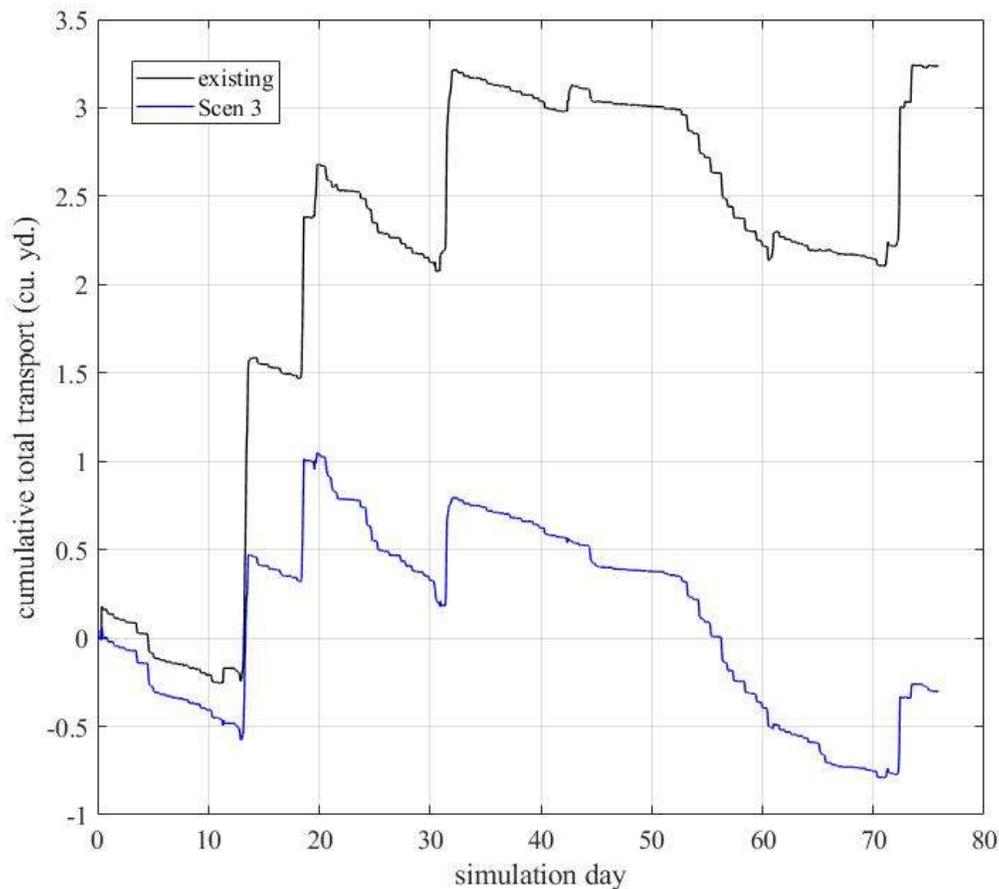


Figure 3.38 Cumulative sediment transport across the cross-shore transect line 2 indicated in Figure 3.10. Positive flux is directed towards the planned boat basin, while negative flux is directed to the northeast (toward Lagoon Pond inlet).

3.4 Scenario 4: Wave Fence Modification

Scenario 4 included all three structures, with modifications to the wave fence. Under this scenario, the wave fence would be constructed with a three-foot gap off of the seafloor. This would allow flow and some sediment to travel underneath and into the wave basin. The bulkhead would be designed to an elevation of six feet NAVD88, and extend from the base of the pier to the end of the wave fence. Flow would therefore be unable to pass through, thus disrupting existing circulation patterns. Finally, the existing solid fill pier would be reconstructed to modify the existing footprint.

Some noticeable differences between flow under the existing conditions (scenario 1) and scenario 4 are increased velocities around the northwestern end of the wave fence (Figure 3.39) during a strong flooding tide. There is some modification in flow within the boat basin, however, the gap in the wave fence allows for flow to pass into the basin. During a max ebb tide, there is not much difference between scenario 4 and existing

conditions, as flows are minimal. It is worth noting that any noticeable difference is only observed during periods of greater than average tides, and that the velocities are insignificant in terms of their ability to mobilize and transport sediment.

Looking at the results of the conservative tracer study, there is a significant improvement in circulation when compared to scenario 2. Due to the gap at the base of the wavefence structure, flow is able to pass underneath, flushing out the wave basin from the north. Comparison of the conservative tracer concentration at the four (4) locations depicted in Figure 3.5 within the region of the boat basin are shown in Figures 3.41 through 3.44, respectively. There is still a slight inhibition in flow and circulation for this scenario as it takes a few hours to reduce the initial concentration in the corner between the solid fill pier and the bulkhead (compared to less than 1 hour for existing conditions). Visually, this inhibited dispersion is illustrated in Figures 3.45 through 3.48, for times 0, 60 minutes, 120 minutes, and 360 minutes, respectively. Section 4 includes a further discussion of the tracer study between scenarios.

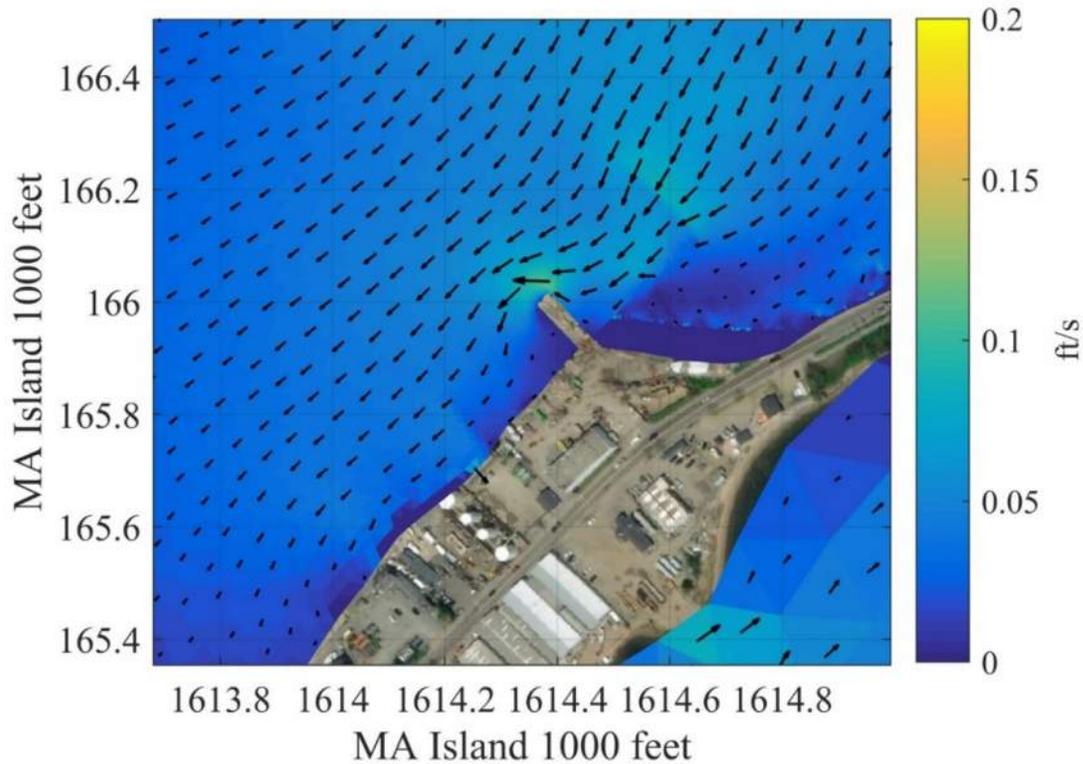


Figure 3.39 Flow velocity magnitude and direction during the May 7th, 2004 14:00:00 timestep (max flood tide) for scenario 4 conditions.

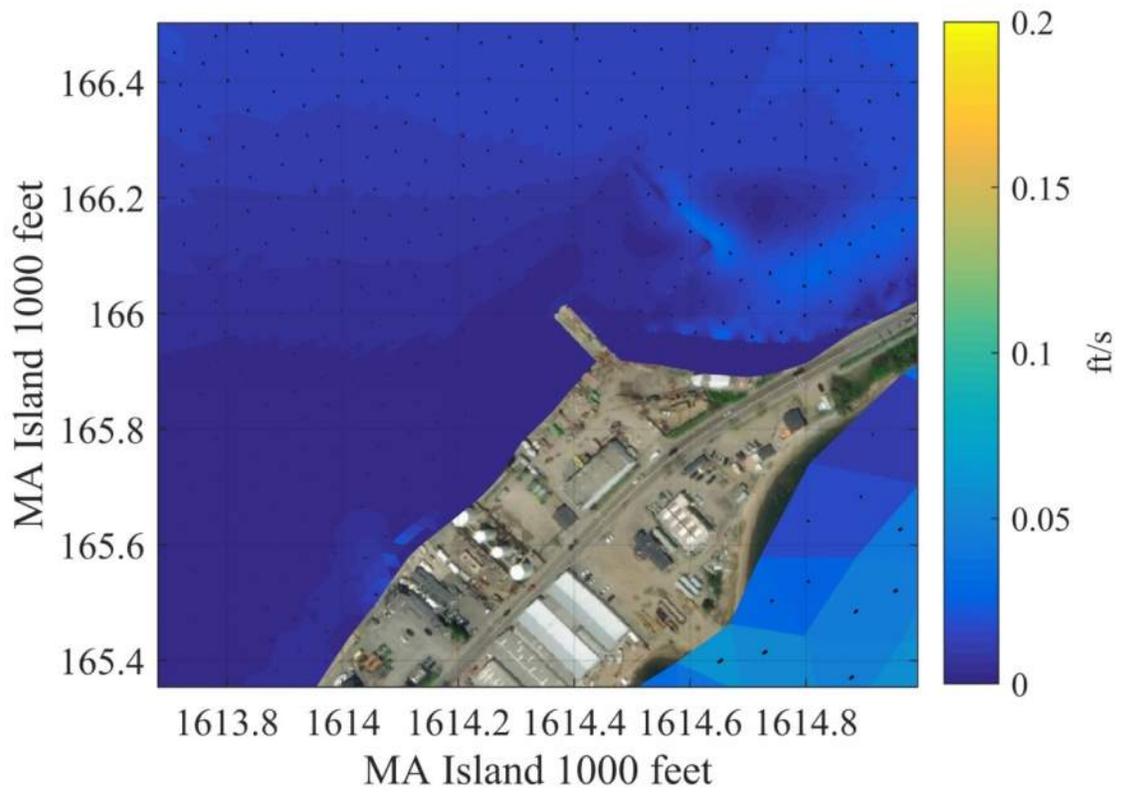


Figure 3.40 Flow velocity magnitude and direction during the May 7th, 2004 08:20:00 timestep (max ebb tide) for scenario 3 conditions.

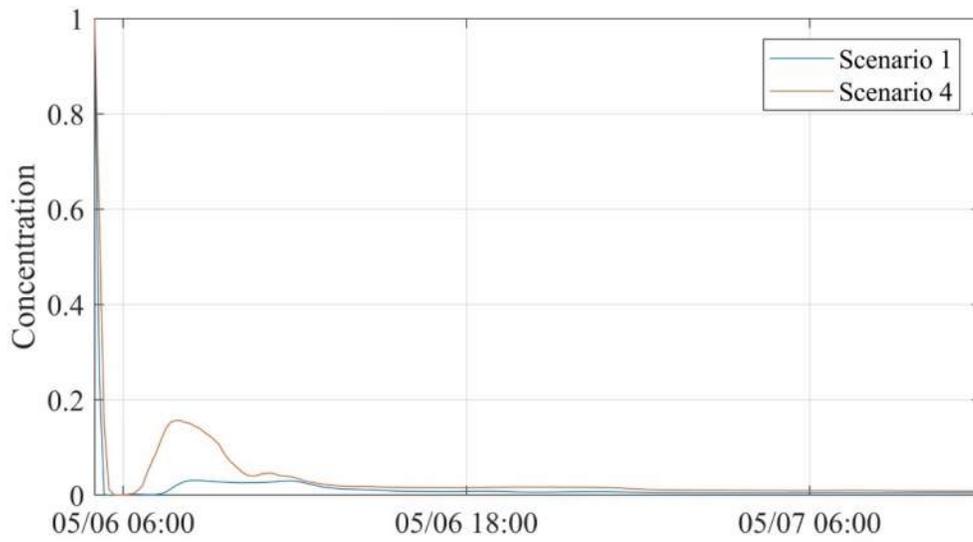


Figure 3.41 Conservative tracer time series results as observed from the included observation point A along the seaward edge of the boat basin (Figure 3.5).

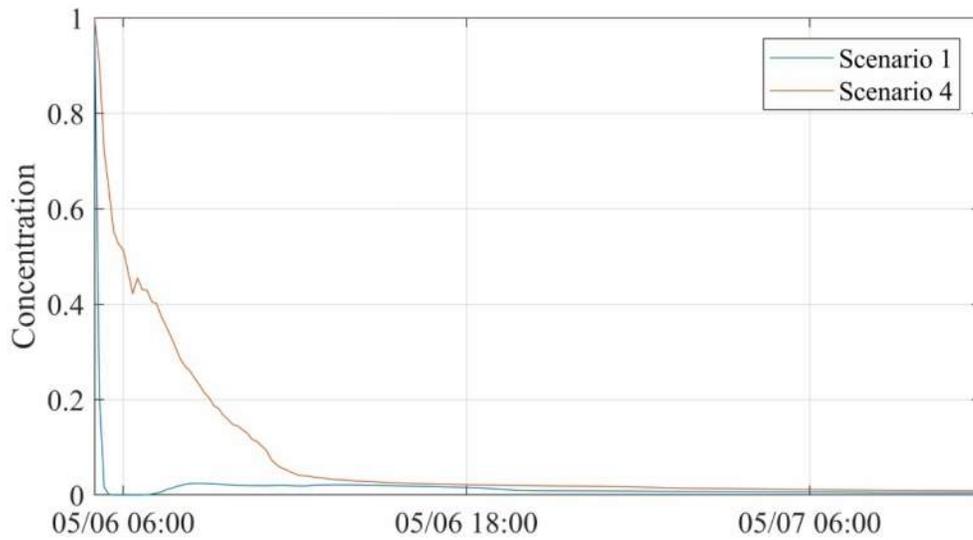


Figure 3.42 Conservative tracer time series results as observed from the included observation point B along the seaward edge of the boat basin (Figure 3.5).

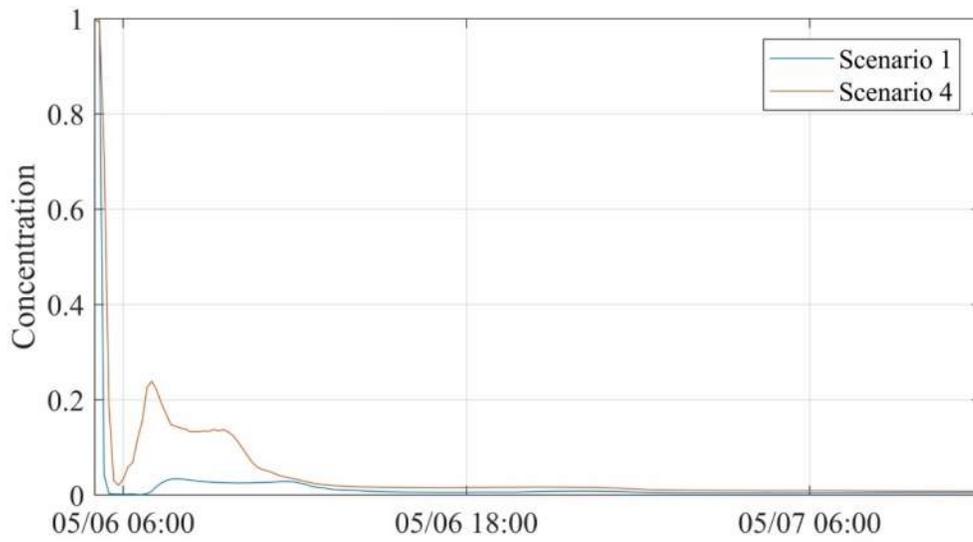


Figure 3.43 Conservative tracer time series results as observed from the included observation point C along the seaward edge of the boat basin (Figure 3.5).

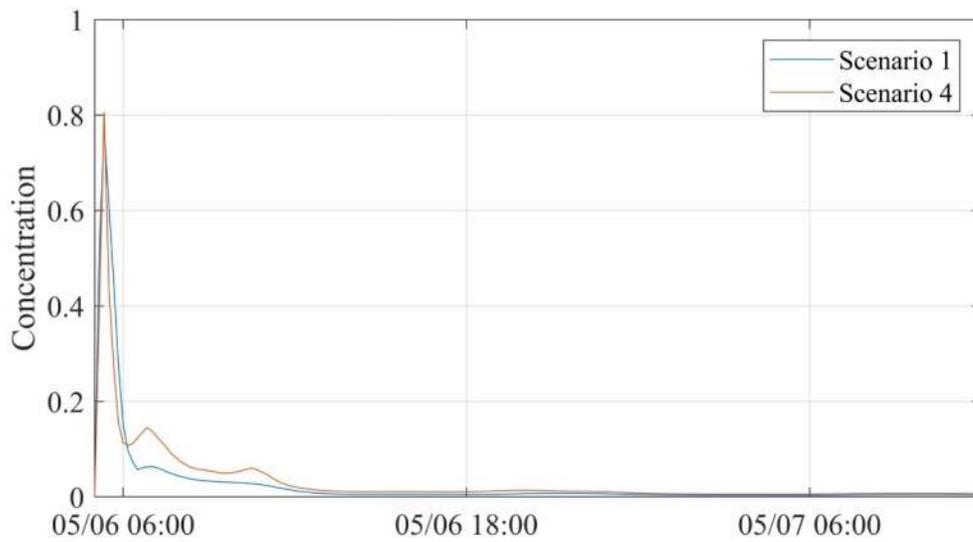


Figure 3.44 Conservative tracer time series results as observed from the included observation point D along the seaward edge of the boat basin (Figure 3.5).



Figure 3.45 Initial “salinity” value prescribed to the model with scenario 4 structures included. The red region was provided a value of 1 and the surrounding blue region contains a value of 0.

