ADDENDUM NO. 2, October 4, 2024

Proposal No. 605311-128035

Marion-Warham Bridges Over Weweantic River along Route 6

Bridge No. M-05-001=W-06-013 and W-06-016

Project No. 605311

Final Hydraulic Report

Marion/Warham, MA

September 30, 2024

Massachusetts Department of Transportation
Highway Division

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1 Executive Summary

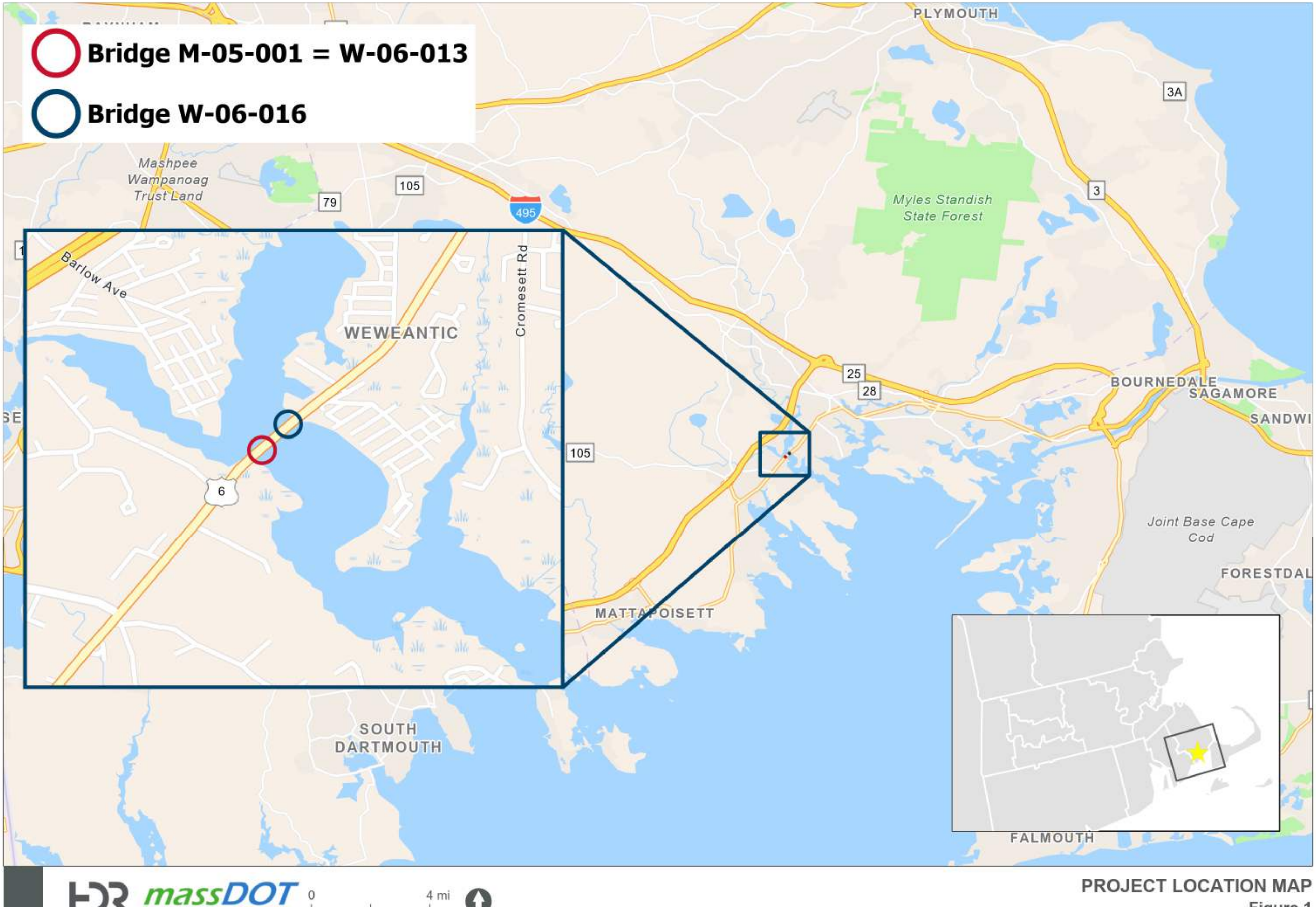
The Massachusetts Department of Transportation (MassDOT) is replacing the Marion-Warham Bridges over the Weweantic River (Bridge Numbers M-05-001=W-06-013 and W-06-016) along Route 6, also known as Marion Road or Wareham Road. HDR prepared this report to provide documentation of the hydraulic engineering analysis and calculations. Numerical modeling of storm surge and associated currents was performed using MIKE21 HD FM. The modeling evaluated the 2-percent (50-year) and the 1-percent (100-year) annual exceedance probability (AEP) storm events and assessed the impact of Relative Sea Level Rise (RSLR) on storm surge and current velocities. Numerical modeling was also performed for waves using the MIKE21 SW model. Wave modeling was performed for the 100-year wind speeds and AEP water levels with and without RSLR. Variables for these calculations were extracted from hydrodynamic and spectral wave analysis models simulating multiple design storm events. A two-dimensional hydraulic model, SRH-2D, was utilized to model and calculate the water surface elevations, depths, and velocities of the 25-year, 50-year, and 100-year riverine floods, as well as the 50-year and 100-year coastal floods with and without RSLR. SRH-2D was used to obtain variables for the 50-year and 100-year riverine and the 25-year, 50-year, and 100-year coastal surge storm events in order to estimate total scour at the proposed bridges.

The existing bridges have a documented history of scour, and both are currently experiencing scour related issues. Review of historical documents reveal scour countermeasures have been installed on multiple occasions. This has caused a hardening of the riverbed directly under the bridges. This can be clearly seen in the bathymetry data. There are large contraction scour holes upstream and downstream of both bridges, but the riverbed is significantly higher under the bridges. This hardening of the bed is currently preventing scour under the bridge to the magnitude it is observed upstream and downstream. However, there is no way to know how long this hardening will last. This also makes the scour calculations difficult, as there is no way to take the hardening into account. Therefore, this scour investigation of the proposed bridge designs determined the proposed bent piles should be placed either to a depth below the scour and check-scour envelope, securely embedded into bedrock, or have scour countermeasures designed, installed, maintained, and checked after every large storm event.

2 Project Description

This project involves the replacement of the existing Bridges M-05-001=W-06-013 (Lat. 41° 44’ 15.01” N; Long. 70° 44’ 50.66” W) and W-06-016 (Lat. 41° 44’ 19.06” N; Long. 70° 44’ 45.90” W). The structure was built in 1929 and then widened in 1957. The bridges carry Route 6 (Marion Road / Wareham Road) over Weweantic River, an estuary separating the towns of Marion and Wareham in Massachusetts.

The MassDOT 2013 LRFD Bridge Manual, revised in 2020, was used as a guide to follow hydrologic and hydraulic design criteria for the construction of new bridge structures in the Commonwealth of Massachusetts. The project location is shown in Figure 1.



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PROJECT LOCATION MAP
Figure 1

MARION - WAREHAM BRIDGE REPLACEMENTS OVER WEWEANTIC RIVER ALONG ROUTE 6

2.1 Existing Structure

The existing Weweantic River bridges were first constructed in 1929 and then widened in 1957. Both bridges have a 44-foot-wide curb-to-curb width consisting of four travel lanes and a concrete sidewalk with an outside bridge railing is provided along both the north and south side of the bridge. The full bridge width (out to out) is 58.5 feet.

The existing West bridge, Bridge M-05-001 = W-06-013, consists of two spans with a single pier wall at the center of the bridge. The pier wall is supported by a 10-foot-wide pier footing that is 70 feet long. The bridge is approximately 140 feet long and has a low chord of +6.32 ft. NAVD88 at the west abutment. The bridge was designed to provide a vertical clearance of approximately 5-feet 6-inches above the mean high water surface elevation (+0.79 ft. NAVD88) of the Weweantic River at the time of reconstruction in 1956.

The existing East Bridge, Bridge W-06-016, consists of three spans with two pier walls. The pier walls are spaced 51.5 feet apart (center to center) and they are each supported by a 10-foot-wide pier footing that is 70 feet long. The bridge is approximately 178 feet long and has a low chord of +8.42 ft. NAVD88 at the east abutment. The bridge was designed to provide a vertical clearance of approximately 7-feet 8-inches above the mean high water surface elevation (+0.79 ft. NAVD88) of the Weweantic River at the time of reconstruction in 1956.

The National Bridge Inspection Standards (NBIS) uses Item 113 to describe a structure's vulnerability to scour. According to National Bridge Inventory (NBI), both bridges are coded with a rating factor of three (3). The Federal Highway Administration's (FHWA) *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* defines this rating as: "Bridge is scour critical; bridge foundations determined to be unstable for calculated scour conditions."

2.1.1 Crossed Waterway at the Bridge Location

FHWA's Hydraulic Engineering Circular Number 20, Stream Stability at Highway Structures, (HEC-20) was utilized to assist in the analysis of the project's environment. These parameters can provide useful insight of the geomorphic activities occurring in an area.

2.1.1.1 Watershed

The Weweantic River merges with the Sippican River just upstream of the bridges. The flow discharges downstream of the bridges into Buzzards Bay. The hydrologic information available at the project location on the United States Geological Survey's (USGS) Website, StreamStats, is not sufficient since the delineation point is located in an excluded area. Therefore, the drainage area of the Route 6 crossings were delineated using the hydrological tools in ArcGIS Pro. ArcGIS Pro has internal geoprocessing tools that can compute flow directions and accumulations based on raster elevation data. The elevation data used to delineate the watershed is discussed in Section 3.1.

The watershed delineated using ArcGIS Pro proved to be more accurate when compared to the watershed produced from StreamStats. The watershed produced from ArcGIS Pro utilized newer and better elevation data and was able to ascertain hydrologic flow lines that StreamStats did not.

Figure 2 displays the drainage area delineation. Table 1 summarizes the watershed basin characteristics determined from the watershed delineation. This information was used to calculate the hydrology further discussed in Section 4.1.

Table 1. Total Watershed Basin Characteristics

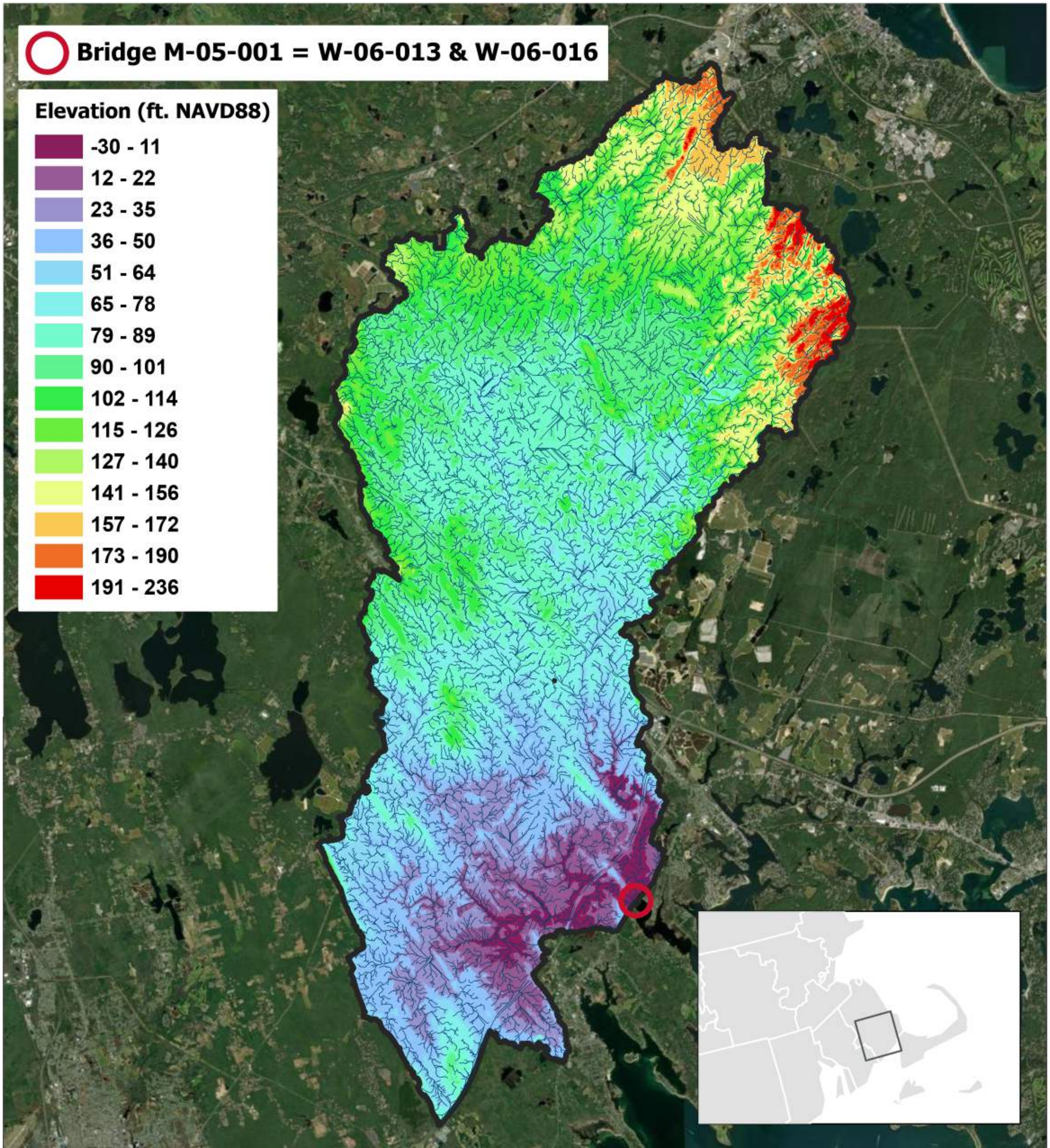
Basin Characteristic	Value	Unit
Drainage Area	89.5	sq. miles
Maximum Elevation	+235.3	ft (NAVD88)
Minimum Elevation	-30.7	ft (NAVD88)

The total watershed that drains to the bridges consists of two independent watersheds converging just upstream of the bridges: the Sippican River watershed and the Weweantic River watershed.

Table 2 shows the drainage area of each watershed and the percent of the total watershed. This information was used to generate the inflow boundary conditions for the riverine hydraulic simulations and is further discussed in Section 4.3.1.5.

Table 2. Sippican River Watershed & Weweantic River Watershed

Watershed	Drainage Area (sq. miles)	% of Total Watershed
Sippican River	30.86	34.5%
Weweantic River	58.67	65.5%



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0 1.3 mi

PROJECT LOCATION WATERSHED DELINEATION

Figure 2

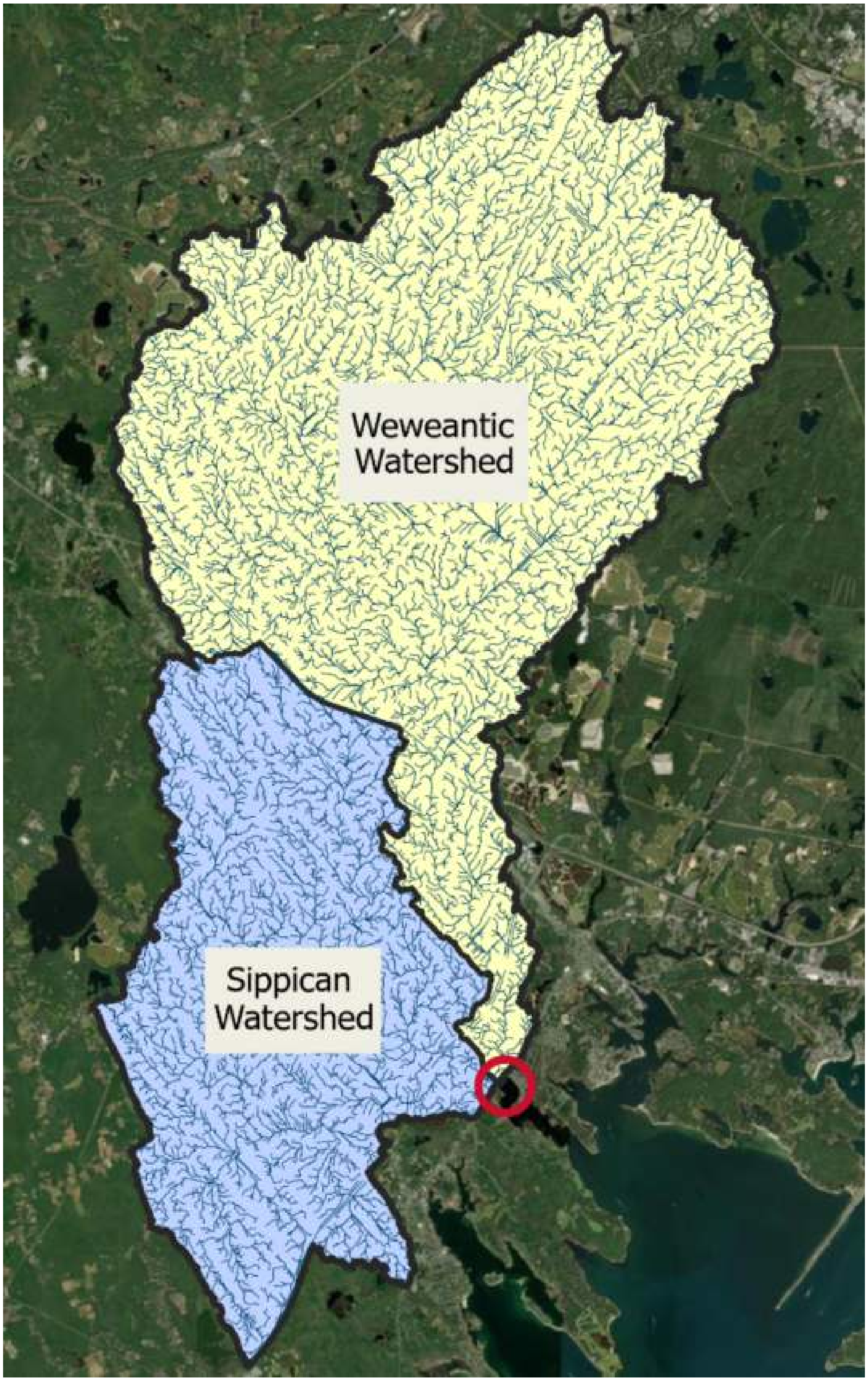


Figure 3. Sippican River Watershed & Weweantic River Watershed

2.1.1.2 Land Cover

Utilizing the National Land Cover Database (NLCD) provided by the Multi-Resolution Land Characteristics (MRLC) Consortium, a comparison of the land use of the Route 6 crossing Weweantic River watershed from 2001 to 2021 was developed.

Table 3 displays the percent change in the land use types within the watershed from 2001 to 2021. Over the last two decades, there has been no more than a 2% change in any individual type of land use. Therefore, it was concluded that there is no significant rapid urbanization or major changes in the watershed.

Table 3. Land Use Change from 2001 to 2021

Land Use Type	2001	2021	Difference
Barren Land	4.44%	4.02%	-0.42%
Cultivated Crops	4.38%	4.32%	-0.06%
Deciduous Forest	6.35%	6.64%	0.29%
Developed, High Intensity	3.00%	3.79%	0.79%
Developed, Low Intensity	0.47%	0.81%	0.34%
Developed, Medium Intensity	0.85%	1.15%	0.30%
Developed, Open Space	1.39%	1.10%	-0.30%
Emergent Herbaceous Wetlands	27.42%	25.43%	-1.99%
Evergreen Forest	11.88%	11.55%	-0.33%
Hay/Pasture	0.13%	0.28%	0.16%
Herbaceous	1.31%	2.15%	0.84%
Mixed Forest	1.67%	1.56%	-0.11%
Open Water	16.18%	16.10%	-0.08%
Shrub/Scrub	19.64%	19.70%	0.06%
Woody Wetlands	0.90%	1.40%	0.50%

2.1.1.3 Sinuosity

A stream's sinuosity is the ratio of the channel length at the thalweg to the valley's length. Typically, a stream’s sinuosity is indicative of its lateral stability. However, the two bridges being analyzed are in the coastal environment, not the typical stream environment. Therefore, sinuosity was not calculated for either river.

2.1.1.4 Approach Channel Slope

Based on bathymetric data, The Weweantic River channel slope was measured to be 0.0027 feet per foot through Bridge W06016 and the Sippican River channel slope was measured to be 0.0025 feet per foot through Bridge M01013. These calculations stopped short of and does not include the scour hole upstream of each bridge.