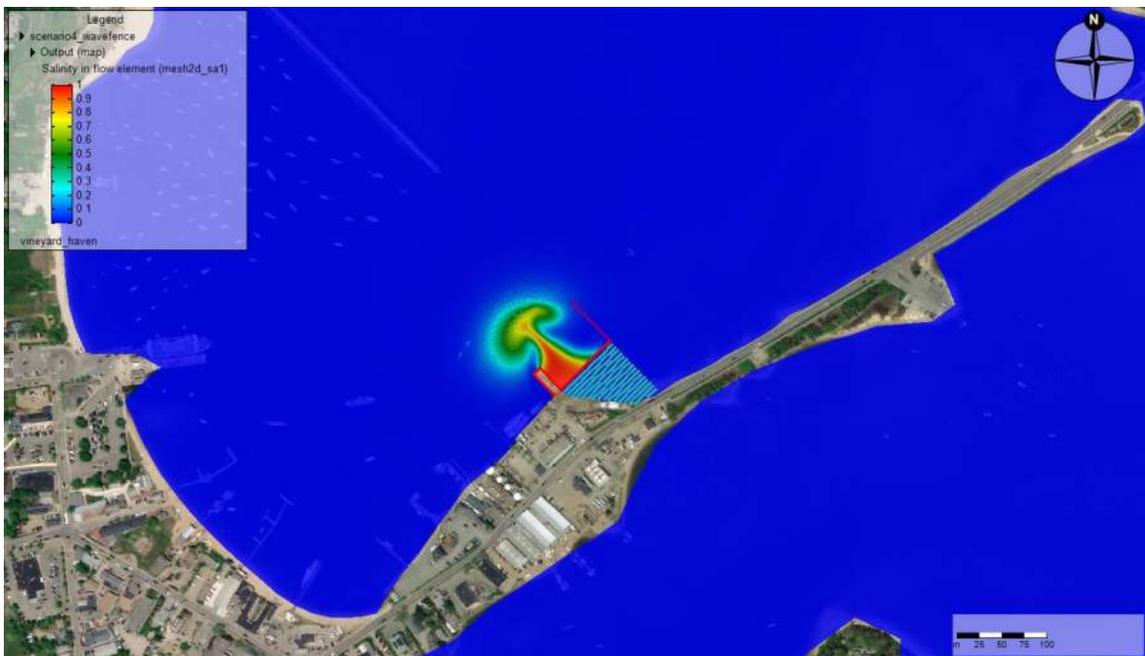


Figure 3.46 “Dye Test” results for the partial structure scenario following a 30-minute period of tidal flow.

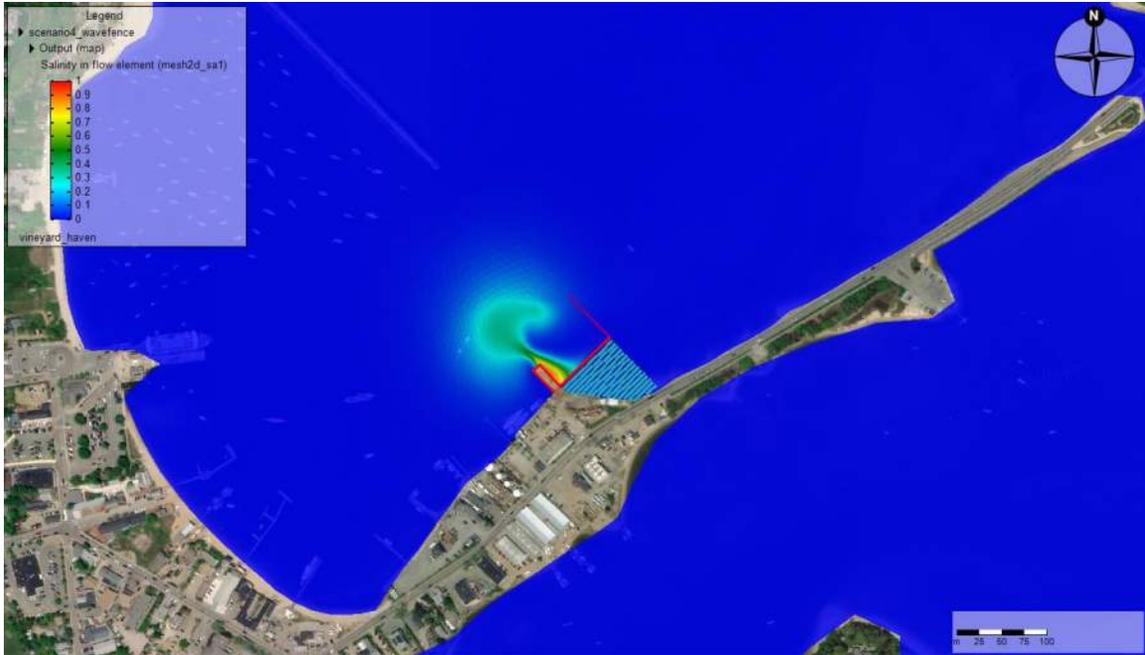


Figure 3.47 “Dye Test” results for the partial structure scenario following a 60-minute period of tidal flow. Although there is some concentration remaining within the basin, most of the “dye” has been flushed out from the lowered bulkhead section.



Figure 3.48 “Dye Test” results for the partial structure scenario following a 360-minute period of tidal flow.

In addition to changes in circulation, wave-induced sediment transport was also evaluated for Scenario 4. Although hydrodynamic related sediment transport was included

as well, the values alone were determined to be negligible based on a D-Flow model simulation without waves. Sediment transport results for Scenario 4 when compared with existing conditions for the modeled year indicates that there are small changes in sediment movement that are mostly due to wave sheltering by the new bulkhead. Figure 3.51 shows that no bathymetric change occurs within the dock facility. Southeast of the bulkhead, reduced wave heights in the area behind the bulkhead cause nearly zero change in the beach elevation within 50 feet of the wall. Beyond that point, the magnitude of change is the same as what occurs for existing conditions. This can be seen in the bathymetric change difference plot provide in Figure 3.50. In this plot, the surface of bathymetric change computed for Scenario 4 is subtracted from the surface of change computed for Scenario 1 existing conditions. It can be seen that the largest differences in this plot are southeast of the bulkhead, but these differences are small and strictly related to wave sheltering from the new structures that are a part of this scenario.

The small magnitude of changes that result from the Scenario 4 configuration is also evident in the sediment transport rates determined for the beach for existing conditions and this scenario. Figure 3.51 and Figure 3.52 show a trace of cumulative sediment flux at cross-shore transect line placed both east of the project area and alongside the bulkhead, respectively (Figure 3.10 for locations of these transects). These sediment transport rates were computed for the duration of the annual simulation. The results show that net transport for existing conditions has a very small west-directed magnitude of 3.2 cubic yards per year. With the wave sheltering provided by the array of the structural components that are a part of Scenario 4, the net transport magnitude is reduced to 0.5 cubic yards per year, and is directed now to the east. The small net transport flux of both scenarios indicates that alongshore transport is small even during active periods like the modeled year. Overall, the actual transport rates are almost negligible for both existing conditions and for the Wave Fence Modification alternative (Scenario 4)

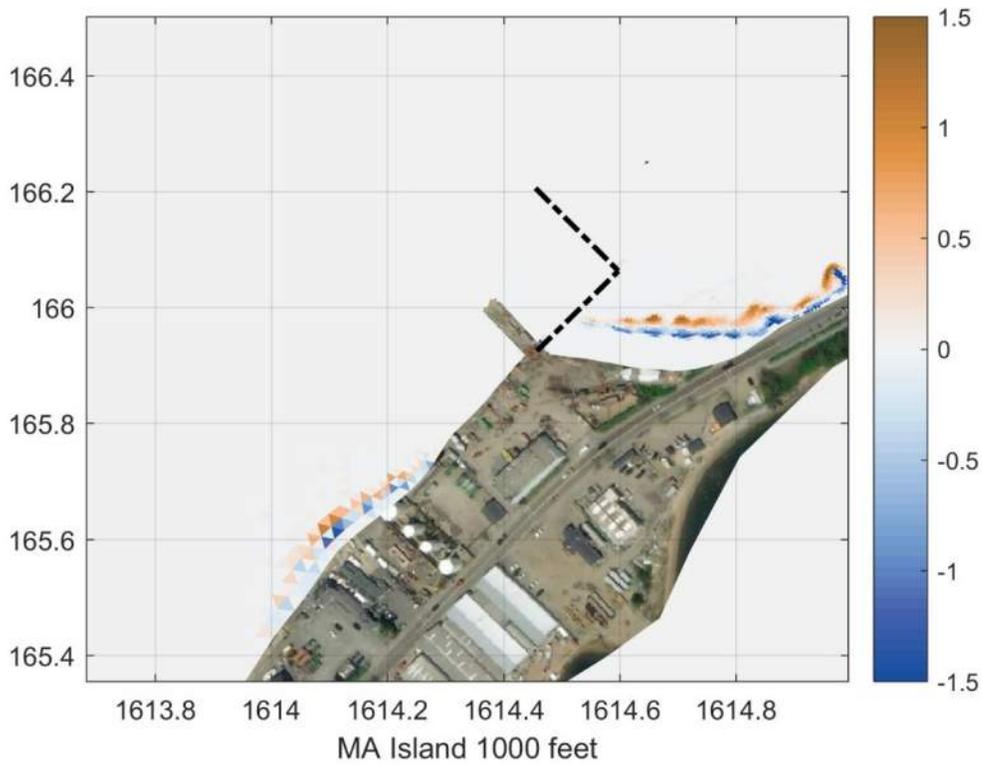


Figure 3.49 Plot of morphology change for the annual run of Scenario 4 conditions, contours in feet. Blue shades indicate erosion and brown shades indicate accretion. The location of the new wave fence and bulkhead are indicated using the black dot-dash line.

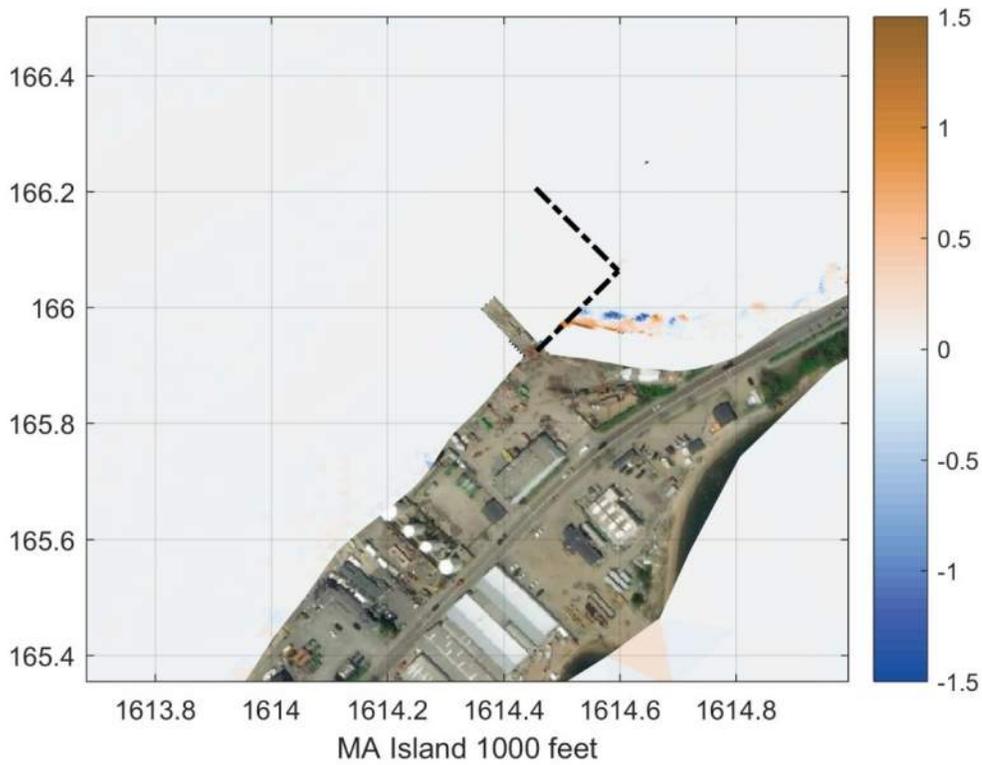


Figure 3.50 Color contour plot of difference in elevation change (in meters) between existing conditions and Scenario 4 plan. Brown shades indicate areas of less erosion in Scenario 4, while blue shades indicate areas of less accretion in Scenario 4. The location of the new wave fence and bulkhead are indicated using the black dot-dash line.

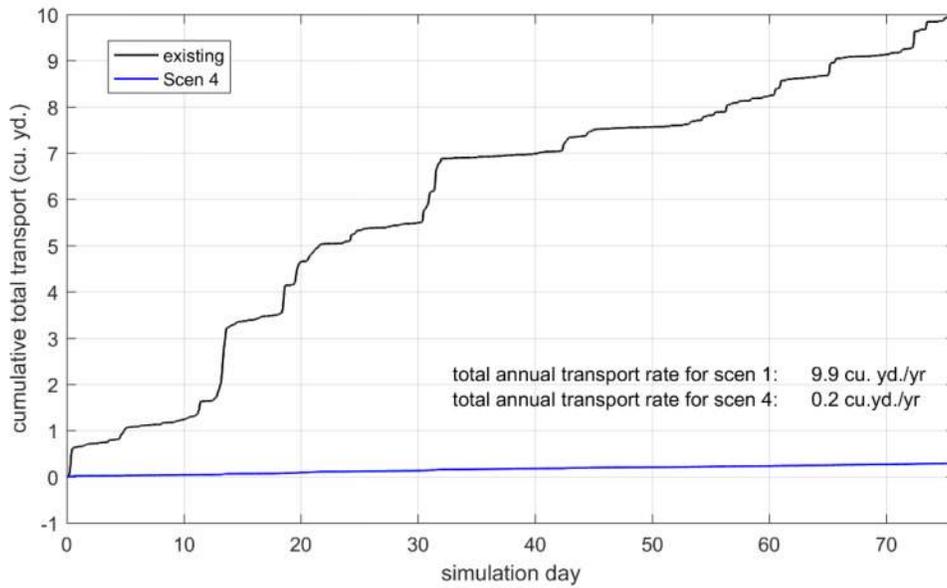


Figure 3.51 Cumulative sediment transport across the cross-shore transect line 1 indicated in Figure 3.10. Positive flux is directed to the southwest, while negative flux is directed to the northeast (toward Lagoon Pond inlet).

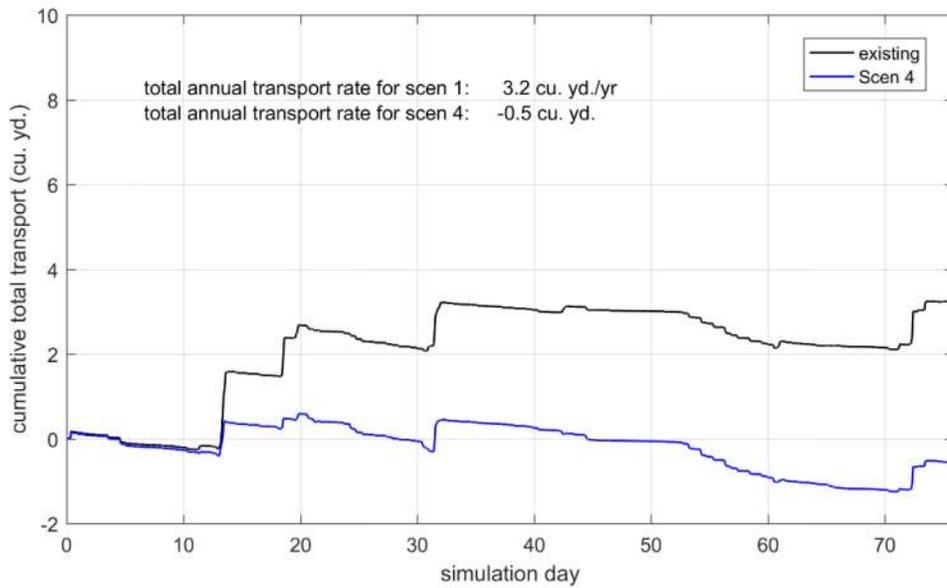


Figure 3.52 Cumulative sediment transport across the cross-shore transect line 3 indicated in Figure 3.10. Positive flux is directed towards the planned boat basin, while negative flux is directed to the northeast (toward Lagoon Pond inlet).

3.5 Scenario 5: Bulkhead Window Modification

Scenario 5 was a modified version of Scenario 3, where the bulkhead section contains a 'window' along the seaward (northeast) 80 ft section adjacent to the proposed wave fence (Figure 3.53). The location of the transect along which the bulkhead and window can be viewed is provided in Figure 3.53. The landward section of the bulkhead remains full height and serves to inhibit any beach sediment from the south side of the bulkhead from migrating into the dredged basin during severe storms. In addition, Scenario 5 would slightly reduce the influence of waves within the basin by blocking wave action along the landward (southwestern) portion of the bulkhead, while still allowing flow to circulate across the structure and into the dredged basin during all phases of the tide. The wave fence would still be designed to the seafloor so that flow would be unable to pass through, resulting in a minor reduction in tidal circulation within the basin. Finally, similar to all other scenarios evaluated, the existing solid fill pier would be reconstructed to modify the existing footprint.



Figure 3.53 Location of transect A-A' used to show a cross section observation of the bulkhead and bulkhead window in Figure 3.54.

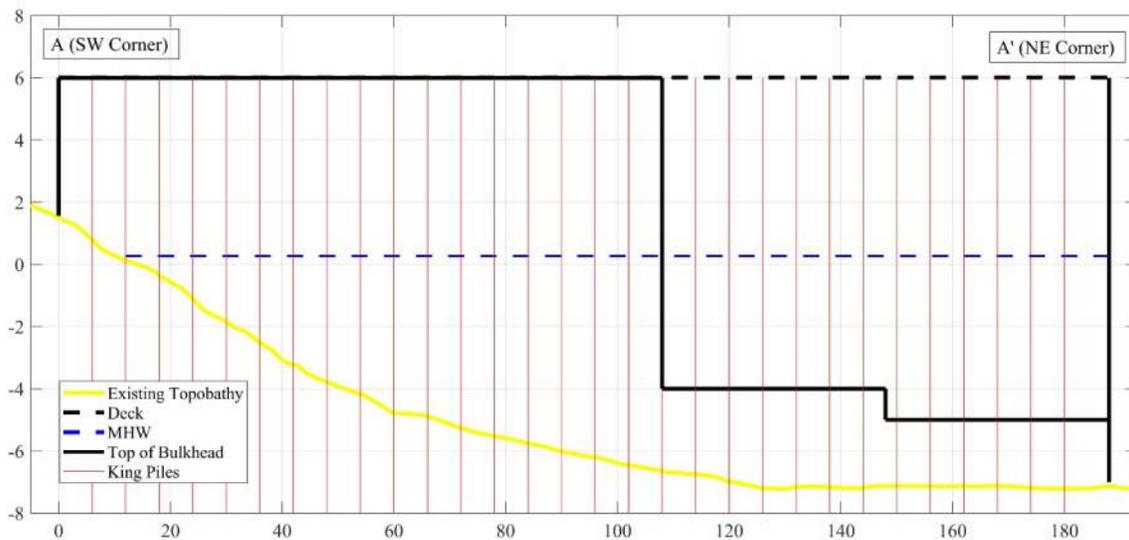


Figure 3.54 Cross sectional view of the bulkhead and window for Scenario 5.

Some noticeable differences between flow under the existing conditions (Scenario 1) and Scenario 5 are increased velocities around the northwestern end of the wave fence (Figure 3.55) during the peak flooding tide. This increase is a result of flow obstruction by the wave fence. There is slight reduction in flow compared to Scenario 3 due to the reduced bulkhead opening; however, these differences in tidal flow velocities are minimal (< 10%). During maximum ebb tide, there is not much difference between Scenario 5 and existing conditions, as flows are minimal and therefore changes in flow are minimal, as well. It is worth noting that any noticeable difference is only observed during periods of greater than average tides, and that the velocities are insignificant in terms of their ability to mobilize and transport sediment.

A review of the results of the conservative tracer study indicate there is a significant improvement in circulation when compared to Scenario 2. Due to the 80-foot window in the bulkhead, flow is able to pass unimpeded along the shoreline and into the dredged basin. Comparison of the conservative tracer concentration at the four (4) locations depicted in Figure 3.5 within the region of the boat basin are shown in Figure 3.57 through 3.58, respectively. When compared to Scenario 3, there are minimal differences in dispersion of the conservative tracer, as the 80-foot window does not obstruct the flow significantly in comparison to the bulkhead that is 2 feet above the seafloor for its full length (Scenario 3). Visually, this slightly inhibited dispersion is illustrated in Figures 3.59 through 3.62, for times 0, 60 minutes, 120 minutes, and 360 minutes, respectively, for Scenarios 3 and 5 relative to existing conditions (Scenario 1). Section 4 includes a further discussion of the tracer study between scenarios.

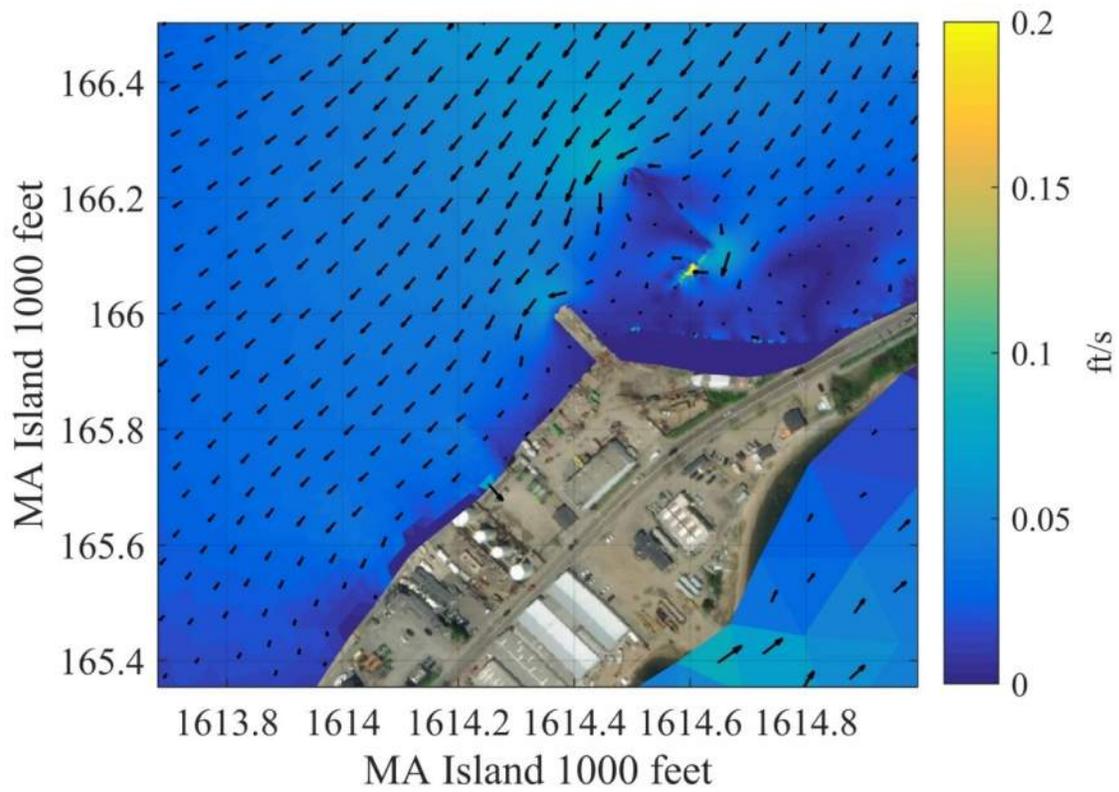


Figure 3.55 Flow velocity magnitude and direction during the May 7th, 2004 14:00:00 timestep (max flood tide) for scenario 5 conditions.

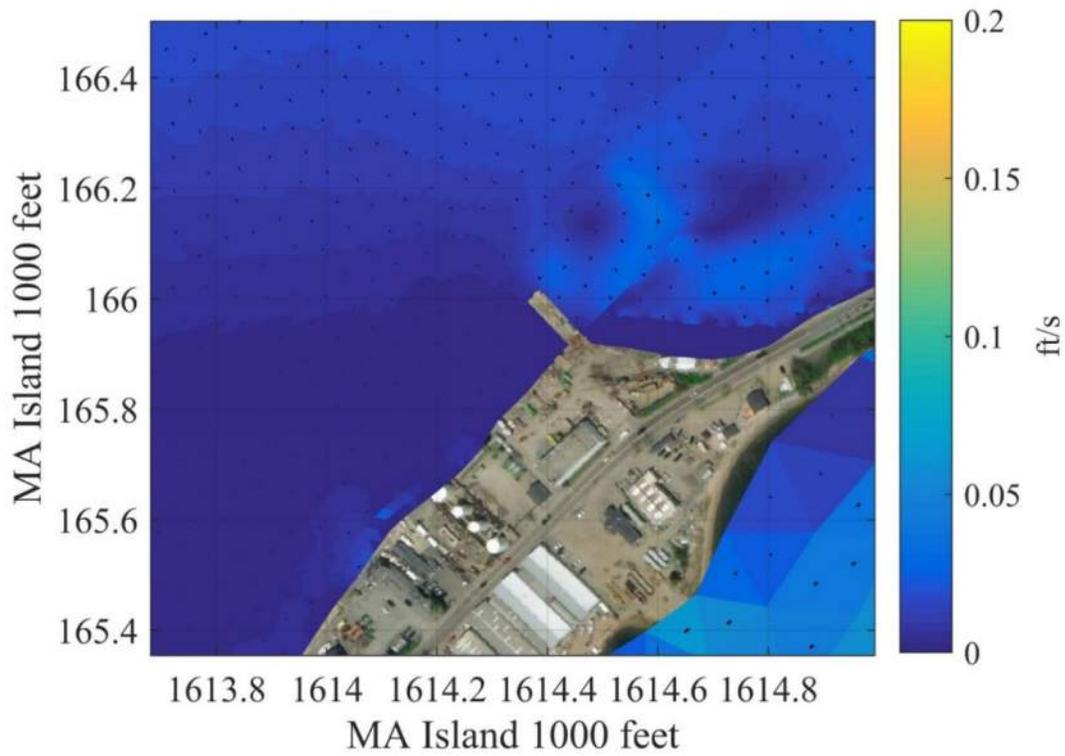


Figure 3.56 Flow velocity magnitude and direction during the May 7th, 2004 08:20:00 timestep (max ebb tide) for scenario 5 conditions.

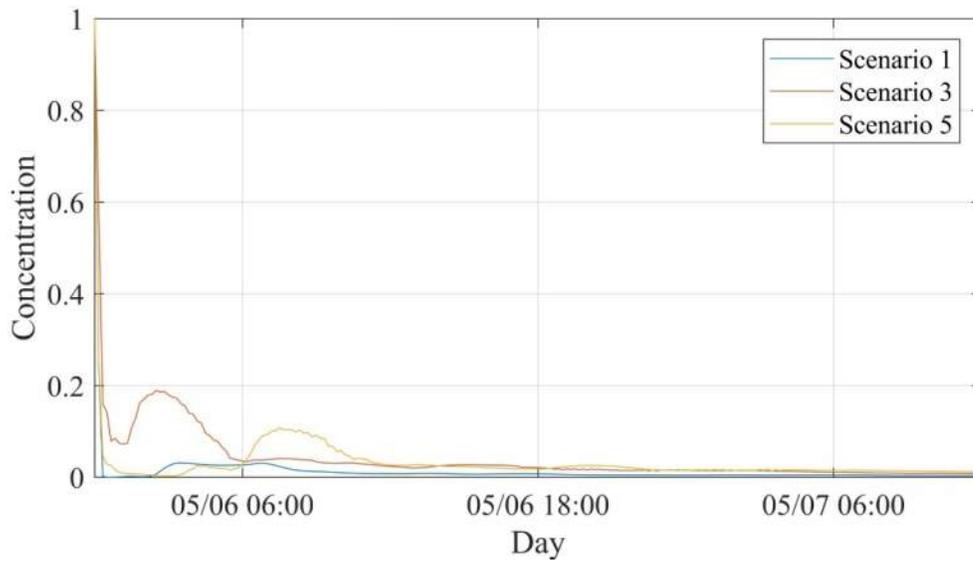


Figure 3.57 Conservative tracer time series results as observed from the included observation point A along the seaward edge of the boat basin comparing Scenario 1, 3 and 5 (Figure 3.5).

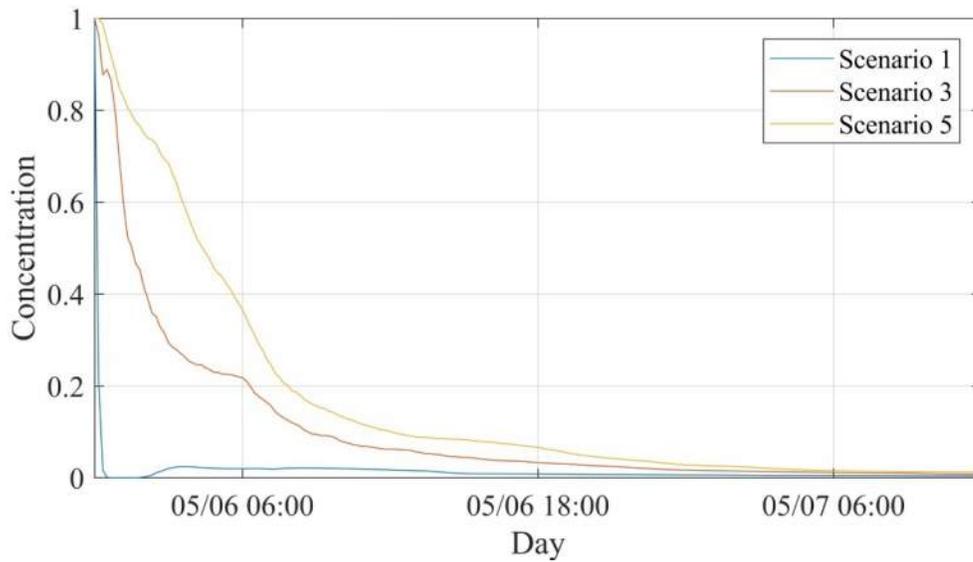


Figure 3.58 Conservative tracer time series results as observed from the included observation point B along the seaward edge of the boat basin comparing Scenario 1, 3 and 5 (Figure 3.5).

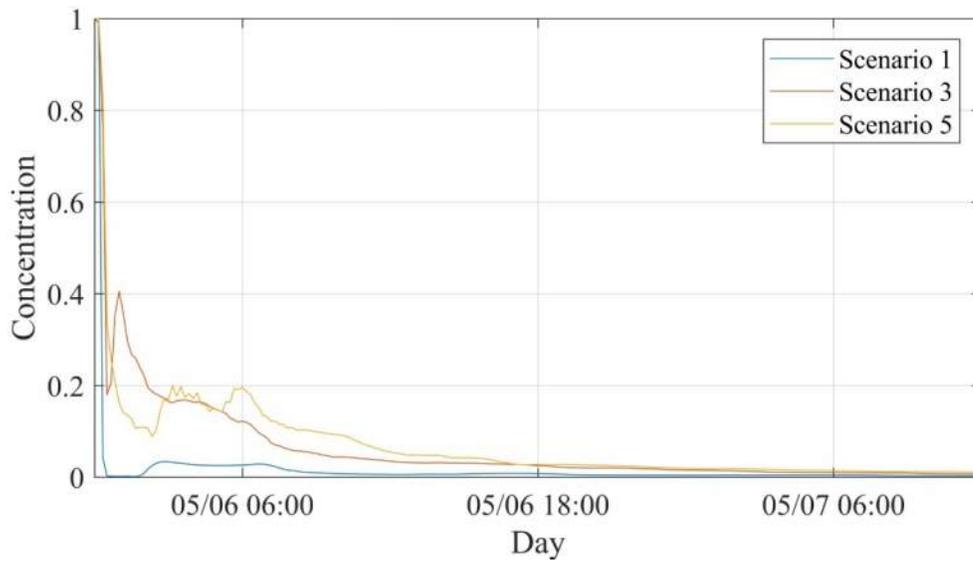


Figure 3.59 Conservative tracer time series results as observed from the included observation point C along the seaward edge of the boat basin comparing Scenario 1, 3 and 5 (Figure 3.5).

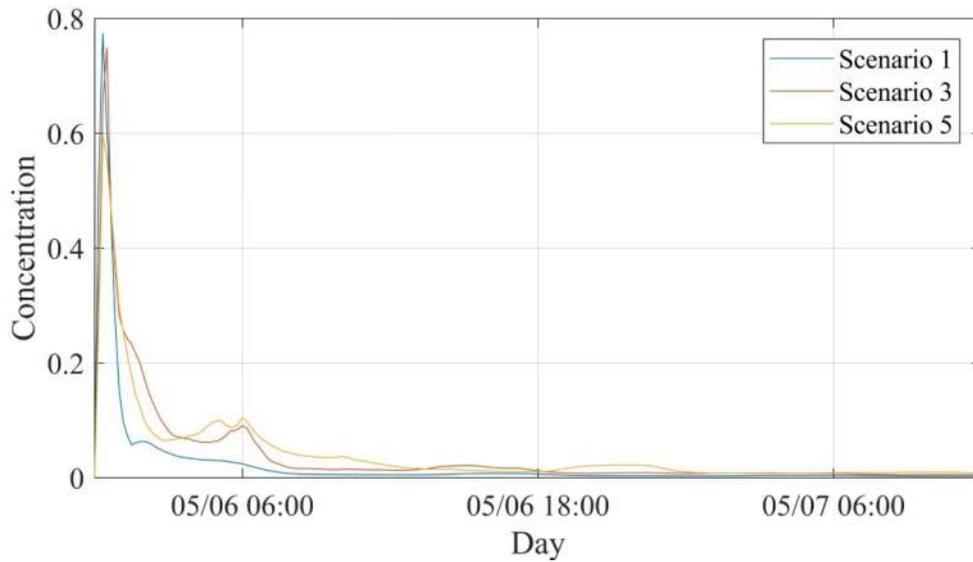


Figure 3.60 Conservative tracer time series results as observed from the included observation point D along the seaward edge of the boat basin comparing Scenario 1, 3 and 5 (Figure 3.5).



Figure 3.61 Initial “salinity” value prescribed to the model with scenario 5 structures included. The red region was provided a value of 1 and the surrounding blue region contains a value of 0.

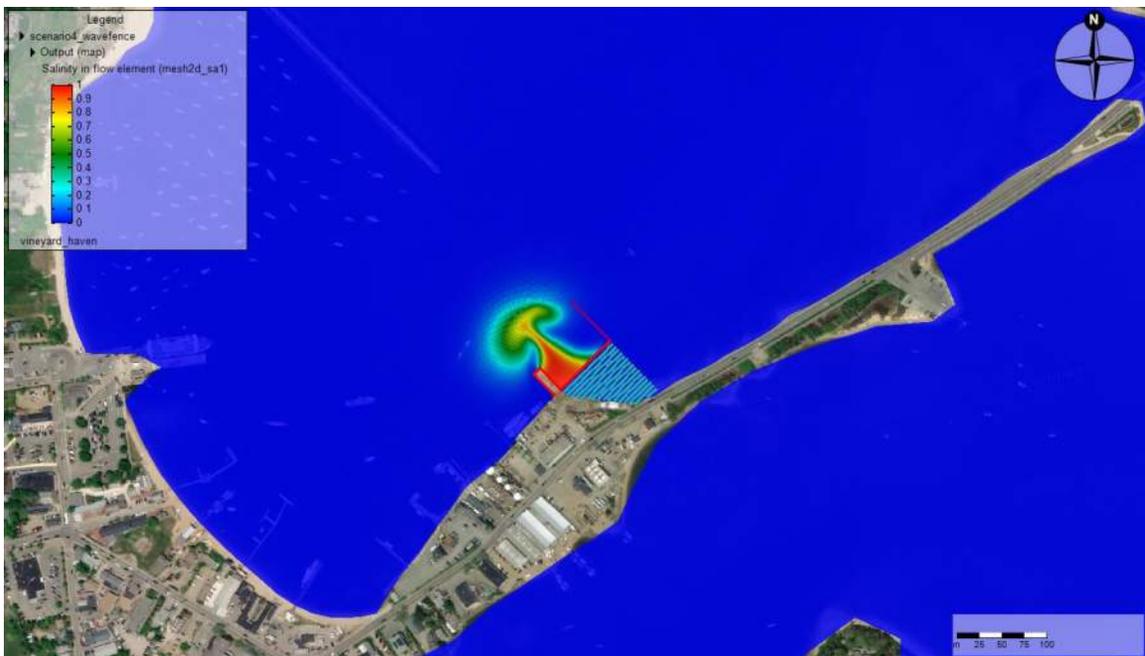


Figure 3.62 “Dye Test” results for the partial structure scenario following a 30-minute period of tidal flow.

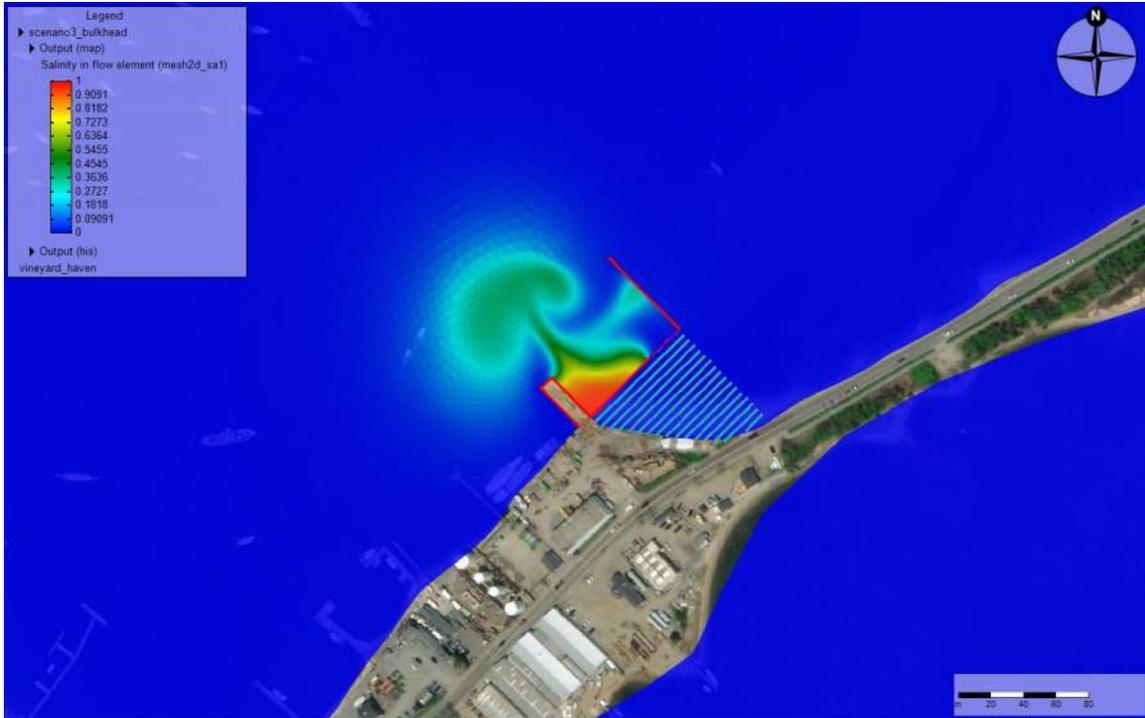


Figure 3.63 “Dye Test” results for the partial structure scenario following a 60-minute period of tidal flow. Although there is some concentration remaining within the basin, most of the “dye” has been flushed out from the lowered bulkhead section.