2.1.1.5 Topography

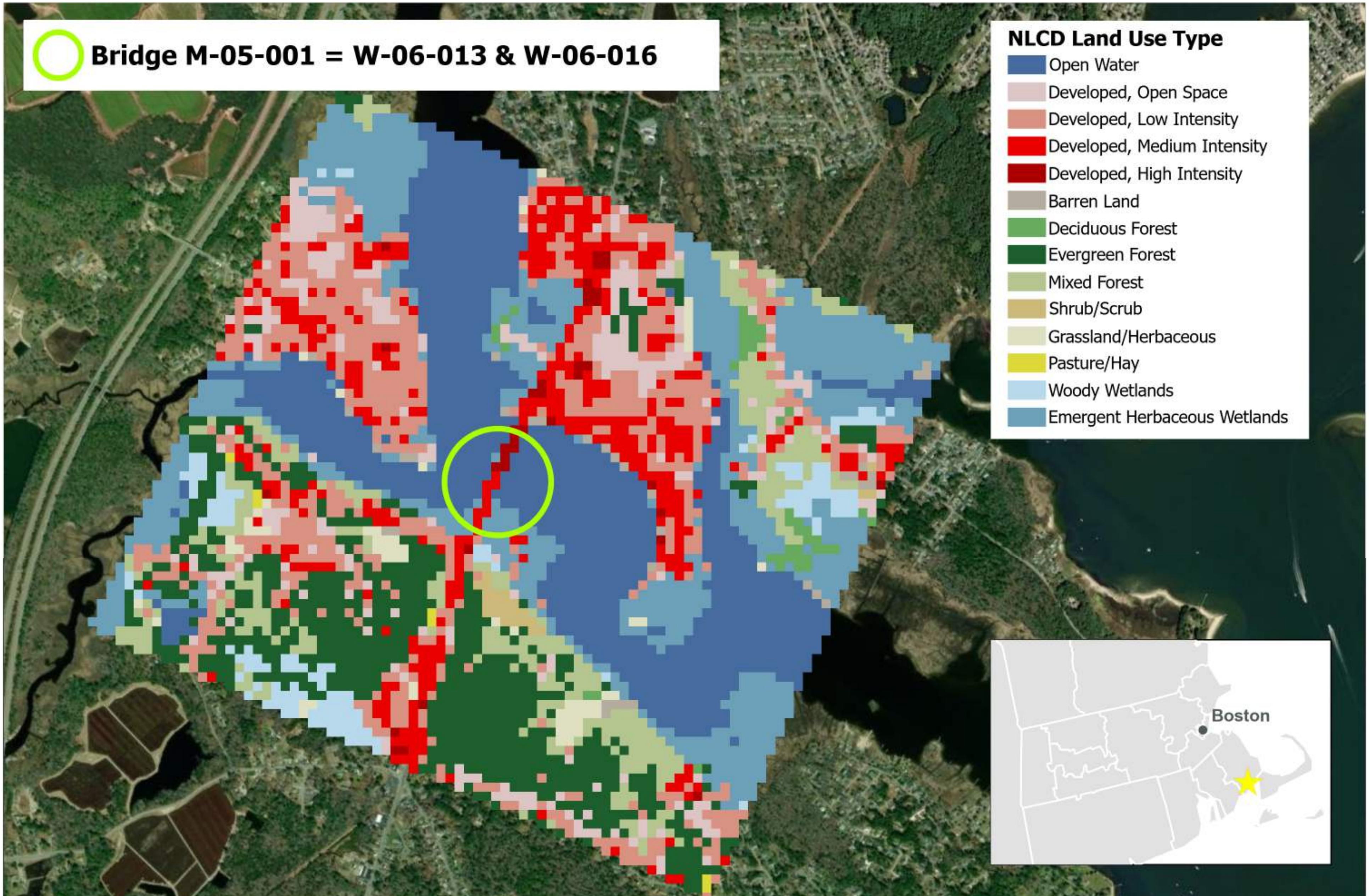
The topography of this region is mostly a low-lying coastal area. Compared to the rest of the state, this location has relatively low elevations and relief. The highest elevation in the watershed is +235.3 feet NAVD88. The Weweantic River's channel is below sea level.

2.1.2 Highway Conveyed

Route 6 is a part of the highway functional classification, Urban Minor Arterial.

2.1.3 Land Use in the Vicinity of the Bridges

In the immediate vicinity of the bridge, the area consists mostly of water bodies and wetlands (45%), developed areas (30%), and forests (22%).Figure 4 shows the land use types near Route 6 crossing Weweantic River. Residencies make up the majority of the developed areas near the bridges.



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LAND USE IN THE IMMEDIATE VICINITY OF THE BRIDGES
Figure 4

2.1.4 Special Site Considerations

The Federal Emergency Management Agency (FEMA) produced a Flood Insurance Study (FIS) for Plymouth County, Massachusetts. The current FIS Number containing the detailed studies of Plymouth County is 25023CV001D, 25023CV002D, 25023CV003D, 25023CV004D, and 25023CV005D. The Flood Insurance Rate Map (FIRM) Number 25023C0576K contains the area where Route 6 crosses the Weweantic River. The FIRM classifies this area of interest as Zone VE.

Zone VE is a part of a Special Flood Hazard Area (SFHA) subject to inundation by the 1% annual chance flood, also known as the base flood, that has a 1% chance of being equaled or exceeded in any given year. Zone VE is defined as a coastal flood zone with velocity hazard (wave action) with base flood elevations (BFE) determined.

The FIS does not use any riverine flows from Sippican River or Weweantic River. The elevations provided by the FEMA study are stillwater elevations derived from coastal flood analysis to determine base flood elevations, which includes extreme tides and storm surge with the addition of overland wave effects. The Weweantic River in the project area does not contain a regulatory floodway as defined by the National Flood Insurance Program (NFIP). According to the FIRM for the project area, the BFE at the bridges is between +18 and +19 feet (NAVD88), as shown in Figure 5. Table 4 shows the parameters for the Coastal Transect 167, just downstream of the bridge and the closest transect to the bridge, extracted from Table 4 of the FIS (25023CV002D). The table list the flooding source as Buzzards Bay.

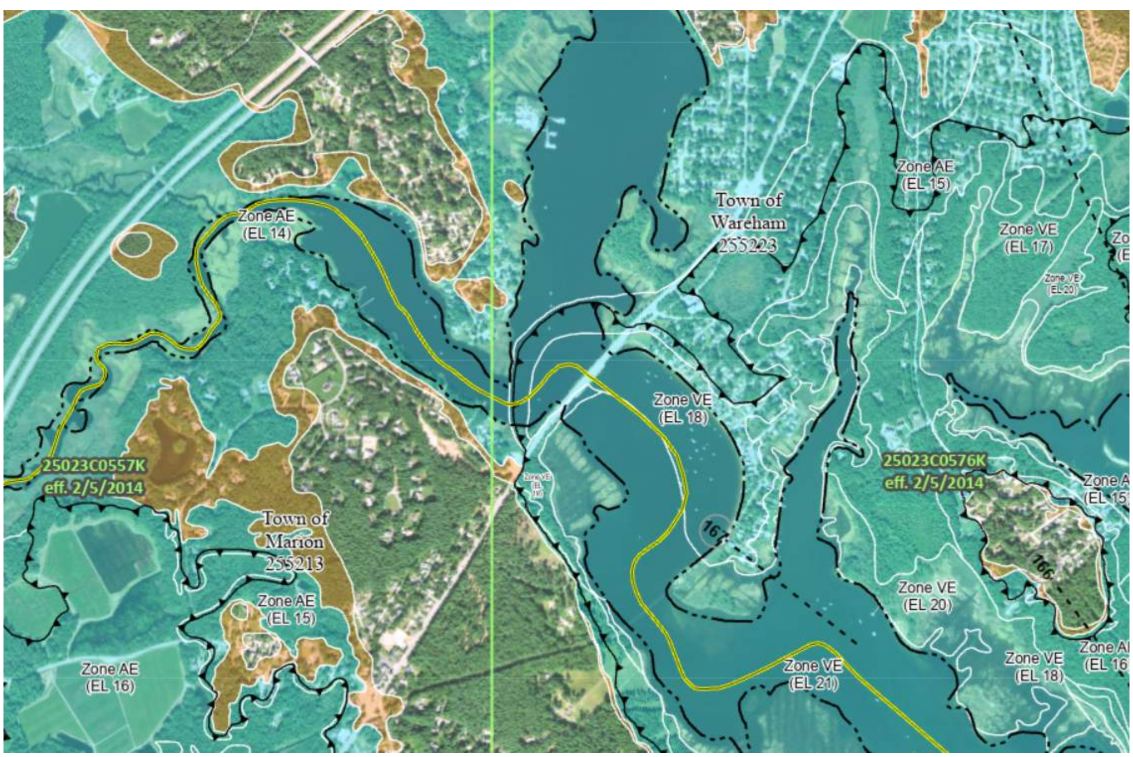

Figure 5. FEMA's FIRM for the Route 6 over Weweantic River

Table 4. Coastal Transect 167 Parameters

Flood Source	Coastal Transect	Starting Wave Conditions for the 1% Annual Chance		Starting Stillwater Elevations (ft NAVD88) Range of Stillwater Elevations (ft. NAVD88)				
		Significant Wave Height (ft)	Peak Wave Period (sec)	10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Buzzards Bay	167	20.57	*	+7.8	*	+11.8	+13.7	+17.7

**Not calculated for this Flood Risk Project*

2.2 Proposed Conditions

Based on the existing and future traffic volume projections for Route 6 roadway segment, the decision was made to not widen the Route 6 corridor to provide additional vehicular capacity. However, additional considerations include providing shoulders for extra space for emergency vehicle or stopped vehicles, and additional room for pedestrians and handrails that separate traffic from the sidewalks. Both proposed bridge designs call for four 11-foot lanes, a 4-foot shoulder on each side, and a 10-foot shared use path (SUP) on both the upstream and downstream sides of the bridge. The existing abutments and pier footings of the existing bridges will be cutoff and left in place.

Proposed Bridge M-05-001 = W-06-013 is two spans (one @ 74’ and one @ 62’) with one complex pier that consists of six, four-foot pile columns, two pile caps (one for every three columns), and six, five-foot drilled shafts. The bridge is designed to provide a vertical clearance of approximately 8-feet 4-inches above the mean high water surface elevation (MHW EL = +1.65’).

Proposed Bridge W-06-016 is three spans (one @ 56’, one @ 72’, and one @ 56’) with two complex piers that each consist of six, four-foot pile columns with two pile caps (one for every three columns), and six, five-foot drilled shafts. The bridge is designed to provide a vertical clearance of approximately 8-feet 6-inches above the mean high water surface elevation (MHW EL = +1.65’).

3 Data Collection

3.1 Elevation Data

Elevation data used in developing numerical models for the hydraulic study was obtained from several sources. The data used in the hydraulic analyses is shown in Figure 6 and is further discussed below. All data was obtained with or converted to the North American Vertical Datum of 1988 (NAVD88) in U.S. Survey Feet.

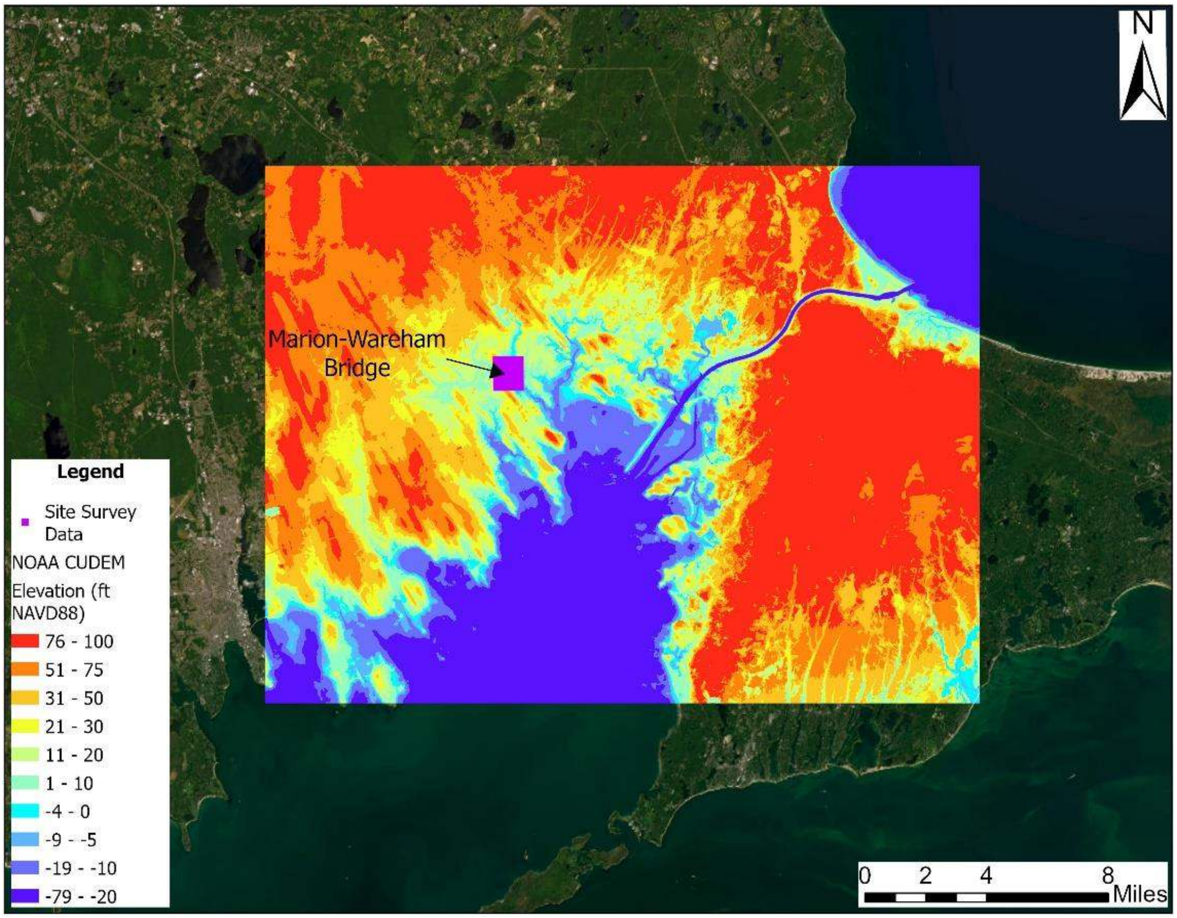

Figure 6. Project Elevation Data

3.1.1 NOAA CUDEM

The National Oceanic and Atmospheric Administration (NOAA) houses the Continuously Updated Digital Elevation Model (CUDEM) online, which is available for download to the public. The CUDEM is a ninth arc-second 9 (~3 meters) resolution bathymetric-topographic dataset and was used as the primary elevation data source, as it can be downloaded for large areas along the coast (CIRES, 2023).

3.1.2 Project Survey Data

Survey elevation data was collected for the roadway and bridge by Surveying and Mapping Consultants on 3/18/2018. Additional bathymetric survey was provided by MassDOT on 2/7/2024.

The existing elevation data at the bridges containing the bathymetric survey, additional LiDAR, and the survey of the roadway and causeway is shown in Figure 7.

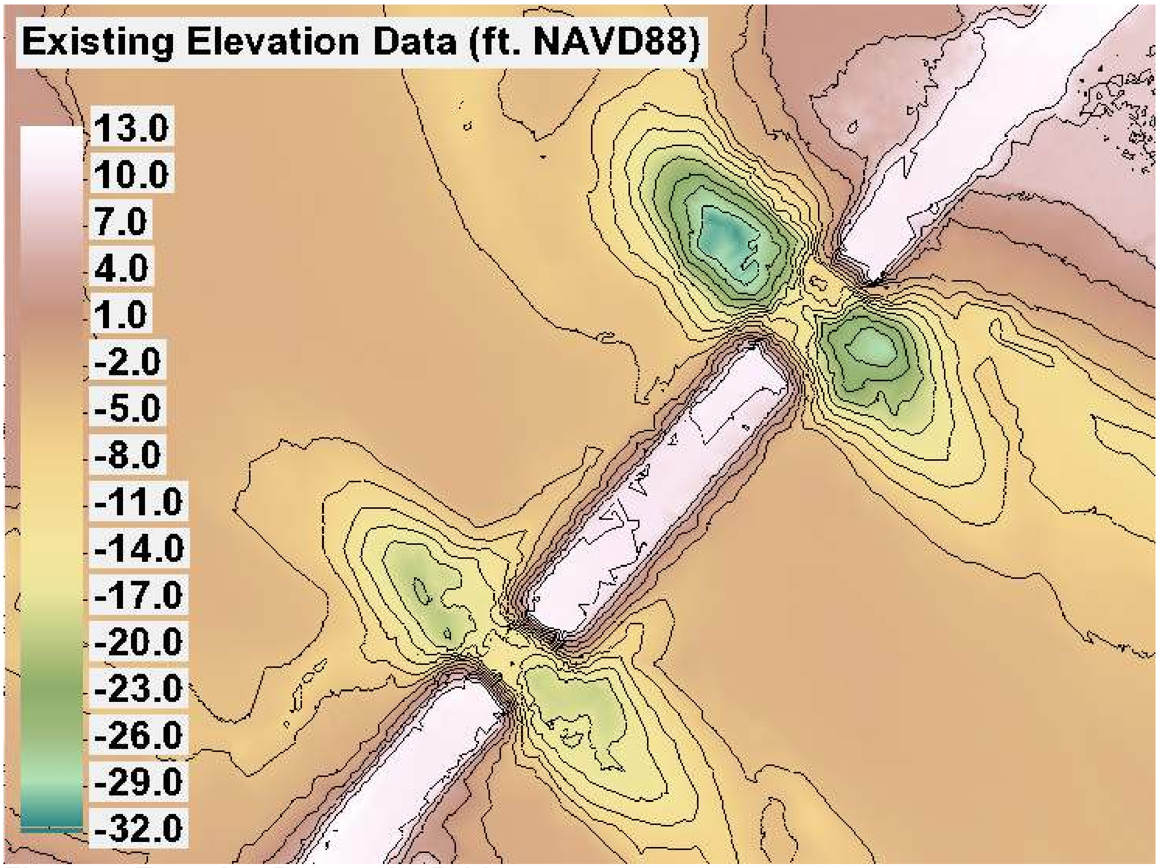

Figure 7. Elevation Data at the Bridges

3.2 Sediment Sampling Results

Sediment Sampling completed in 2018 by Haley Aldrich was provided to HDR. The median bed-material sediment particle size, the size larger than 50% of the sediment particles, also known as the D_{50}, from each sample taken at the bridges are listed in Table 5 and Table 6 below.

These D_{50} values were used to calculate an average D_{50} at each bridge. Certain soil samples were omitted from the calculation of the average D_{50} value because they appeared to be outliers and are likely not representative of the channel bottom. The average D_{50} at Bridge M-05-001 and Bridge W-06-016 was determined to be 0.5 mm and 0.4 mm, respectively.

A D_{50} of 0.4 mm was used in the scour calculations for both bridges in order to be conservative and consistent.

Table 5. Marion - Bridge M-05-001 Sediment Soil Sampling

Marion – Bridge M-05-001		
Location	**Depth of Sample**	**D50 (mm)**
Channel at the Upstream Face of the Bridge (BB-3)	0-2 feet	0.37
	4-6 feet	0.60
	8-10 feet	0.73
Channel at the Downstream Face of the Bridge (BB-4)	0-2 feet	2.3 *
	4-6 feet	1.7 *
	10-11.7 feet	1.05 *
Upstream of the Channel (SS-1)	3-9 inches	0.37
Marion Overbank	0-12 inches	0.43
Average D50 =		**0.50 mm**
**not included in the average D50*		

Table 6. Wareham - Bridge W-06-016 Sediment Soil Sampling

Wareham – Bridge W-06-016		
Location	**Depth of Sample**	**D50 (mm)**
Channel at the Upstream Face of the Bridge (BB-9)	0 - 2 feet	0.90
	4.5 - 6.5 feet	4.75 *
	6.5 - 8.5 feet	0.02
	12.5 - 14.5 feet	0.01
Channel at the Downstream Face of the Bridge (BB-10)	0 - 2 feet	5.00 *
	4 - 6 feet	0.43
	8 - 10 feet	0.73
Upstream of the Channel (SS-2)	3 - 9 inches	0.50
Wareham Overbank	0 - 1 feet	0.22
Average D50 =		**0.40 mm**
***not included in the average D50**		

4 Engineering Methods

4.1 Hydrologic Analysis

The project's drainage basin was not studied in the NFIP, and a large percentage of the watershed falls in an undefined region of USGS's website application, StreamStats.

The USGS Scientific Investigations Report (SIR) 2016-5156 was used as a guide to calculate the peak flood discharges. Table 7 shows the variables that were used to calculate the peak flood discharges using equations from the SIR. Table 8 shows the peak flood discharges for the 10-, 25-, 50-, 100-, 200-, and 500-year return frequencies for riverine floods.

Table 7. Basin Characteristics

Basin Characteristic	Value
Drainage Area, sq. miles	89.53
Storage, % of basin area consisting of open water and wetlands	25.13
Mean Basin Elevation, feet (NAVD88)	+83.90

Table 8. Summary of Peak Flood Discharges

Total Flow	Drainage Area (sq. miles)	10% Annual Chance [10-yr] (cfs)	4% Annual Chance [25-yr] (cfs)	2% Annual Chance [50-yr] (cfs)	1% Annual Chance [100-yr] (cfs)	0.5% Annual Chance [200-yr] (cfs)	0.2% Annual Chance [500-yr] (cfs)
Marion-Wareham	89.53	1,698	2,217	2,639	3,074	3,538	4,190

4.2 Relative Sea Level Rise Analysis

Relative sea level rise (RSLR) was assessed using the 2022 Sea Level Rise Technical Report from NOAA. NASA developed a tool for viewing the results developed in the NOAA report, which was applied to assess Relative Sea Level Rise at the Woods Hole, MA gauge. The results from the sea level rise viewer are shown in Figure 8.

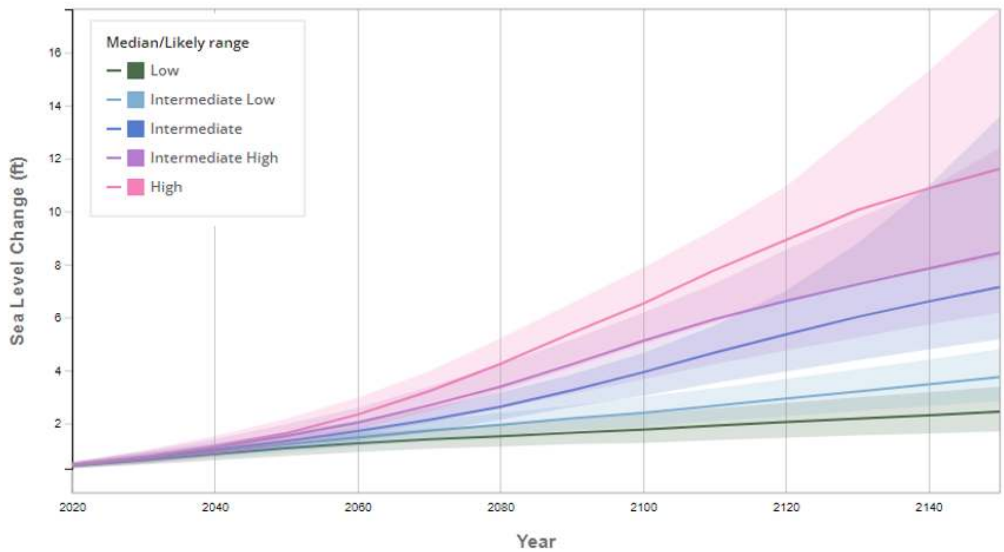

Figure 8. Relative Sea Level Change at Woods Hole, MA

The figure shows the relative sea level change for five scenarios. The five scenarios relate to the risk for each curve, with “high” representing an extreme case (lower risk). Based on discussions with MassDOT during a design meeting on August 8, 2023, for the Dennis Yarmouth Bridge Project, the project team agreed to assume an “intermediate high” risk level at year 2100 for the project analysis, which equates to 5.11 feet of RSLR.