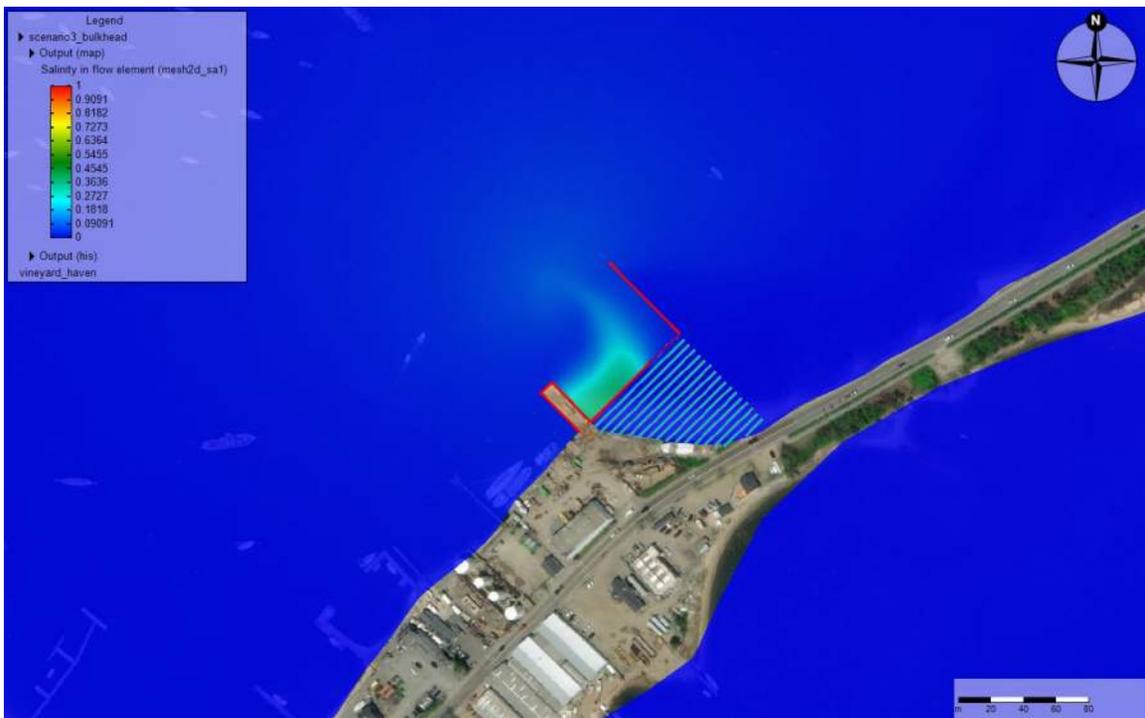


Figure 3.64 “Dye Test” results for the partial structure scenario following a 360-minute period of tidal flow.

In addition to changes in circulation, wave-induced sediment transport was also evaluated for Scenario 5. Although hydrodynamic related sediment transport was included as well, the values alone were determined to be negligible based on a D-Flow model simulation without waves. Sediment transport results for Scenario 5 when compared with existing conditions for the modeled year indicates that there are small changes in sediment movement that are mostly due to wave sheltering by the proposed bulkhead and wave fence. Figure 3.63 shows that no bathymetric change occurs within the dock facility. Southeast of the bulkhead, reduced wave heights in the area behind the bulkhead cause nearly zero change in the beach elevation within 50 feet of the bulkhead. Beyond that point, the magnitude of change is the same as what occurs for existing conditions. This can be seen in the bathymetric change difference plot provide in 3.63. In this plot, the surface of bathymetric change computed for Scenario 5 is subtracted from the surface of change computed for Scenario 1 existing conditions. It can be seen that the largest differences in this plot are southeast of the bulkhead, but these differences are small and strictly related to wave sheltering from the new structures that are a part of this scenario.

The small magnitude of changes that result from the Scenario 5 configuration is also evident in the sediment transport rates determined for the beach for existing conditions and this scenario. Figure 3.64 and figure 3.65 show a trace of cumulative sediment flux at cross-shore transect line placed both east of the project area and alongside the bulkhead, respectively (Figure 3.10 for locations of these transects). These sediment transport rates were computed for the duration of the annual simulation. The results show that net transport for existing conditions has a very small west-directed magnitude of 3.2 cubic yards per year. With the wave sheltering provided by the array of the structural components that are a part of Scenario 5, the net transport magnitude is reduced to 0.5 cubic yards per year, and is directed now to the east. The small net transport flux of both scenarios indicates that alongshore transport is small even during active periods like the modeled year. Overall, the actual transport rates are almost negligible for both existing conditions and for the Wave Fence Modification alternative (Scenario 5). There were no observed differences in transport between Scenario 3 and Scenario 5.

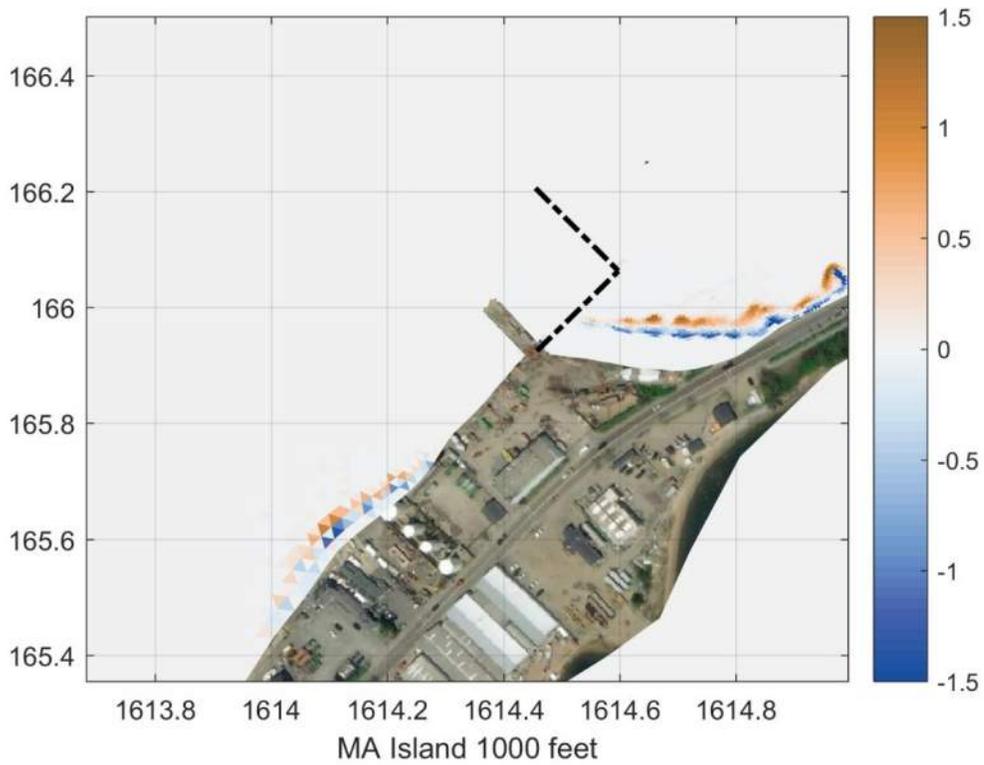


Figure 3.65 Plot of morphology change for the annual run of Scenario 5 conditions, contours in feet. Blue shades indicate erosion and brown shades indicate accretion. The location of the new wave fence and bulkhead are indicated using the black dot-dash line.

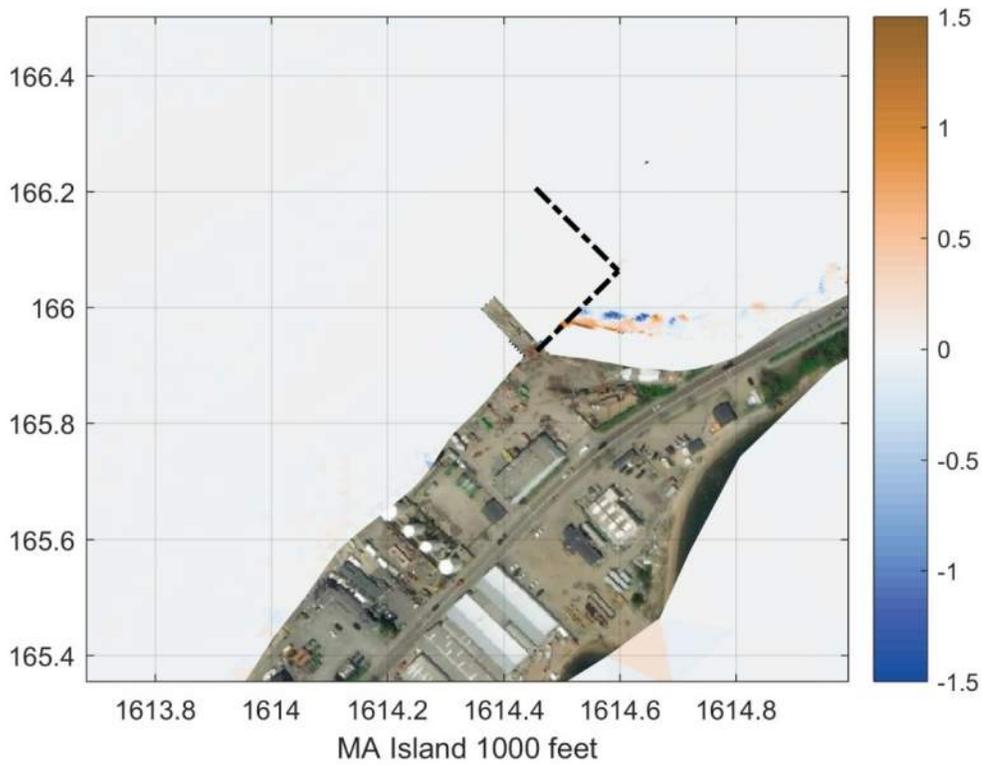


Figure 3.66 Color contour plot of difference in elevation change (in meters) between existing conditions and Scenario 5 plan. Brown shades indicate areas of less erosion in Scenario 5, while blue shades indicate areas of less accretion in Scenario 5. The location of the new wave fence and bulkhead are indicated using the black dot-dash line.

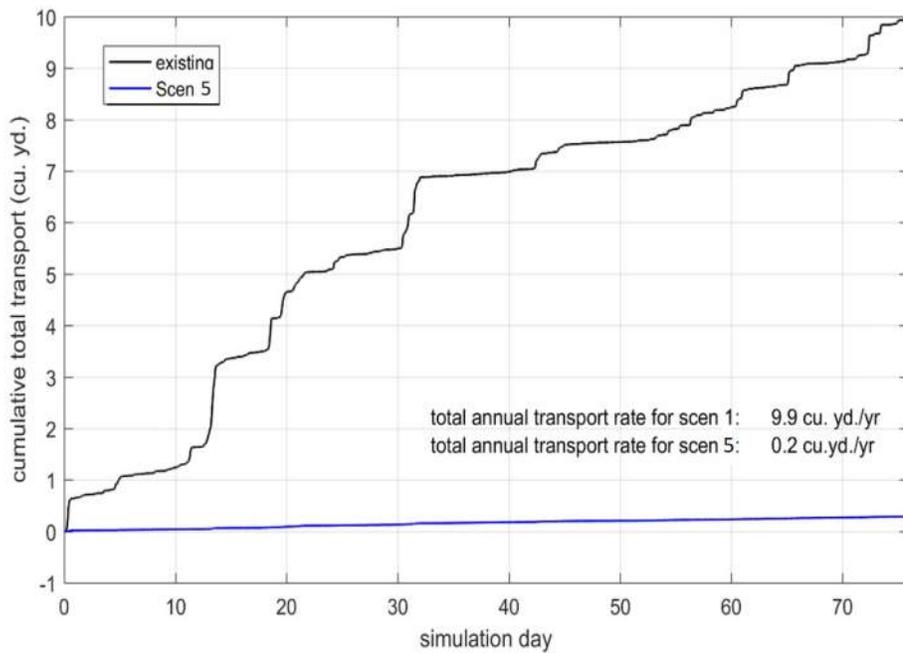


Figure 3.67 Cumulative sediment transport across the cross-shore transect line 1 indicated in Figure 3.10. Positive flux is directed to the southwest, while negative flux is directed to the northeast (toward Lagoon Pond inlet).

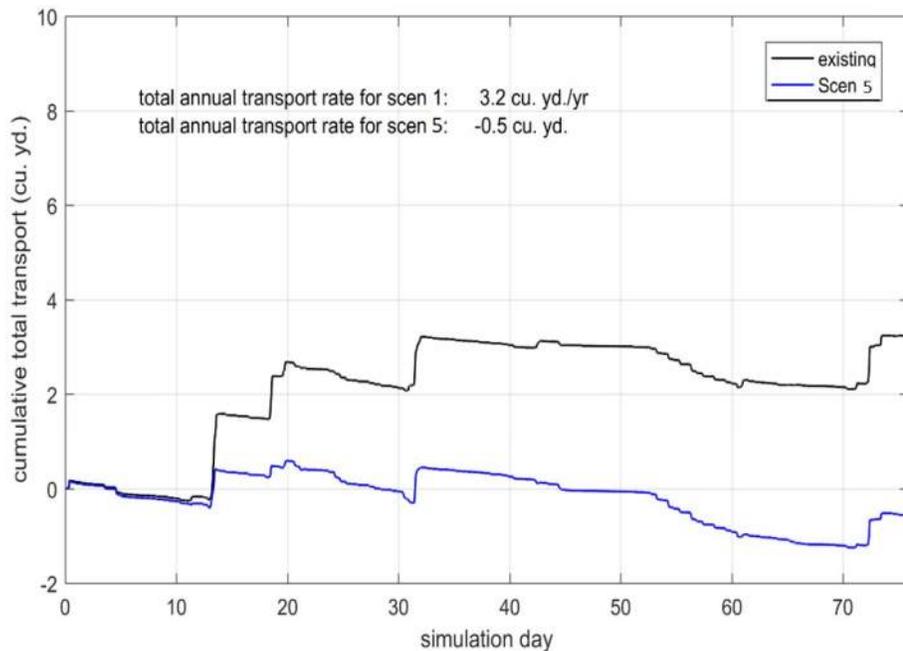


Figure 3.68 Cumulative sediment transport across the cross-shore transect line 3 indicated in Figure 3.10. Positive flux is directed towards the planned boat basin, while negative flux is directed to the northeast (toward Lagoon Pond inlet).

4.0 DISCUSSION OF MODEL RESULTS AND RECOMMENDATIONS

As described above, Applied Coastal performed an in-depth numerical analysis of tidal hydrodynamics, waves, and sediment transport for the Vineyard Haven Harbor area, with a focused effort on the Packer Facility. Several engineering alternatives were considered to meet the requirements for expanding the Packer Facility to the east for the purpose of providing additional slips for commercial vessels. To provide needed wave protection, a series of coastal engineering structures were evaluated relative to potential alterations to both sediment transport patterns and tidal circulation. Specifically, the evaluation of alternatives provided quantification of any changes to coastal processes associated with engineering alterations.

4.1 Alternatives Evaluated

In addition to providing information regarding existing conditions, the modeling tools also provided the ability to evaluate various engineering alternatives. For the Packer Facility evaluation, the model grid was modified to include the bulkhead, solid fill pier, pile-supported platform, and wave fence depicted on the conceptual project plans provided by Foth and shown in Figure 1.1. Specifically, the following alternatives were evaluated:

1. *Scenario 1*: Existing conditions
2. *Scenario 2*: Conceptual design as presented in the Proposed Site Plan dated November 26, 2019 that extends the existing Packer Facility basin to the east at a facility depth of -18.4 feet NAVD. This proposed design includes the following four (4) elements:
 - a. Solid fill pier reconfigured with a slightly reduced footprint
 - b. New bulkhead extending ~190 feet to the east to from a contiguous shoreline with the existing Packer Marine Facility bulkhead. The elevation of this bulkhead is +6 feet NAVD.
 - c. Incorporation of a solid wave fence extending from the eastern end of the new bulkhead for a distance of ~212 feet to the north
 - d. Incorporation of a pile-supported platform south of the new bulkhead, where the 1-ft diameter wood piles are spaced on a 15-ft grid. The landward edge of the platform will consist of a wall feature that is landward of the active sediment transport zone; therefore, there is no interaction of this wall with wave action.
3. *Scenario 3*: Implementation of *Scenario 2*, with a revised bulkhead elevation that lowers the top height of the bulkhead to an elevation that is 2 feet above the existing grade to the adjacent area south of the bulkhead. This potentially could improve tidal circulation by allowing tidal flow to circulate over the structure.
4. *Scenario 4*: Implementation of *Scenario 3*, with the wave fence designed in a manner that allows a gap along the seafloor to potentially improve tidal circulation. The bottom of the wave fence for this simulation was 3 feet above the seafloor, with a transition from -18.4 feet NAVD to the existing grade at a 1:3 (v:h) slope from the Wave Fence extending in an easterly direction.
5. *Scenario 5*: Implementation of *Scenario 2*, with a revised bulkhead elevation that lowers the top height of the bulkhead to an elevation of -5 feet NAVD for a distance of 40 feet southwest of the wave fence end and -4 feet NAVD for an additional 40

feet southwest of that (see Figure 3.54 for a detailed cross-section). This bulkhead lowering creates an 80 foot “window” opening of the bulkhead to allow flow to pass. This provides unimpeded tidal circulation by allowing tidal flow over the structure during all phases of the tide, while still reducing infilling of the boat basin by preventing beach material from migrating into the dredged basin.

For each of the alternative configurations above, the modeling was conducted for the same input parameters utilized for the existing conditions run (i.e. simulation of annual transport rates and typical tidal circulation conditions). A comparison of modeling results for the various scenarios are describe in the following section.

4.1.1 Circulation

As described in Section 3 of this report, tidal circulation (i.e. the strength of tidal currents) in the project area is relatively weak, due to the microtidal conditions within the harbor system, where the maximum astronomical tides range is about 3 feet. Maximum modeled currents under existing conditions in the vicinity of the project area are around 8 centimeters per second or 0.25 feet per second, which are not of sufficient magnitude to mobilize *in situ* sediments. Therefore, local tidal currents alone are not able to transport sediment. Additionally, numerical modeling of tidal hydrodynamics indicated that none of the alternatives evaluated generate tidal currents sufficient to mobilize sediment at the facility. Therefore, it is only the combination of waves and tidal currents together that can mobilize sediment, and the evaluation of sediment transport was restricted to the evaluation of these combined effects.

Changes to circulation patterns in the system were also evaluated by numerical simulation of a conservative tracer. Delft3D provides a ‘salinity’ variable within the flow model, in which a conservative tracer value can be prescribed spatially throughout the model domain. A value of 1 was prescribed to the area of the proposed boat basin expansion (see red area in Figure 4.1) and a value of 0 was included throughout the rest of the model domain (blue region).