4.3 Hydraulic Analysis

4.3.1 Riverine Hydrodynamic Analysis

A two-dimensional (2D) hydrodynamic numerical model was created to perform a hydraulic analysis of the project site to determine water depths and velocities for scour evaluation. The SRH-2D (Sediment and River Hydraulics 2D model) software, version 3.6.5, was chosen for this evaluation because it contains a very robust solving routine and is a currently accepted standard 2D modeling software for bridges. This software is recommended by the Federal Highway Administration (FHWA) for 2D hydraulic bridge analysis. The results were analyzed using Aquaveo’s Surface Water Modeling Software (SMS), version 13.3.10.

4.3.1.1 Horizontal and Vertical Datum

All riverine modeling was conducted in a horizontal datum of NAVD83 (2011) State Plane, Massachusetts, in units of U.S. survey feet, and a vertical datum of NAVD88 in units of U.S. survey feet.

4.3.1.2 Elevation Data

The elevation data used in the riverine hydrodynamic model consists of the CUDEM downloaded from NOAA merged with the survey data provided, as discussed in Section 3.1 above. The SRH-2D model utilized two sets of elevation data: one for the existing conditions and another for the proposed conditions.

4.3.1.2.1 Existing Conditions

The dataset depicting existing conditions underwent modifications to accurately portray the pier footings at the existing bridges. This adjustment was implemented specifically where the bathymetric data fell short of encompassing the top of the pier footing. Figure 9 shows the existing elevation data at the bridges.,

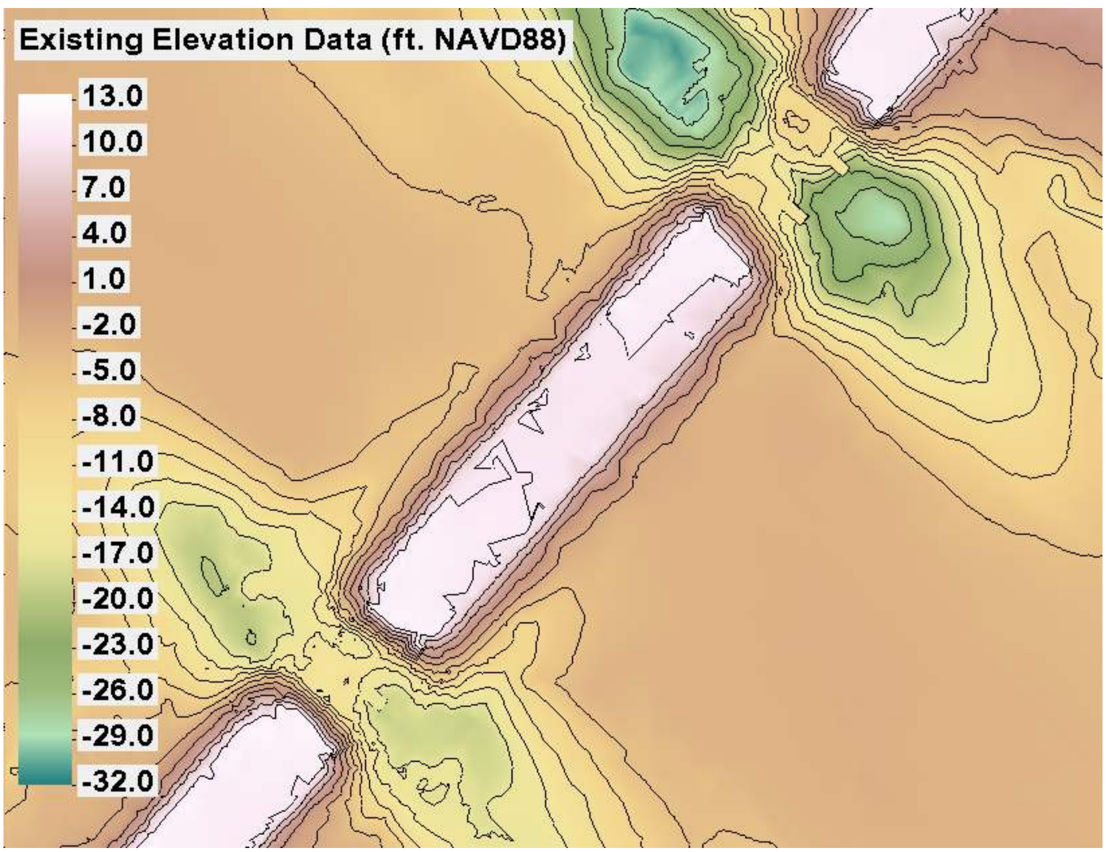

Figure 9. SRH-2D Existing Conditions Elevation Data

4.3.1.2.2 Proposed Conditions

The elevation data for the proposed conditions was altered to represent the intended finished grade, proposed abutments, proposed concrete retaining walls, the projected elevation for the top of the riprap, and the projected bank slopes underneath the bridge. Figure 10 shows the proposed elevation data at the bridges.

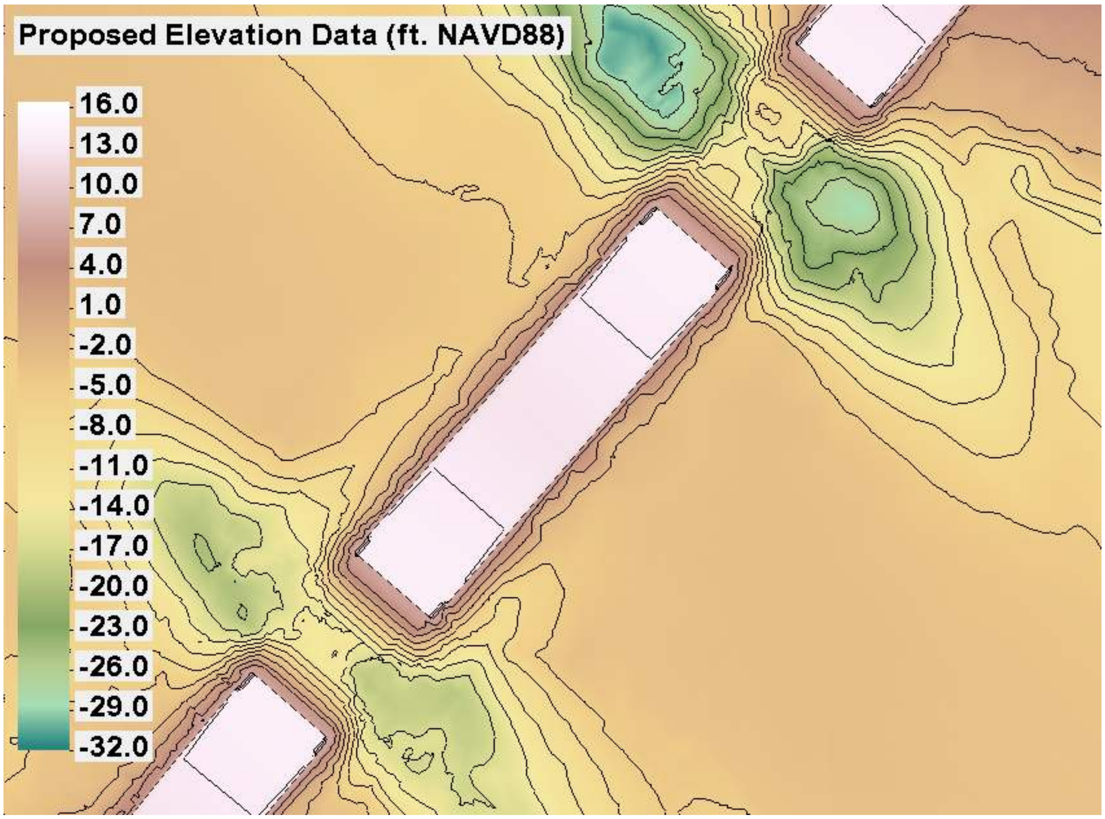

Figure 10. SRH-2D Proposed Conditions Elevation Data

4.3.1.3 Model Domain

The first step in the 2D model development was determining the model domain, i.e., the spatial extent of the modeled area. The model domain should be large enough that the boundary conditions applied to the model are far enough away from the area of interest that any anomalies, which may be caused by the boundary conditions, will not influence model results in the area of interest. This can occur because the model results, such as flow distributions, velocities, and water surface elevations at the boundaries, may be inaccurate. Placing the model boundaries away from the area of interest enables these irregularities to be worked out by the model from the area of interest. The extent of the domain's limits is shown in Figure 11.

Figure 11. SRH-2D Model Domain Limits

4.3.1.4 Land Use and Roughness Coefficients

Land use was analyzed to select roughness coefficients for the hydraulic analysis. Land use was determined from aerial imagery and the National Landcover Database (NLCD). The land use types and corresponding roughness coefficients, Manning's n values, are listed in Table 9. Figure 12 is a map detailing the limits of each land use type in the model domain for the existing conditions. The existing materials coverage was duplicated and altered at the bridge site to represent the proposed conditions. Figure 13 is a map detailing the land use type at the bridges for the proposed conditions based on the proposed bridge plans.

Table 9. Land Use Type Roughness Coefficient

Land Use Type	Roughness Coefficient
Channel / Open Water	0.035
Wetlands	0.065
Road / Parking Lots	0.02
Developed Residential Areas	0.1
Buildings	0.18
Trees	0.15
Grass	0.045
Riprap Bank Protection	0.06

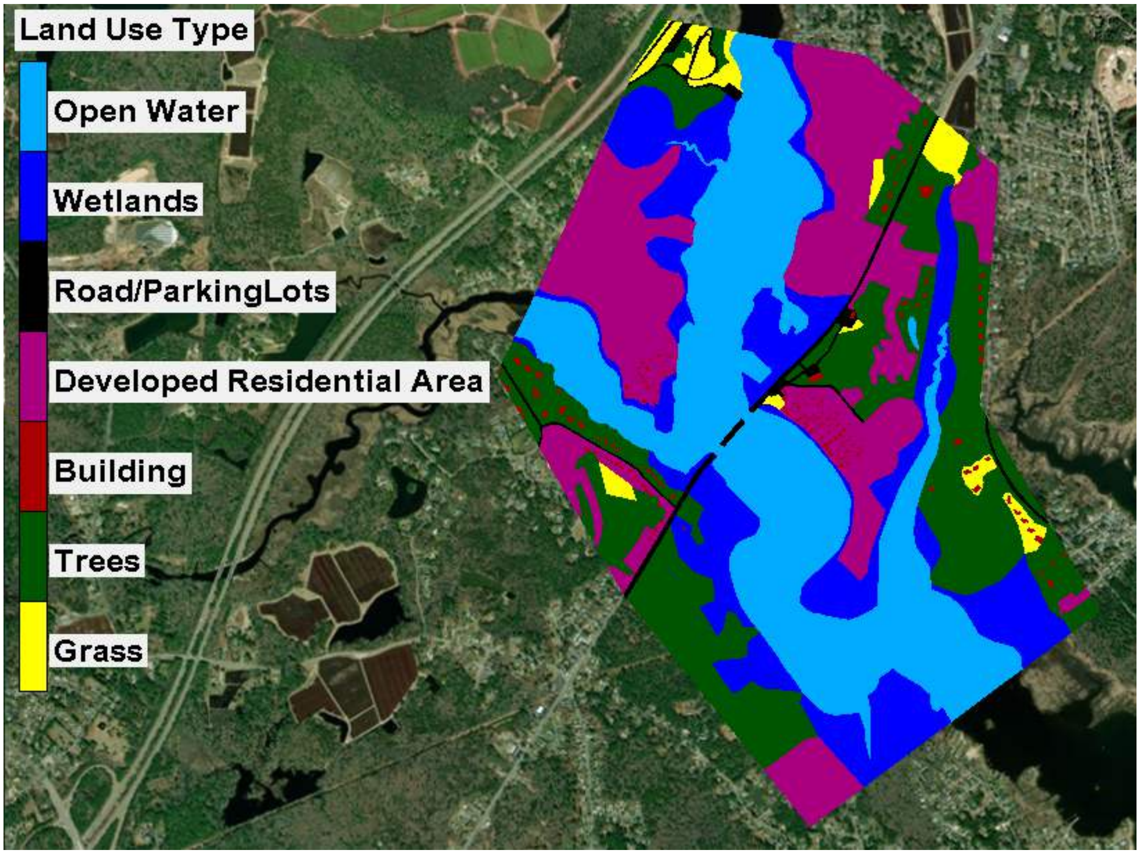

Figure 12. Land Use Type Map in the Model Domain for the Existing Conditions

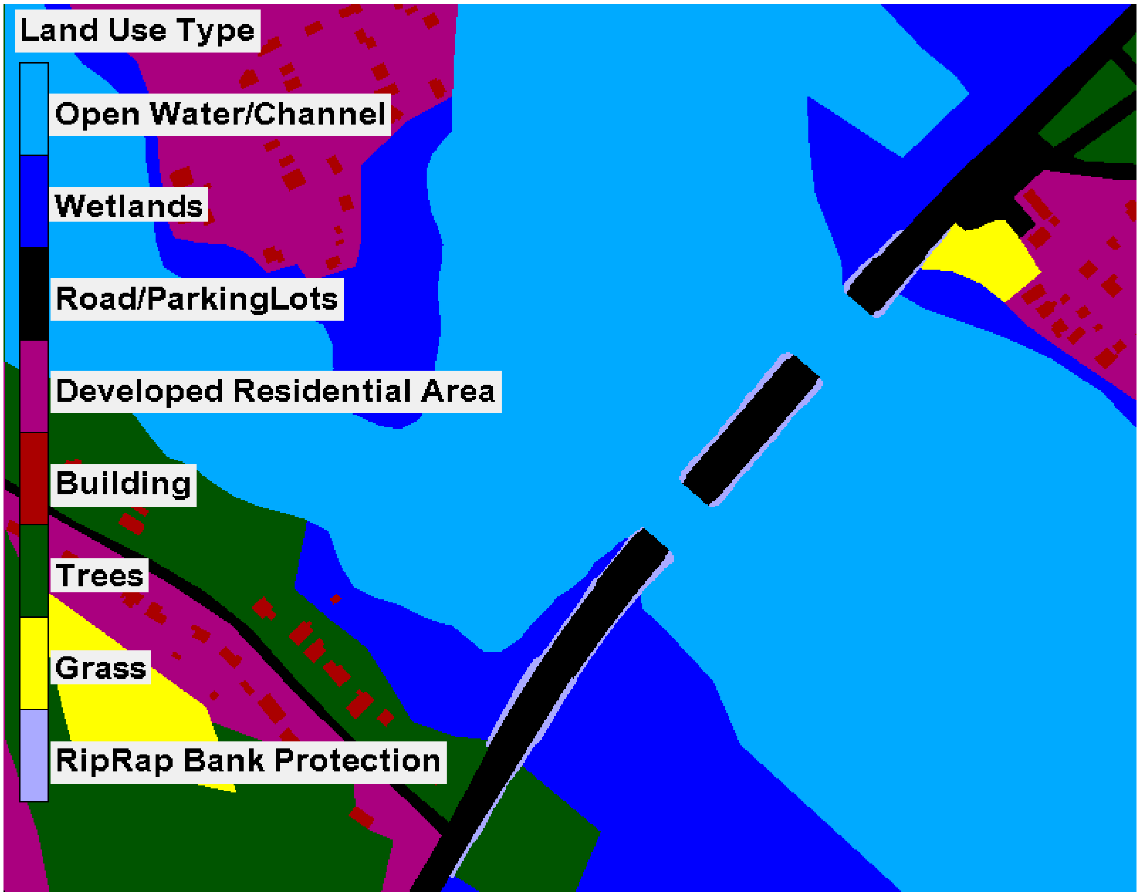

Figure 13. Land Use Type Map at the Bridges for the Proposed Conditions

4.3.1.5 Riverine Model Boundary Conditions

The riverine SRH-2D model consists of two inflow boundary condition locations, the Weweantic River and the Sippican River, and one exit boundary condition into Buzzard's Bay. The locations of these model boundary conditions are shown in Figure 14 .

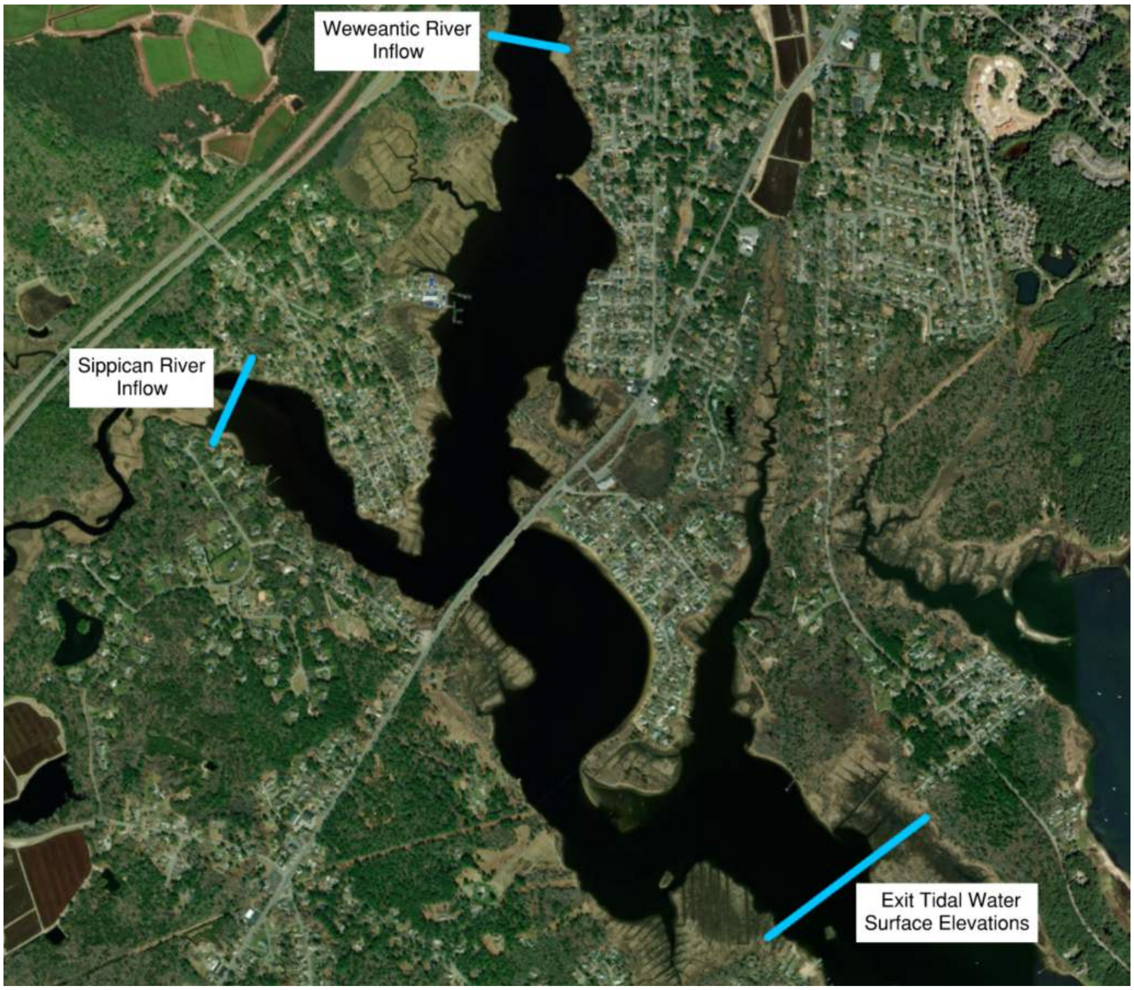

Figure 14. Riverine Model Boundary Conditions

4.3.1.5.1 Inflow Boundary Conditions

The inflow of the riverine SRH-2D model was controlled by two inflow boundary conditions: Sippican River Inflow and Weweantic River Inflow. The Sippican River Inflow and Weweantic River inflow boundary conditions (Inlet-Q in SRH-2D terminology) was established approximately 0.6 and 1 stream miles (following the meandering channel) upstream of the crossing, respectively.

The hydrology computed in Section 4.1 above was split into the Sippican River Inflow and the Weweantic River Inflow based on the ratio of each river's drainage area to total drainage area. The Sippican River and Weweantic River Inflows are listed in Table 10.

Table 10. Riverine SRH-2D Model Inflows

AEP	Year	Sippican River Inflow (cfs)	Weweantic River Inflow (cfs)
50 %	2	289	550
20 %	5	458	870
10 %	10	585	1,113
4 %	25	764	1,453
2 %	50	910	1,730
1 %	100	1,059	2,014
0.5 %	200	1,219	2,318
0.2 %	500	1,444	2,746

4.3.1.5.2 Outflow Boundary Conditions

The outflow boundary was established approximately 1.1 stream miles downstream of the bridge crossings. A time series boundary condition (EXIT-H in SRH-2D terminology) was applied at the outflow boundary condition. Tidal water levels were used as the exit boundary condition for all AEP floods due to the bridges' location in the coastal area. The tidal water levels used in the exit boundary condition are from January 27 - January 31, 2024, at Woods Hole (8447930). These dates were chosen to represent recent, normal tidal conditions. This information was found on NOAA's tides and currents website. The tidal data used as the outlet in the SRH-2D riverine model is shown in Figure 15.

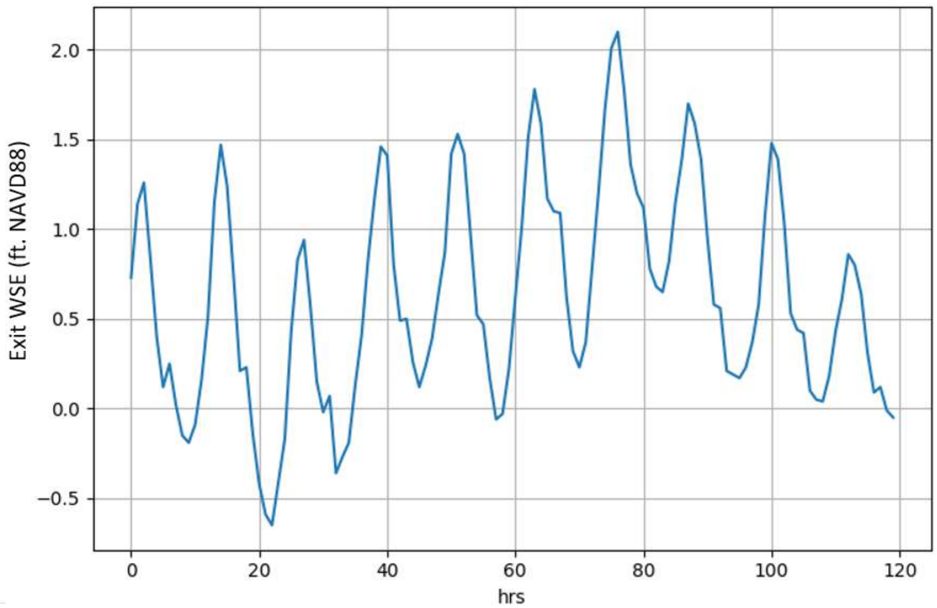

Figure 15. Riverine SRH-2D Model Outlet Tidal Water Surface Elevations

4.3.1.6 Mesh Development

The next step in the 2D model development was the creation of two model meshes, one for the existing conditions and one for the proposed conditions. The mesh is not only the representation of the terrain surface used by SRH-2D, but also the framework for all the model calculations. Therefore, the model mesh is critically important to the success of the 2D model.

The meshes were constructed so that the elements are larger in the fringes of the floodplain and in higher elevations that are less likely to be wet during the model simulations. The element density is generally the greatest at bridge openings. A denser mesh (more nodes and elements) will lead to longer model runtimes, so the mesh should be dense enough to produce acceptable results but not so dense to cause excessive runtimes.

Stream channels are generally represented with quadrilateral elements that are usually elongated in the direction of flow along the channels. Likewise, roadway embankments are generally represented by quadrilateral elements, except adjacent to bridge openings. The rest of the mesh is composed of triangular elements. Arcs, which function as breaklines in the mesh, were drawn along the thalwegs of smaller channels to ensure that the channels were represented in the mesh. Arcs were also used to define significant breaks in topography, and to adjust the mesh density.

It is good practice to use the SMS Mesh Quality display option to identify possible mesh problem areas related to element interior angles, element connectivity, and adjacent element size differential. An effort was made to resolve mesh issues related to element interior angles and element connectivity. Adjacent element size differentials often occur at the interface between rectangular patches and adjacent triangles, as well as near bridge foundation elements, so these issues were generally discounted, as experience has shown that they rarely cause problems with model simulations.

As mesh development progressed, it was checked for fidelity to the elevation data by overlaying the contour lines over the mesh contour lines. There will necessarily be some loss of detail of the mesh relative to the scatter data, since the mesh will typically be significantly less dense than the underlying topography, most of which is based on the LiDAR data. However, it is important to evaluate whether the mesh adequately depicts significant topographic features that affect the flow.

Voids are used in the mesh in order to represent areas where there will be no flow, which is common practice for an SRH-2D model. Voids were used to represent the sub-structure and the buildings that are located in the additional LiDAR that was provided due to the voids in the LiDAR dataset.

4.3.1.6.1 Existing Mesh

The existing mesh was used to run the simulations for the existing conditions. This mesh uses the existing elevation data, as discussed in Section 4.3.1.2.1. The existing mesh contained a total of 22,047 elements: 13,537 triangular elements and 8,510 quadrilateral elements. Figure 16 the elevation contours of the mesh. Figure 17 shows the mesh elevations and elements for the entire domain. Figure 18 shows the meshing and elevation through the bridge opening.

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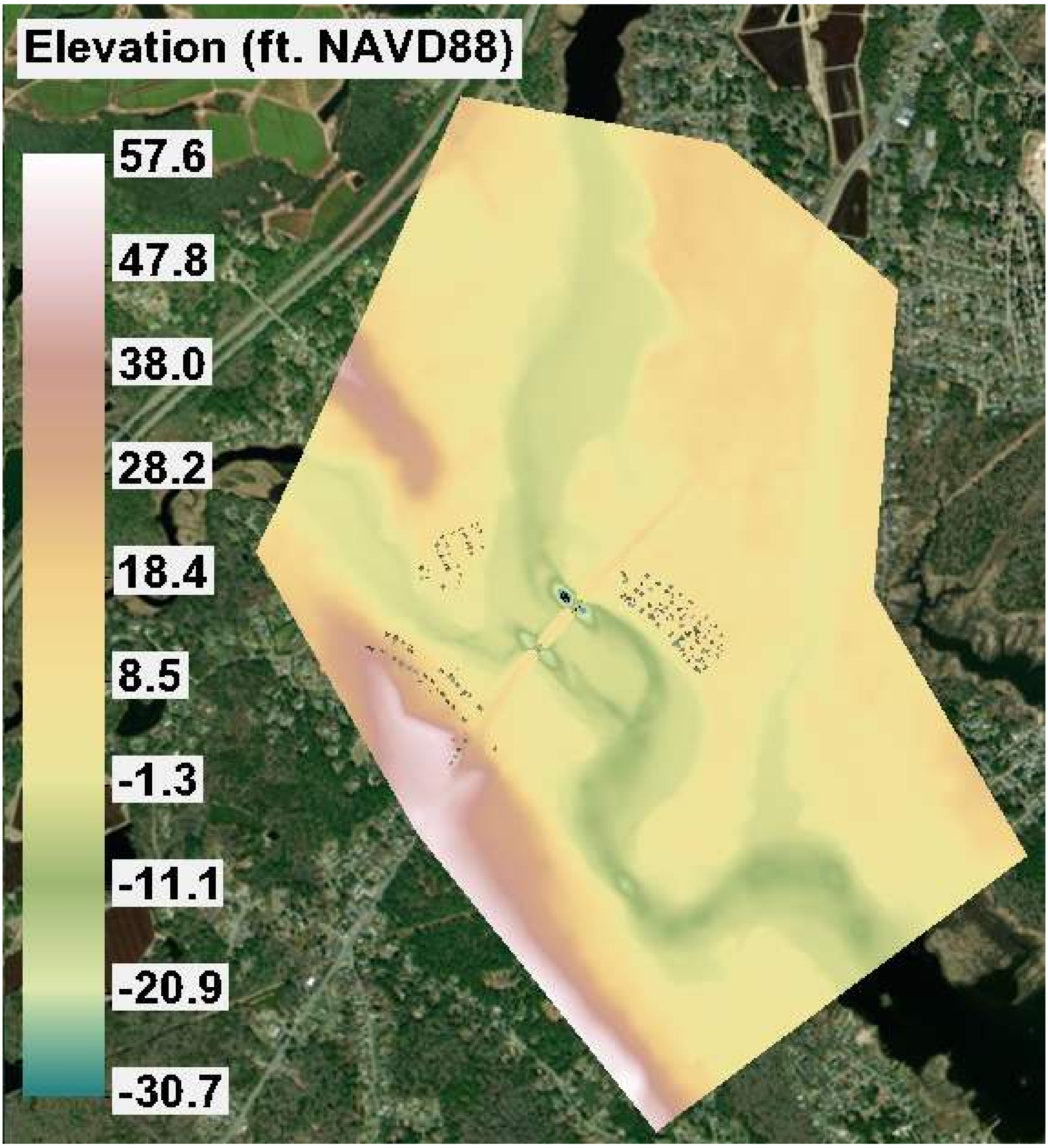

Figure 16. SRH-2D Existing Mesh Elevation

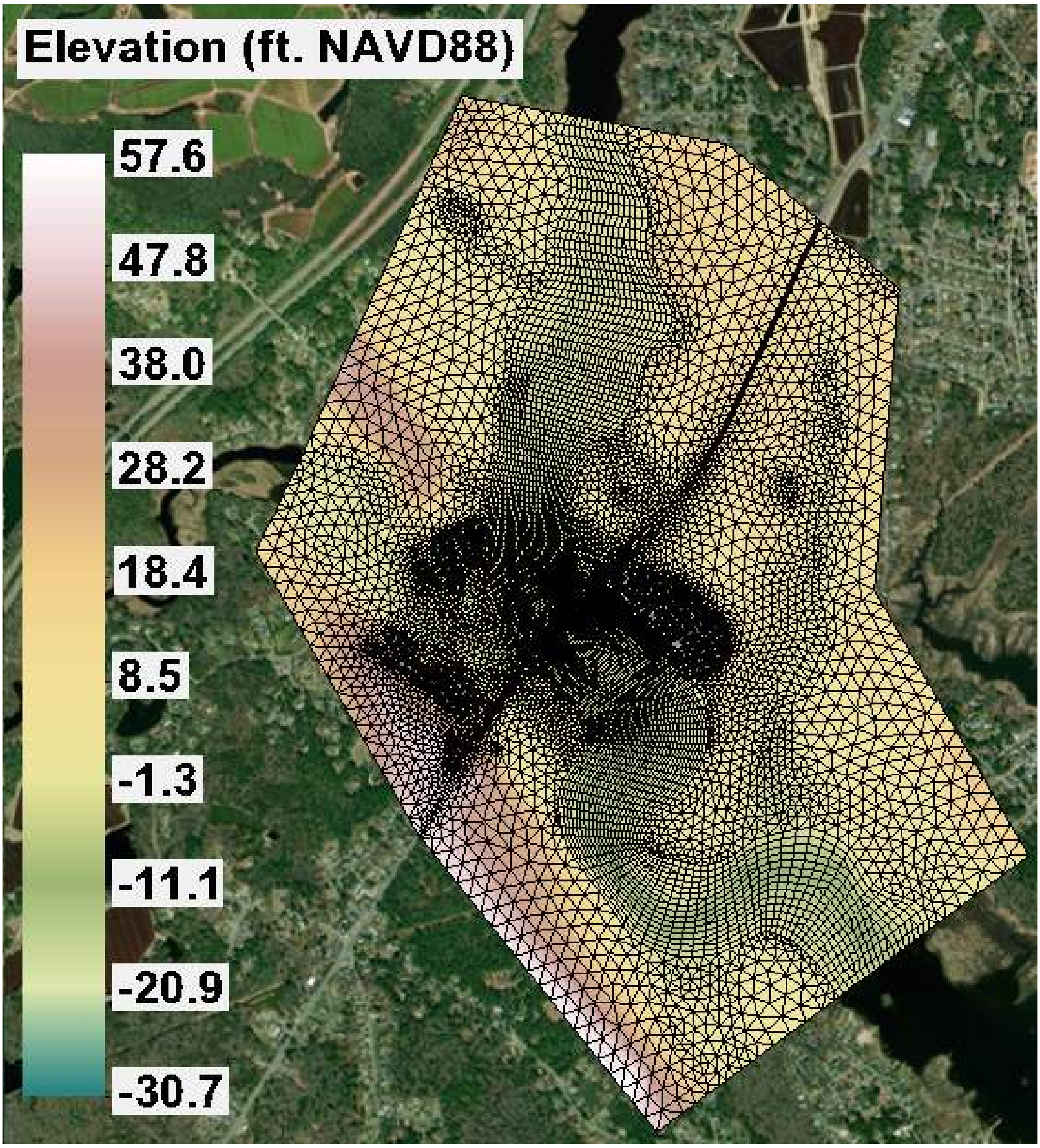

Figure 17. SRH-2D Existing Mesh Elevation Data with Elements

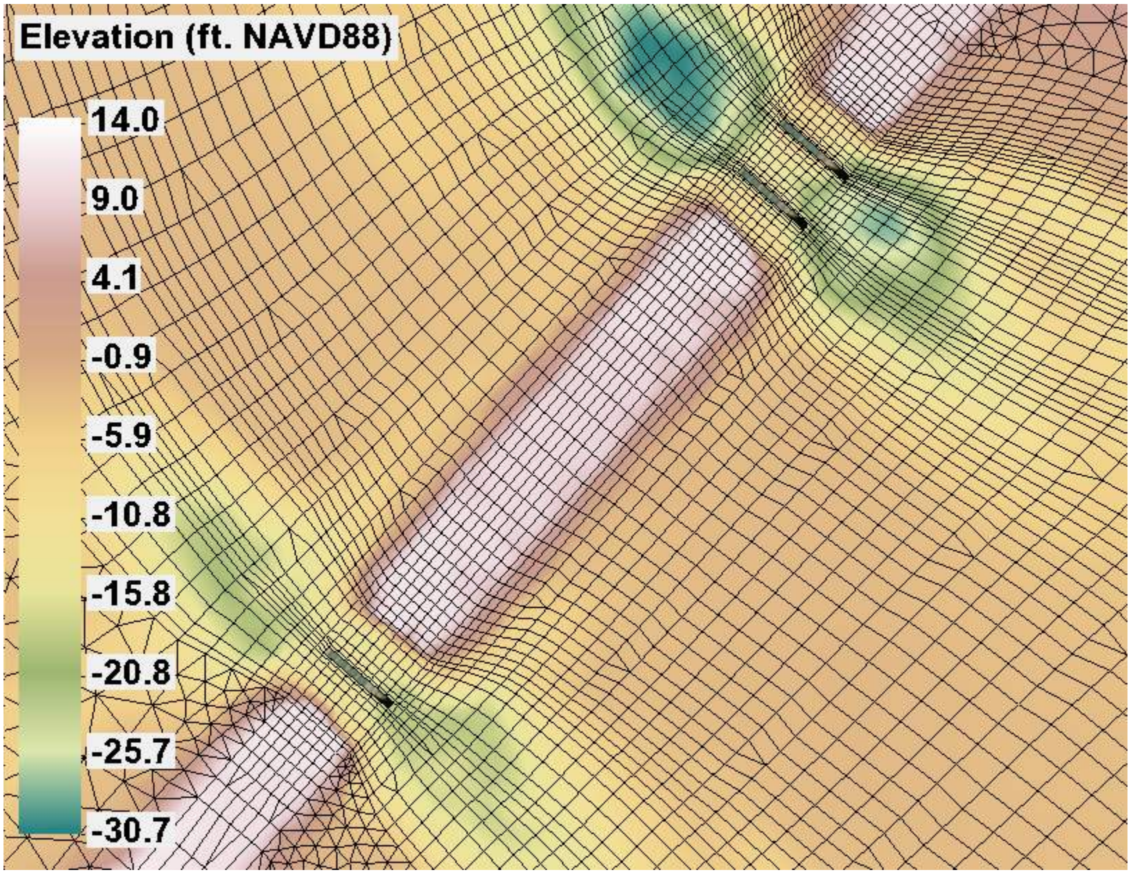

Figure 18. SRH-2D Existing Mesh Through the Bridge Openings

4.3.1.6.2 Proposed Mesh

The proposed mesh was used to run the simulations for the proposed conditions. This mesh uses the proposed elevation data, as discussed in Section 4.3.1.2.2 . The proposed mesh contained a total of 20,484 elements: 11,906 triangular elements and 8,578 quadrilateral elements. Figure 19 shows the elevation contours of the mesh. Figure 20 shows the mesh elevations and elements for the entire domain. Figure 21 shows the meshing and elevation through the bridge opening.

For the proposed mesh, 3D structures were used to represent the proposed bridges. The 3D structure coverage is a new feature in SRH-2D that gives a better visualization of a structure and generates plots of the water-surface elevations and flows at the structure. The plots show the high chord of the bridge, the water-surface elevation at the bridge, and displays how much flow that overtops the bridge, if applicable.

The bridge's stationing, finished grade, or high chord, low chord, bridge width, and substructure is input into the 3D structure coverage. The 3D structure coverage is incorporated into the mesh generator and the simulations.

Due to the limitations with the new 3D structure coverage, the piers were represented as one long pier, rather than six individual columns. Proposed Bridge M-05-001 and proposed Bridge W-06-016 are shown in Figure 22.