Details regarding dispersion of the conservative tracer for each scenario are described in Section 3. A comparison of tracer concentrations for all five (5) scenarios the four (4) locations (A, B, C, and D in Figure 4.1) in the region of the proposed dredged basin are shown in Figures 4.2, 4.3, 4.4, and 4.5, respectively. In general, near complete dispersion of the tracer occurs within 1-to-4 hours at all four locations for all scenarios, except Scenario 2 with the full-height bulkhead. For Scenario 2, the combination of the full-height bulkhead and wave fence creates an area where circulation is inhibited (see Figures 4.2 and 4.3 for observation points A, and B, respectively). For the other scenarios, tidal circulation remained similar to existing conditions, indicating negligible impacts to tidal circulation for both Scenarios 3, 4, and 5 relative to existing conditions.



Figure 4.1 Initial 'salinity' value prescribed to the model. The red region was provided a value of 1 and the surrounding blue region contains a value of 0. The four observation points, A - D, were used to monitor the dispersion of the initial concentration.

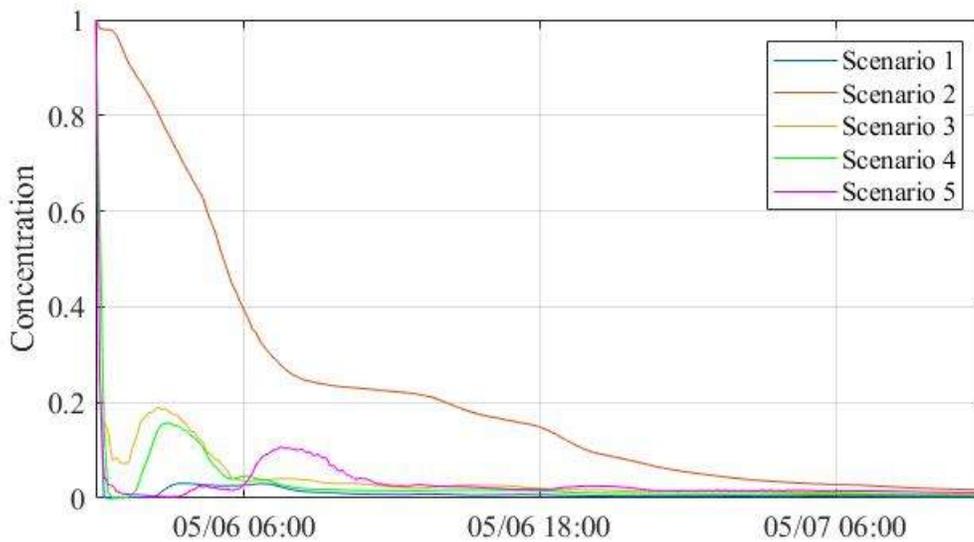


Figure 4.2 Tidal conservative tracer concentrations over time at observation point A (Figure 4.1) for each of the four (5) scenarios.

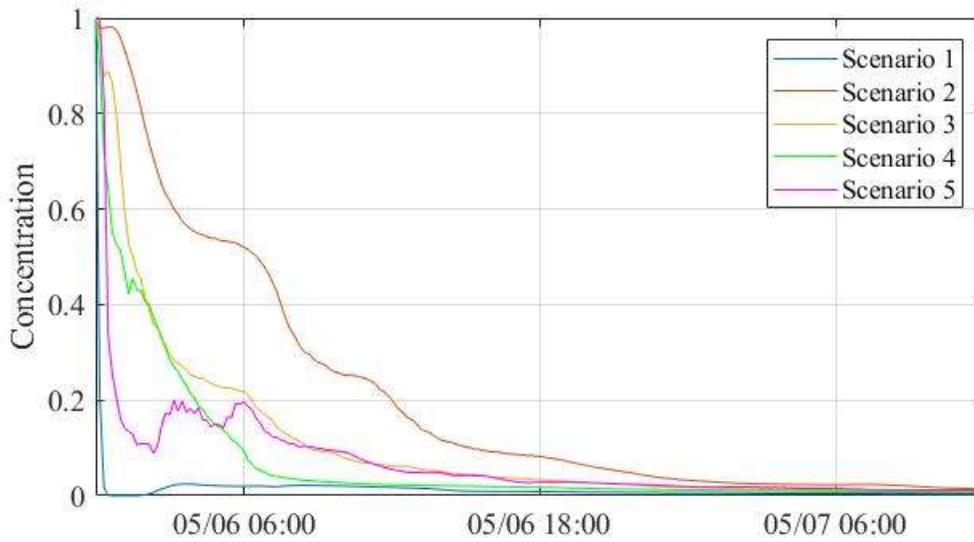


Figure 4.3 Tidal conservative tracer concentrations over time at observation point B (Figure 4.1) for each of the four (5) scenarios.

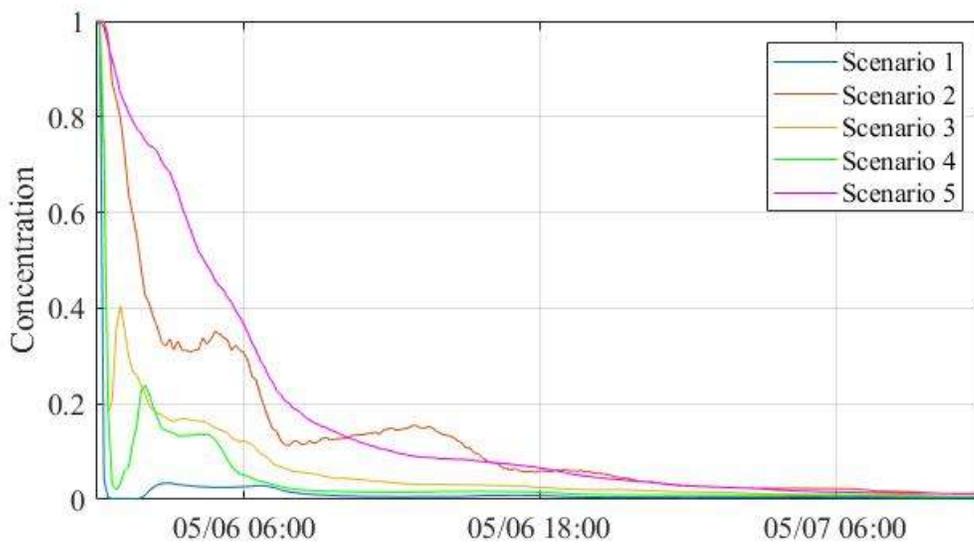


Figure 4.4 Tidal conservative tracer concentrations over time at observation point C (Figure 4.1) for each of the four (5) scenarios.

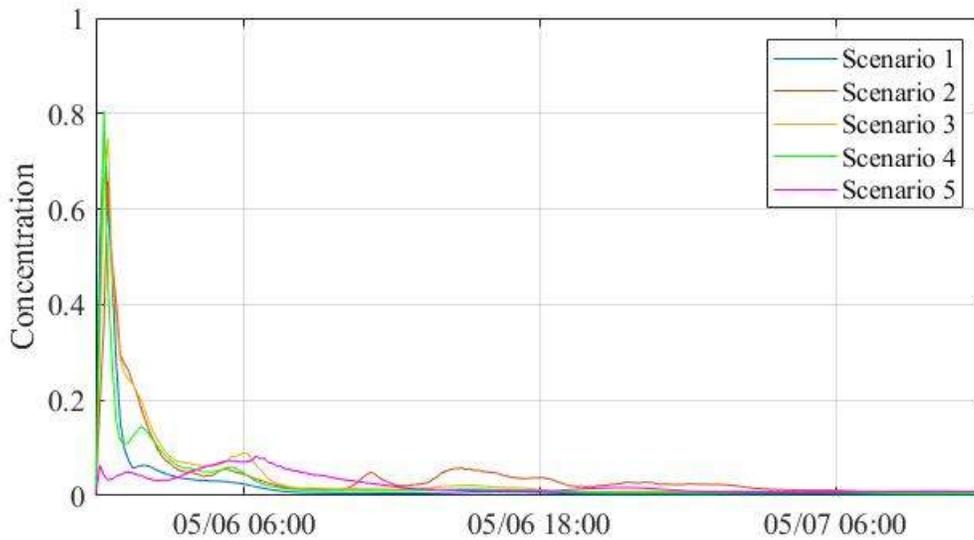


Figure 4.5 Tidal conservative tracer concentrations over time at observation point D (Figure 4.1) for each of the four (5) scenarios.

4.1.2 Wave Induced Sediment Transport

The high-resolution wave and tidal conditions were simulated within Vineyard Haven Harbor as the combined driving forces responsible for sediment transport, as tidal forces alone were not sufficient to mobilize *in situ* sediments. Due to the weak tidal currents and relatively mild wave climate within the confines of Vineyard Haven Harbor, sediment transport is limited to the shallow beach areas of the harbor. For all scenarios, the dominant area of sediment movement was along the beach face and shallow nearshore areas at the eastern extent of the existing Packer Facility. Sediment movement elsewhere in the project region was negligible.

In addition to morphology change, the sediment transport model also computed sediment flux (i.e. net sediment transport rates) along the beach. For this analysis, sediment flux was computed across three (3) shore-perpendicular cross-section along the beach, as shown in Figure 3.10. Sediment transport across each of these lines was computed for existing conditions and serves as the baseline for comparison of the various scenarios with different structural alternatives. It should be noted that the mild wave conditions in the harbor prevent significant volumes of sediment transport along the beach. In this case, the annual transport rates are generally below 10 cubic yards per year, which is several orders of magnitude lower than beaches that are exposed to either Nantucket Sound or open Atlantic Ocean waves.

Figures 4.6, 4.7, and 4.8 illustrate the annual sediment flux across Cross-Sections 1, 2, and 3, respectively. Positive sediment transport flux is to the west and north along the beach. At Cross-Section 1 (Figure 4.6), transport along the existing beach is slightly over 10 cubic yards per year (i.e. about one dump truck load). As shown, over the simulation period that represents one-year of transport, movement of sediment along the beach is dominated by short periods of increased transport rates associated with storm wave activity. During relatively quiescent wave conditions, sediment flux approaches zero. Due to direct wave sheltering provided by the wave fence and bulkhead, sediment

transport flux immediately south of the bulkhead is near zero for Scenarios 2, 3, 4, and 5 (Figure 4.6).

For Cross-Sections 2 (Figure 4.7) and 3 (Figure 4.8), sediment transport flux is altered slightly for each of the three scenarios that represent structural improvements to the Packer Facility. These alterations to sediment flux are insignificant due to the very low transport rates along the beach, where the flux changes from about +5 cubic yards per year for existing to a maximum of -3 cubic yards per year for Scenario 4 at Cross-Section 2. The flux changes from about +3 cubic yards per year for existing conditions to a maximum of -1.5 cubic yards per year for Scenario 3 and 5 at Cross-Section 3. In general, the rate of sediment transport along the beach is minimal for all scenarios; therefore, alterations to the beach associated with the four (4) scenarios that included alternatives are negligible relative to existing conditions. The slight reversals in sediment transport flux at Cross-Sections 2 and 3 caused by the structural enhancements would actually create a wider beach along the shoreline area sheltered by the combined bulkhead and wave fence.

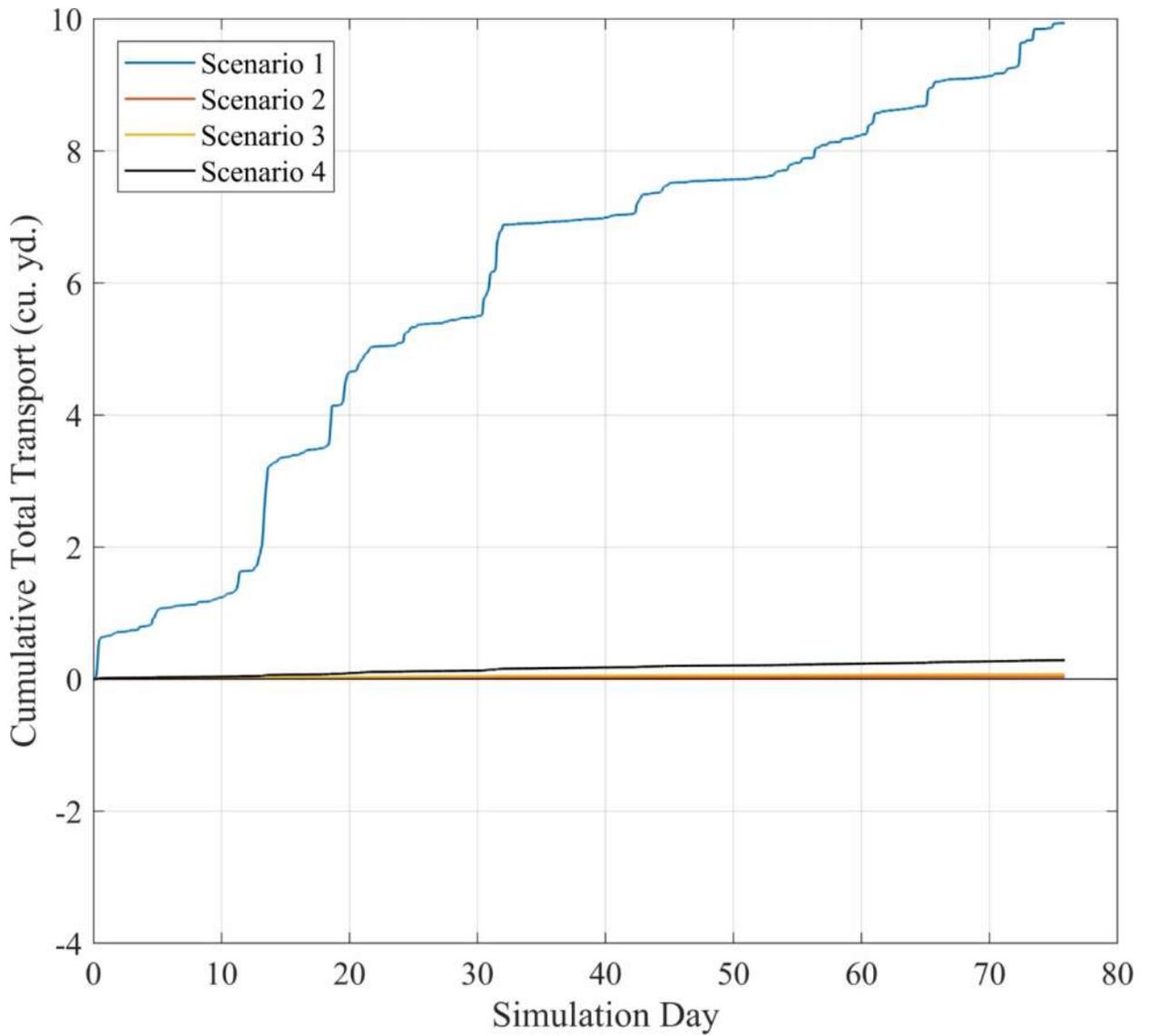


Figure 4.6 Sediment transport flux across Cross-Sections 1 (Figure 3.10) for Scenarios 1 through 4. There was no difference in sediment transport between Scenario 3 and 5.

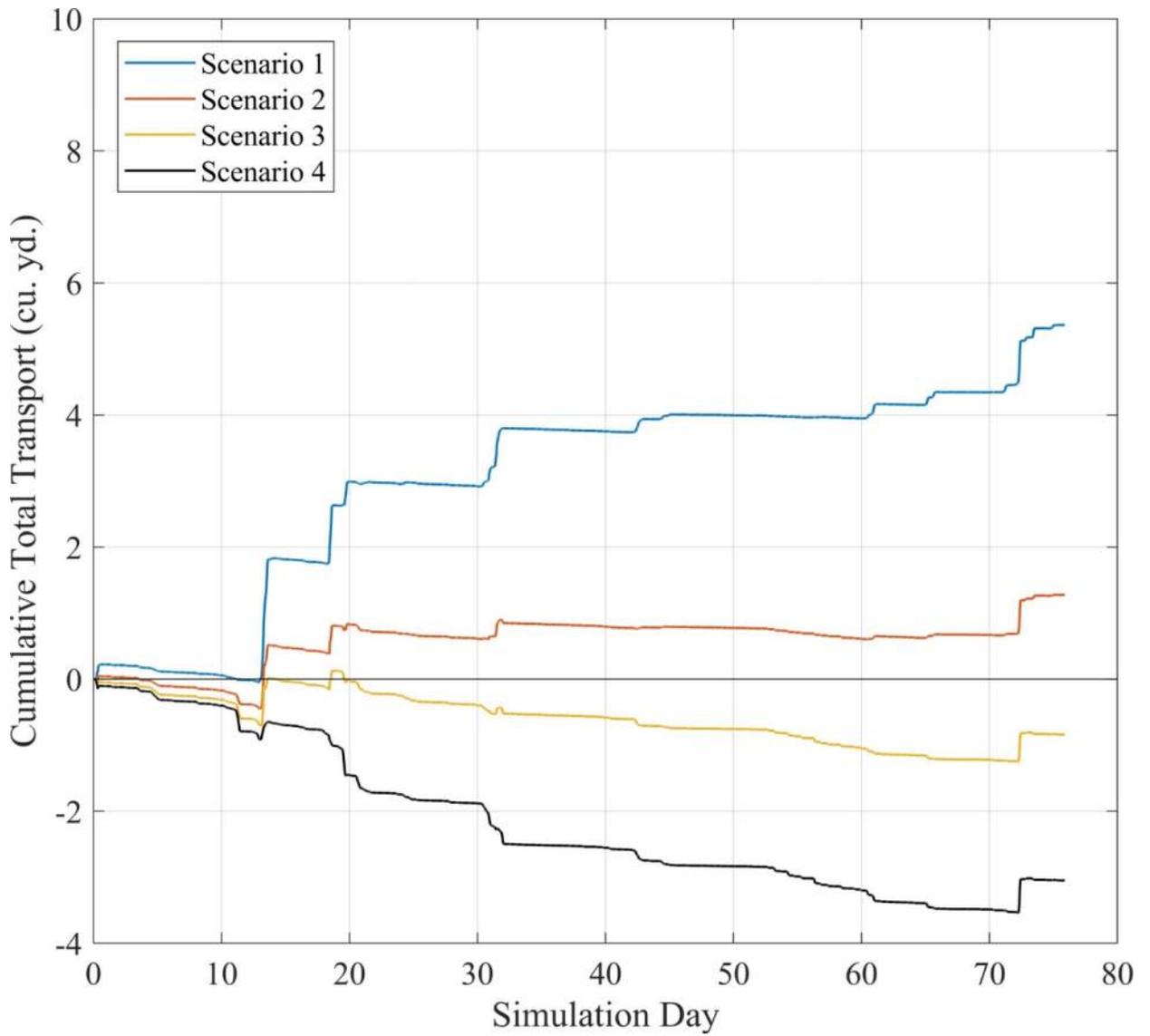


Figure 4.7 Sediment transport flux across Cross-Sections 2 (Figure 3.10) for Scenarios 1 through 4. There was no difference in sediment transport between Scenario 3 and 5.

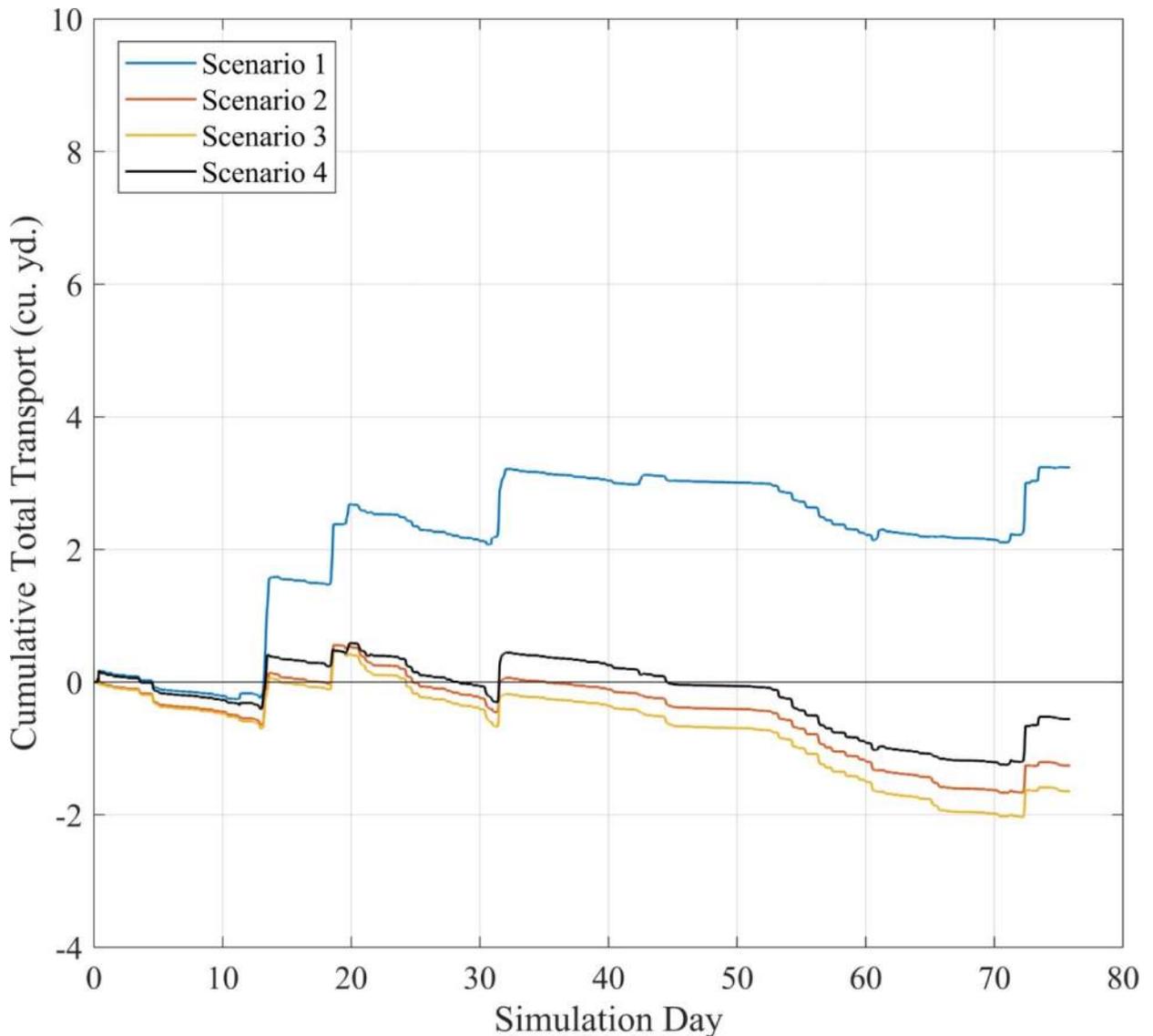


Figure 4.8 Sediment transport flux across Cross-Sections 3 (Figure 3.10) for Scenarios 1 through 4. There was no difference in sediment transport between Scenario 3 and 5.

4.2 Overall Recommendations

Based on the evaluation of coastal processes, it is clear that the wave-induced sediment transport along the beach system at the eastern extent of the Packer Facility is minimal and structural alterations including construction of the proposed bulkhead and wave fence will have similar minimal effects on the stability and form of the overall beach system. The highest annual sediment transport rates along the beach were approximately 10 cubic yards, or about one small truckload. Due to the low sediment transport rates along the beach, it was determined that none of the structural alternatives evaluated would have a discernable impact the stability or form of the beach.

In general, the typical tidal currents in the vicinity of the Packer Facility do not exceed about 0.25 feet per second, and in most areas are only a fraction of this relatively low

velocity. Construction of the full-height wave fence and bulkhead creates a basin that is enclosed on three (3) sides. The conservative tracer evaluation indicates that this design (Scenario 2) markedly reduces tidal circulation within the dredged basin proposed for the project, especially in the basin area at the confluence of the breakwater and wave fence. Therefore, it was concluded that Scenario 2 was not the optimal design to eliminate potential impacts to tidal circulation.

Scenarios 3, 4, and 5 generated similar circulation characteristics to existing conditions. However, Scenario 4 included a wave fence designed in a manner that allows a gap along the seafloor to potentially improve tidal circulation. Based on the conservative tracer simulation, the gap along the seafloor does not provide a measurable improvement in tidal circulation relative to the full height wave fence (Scenarios 3 and 5); however, the gap along the seafloor creates two additional design issues:

- a) The gap allows wave energy, especially from long-period swell) to propagate beneath the wave fence, likely reducing the functionality of the proposed facility. The amount of wave energy propagating under the wave fence was estimated to be over 40%.
- b) By creating a gap under the wave fence, the overall dredged 'footprint' increases as a result of having a designed slope to the east of the wave fence. If the wave fence is constructed into the seafloor, a vertical slope is can be constructed; however, if a gap exists under the wave fence, a 1:3 (v:h) slope will be required, increasing the overall dredged 'footprint' for the project.

The results for the remaining two scenarios (Scenario 3 and 5) are similar, where the model results indicate only slight changes in tidal circulation relative to existing conditions (Scenario 1). Since Scenario 5 provides a full height bulkhead along the landward (southwest) portion of the structure, this scenario provides a slight advantage over Scenario 3, as beach sediments would be prevented from migrating north across the bulkhead even during severe storms for Scenario 5. For this reason, Scenario 5 is recommended with the following elements:

- a) Solid fill pier reconfigured with a slightly reduced footprint
- b) New bulkhead extending ~190 feet to the east to from a contiguous shoreline with the existing Packer Marine Facility bulkhead. The revised bulkhead elevation would include an 80-foot window with two 40-foot subsections. The first of which would lower the bulkhead to -5 feet NAVD beginning at the eastern end of the wave fence and extend 40 feet south. The next section would extend another 40 feet south and lower the bulkhead to -4 feet NAVD. The cross-section details are provided in Figure 3.54.
- c) Incorporation of a solid wave fence extending from the eastern end of the new bulkhead for a distance of ~212 feet to the north
- d) Incorporation of a pile-supported platform south of the new bulkhead, where the 1-ft diameter wood piles are spaced on a 15-ft grid. The landward edge of the platform will consist of a wall feature that is landward of the active sediment transport zone; therefore, there is no interaction of this wall with wave action.

5.0 REFERENCES

Howes B.L., J. S. Ramsey, R.I. Samimy, D.R. Schlezinger, E.M. Eichner (2010). Linked Watershed-Embayment Model to Determine the Critical Nitrogen Loading Threshold for the Lagoon Pond System, Oak Bluffs/Tisbury, Massachusetts. SMAST/DEP Massachusetts Estuaries Project, Massachusetts Department of Environmental Protection. Boston, MA.

6.0 APPENDICES

Appendix A – Grain Size Analysis
(following page)



Briggs Engineering & Testing

A DIVISION OF PK ASSOCIATES, INC.

Foth-CLE Engineering, Inc.
15 Creek Road
Marion, MA 02738
Attn: Ms. Christine Player

Report Date: 1/10/20

Project: Tisbury Marine Terminal
Briggs #: 30428

Tested: 1/10/20
Received: 12/23/20

1. Sample No.	Description	Source of Material
M-31231	TMT-B5 S1 (0-2')	Tisbury Marine Terminal

2. Sieve Analysis {ASTM C 136, and ASTM C 117}

Sieve Size		Results	Specifications
Standard	Alternate	{% Passing by Wt.}	No Spec Provided
100 mm	4"	100	
90 mm	3-1/2"	100	
75 mm	3"	100	
63 mm	2-1/2"	100	
50 mm	2"	100	
37.5 mm	1-1/2"	100	
25 mm	1"	100	
19 mm	3/4"	100	
12.5 mm	1/2"	92	
9.5 mm	3/8"	87	
4.75 mm	#4	80	
2.38 mm	#8	79	
1.190 mm	#16	74	
0.595 mm	#30	40	
0.297 mm	#50	6	
0.150 mm	#100	2	
0.075 mm	#200	1.2	

BRIGGS ENGINEERING & TESTING
A Division of PK Associates, Inc.

Sean Skorohod
Director of Testing Services
Construction Technology Division

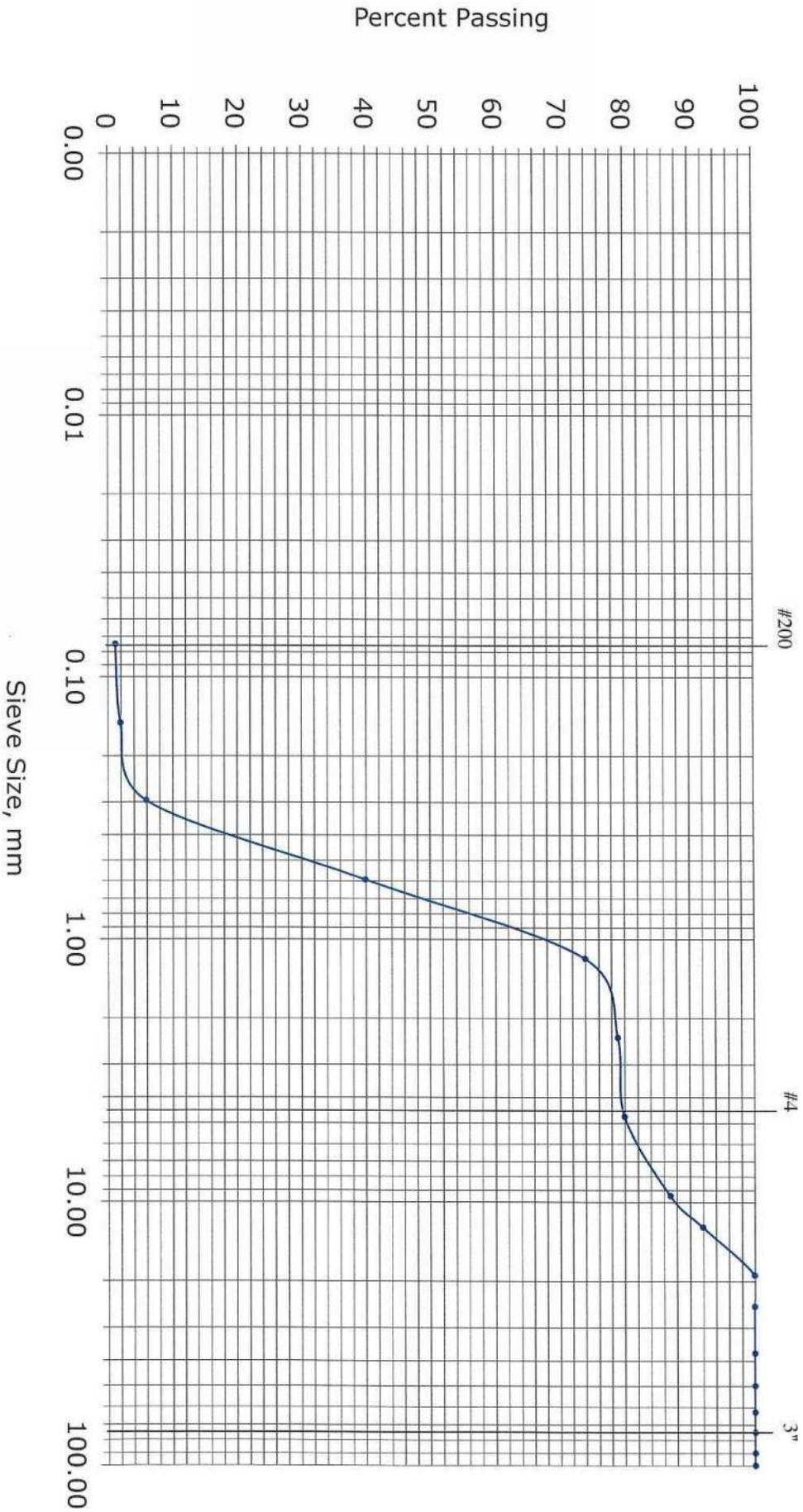
www.briggsengineering.com



Briggs Engineering & Testing
A Division of PK Associates, Inc.

Project:	Tisbury Marine Terminal
Date Tested:	1/10/20
Lab Ref. No.:	M-31231

Sieve Analysis





Briggs Engineering & Testing

A DIVISION OF PK ASSOCIATES, INC.

Foth-CLE Engineering, Inc.
15 Creek Road
Marion, MA 02738
Attn: Ms. Christine Player

Report Date: 1/10/20

Project: Tisbury Marine Terminal
Briggs #: 30428

Tested: 1/10/20
Received: 12/23/20

1.	<u>Sample No.</u>	<u>Description</u>	<u>Source of Material</u>
	M-31232	TMT-B9 S1 (0-2')	Tisbury Marine Terminal

2. Sieve Analysis {ASTM C 136, and ASTM C 117}

<u>Sieve Size</u>		<u>Results</u>	<u>Specifications</u>
Standard	Alternate	{% Passing by Wt.}	No Spec Provided
100 mm	4"	100	
90 mm	3-1/2"	100	
75 mm	3"	100	
63 mm	2-1/2"	100	
50 mm	2"	100	
37.5 mm	1-1/2"	100	
25 mm	1"	100	
19 mm	3/4"	35	
12.5 mm	1/2"	35	
9.5 mm	3/8"	32	
4.75 mm	#4	26	
2.38 mm	#8	21	
1.190 mm	#16	18	
0.595 mm	#30	11	
0.297 mm	#50	5	
0.150 mm	#100	3	
0.075 mm	#200	1.8	

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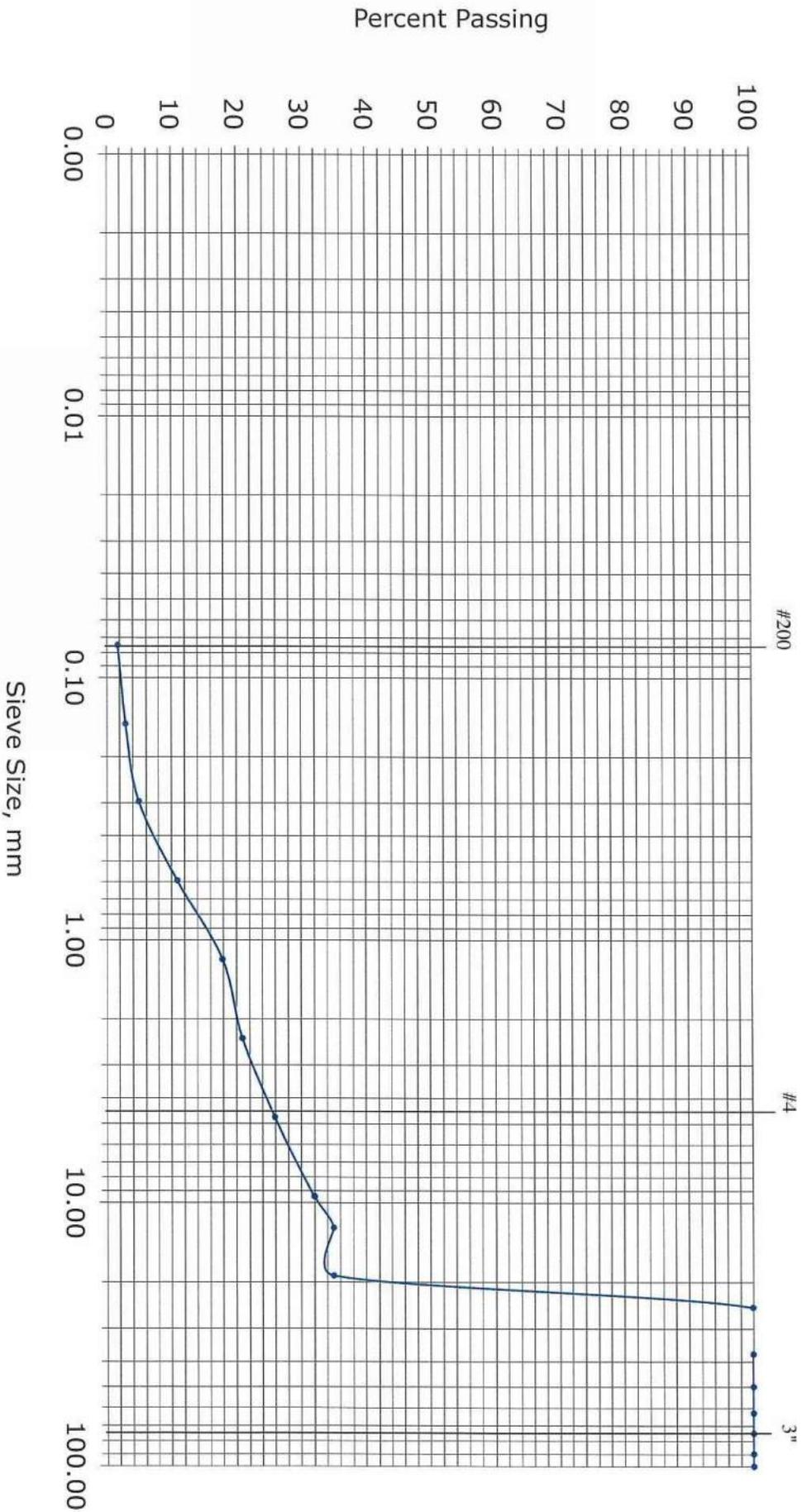
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Briggs Engineering & Testing
A Division of PK Associates, Inc.

Project:	Tisbury Marine Terminal
Date Tested:	1/10/20
Lab Ref. No.:	M-31232

Sieve Analysis





Briggs Engineering & Testing

A DIVISION OF PK ASSOCIATES, INC.

Foth-CLE Engineering, Inc.
15 Creek Road
Marion, MA 02738
Attn: Ms. Christine Player

Report Date: 1/10/20

Project: Tisbury Marine Terminal
Briggs #: 30428

Tested: 1/10/20
Received: 12/23/20

1. Sample No.	Description	Source of Material
M-31233	TMT-B9 S2 (5'-7')	Tisbury Marine Terminal

2. Sieve Analysis {ASTM C 136, and ASTM C 117}

Sieve Size		Results	Specifications
Standard	Alternate	{% Passing by Wt.}	No Spec Provided
100 mm	4"	100	
90 mm	3-1/2"	100	
75 mm	3"	100	
63 mm	2-1/2"	100	
50 mm	2"	100	
37.5 mm	1-1/2"	100	
25 mm	1"	100	
19 mm	3/4"	78	
12.5 mm	1/2"	65	
9.5 mm	3/8"	58	
4.75 mm	#4	48	
2.38 mm	#8	38	
1.190 mm	#16	30	
0.595 mm	#30	22	
0.297 mm	#50	13	
0.150 mm	#100	7	
0.075 mm	#200	5.0	

BRIGGS ENGINEERING & TESTING
A Division of PK Associates, Inc.