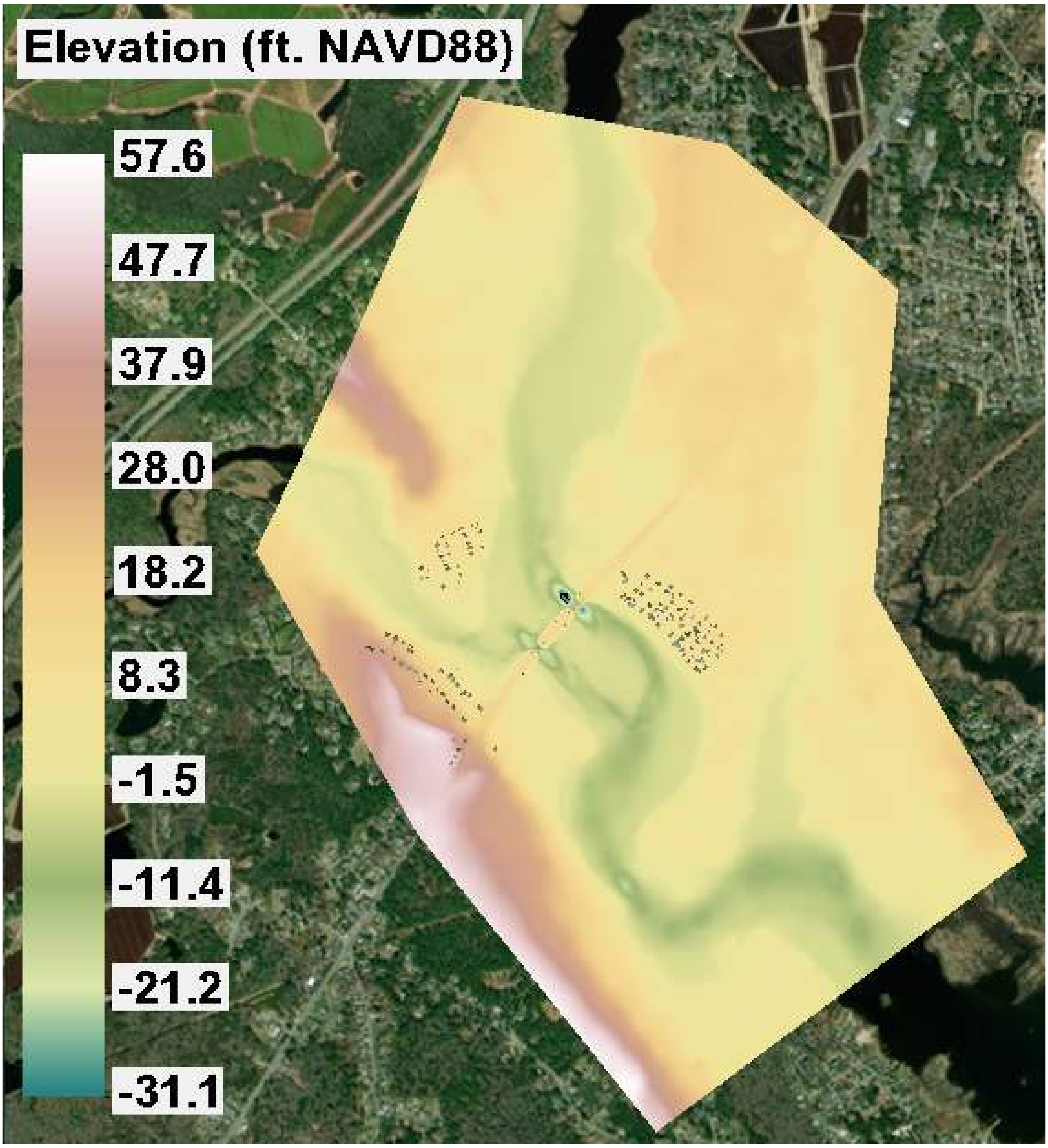

Figure 19. SRH-2D Proposed Mesh Elevation

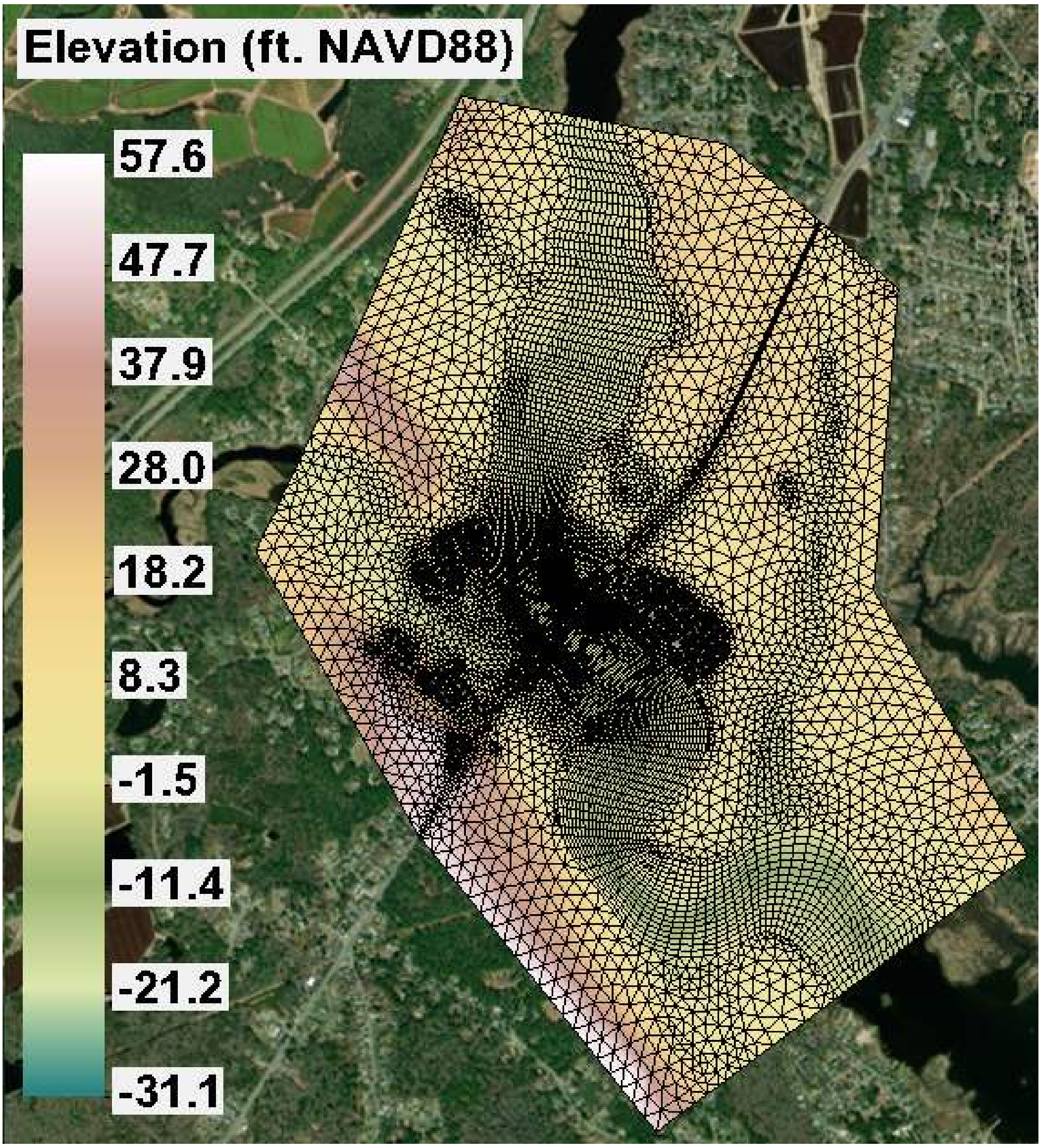

Figure 20. SRH-2D Proposed Mesh Elevation Data with Elements

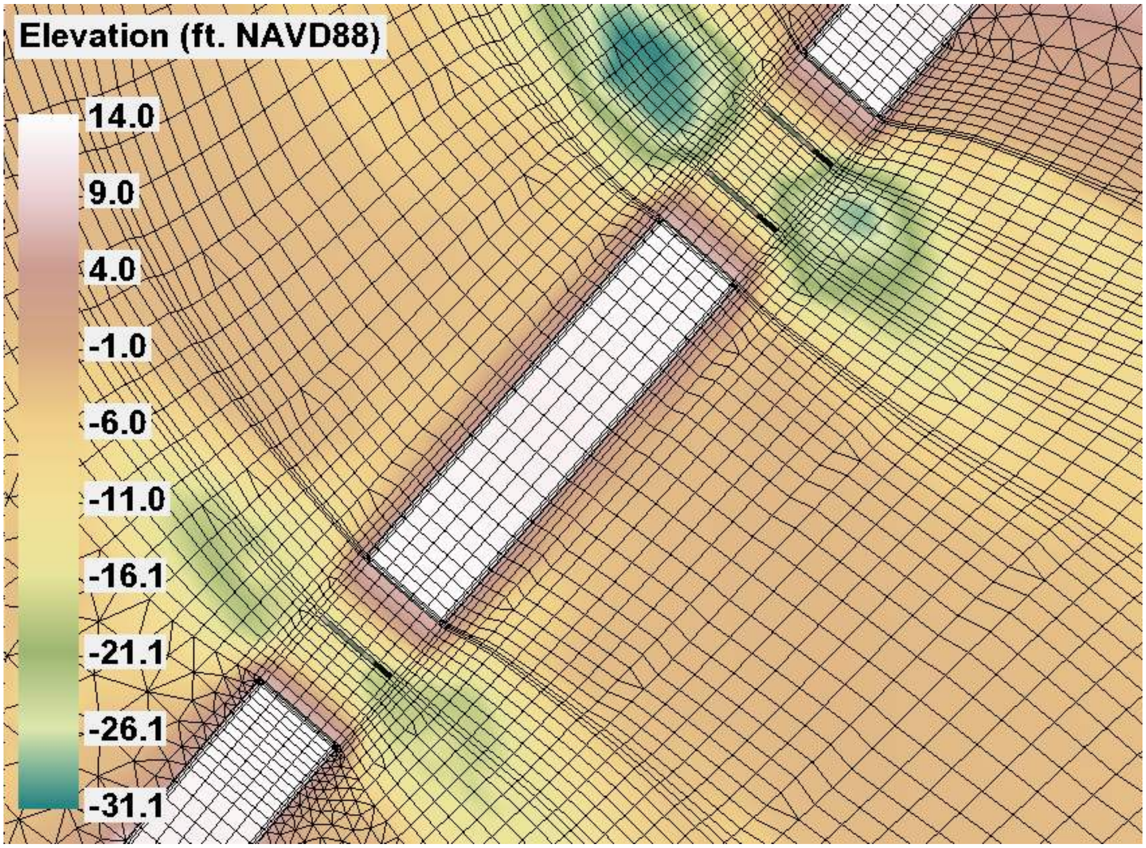

Figure 21. SRH-2D Proposed Mesh Through the Bridge Openings

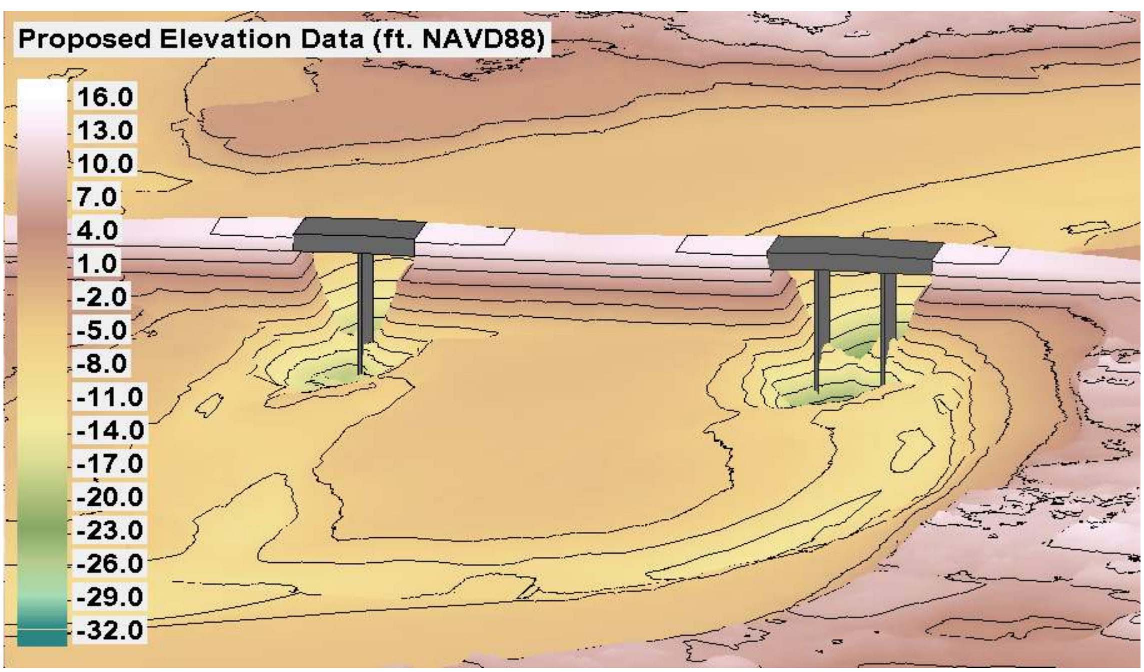

Figure 22. Proposed 3D Structures

4.3.1.7 Model Sensitivity Analysis

A model sensitivity analysis was run on the existing 50-year riverine simulation. The manning's n value of the channel was increased and decreased by 25% (+0.00875 and -0.00875) in order to determine if the model is sensitive based on the manning's n value of the channel. The maximum WSE at the upstream face (north side) of each bridge produced from changing the channel manning's n value are displayed in Table 11 below.

Table 11. SRH-2D Sensitivity Analysis - Channel Manning's n Value

Max WSE at the Upstream Face of the Bridge (ft. NAVD88)			
Bridge No.	Decreased Channel n = 0.026	Channel n = 0.035	Increased Channel n = 0.044
Bridge M-05-001	2.13	2.14	2.16
Bridge W-06-016	2.13	2.14	2.16

The sensitivity analysis shows the WSE at the bridge is not significantly affected when the model's manning's n value of the channel is changed. Therefore, the model was deemed to not be sensitive and is not dependent on the channel manning's n value.

4.3.1.8 SRH-2D Riverine Model Results

The maximum water surface elevations (WSEs) were extracted at the upstream face of the bridge (the north side) and the maximum velocities were extracted from the bridges. The maximum water-surface elevations and maximum velocities for the existing conditions are listed in Table 12. The 25-Year, 50-Year and 100-Year Riverine Velocities at the Existing Bridges are shown in Figure 23, Figure 24, Figure 25, respectively. The maximum water-surface elevations and maximum velocities for the proposed conditions are listed in Table 13. The 25-Year, 50-Yea,r and 100-Year Riverine Velocities at the Proposed Bridges are shown in Figure 26, Figure 27, Figure 28, respectively.

Table 12. SRH-2D Riverine Hydraulic Analysis for the Existing Conditions

AEP	Year	Peak Flow (cfs)	Existing Bridge M-05-001		Existing Bridge W-06-016	
			Max Water-Surface Elevation (ft. NAVD88)	Max Velocity at the Bridge (ft/sec)	Max Water-Surface Elevation (ft. NAVD88)	Max Velocity at the Bridge (ft/sec)
10%	10	1,698	2.11	1.38	2.11	1.54
4%	25	2,217	2.13	1.62	2.13	1.82
2%	50	2,639	2.14	1.82	2.14	2.04
1%	100	3,074	2.16	2.03	2.16	2.27
0.2%	500	4,190	2.22	2.55	2.22	2.87

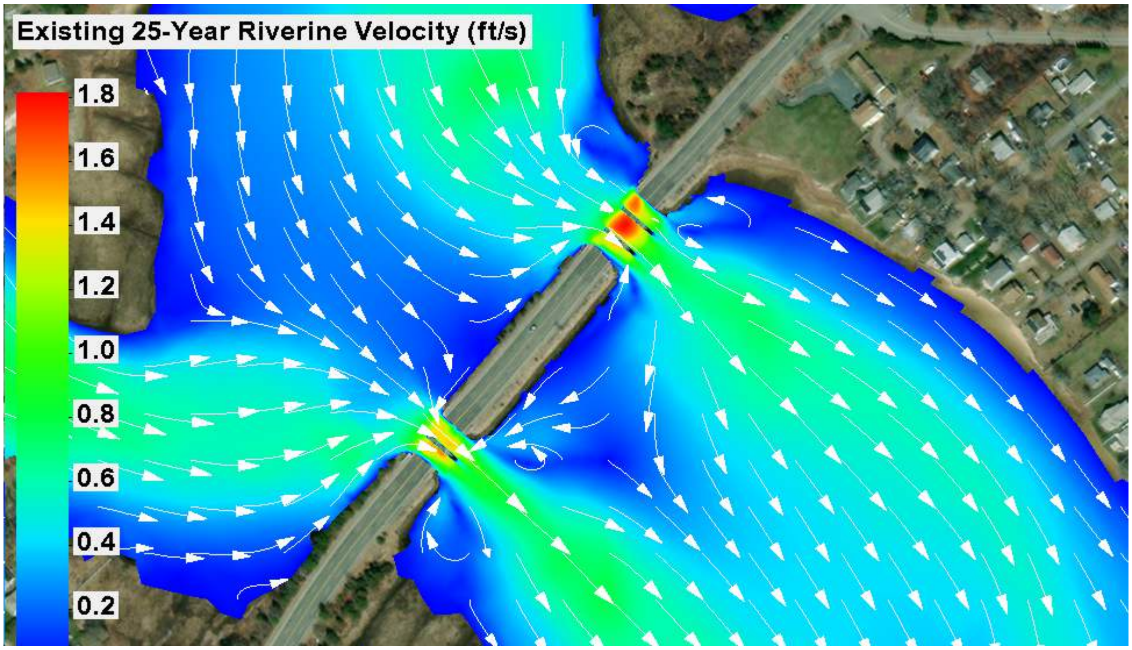

Figure 23. Existing 25-Year Riverine Maximum Velocities

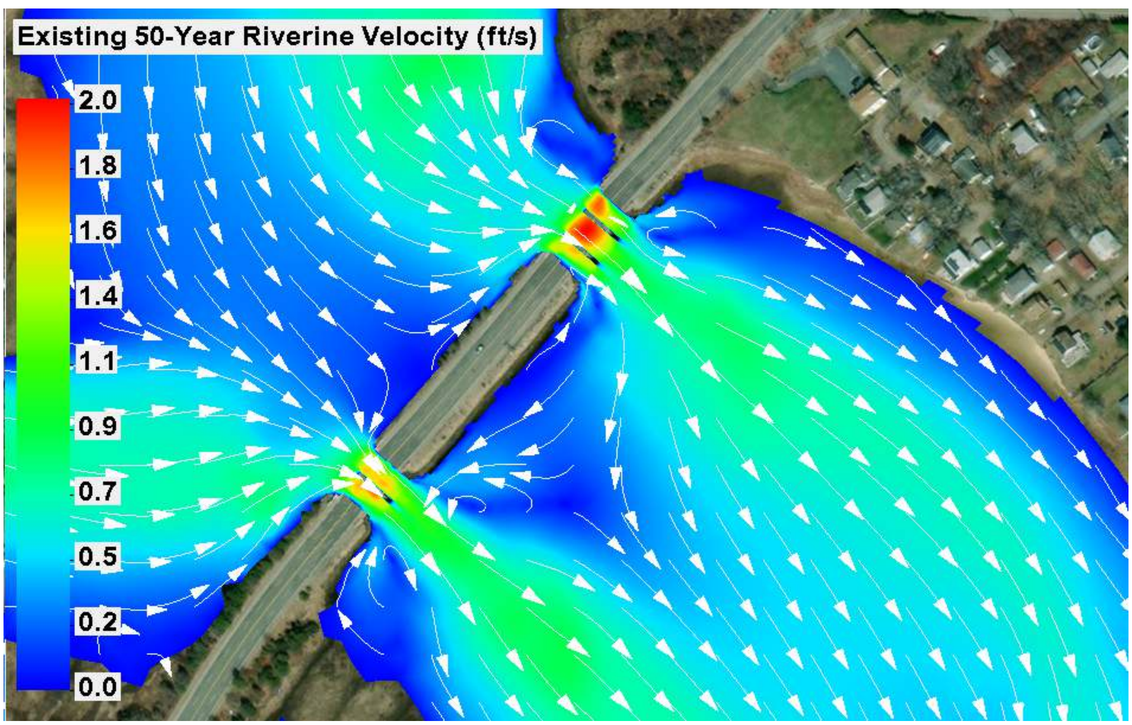

Figure 24. Existing 50-Year Riverine Maximum Velocities

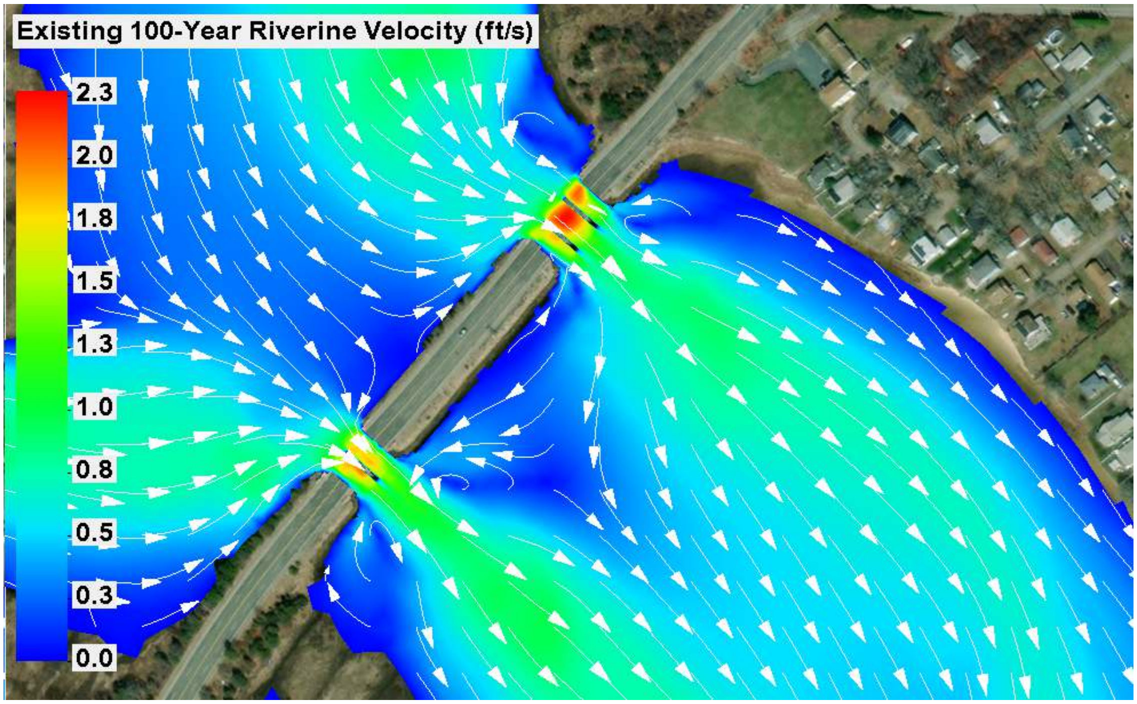

Figure 25. Existing 100-Year Riverine Maximum Velocities

Table 13. SRH-2D Riverine Hydraulic Analysis for the Proposed Conditions

AEP (%)	Year	Peak Flow (cfs)	Proposed Bridge M-05-001		Proposed Bridge W-06-016	
			Max Water-Surface Elevation (ft. NAVD88)	Max Velocity at the Bridge (ft/sec)	Max Water-Surface Elevation (ft. NAVD88)	Max Velocity at the Bridge (ft/sec)
10%	10	1,698	2.11	1.29	2.11	1.42
4%	25	2,217	2.12	1.52	2.12	1.68
2%	50	2,639	2.14	1.70	2.14	1.88
1%	100	3,074	2.16	1.90	2.15	2.10
0.2%	500	4,190	2.22	2.40	2.21	2.65

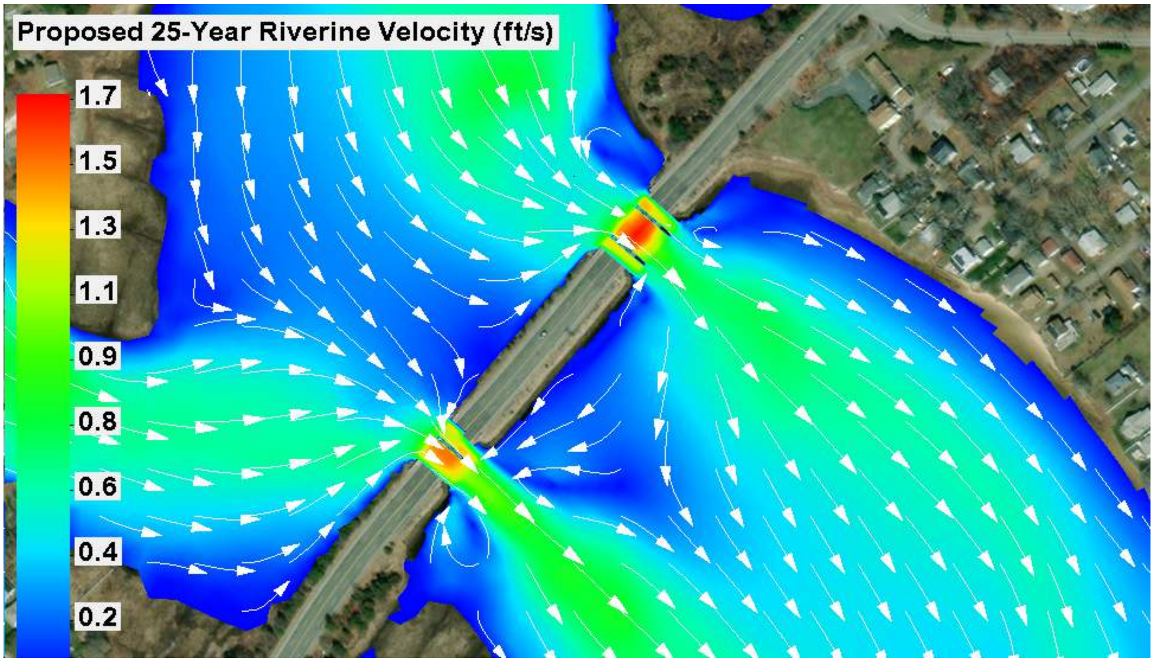

Figure 26. Proposed 25-Year Riverine Maximum Velocities

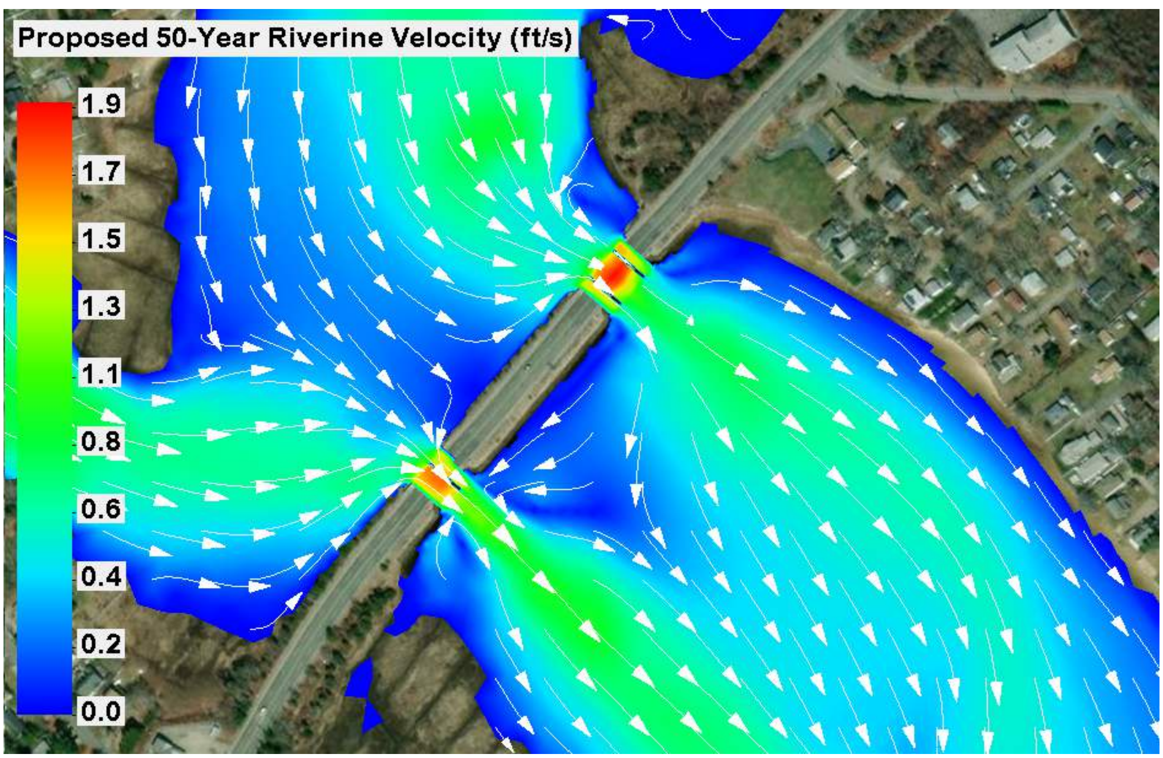

Figure 27. Proposed 50-Year Riverine Maximum Velocities

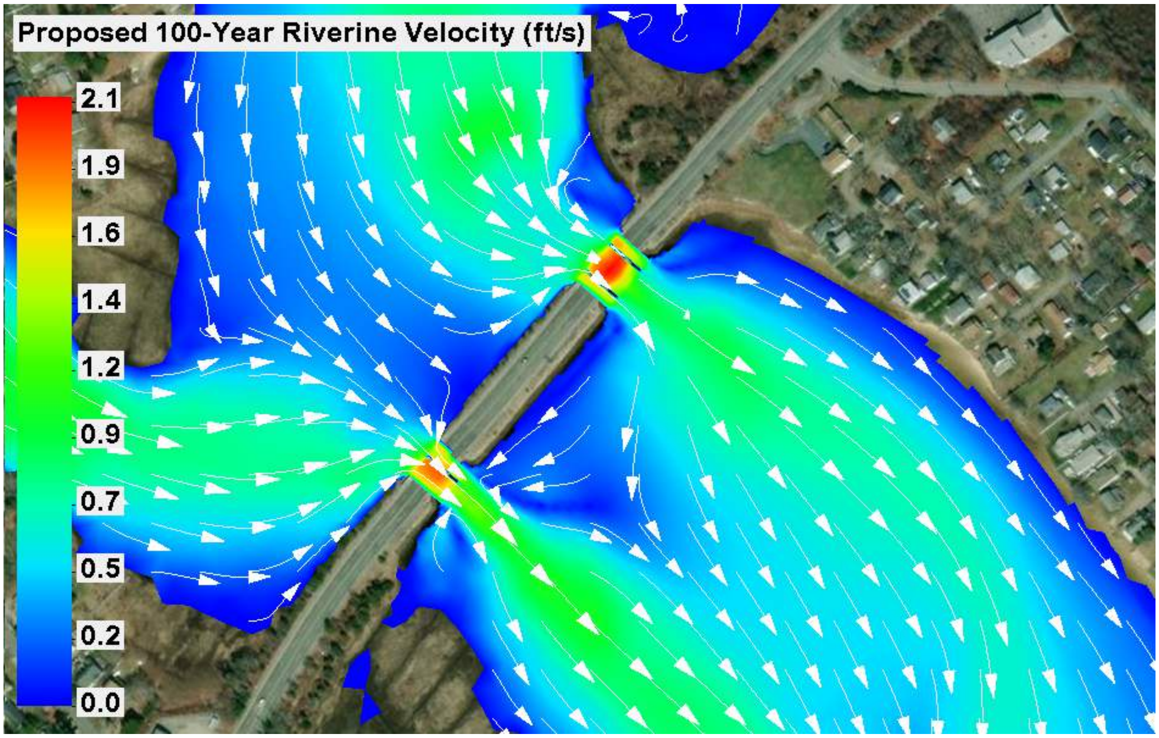

Figure 28. Proposed 100-Year Riverine Maximum Velocities

4.3.2 Coastal Hydrodynamic Analysis

A two-dimensional (2D) hydrodynamic numerical model was developed to perform a coastal hydraulic analysis of the project site and nearby area to determine water depths and velocities for scour evaluation and analyze hydrodynamic loads on the proposed structure. The coastal hydraulic model was developed to compliment the riverine hydrodynamic model in Section 4.3.1 to improve confidence in the selected design velocities. MIKE21 Hydrodynamic (HD) Flexible Mesh (FM) software was applied for this modeling effort. MIKE21 HD FM is a state-of-the-art commercial software distributed by Danish Hydraulics Institute (DHI). Numerical models developed with this software simulate hydrodynamics based on oceanic tidal boundary conditions and meteorological forcing (wind and pressure). Notably, this software is on the FEMA list of “Hydraulic Numerical Models Meeting the Minimum Requirement of National Flood Insurance Program.” Models developed with this software are 2D in the sense they simulate hydrodynamics using depth-averaged equations.

4.3.2.1 Horizontal and Vertical Datum

All coastal hydrodynamic modeling was conducted in a horizontal datum of NAD83 (2011) State Plane, Massachusetts, in units of meters, and a vertical datum of NAVD88 in units of meters.

4.3.2.2 Elevation Data

Elevation data for the coastal hydrodynamic model are discussed in Section 3.1.

4.3.2.3 Model Domain

The model domain for the coastal hydrodynamic modeling utilized two boundary conditions, with one across Buzzards Bay (Offshore Boundary) and one within the Cape Cod Canal (Channel Boundary). The Offshore Boundary location was chosen to include the -10-meter (-32.8 feet) NAVD88 contour and is approximately 8 miles wide. The Canal Boundary was placed across the Cape Cod Canal to allow for water flow in and out of the model domain. The Cape Cod Canal is approximately -14 meters (-46 feet) NAVD88 deep and approximately 0.15 miles wide. Interior land boundaries connect to the forcing boundaries and extend beyond the furthest inland surge propagation extent1. The model domain is shown in Figure 29. Topography and bathymetry around the project site are shown in Figure 30.

[1] Model inland extent was based approximately on the +26 ft NAVD88 contour, which represents higher elevations than the 500-year water surface elevation plus RSLR.

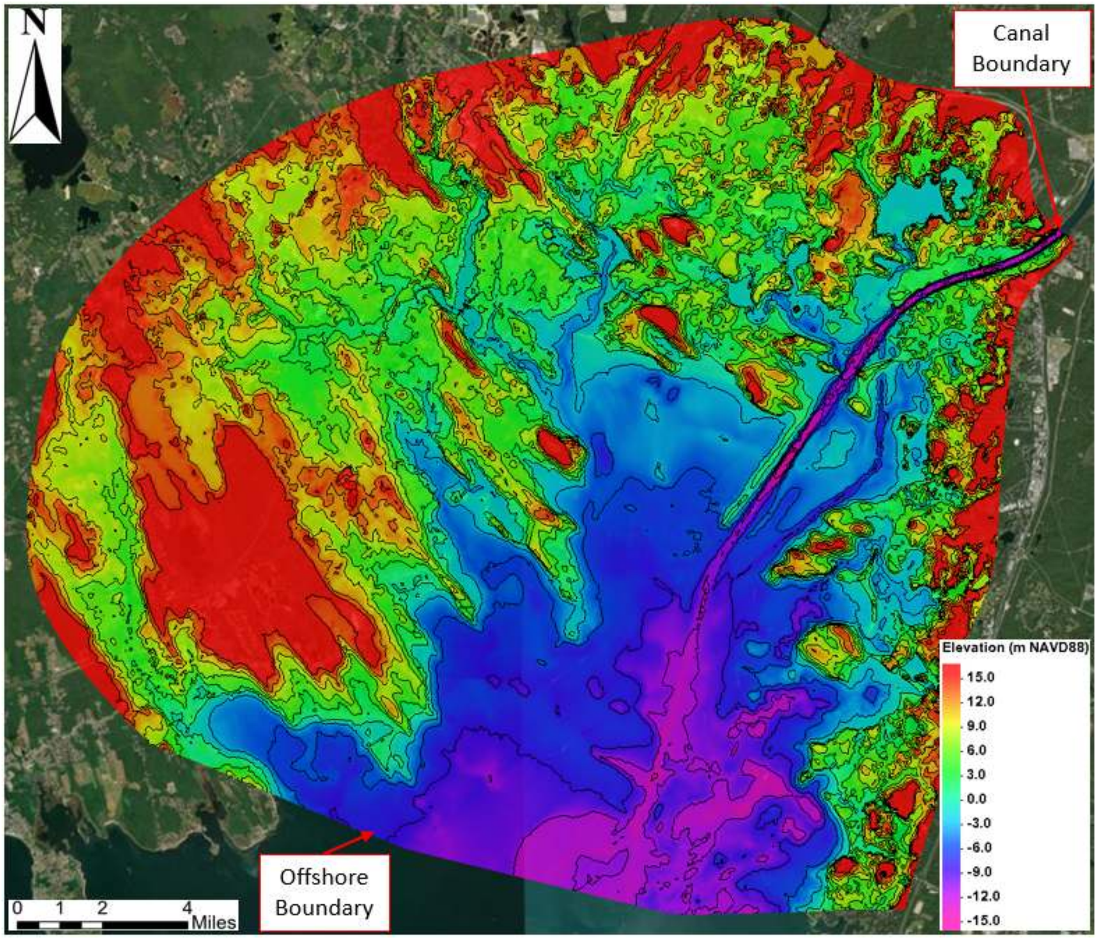

Figure 29. MIKE21 HD FM Model Domain

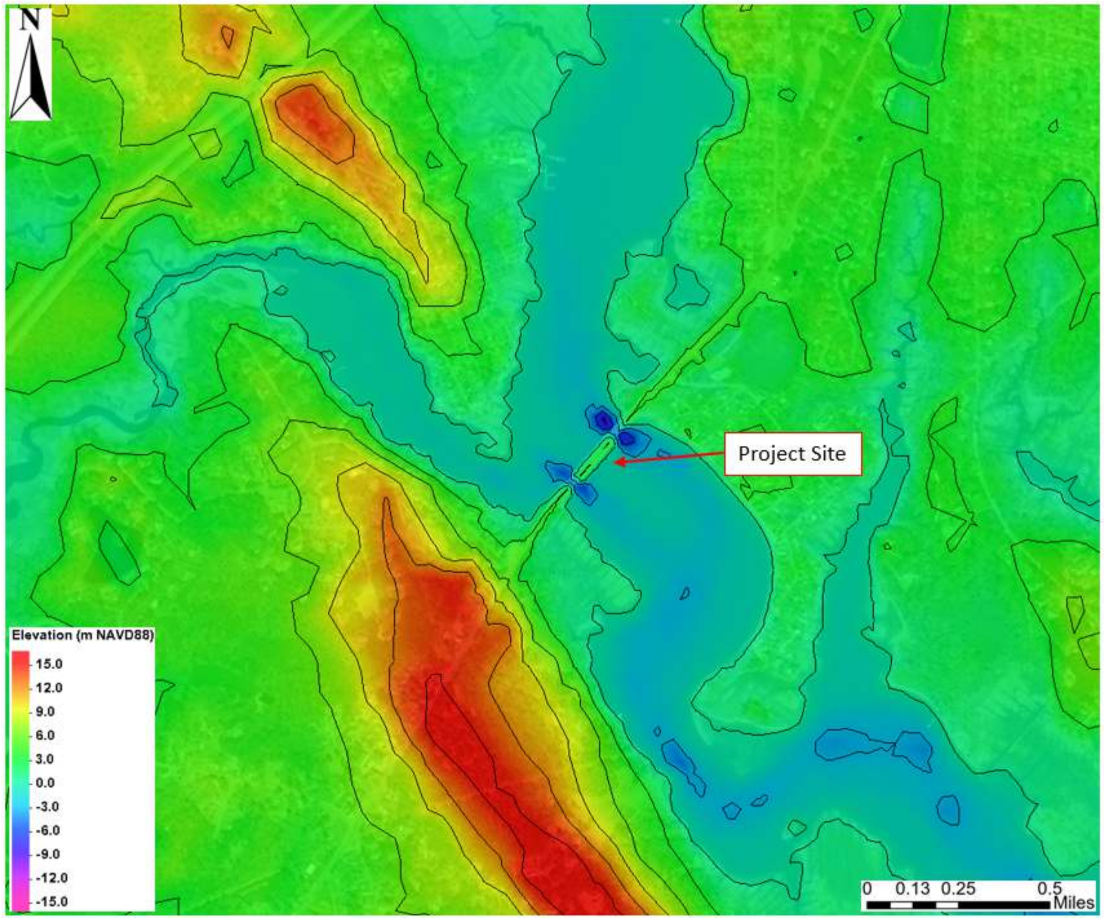

Figure 30. Model Topography and Bathymetry around Project Site

Model elements were refined to capture both bathymetric and topographic features of importance as well as anticipated hydrodynamic processes, such as changes in velocity. Channels and flow pathways were developed with smaller element sizes, whereas open water was developed with larger element sizes. This approach allows reduced model computational time while resolving the features and processes important when simulating the storm surge inundation and velocities. The mesh had a total of 150,565 elements ranging from approximately 36 to 5,000 square meters. Figure 31 shows the mesh elements at the bridge site.

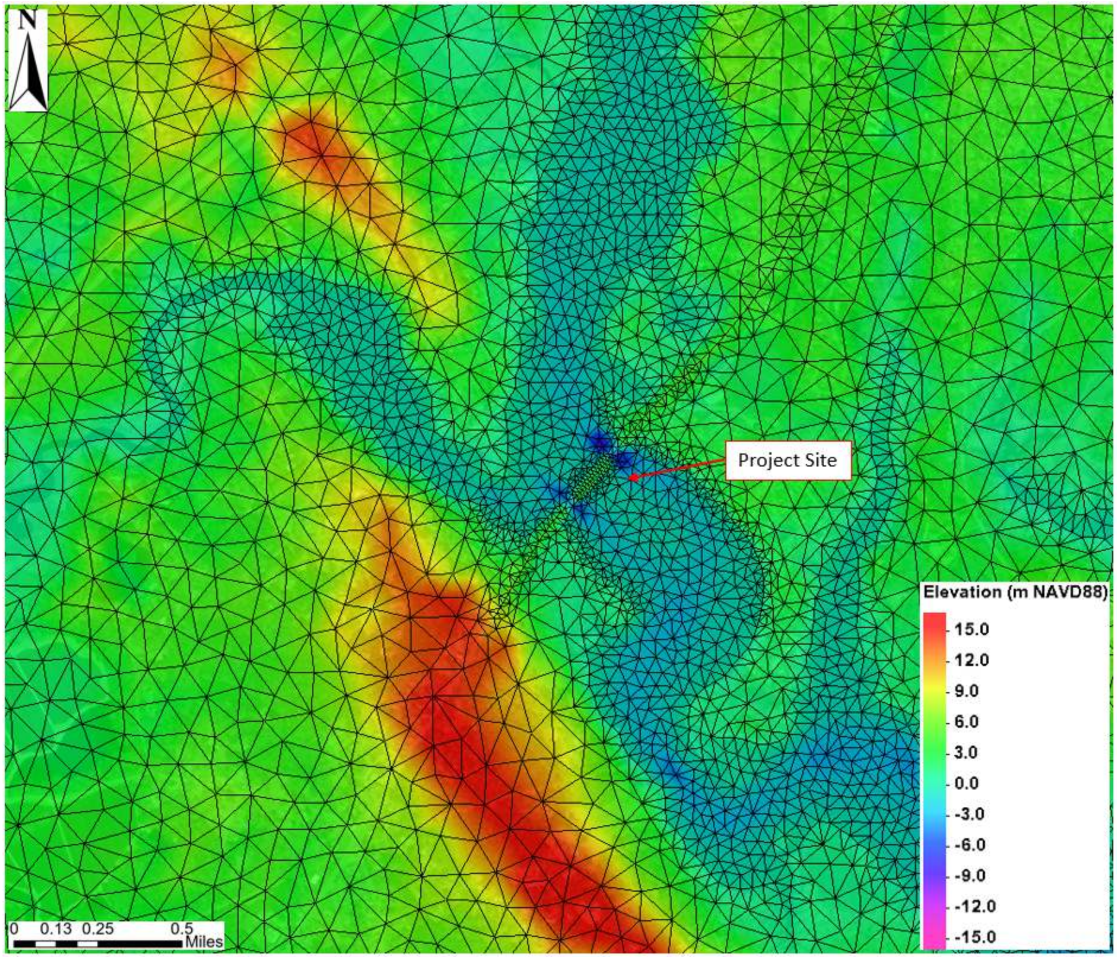

Figure 31. MIKE21 HD FM Mesh at the Project Site

4.3.2.4 Model Boundary Conditions

Model forcing applied WSE time series at the offshore and channel boundaries while also applying a time and spatially varying cyclonic wind field. The forcings were derived from numerical modeling performed by the U.S. Army Corps of Engineers (USACE) for the North Atlantic Comprehensive Coastal Study (NACCS). The USACE modeling assessed coastal storm and flood risk in the U.S. North Atlantic Region using coupled ADCIRC and SWAN modeling. The study developed synthetic tropical cyclones and used historical extratropical events covering the region to perform basin scale numerical modeling covering subregions of the Atlantic Ocean. The results of the modeling are available at “save points” throughout the study area and can be downloaded from the USACE Coastal Hazards System (CHS (dren.mil)). The save points located near the project site are shown in Figure 32.

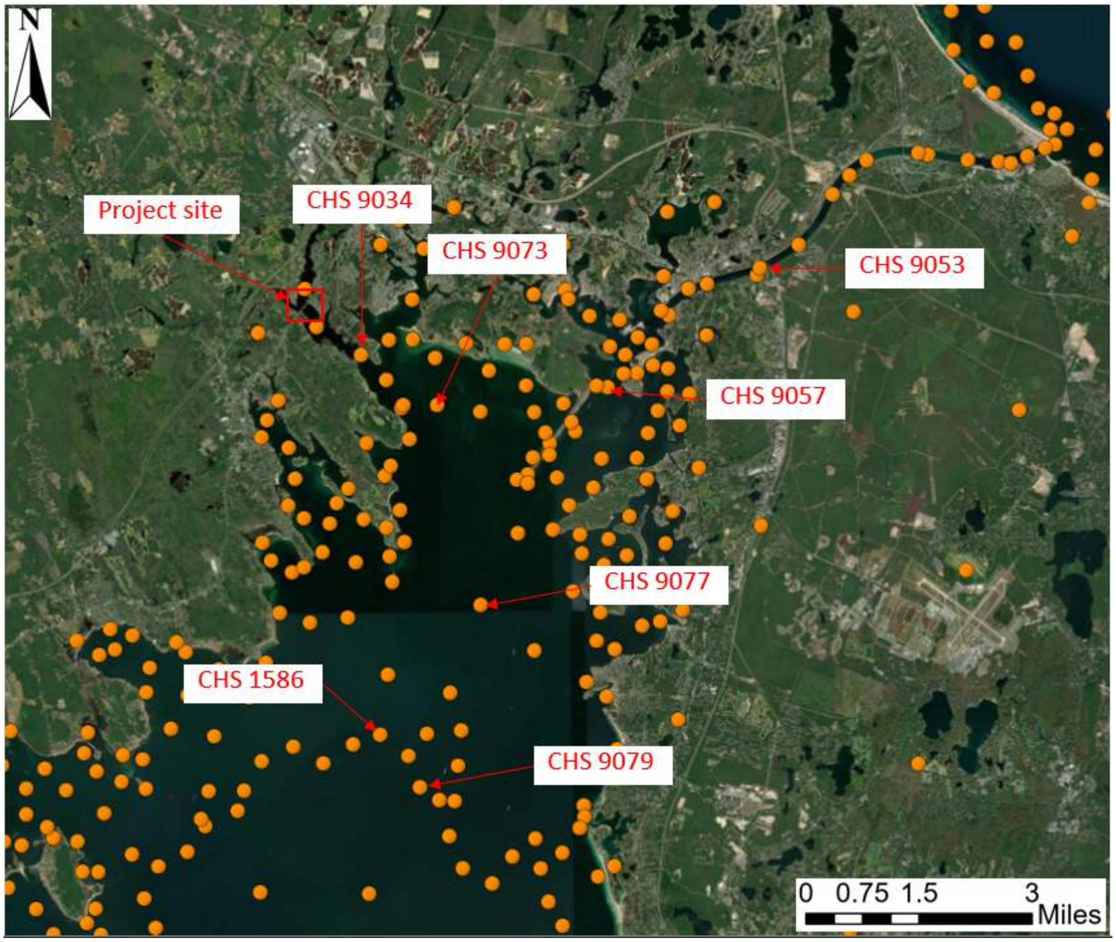

Figure 32. Save Point Locations from USACE NACCS near the Project Site

Save points near the project site provide water surface elevation time series for all modeled storms. Data from save point 9034, closest to the project site and within the Weweantic River, was reviewed to select a storm to apply in the bridge hydrodynamic modeling. Peak water levels extracted from all storms in the NACCS at Save Point 9034 were compared to the AEP water surface elevations (reference Section 2.1.4) to identify a storm that represented a 25-year, 50-year and 100-year. Once a storm matching the AEP value was found, a time series of the representative storm was chosen from save points along the offshore boundary. Table 14 shows the save points and corresponding water levels used for model forcing from the offshore boundary. Results from extratropical events, or nor'easters, were also assessed, but the largest water surface elevation produced by nor'easters was +7.0 feet NAVD88, which was nearly 7 feet below the 100-year water surface elevation produced by tropical cyclones. Therefore, the mode ling focused on tropical cyclones which produce the most conservative water surface elevations.