Sean Skorohod
Director of Testing Services
Construction Technology Division

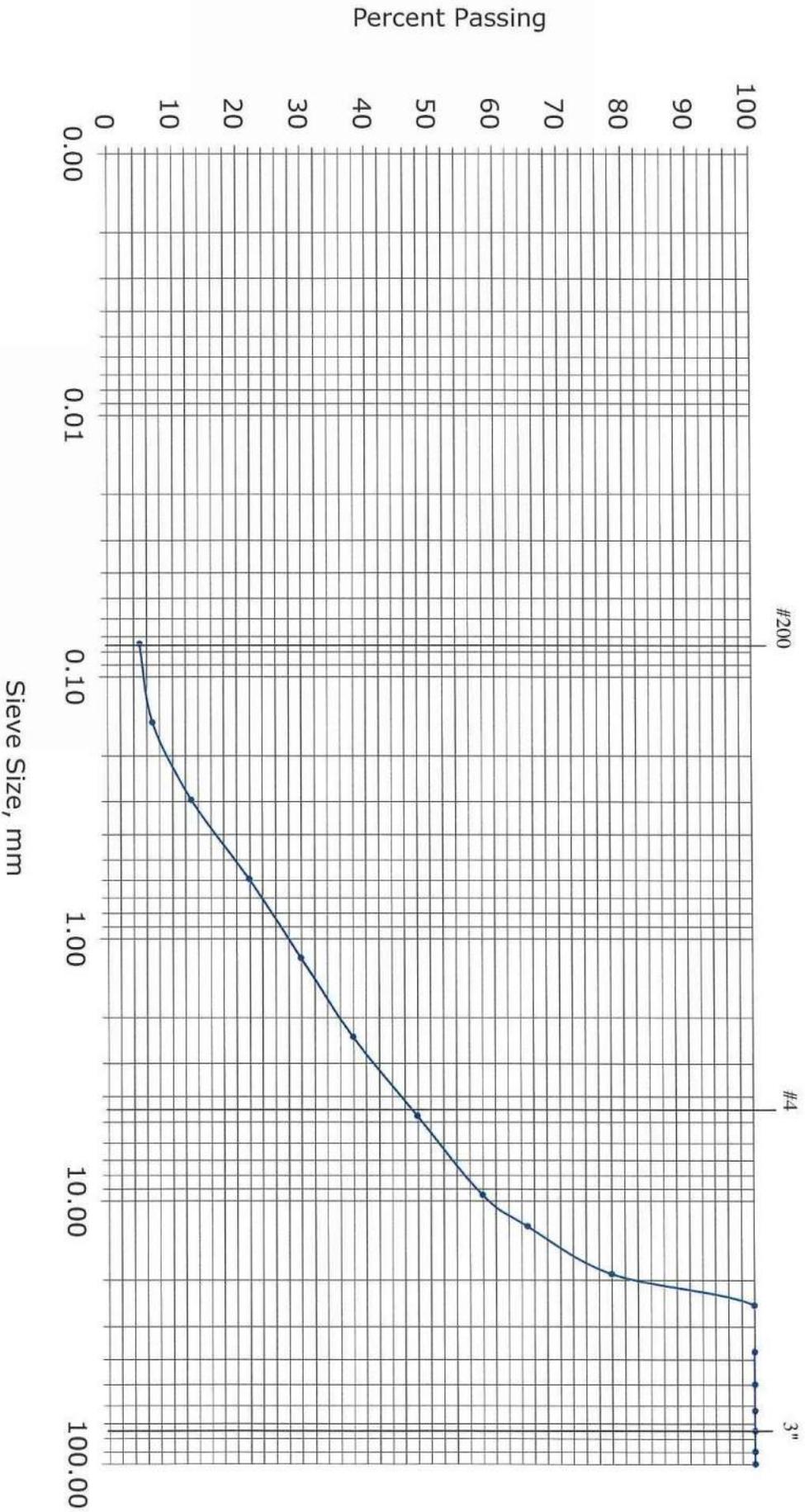
www.briggsengineering.com



Briggs Engineering & Testing
A Division of PK Associates, Inc.

Project:	Tisbury Marine Terminal
Date Tested:	1/10/20
Lab Ref. No.:	M-31233

Sieve Analysis





Briggs Engineering & Testing

A DIVISION OF PK ASSOCIATES, INC.

Foth-CLE Engineering, Inc.
15 Creek Road
Marion, MA 02738
Attn: Ms. Christine Player

Report Date: 1/10/20

Project: Tisbury Marine Terminal
Briggs #: 30428

Tested: 1/10/20
Received: 12/23/20

1. Sample No.	Description	Source of Material
M-31234	TMT-B15 S1 (0-2')	Tisbury Marine Terminal

2. Sieve Analysis {ASTM C 136, and ASTM C 117}

Sieve Size		Results	Specifications
Standard	Alternate	{% Passing by Wt.}	No Spec Provided
100 mm	4"	100	
90 mm	3-1/2"	100	
75 mm	3"	100	
63 mm	2-1/2"	100	
50 mm	2"	100	
37.5 mm	1-1/2"	100	
25 mm	1"	100	
19 mm	3/4"	87	
12.5 mm	1/2"	79	
9.5 mm	3/8"	74	
4.75 mm	#4	64	
2.38 mm	#8	57	
1.190 mm	#16	49	
0.595 mm	#30	26	
0.297 mm	#50	6	
0.150 mm	#100	4	
0.075 mm	#200	2.7	

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A Division of PK Associates, Inc.

Sean Skorohod
Director of Testing Services
Construction Technology Division

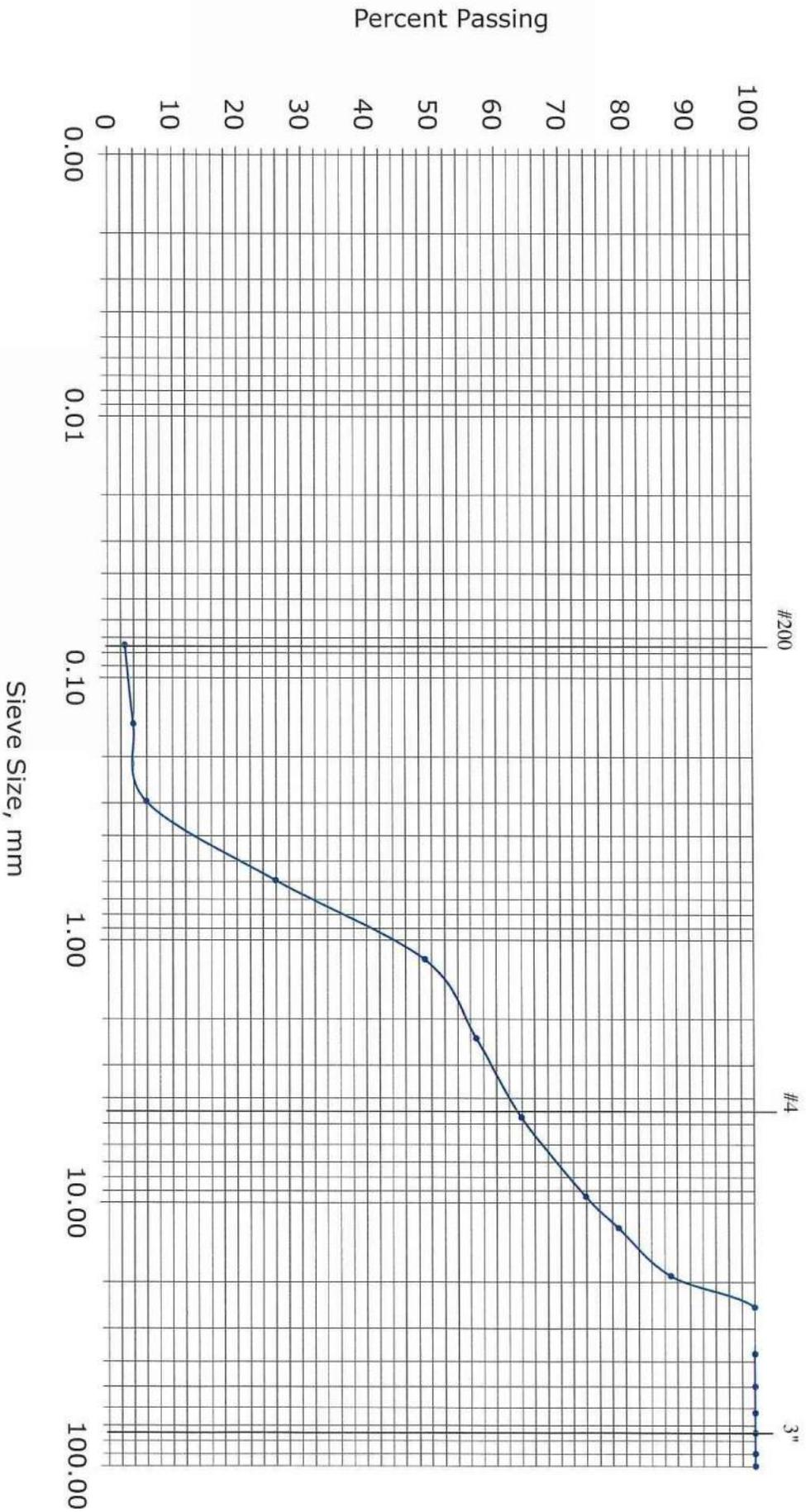
www.briggsengineering.com



Briggs Engineering & Testing
A Division of PK Associates, Inc.

Project:	Tisbury Marine Terminal
Date Tested:	1/10/20
Lab Ref. No.:	M-31234

Sieve Analysis





Briggs Engineering & Testing

A DIVISION OF PK ASSOCIATES, INC.

Foth-CLE Engineering, Inc.
15 Creek Road
Marion, MA 02738
Attn: Ms. Christine Player

Report Date: 1/10/20

Project: Tisbury Marine Terminal
Briggs #: 30428

Tested: 1/10/20
Received: 12/23/20

1. Sample No.	Description	Source of Material
M-31235	TMT-B12 S1 (0-2')	Tisbury Marine Terminal

2. Sieve Analysis {ASTM C 136, and ASTM C 117}

Sieve Size		Results	Specifications
Standard	Alternate	{% Passing by Wt.}	No Spec Provided
100 mm	4"	100	
90 mm	3-1/2"	100	
75 mm	3"	100	
63 mm	2-1/2"	100	
50 mm	2"	100	
37.5 mm	1-1/2"	100	
25 mm	1"	100	
19 mm	3/4"	87	
12.5 mm	1/2"	87	
9.5 mm	3/8"	86	
4.75 mm	#4	80	
2.38 mm	#8	73	
1.190 mm	#16	63	
0.595 mm	#30	43	
0.297 mm	#50	20	
0.150 mm	#100	5	
0.075 mm	#200	3.4	

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Sean Skorohod
Director of Testing Services
Construction Technology Division

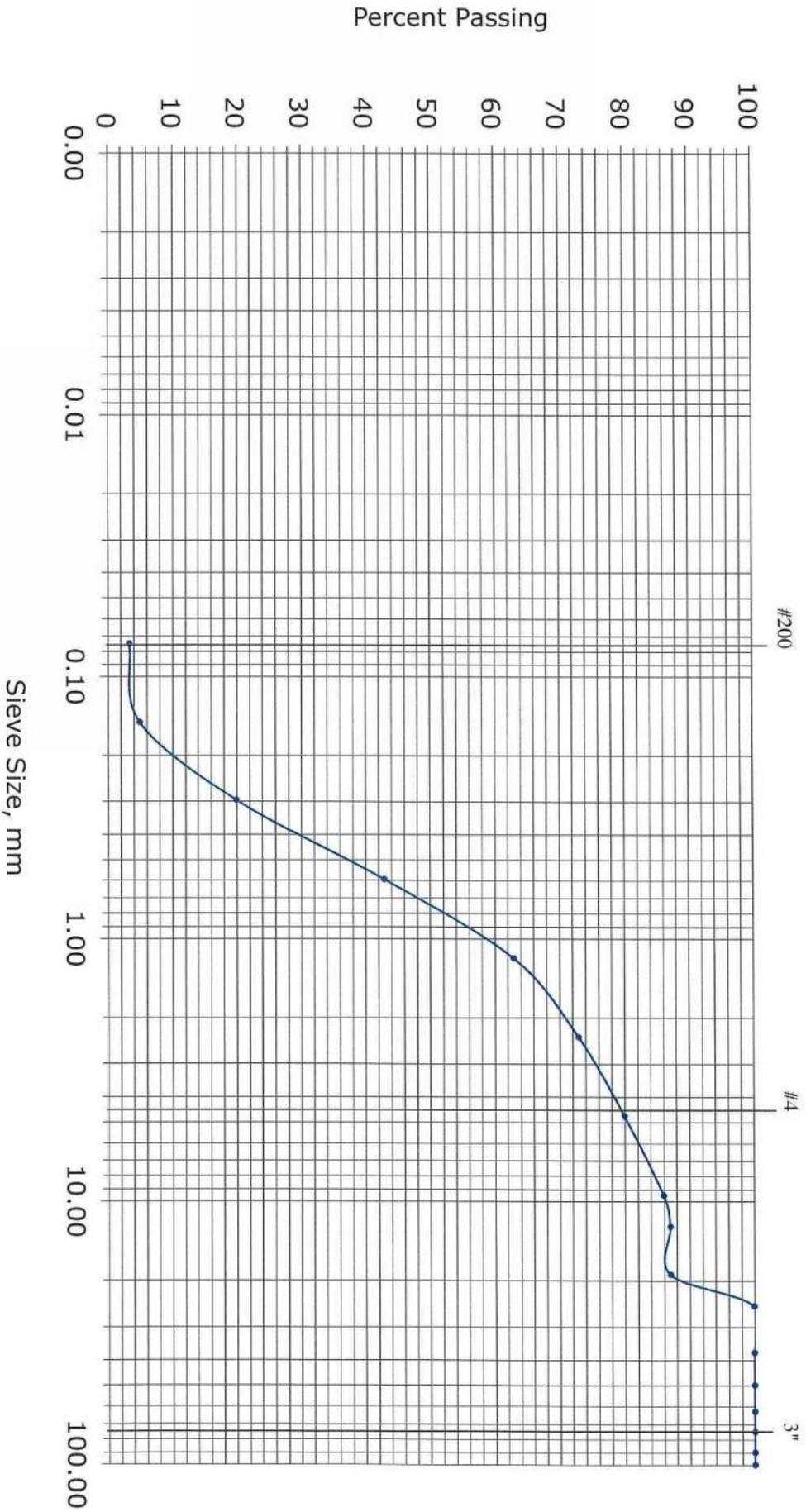
www.briggsengineering.com



Briggs Engineering & Testing
A Division of PK Associates, Inc.

Project:	Tisbury Marine Terminal
Date Tested:	1/10/20
Lab Ref. No.:	M-31235

Sieve Analysis





Briggs Engineering & Testing

A DIVISION OF PK ASSOCIATES, INC.

Foth-CLE Engineering, Inc.
15 Creek Road
Marion, MA 02738
Attn: Ms. Christine Player

Report Date: 1/10/20

Project: Tisbury Marine Terminal
Briggs #: 30428

Tested: 1/10/20
Received: 12/23/20

1.	<u>Sample No.</u>	<u>Description</u>	<u>Source of Material</u>
	M-31236	TMT-B11 S1 (0-2')	Tisbury Marine Terminal

2. Sieve Analysis {ASTM C 136, and ASTM C 117}

<u>Sieve Size</u>		<u>Results</u>	<u>Specifications</u>
Standard	Alternate	{% Passing by Wt.}	No Spec Provided
100 mm	4"	100	
90 mm	3-1/2"	100	
75 mm	3"	100	
63 mm	2-1/2"	100	
50 mm	2"	100	
37.5 mm	1-1/2"	100	
25 mm	1"	100	
19 mm	3/4"	100	
12.5 mm	1/2"	100	
9.5 mm	3/8"	100	
4.75 mm	#4	100	
2.38 mm	#8	99	
1.190 mm	#16	91	
0.595 mm	#30	73	
0.297 mm	#50	32	
0.150 mm	#100	6	
0.075 mm	#200	3.7	

BRIGGS ENGINEERING & TESTING
A Division of PK Associates, Inc.

Sean Skorohod
Director of Testing Services
Construction Technology Division

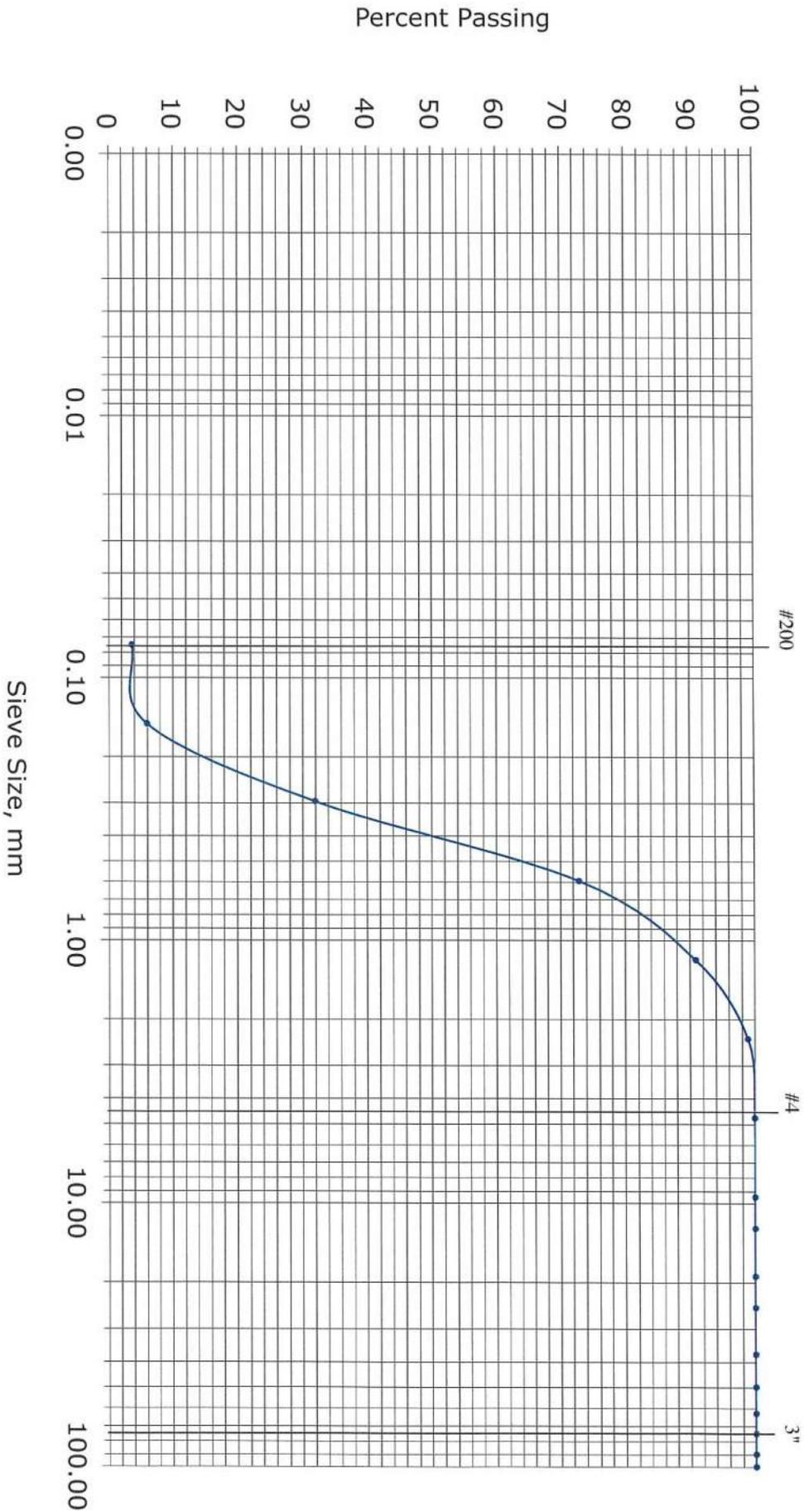
www.briggsengineering.com



Briggs Engineering & Testing
A Division of PK Associates, Inc.