Table 14. Offshore Boundary Model Forcings

AEP (Return Period)	FEMA Water Level (ft NAVD88)	CHS Storm ID	CHS Water Level (ft NAVD88)
4% (25-year)	10.1	511	10.1
2% (50-Year)	11.8	457	12.0
1% (100-Year)	13.7	1009	13.7

The Canal Boundary was placed across the Cape Cod Canal to maintain water flow through the model domain. The Canal Boundary was forced using the storm surge hydrographs from SP 9053 for all storms. Although the NACCS data were reported relative to mean sea level, all modeling for this project was performed relative to NAVD88; therefore, the WSEs were converted to NAVD88 using factors provided by USACE. The conversion factor from MSL to NAVD88 is -0.115 for SP 457, -0.1166 for SP 9079, -0.1198 for SP 1566 and -0.1827 for SP 9053. To convert the time series to NAVD88 the provided conversion factors were subtracted from the full water level time series. Figure 33 shows the time series applied at the offshore and channel boundaries for the 25-year, 50-year and 100-year WSEs.

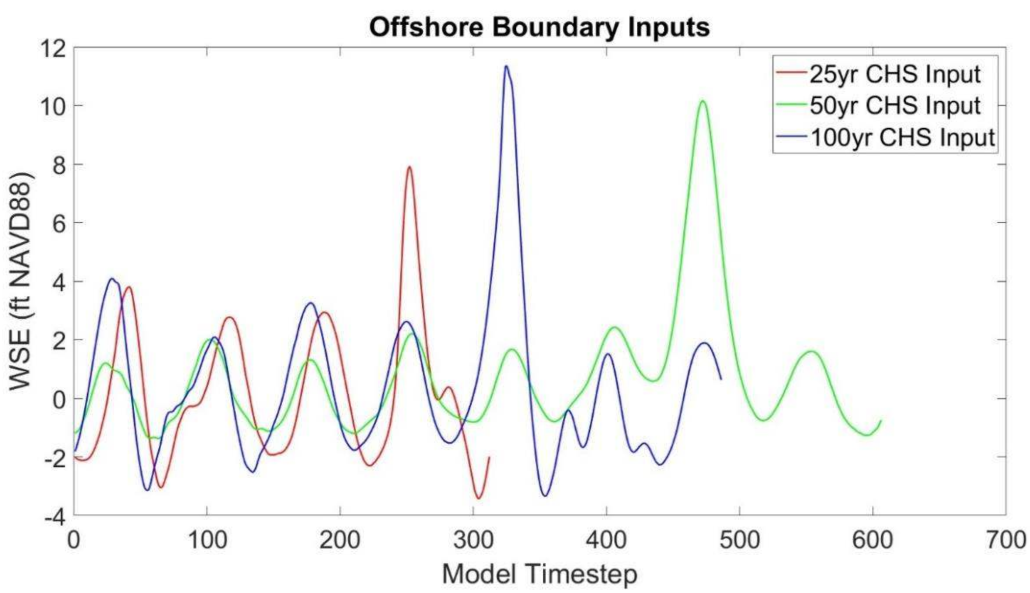

Figure 33. Boundary Condition Timeseries

Wind was also applied to the model. Through communications with USACE, the project team obtained the wind time series data for the NACCS storms. The time series included the storm center coordinates, radius of maximum wind, and central pressure deficit. These parameters were used in DHI's Cyclone Wind Generation tool for developing cyclonic wind fields. The resulting wind field files had spatially and temporally varying winds representing synthetic storms for the 25-year, 50-year and 100-year AEPs. These wind fields were applied to the respective model runs.

Model runs were also performed for the RSLR condition discussed in Section 4.2. For these model runs, WSEs for both boundary conditions were increased uniformly by 5.11 feet to account for RSLR.

4.3.2.5 Model Validation Run

Field data collection was not included in this modeling effort. However, an additional model run was performed to help validate the model. The validation effort was performed using Hurricane Sandy, which was the largest historical event modeled in the NACCS in terms of peak WSE. The WSE at the offshore boundary and channel boundary used the time series from Save Points 9079 and 9053, respectively, as described in Section 4.3.2.4. Because Hurricane Sandy made landfall approximately 250 miles south of the project site, the wind in the region of the model domain was relatively constant spatially. Therefore, a spatially constant, but temporally varying, wind field was applied to the validation model.

Figure 34 shows a comparison of the MIKE21 validation model output to data extracted at the project site and CHS point closest to the project site (SP 9034).

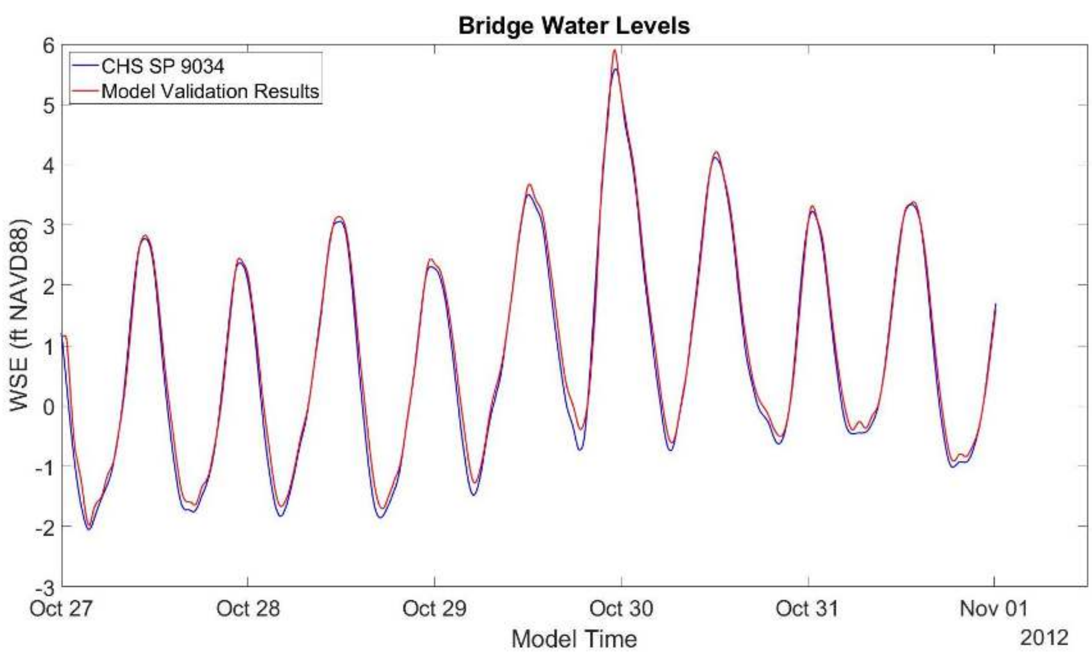

Figure 34. Comparison of MIKE21 Validation Model Output to CHS SP 9034

The figure shows the model is close in replicating the CHS modeling in phase of water surface oscillations. Overall water levels are seen to be slightly higher than the results from the CHS model, approximately +0.4 feet at the peak water level. However, CHS SP 9034 is located at the mouth of the Weweantic river with flow being less constricted than at the bridge site, which could lead to the minor difference in results. In addition to comparing the MIKE21 model to NACCS model data, the MIKE21 results were compared to high water mark data published by the US Geological Survey (USGS). These comparisons are shown in Table 15.

Table 15. Comparison of MIKE21 Model Results to USGS Mean High Water Marks for Hurricane Sandy

Points	X (State Plane, meters)	Y (State Plane, meters)	USGS Recorded High Water Mark (ft NAVD88)	MIKE21 Model Highwater Mark (ft NAVD88)
Point 1	262776	832364	+5.7	+5.9

The comparison in Table 15 shows good agreement between the recorded and modeled high water marks for Hurricane Sandy. The difference of 0.2 ft could be attributed to inaccuracies in recording of the high-water mark elevations, small differences in horizontal position, smaller local features present in real world that are not captured in the model, or other model limitations. In addition, the WSE boundary conditions utilized NACCS model results rather than measured data because no measured data were available at the model boundaries. In the absence of actual recorded water levels at the model boundaries or interior, the model appears to function well and was considered suitably validated.

4.3.2.6 Model Sensitivity Analysis

Because recorded observations were not available to perform a detailed model calibration exercise, model sensitivity to bottom roughness and offshore storm surge amplitude was conducted.

4.3.2.6.1 Bottom Roughness

For bottom roughness, a constant roughness value and variable friction layers were tested. The base condition model roughness utilized a Manning's number of 32, which is the default setting. For the sensitivity analysis, constant Manning's numbers of 40 and 45 were applied to the model domain. A variable friction layer was also tested, using a Manning's coefficient of 45 for all surfaces below MHHW and 25 for all above MHHW. Note a higher Manning's coefficient is a lower bed friction and is expected to yield higher storm surge.

4.3.2.6.2 Water Surface Elevation

The offshore WSE boundary condition was increased and decreased by 20% to check model velocity sensitivity to changes in the offshore water surface elevation.

A summary of the comparisons for the sensitivity model runs is shown in Table 16. The table shows current speed changes up to 21% at the bridge, with the highest current speeds occurring for the sensitivity test with the Manning's coefficient set to 45. Because of the uncertainty with the lack of calibration data, production runs utilized the Manning's coefficient of 45 on all surfaces, which brought the greatest agreement between the model and NACCS data.

Table 16. Summary of Model Sensitivity Runs

Model Runs	Peak WSE (ft NAVD88)	Peak Flood Current Speeds (ft/s)	Peak Ebb Current Speeds (ft/s)	Peak WSE % Change	Flood Current Speed % Change	Ebb Current Speed % Change
Base Model	+13.3	4.3	7.0	--	--	--
+20 % Water Level	+15.8	4.7	7.6	+18.8%	+9.3%	+8.6%
-20% Water Level	+10.7	4.1	6.6	-19.5%	-4.7%	-5.7%
Manning = 40	+13.6	4.3	8.0	+2.3%	+0.0%	+14.3%
Manning = 45	+13.7	4.5	8.5	+3.0%	+4.7%	+21.4%
Manning = variable	+13.7	4.5	8.5	+3.0%	+4.7%	+21.4%
** Percent change was compared with base model result.*						

4.3.2.7 Coastal Surge Model Results in MIKE21

The MIKE21 HD FM Model results were analyzed at both bridge channel openings. Figure 35 shows the current velocities extracted from the model for the 100-year model case. The peak current velocity was 7.1 feet per second and occurred during the storm surge ebb as the water was flowing back toward Buzzards Bay.

It should also be noted that the highest velocities during the flooding portion of the surge events occurred during the 25-year event. This is likely a result of the surge beginning at a WSE of -2.3 feet NAVD88, as seen in Figure 33, where the 50-year and 100-year start from a WSE of +0.5 and -1.5 feet NAVD88, respectively. The lower WSE during the 25-year event would cause a constricted water way within the Weweantic River and lead to higher current velocities. Lower overall water level of this event could also lead to less water flowing over land near the project site and force more water through the bridge channel openings.

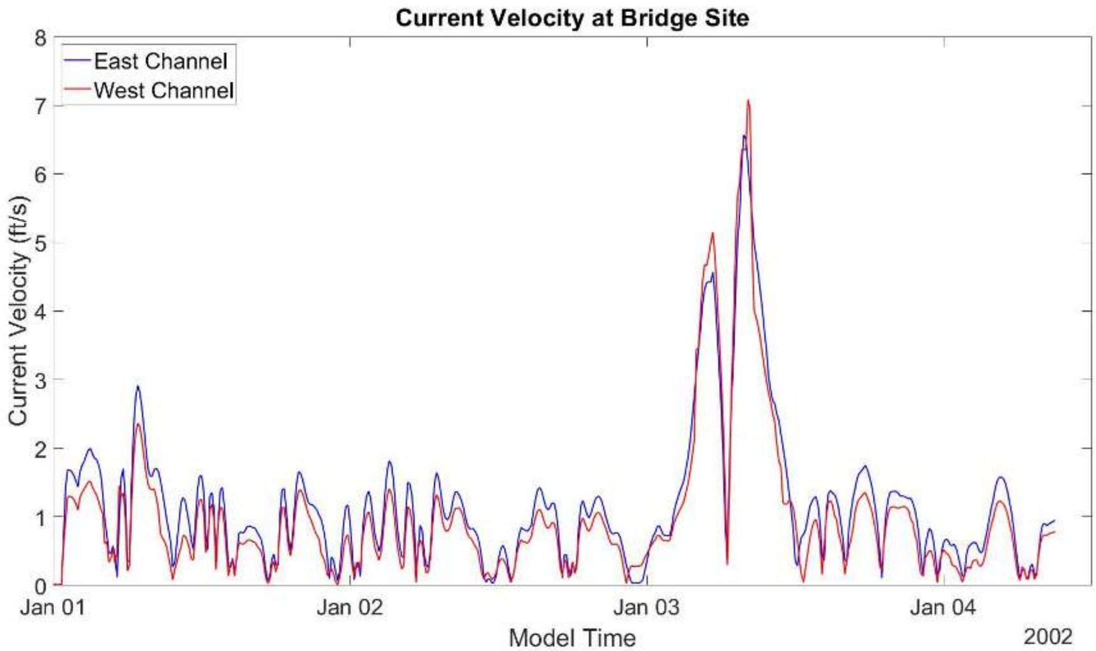

Figure 35. Current Velocities for 100-Year Case

The peak WSE and peak current velocity for the flood and ebb conditions are summarized in Table 17 for the various design conditions modeled. In both 50-year and 100-year cases, the highest velocities occurred during the ebb portion of the surge event.

Table 17. Summary of Model Results at the Project Site

Case	Peak Water Level (ft NAVD88)	Flood Current Velocity (ft/s)	Ebb Current Velocity (ft/s)
25-year	+9.6	7.7	5.1
50-year	+12.0	3.6	3.8
100-year	+16.0	5.2	7.1
RSLR 50-year	+16.9	2.1	3.9
RSLR 100-year	+21.0	2.7	7.4

4.3.2.8 Coastal Hydrodynamic Analysis in SRH-2D

The Coastal Hydrodynamic Analysis was finalized in SRH-2D in order to utilize the SRH-2D bridge scour coverage tool to extract variables used to calculate the scour for the coastal simulations. The SRH-2D mesh has higher resolution at the bridges than the MIKE21 mesh. Unlike the MIKE21 mesh, the SRH-2D mesh also has voids in the mesh that represent the bridge piers, giving more realistic results. Due to the higher resolution and the piers being represented in the mesh, the velocities at the bridges are expected to be higher in the SRH-2D model than in the MIKE21 model due to the greater constriction of the bridge openings.

The coastal simulations were computed using the same SMS SRH-2D model that was used to compute the riverine simulations, as previously discussed in Section 4.3.1. All parameters, such as the existing and proposed meshes and material coverages remained the same except the boundary conditions.

4.3.2.8.1 Coastal Surge Boundary Conditions in SRH-2D

WSEs and discharges were extracted from the MIKE21 model at the SRH-2D domain. The inflow and outflow boundary condition locations are shown in Figure 36. The inflow boundary conditions (Transect 1-3) were input into the model as a timeseries of flows (Inlet-Q). The outflow boundary conditions (Transect 4-5) were input into the model as a timeseries of water-surface elevations (Exit-H). Table 18 displays the maximum flows and WSEs at the SRH-2D model domain. The maximum flood discharge listed in the table is from the surge, when flow is coming from the South/East, flowing inland. The maximum ebb flow listed in the table is when the surge has peaked and is flowing back into Buzzards Bay.

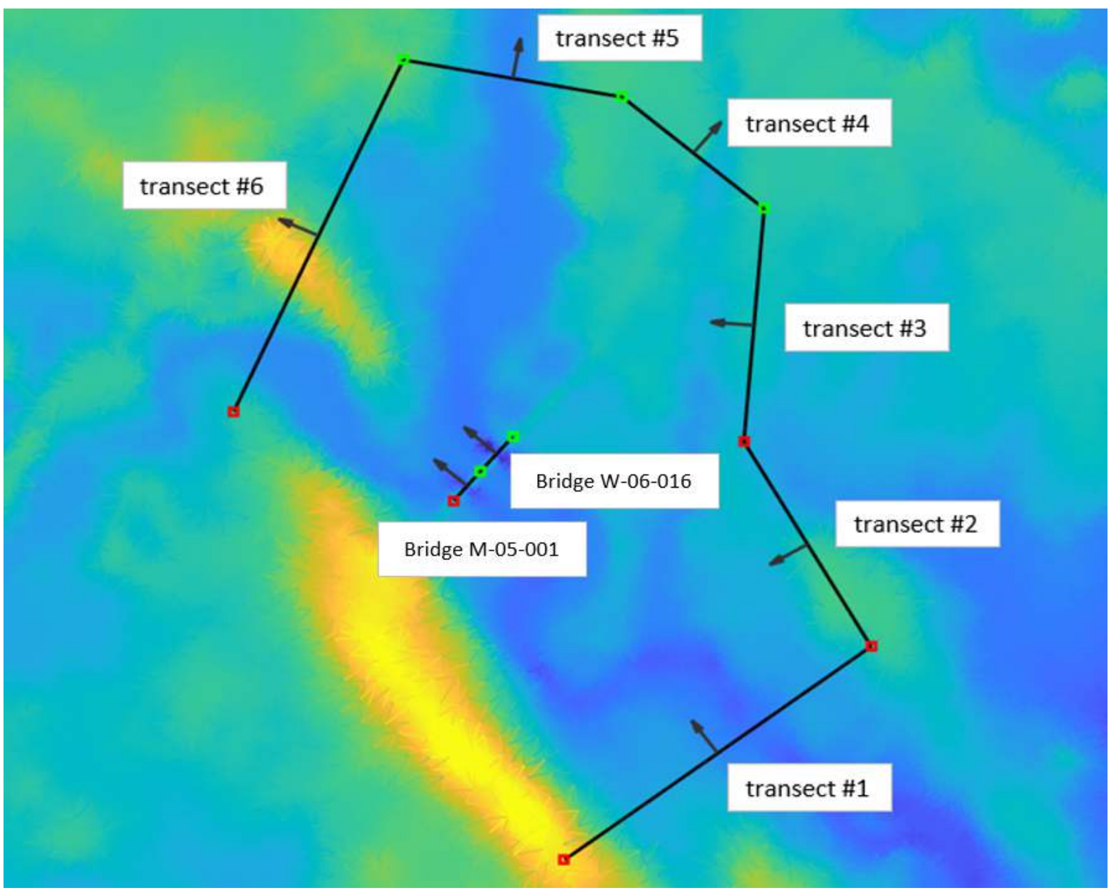

Figure 36. Coastal Boundary Conditions in SRH-2D

Table 18. Coastal Model Boundary Conditions

Case	Transect 1 (Inlet Q)		Transect 2 (Inlet Q)		Transect 3 (Inlet Q)		Transect 4 (Exit H)	Transect 5 (Exit H)	Transect 6 (Exit H)
	Max Flood (cfs)	Max Ebb (cfs)	Max Flood (cfs)	Max Ebb (cfs)	Max Flood (cfs)	Max Ebb (cfs)	Max WSE (ft. NAVD88)	Max WSE (ft. NAVD88)	Max WSE (ft. NAVD88)
25-year	58,550	44,011	446	721	1,300	2,167	Not Wet	9.79	9.54
50-year	31,072	33,217	1,103	1,669	2,914	2,412	10.71	12.21	12.16
100-year	88,796	70,247	12,986	10,453	17,698	11,504	16.05	16.23	15.94
500-year	137,697	95,000	19,586	15,651	30,516	20,680	18.00	18.26	18.21
RSLR 50-year	33,546	55,833	4,784	10,010	17,944	3,313	17.27	17.16	17.08
RSLR 100-year	11,0772	97,438	15,237	20,337	37,520	33,408	21.11	21.10	20.81
RSLR 500-year	15,598	132,228	20,880	26,163	44,782	36,851	23.31	23.34	23.18

4.3.2.8.2 Coastal Surge Model Results in SRH-2D

The maximum WSE, and the maximum velocities from the flood and the ebb produced from the coastal simulations for the existing conditions are displayed in Table 19. The existing bridges are predicted to be in pressure flow with flows overtopping the high chords of both existing bridges for all coastal surges evaluated except the 25-year event.

The 500-year coastal surge with and without RSLR was included in the SRH-2D simulations but will not be included in the model results or scour analysis.