Project:	Tisbury Marine Terminal
Date Tested:	1/10/20
Lab Ref. No.:	M-31236

Sieve Analysis





Briggs Engineering & Testing

A DIVISION OF PK ASSOCIATES, INC.

Foth-CLE Engineering, Inc.
15 Creek Road
Marion, MA 02738
Attn: Ms. Christine Player

Report Date: 1/10/20

Project: Tisbury Marine Terminal
Briggs #: 30428

Tested: 1/10/20
Received: 12/23/20

1.	<u>Sample No.</u>	<u>Description</u>	<u>Source of Material</u>
	M-31237	TMT-B4 S1 (0-2')	Tisbury Marine Terminal

2. Sieve Analysis {ASTM C 136, and ASTM C 117}

<u>Sieve Size</u>		<u>Results</u>	<u>Specifications</u>
Standard	Alternate	{% Passing by Wt.}	No Spec Provided
100 mm	4"	100	
90 mm	3-1/2"	100	
75 mm	3"	100	
63 mm	2-1/2"	100	
50 mm	2"	100	
37.5 mm	1-1/2"	100	
25 mm	1"	100	
19 mm	3/4"	78	
12.5 mm	1/2"	74	
9.5 mm	3/8"	63	
4.75 mm	#4	41	
2.38 mm	#8	30	
1.190 mm	#16	22	
0.595 mm	#30	11	
0.297 mm	#50	4	
0.150 mm	#100	1	
0.075 mm	#200	0.5	

BRIGGS ENGINEERING & TESTING
A Division of PK Associates, Inc.

Sean Skorohod
Director of Testing Services
Construction Technology Division

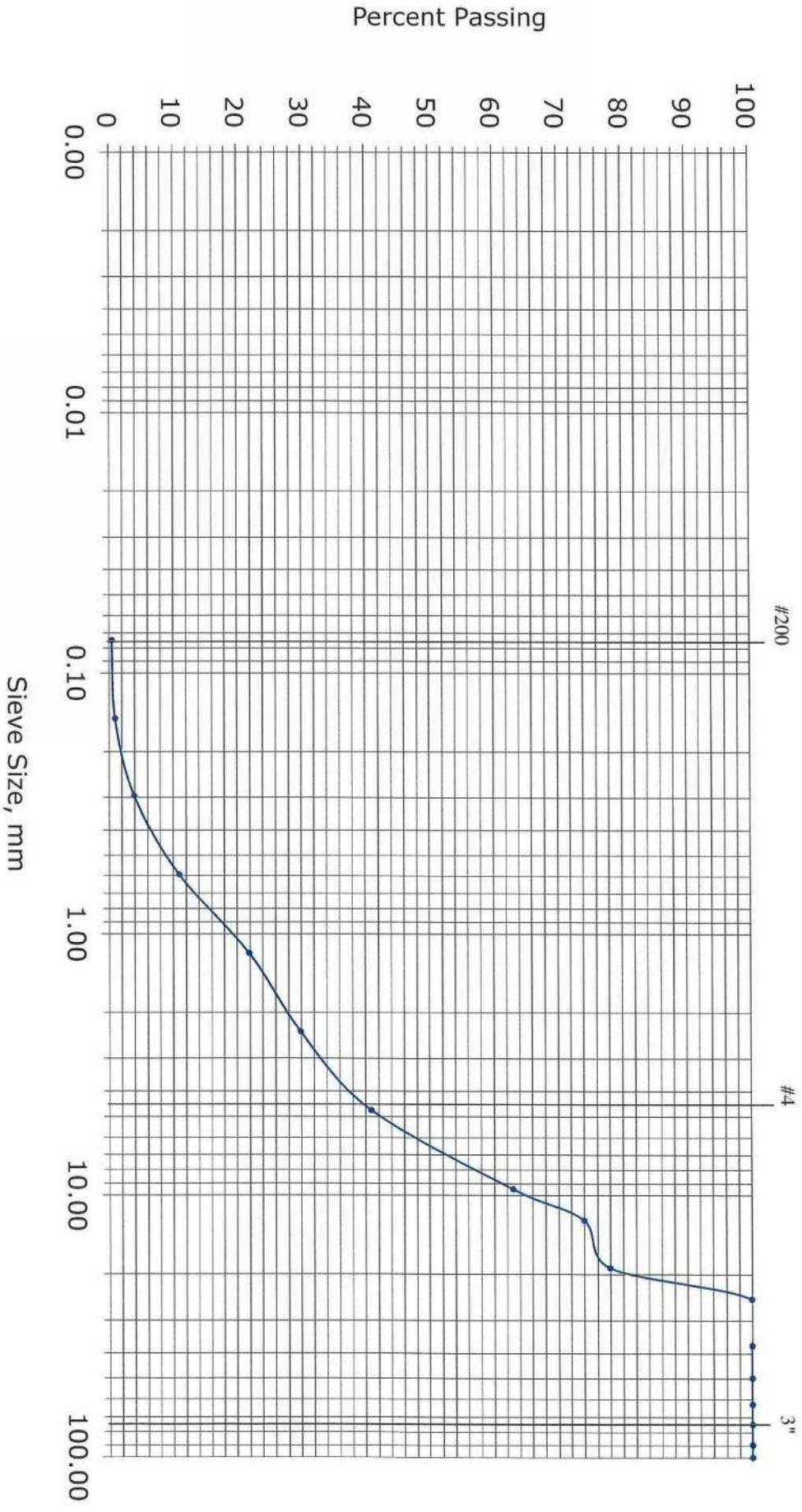
www.briggsengineering.com



Briggs Engineering & Testing
A Division of PK Associates, Inc.

Project:	Tisbury Marine Terminal
Date Tested:	1/10/20
Lab Ref. No.:	M-31237

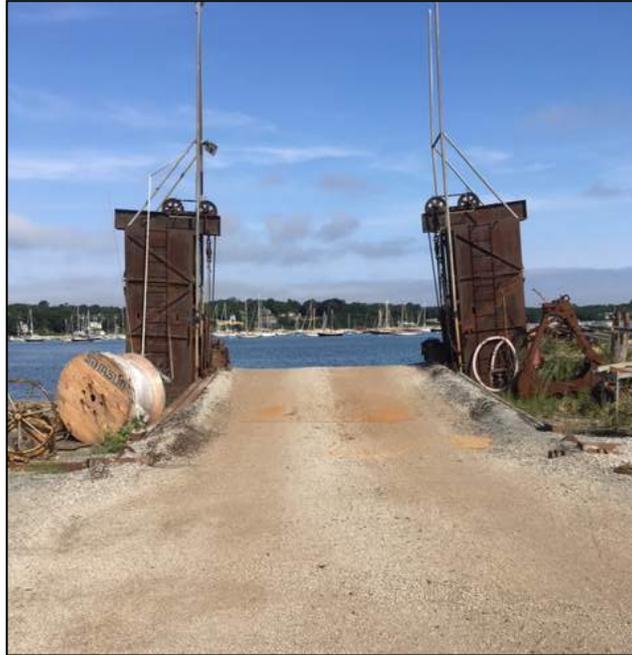
Sieve Analysis



Attachment J

Site Photographic Documentation

Site Photographic Documentation



Existing Barge Ramp



Coastal Barrier Beach Looking East



Looking East Southeast Along Top of Coastal Beach



Looking West Southwest at Existing Bulkhead and Barge Ramp (foreground)



Looking West Along Coastal Barrier Beach to Solid Filled Pier

Attachment K

Filing Fees



Massachusetts Department of Environmental Protection
 Bureau of Resource Protection - Wetlands
NOI Wetland Fee Transmittal Form
 Massachusetts Wetlands Protection Act M.G.L. c. 131, §40

Important: When filling out forms on the computer, use only the tab key to move your cursor - do not use the return key.



A. Applicant Information

1. Location of Project:

190 Beach Road Tisbury
 a. Street Address b. City/Town

 c. Check number d. Fee amount

2. Applicant Mailing Address:

Ralph Packer
 a. First Name b. Last Name
 Tisbury Marine Terminal, LLC
 c. Organization
 190 Beach Road
 d. Mailing Address
 Tisbury (Vineyard Haven) MA 02568
 e. City/Town f. State g. Zip Code
 508 693-0900 rmpacker@vineyard.net
 h. Phone Number i. Fax Number j. Email Address

3. Property Owner (if different):

_____ _____
 a. First Name b. Last Name

 c. Organization

 d. Mailing Address

 e. City/Town f. State g. Zip Code

 h. Phone Number i. Fax Number j. Email Address

B. Fees

Fee should be calculated using the following process & worksheet. **Please see Instructions before filling out worksheet.**

Step 1/Type of Activity: Describe each type of activity that will occur in wetland resource area and buffer zone.

Step 2/Number of Activities: Identify the number of each type of activity.

Step 3/Individual Activity Fee: Identify each activity fee from the six project categories listed in the instructions.

Step 4/Subtotal Activity Fee: Multiply the number of activities (identified in Step 2) times the fee per category (identified in Step 3) to reach a subtotal fee amount. Note: If any of these activities are in a Riverfront Area in addition to another Resource Area or the Buffer Zone, the fee per activity should be multiplied by 1.5 and then added to the subtotal amount.

Step 5/Total Project Fee: Determine the total project fee by adding the subtotal amounts from Step 4.

Step 6/Fee Payments: To calculate the state share of the fee, divide the total fee in half and subtract \$12.50. To calculate the city/town share of the fee, divide the total fee in half and add \$12.50.

To calculate filing fees, refer to the category fee list and examples in the instructions for filling out WPA Form 3 (Notice of Intent).



Massachusetts Department of Environmental Protection
 Bureau of Resource Protection - Wetlands
NOI Wetland Fee Transmittal Form
 Massachusetts Wetlands Protection Act M.G.L. c. 131, §40

B. Fees (continued)

Step 1/Type of Activity	Step 2/Number of Activities	Step 3/Individual Activity Fee	Step 4/Subtotal Activity Fee
Category 3 - Building, Road	2	\$1,050	\$2,100
Category 4 - Dredging	1	\$1,450	\$1,450
Category 5 - Bulkhead	785 LF	\$4/LF	
Category 5 - Solid Fill Pier	110 LF	\$4/LF	
Category 5 - Wave Fence	202 LF	\$4/LF	
Category 5 - Deck (Dock)	246 LF	\$4/LF	
Category 5 - MAX FEE \$2,000			\$2,000
Step 5/Total Project Fee:			\$5,550

Step 6/Fee Payments:

Total Project Fee:	<u>\$5,550</u>
State share of filing Fee:	a. Total Fee from Step 5 <u>\$2,762.50</u>
City/Town share of filing Fee:	b. 1/2 Total Fee less \$12.50 <u>\$2,787.50</u> c. 1/2 Total Fee plus \$12.50

C. Submittal Requirements

- a.) Complete pages 1 and 2 and send with a check or money order for the state share of the fee, payable to the Commonwealth of Massachusetts.

Department of Environmental Protection
 Box 4062
 Boston, MA 02211

- b.) **To the Conservation Commission:** Send the Notice of Intent or Abbreviated Notice of Intent; a **copy** of this form; and the city/town fee payment.

To MassDEP Regional Office (see Instructions): Send a copy of the Notice of Intent or Abbreviated Notice of Intent; a **copy** of this form; and a **copy** of the state fee payment. (E-filers of Notices of Intent may submit these electronically.)

TISBURY MARINE TERMINAL LLC

PO BOX 308
190 BEACH ROAD
TISBURY, MA 02568



Martha's Vineyard

SAVINGS BANK
Edgartown, MA
53-7292/2113



7/14/2020

PAY TO THE ORDER OF COMMONWEALTH OF MASSACHUSETTS NHESP

\$ **300.00

Three Hundred and 00/100***** DOLLARS

COMMONWEALTH OF MASSACHUSETTS



[Handwritten Signature]
AUTHORIZED SIGNATURE

MEMO

MESA Filing Fee

⑈001023⑈ ⑆211372925⑆ 45 190816⑈

TISBURY MARINE TERMINAL LLC

COMMONWEALTH OF MASSACHUSETTS-NHESP

7/14/2020

300.00

Tisbury Marine Termin MESA Filing Fee

300.00

Security features. Details on back.

TISBURY MARINE TERMINAL LLC

PO BOX 308
190 BEACH ROAD
TISBURY, MA 02568



Martha's Vineyard

SAVINGS BANK
Edgartown, MA
53-7292/2113



7/14/2020

PAY TO THE
ORDER OF

Town of Tisbury

\$ **2,827.50

Two Thousand Eight Hundred Twenty Seven and 50/100***** DOLLARS

Town of Tisbury



[Handwritten Signature]
AUTHORIZED SIGNATURE

MEMO

NOI Fee

⑈001022⑈ ⑆211372925⑆ 45 190816⑈

TISBURY MARINE TERMINAL LLC

Town of Tisbury

7/14/2020

1022

2,827.50

Tisbury Marine Termin NOI Fee

2,827.50

Security features. Details on back.

TISBURY MARINE TERMINAL LLC

PO BOX 308
190 BEACH ROAD
TISBURY, MA 02568



Martha's Vineyard

SAVINGS BANK
Edgartown, MA
53-7292/2113



7/14/2020

PAY TO THE
ORDER OF

Commonwealth of MA

\$ **2,762.50

Two Thousand Seven Hundred Sixty Two and 50/100***** DOLLARS

Commonwealth of MA



[Handwritten Signature]
AUTHORIZED SIGNATURE

MEMO

Permitting Fee - NOI Fee

⑈001021⑈ ⑆211372925⑆ 45 190816⑈

TISBURY MARINE TERMINAL LLC

Commonwealth of MA

7/14/2020

1021

2,762.50

Tisbury Marine Termin Permitting Fee - NOI Fee

2,762.50

Security features. Details on back.



Foth Infrastructure & Environment, LLC
 2121 Innovation Court • Suite 300 • P. O. Box 5126
 De Pere, WI 54115-5126

Associated Bank
 Green Bay, Wisconsin
 79-571759

011913

CHECK DATE July 20, 20

PAY TO THE ORDER OF Town of Tisbury
forty dollars 00/100

\$ 40⁰⁰

DOLLARS
 VOID OVER \$500.00

[Signature]
 AUTHORIZED SIGNATURE ^{MP}

VOID AFTER 90 DAYS

Security Check features included. Details on back.

⑈011913⑈ ⑆075900575⑆ 2223 146 891⑈



Foth Infrastructure & Environment, LLC

2121 Innovation Court • Suite 300
 P. O. Box 5126

De Pere, WI
 54115-5126

EMILY BUSINESS FORMS 800.392.6018

011913

ITEM DESCRIPTION	VENDOR NO	VOUCHER NO	AMOUNT	PROJECT	ACCOUNT
ND1				197008.00	

Attachment L

Abutter Documentation

Parcel Number	GIS Number	Cama Number	Property Address	Owner Name	Co-Owner Name	Owner Address	Owner City	Owner State	Owner Zip
10-A-2	10-A-2	10-A-2	BEACH RD	TISBURY TOWN OF	WEST OF BLKHD TO BRIDGE	BOX 1239	TISBURY	MA	02568
10-B-1	10-B-1	10-B-1	199 BEACH RD	PACKER R M CO INC		P O BOX 308	VINEYARD HAVEN	MA	02568
10-B-2	10-B-2	10-B-2	BEACH RD	PACKER RALPH M JR	C/O R M PACKER CO INC	BOX 308	VINEYARD HAVEN	MA	02568
10-B-4	10-B-4	10-B-4	287 BEACH RD	TISBURY TOWN OF	RAMP	BOX 1239	TISBURY	MA	02568
9-B-32	9-B-32	9-B-32	159 BEACH RD	MARTHAS VINEYARD SHIPYARD INC		BOX 1119	VINEYARD HAVEN	MA	02568
9-B-33	9-B-33	9-B-33	173 BEACH RD	MARTHAS VINEYARD SHIPYARD INC		P O BOX 1119	VINEYARD HAVEN	MA	02568
9-B-34	9-B-34	9-B-34	BEACH RD	PACKER R M CO INC		P O BOX 308	VINEYARD HAVEN	MA	02568
9-C-14	9-C-14	9-C-14	158 BEACH RD	TISBURY WHARF CO INC		BOX 758	TISBURY	MA	02568
9-C-15	9-C-15	9-C-15	164 BEACH RD	MARTHAS VINEYARD SHIPYARD INC		BOX 1119	VINEYARD HAVEN	MA	02568
9-C-16	9-C-16	9-C-16	188 BEACH RD	PACKER R M CO INC		BOX 308	VINEYARD HAVEN	MA	02568

Notification to Abutters Under the
Massachusetts Wetlands Protection Act

In accordance with the second paragraph of Massachusetts General Laws Chapter 131, Section 40, you are hereby notified of the following.

- A. The name of the applicant is Tisbury Marine Terminal, LLC
- B. The applicant has filed a Notice of Intent with the Conservation Commission for the municipality of Tisbury seeking permission to remove, fill, dredge or alter an Area Subject to Protection Under the Wetlands Protection Act (General Laws Chapter 131, Section 40).
- C. The address of the lot where the activity is proposed is 190 Beach Road, Tisbury, MA
- D. Copies of the Notice of Intent may be examined at Conservation Office between the hours of 10 and 4pm on the following days of the week: M-F
For more information, call: (508) 696-4260
Check One: This is the applicant , representative , or other (specify):
- E. Copies of the Notice of Intent may be obtained from either (check one) the applicant , or the applicant's representative , by calling this telephone number (508)762-0764 - between the hours of 8am and 5pm on the following days of the week: Monday - Friday
- F. Information regarding the date, time, and place of the public hearing may be obtained from Conservation Office
by calling this telephone number (508) 696-4264 between the hours of 10 and 4p on the following days of the week: M-F
Check One: This is the applicant , representative , or other (specify):

NOTE: Notice of the public hearing, including its date, time, and place, will be published at least five (5) days in advance in the Vineyard Gazette
(name of newspaper)

NOTE: Notice of the public hearing, including its date, time, and place, will be posted in the City or Town Hall not less than forty-eight (48) hours in advance.

NOTE: You also may contact your local Conservation Commission or the nearest Department of Environmental Protection Regional Office for more information about this application or the Wetlands Protection Act. To contact DEP, call:

Central Region: 508-792-7650

Northeast Region: 617-935-2160

Southeast Region: 508-946-2800

Western Region: 413-784-1100