The maximum velocities that occur at the existing bridges from the 25-year, and the 50-year and 100-year coastal surges with and without RSLR are shown in Figure 37 through Figure 41.

The maximum WSE, and the maximum velocities from the flood and the ebb produced from the coastal simulations for the proposed conditions are displayed in Table 20.

The proposed bridges are predicted to be in pressure flow for all of the coastal surge events evaluated except the 25-year event. Flow overtops the high chords of both proposed bridges for all coastal surges evaluated except the 25-year and 50-year surge.

Table 19. SRH-2D Coastal Surge Hydraulic Analysis for the Existing Conditions

Case	Existing Bridge M-05-001			Existing Bridge W-06-016		
	Max Water-Surface Elevation (ft. NAVD88)	Max Flood Velocity (ft/s)	Max Ebb Velocity (ft/s)	Max Water-Surface Elevation (ft. NAVD88)	Max Flood Velocity (ft/s)	Max Ebb Velocity (ft/s)
25-year	10.30	8.55	6.96	10.33	10.65	7.86
50-year	12.26	5.73	5.07	12.31	6.60	6.20
100-year	16.42	8.84	10.73	16.47	10.08	12.33
RSLR 50-year	17.13	6.25	7.14	17.13	7.09	10.98
RSLR 100-year	21.21	6.90	14.96	21.21	11.57	15.27

Table 20. SRH-2D Coastal Surge Hydraulic Analysis for the Proposed Conditions

Case	Proposed Bridge M-05-001			Proposed Bridge W-06-016		
	Max Water-Surface Elevation (ft. NAVD88)	Max Flood Velocity (ft/s)	Max Ebb Velocity (ft/s)	Max Water-Surface Elevation (ft. NAVD88)	Max Flood Velocity (ft/s)	Max Ebb Velocity (ft/s)
25-year	9.84	8.29	6.20	9.84	10.27	7.11
50-year	12.22	5.03	4.23	12.23	6.12	4.92
100-year	16.75	9.28	9.27	16.77	12.05	10.65
RSLR 50-year	17.16	5.97	6.92	17.16	7.00	8.17
RSLR 100-year	21.37	6.97	11.18	21.34	8.20	12.82

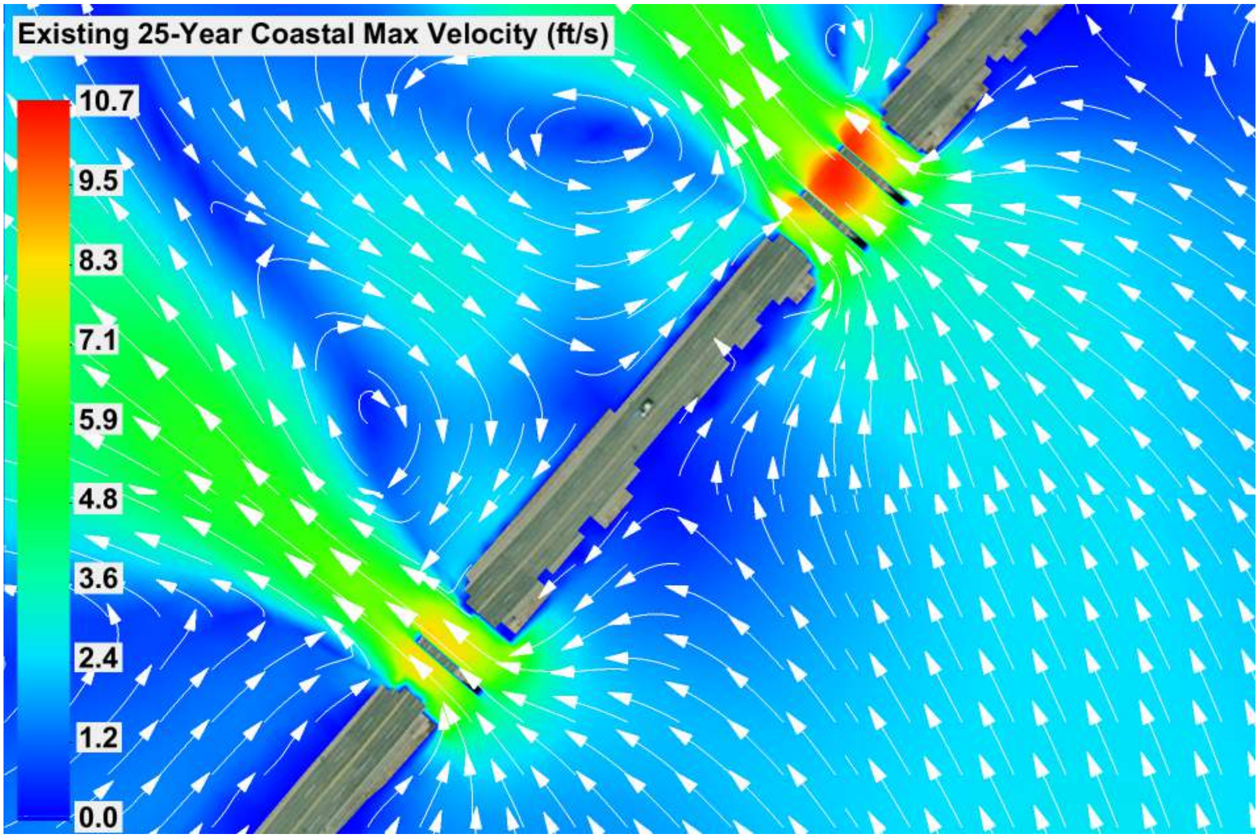

Figure 37. Existing 25-Year Coastal Max Velocities

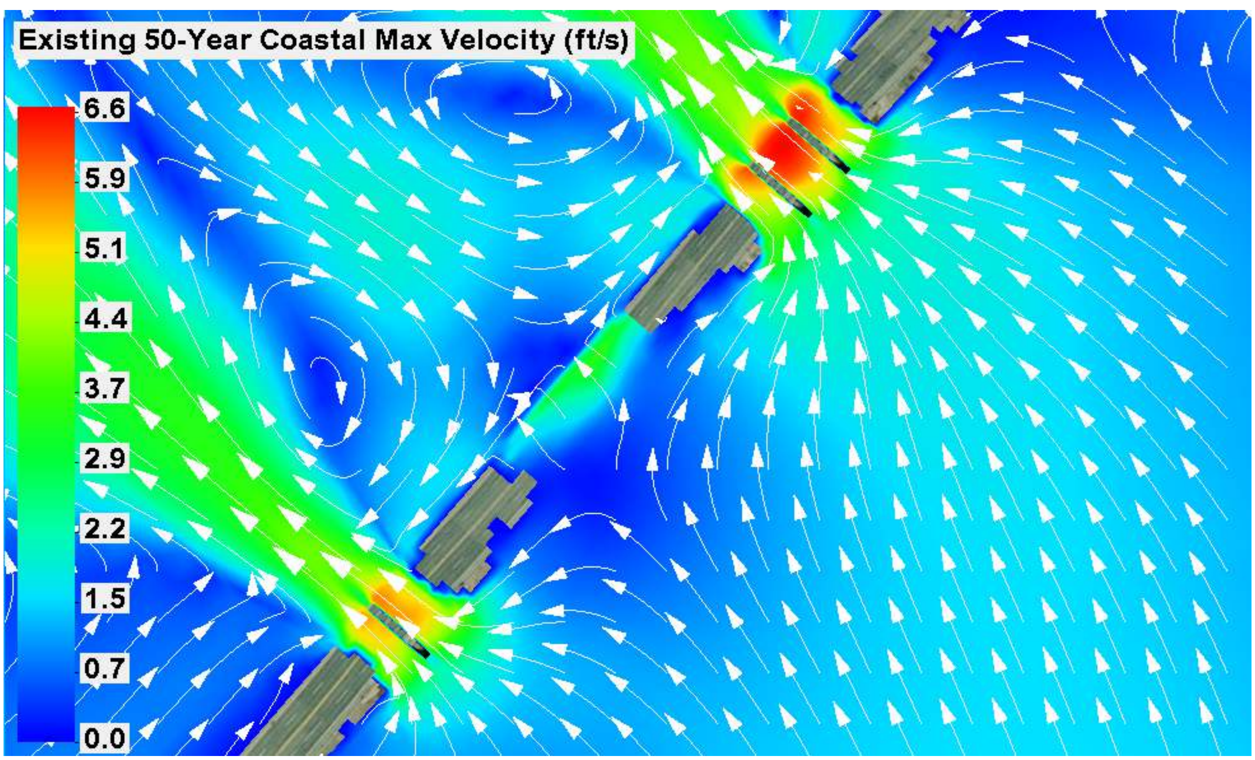

Figure 38. Existing 50-Year Coastal Max Velocities

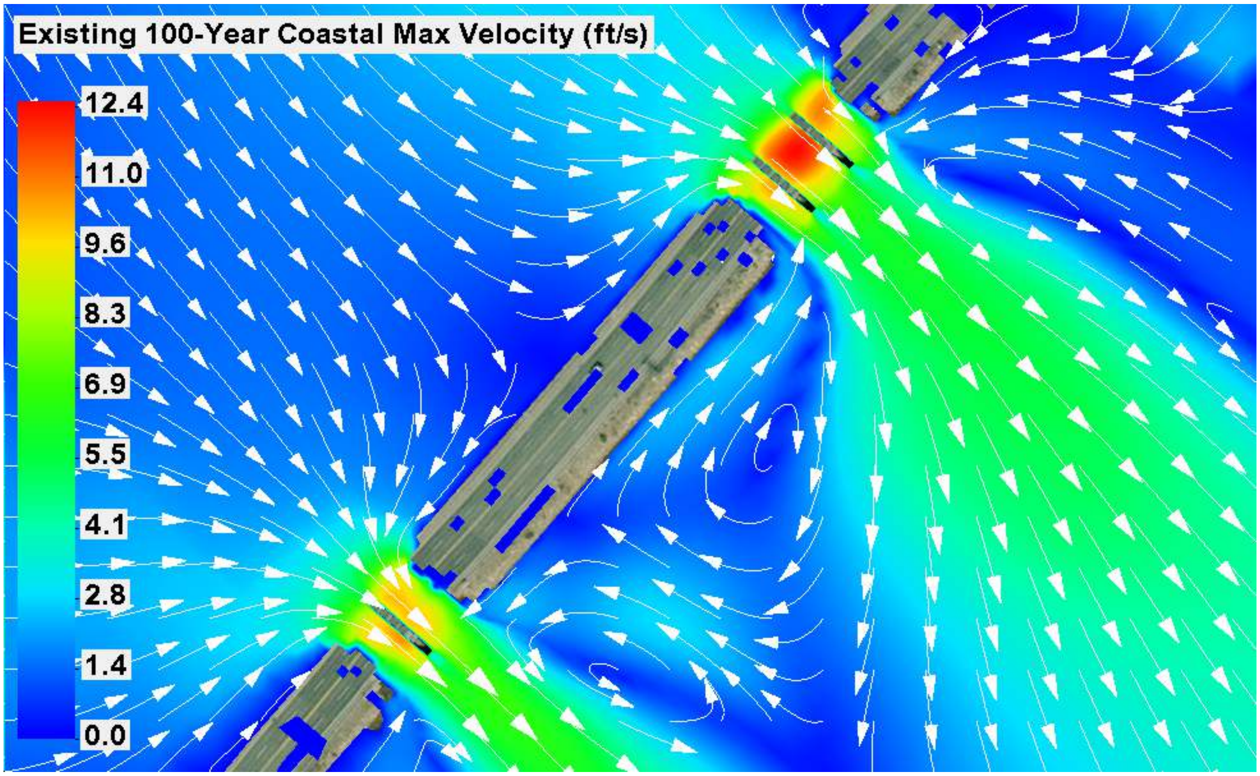

Figure 39. Existing 100-Year Coastal Max Velocities

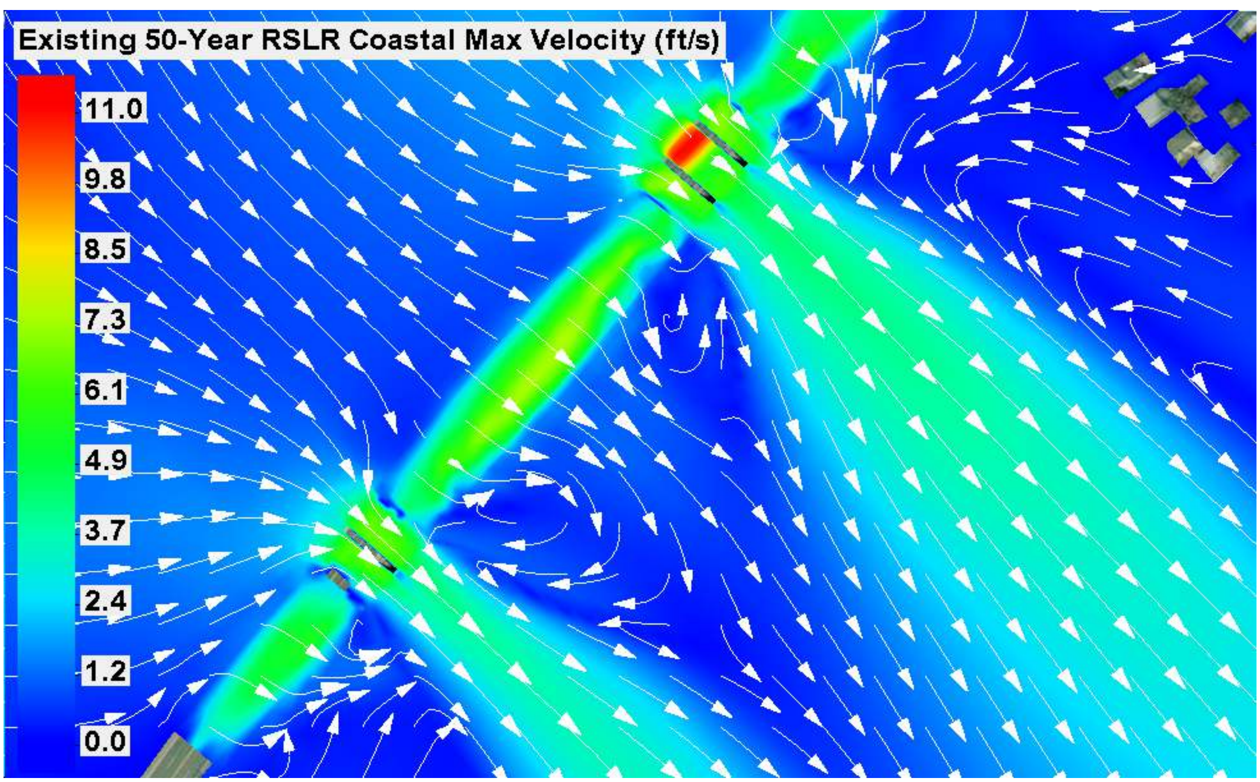

Figure 40. Existing 50-Year RSLR Coastal Max Velocities

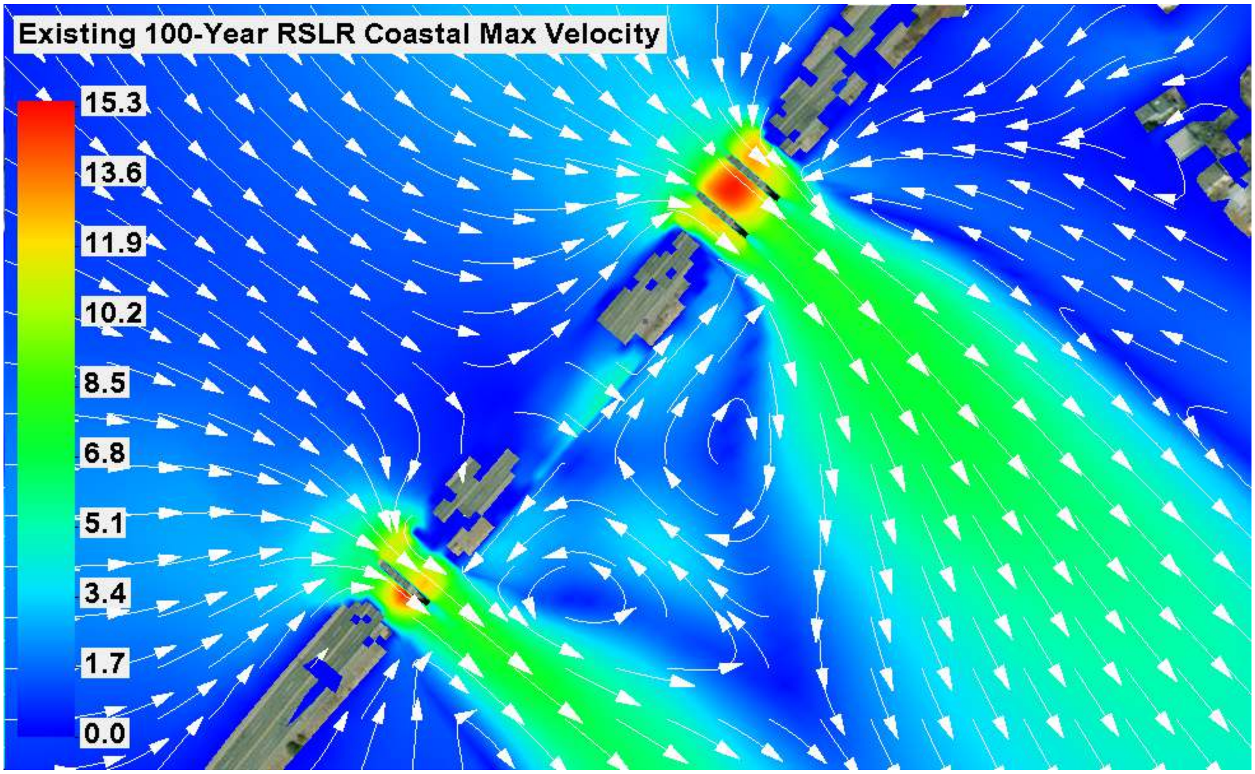

Figure 41. Existing 100-Year RSLR Coastal Max Velocities

The structure plots of the proposed bridges generated from the SRH-2D 3D structure previously discussed in Section 4.3.1.6.2 are shown below.

Figure 42 and Figure 43 display the WSEs produced from the coastal surges at Bridge M-05-001 and Bridge W-06-16, respectively. Figure 44 and Figure 45 display the discharge that overtops the high chord of Bridge M-05-001 and Bridge W-06-016, respectively.

The 25-year and 50-year surges are not displayed in the WSE and discharge structure plots because the SRH-2D 3D coverage only generates plots for simulations that produce flows that overtop the high chord of the structure.

The 25-year coastal surge maximum velocities at the bridges occur when the surge is flowing inland, and the bridges never go into pressure flow. The 50-year coastal surge maximum velocities at the bridges occur when the surge is flowing inland, before the bridge reaches pressure flow. The 100-year coastal surge maximum velocities at the bridges occur when the surge is flowing inland and the bridges are in pressure flow, but flow is not overtopping. The 50-year RSLR coastal surge maximum velocities occur when the bridges are in pressure flow, and the flow is flowing back towards Buzzard's Bay. The 100-year RSLR coastal surge maximum velocities occur when the flow is flowing back towards Buzzard's Bay after the bridges get out of pressure flow.

The maximum velocities that occur at the proposed bridges from the 50-year and 100-year coastal surges with and without RSLR are shown in Figure 46 through Figure 50.

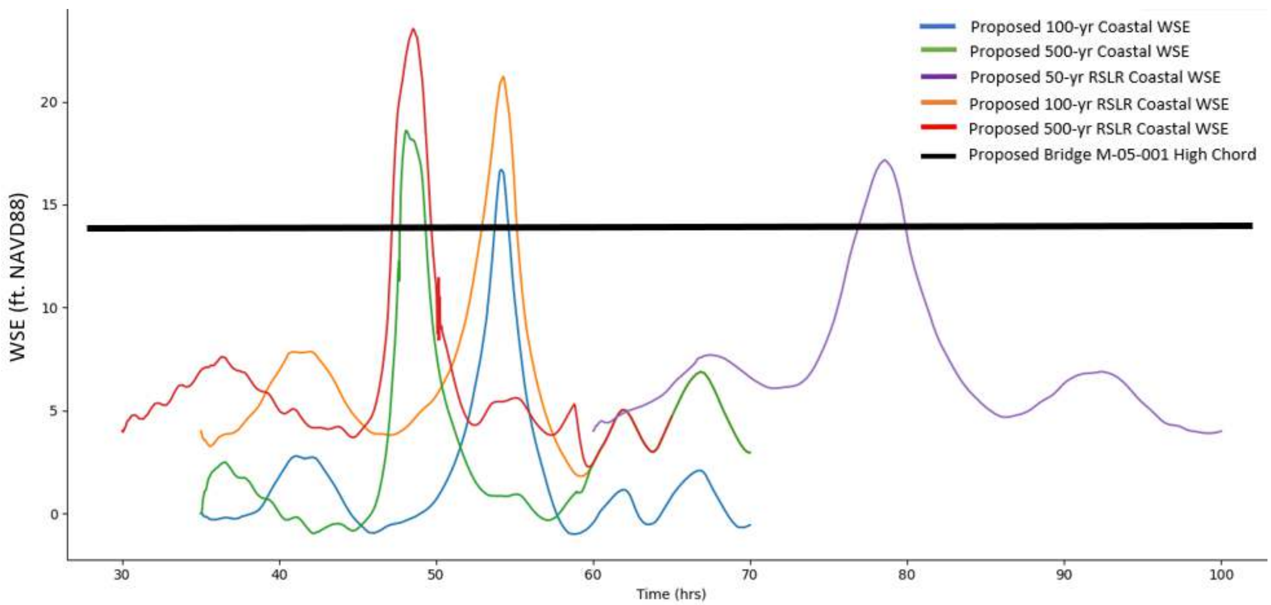

Figure 42. Coastal WSE at Proposed Bridge M-05-001

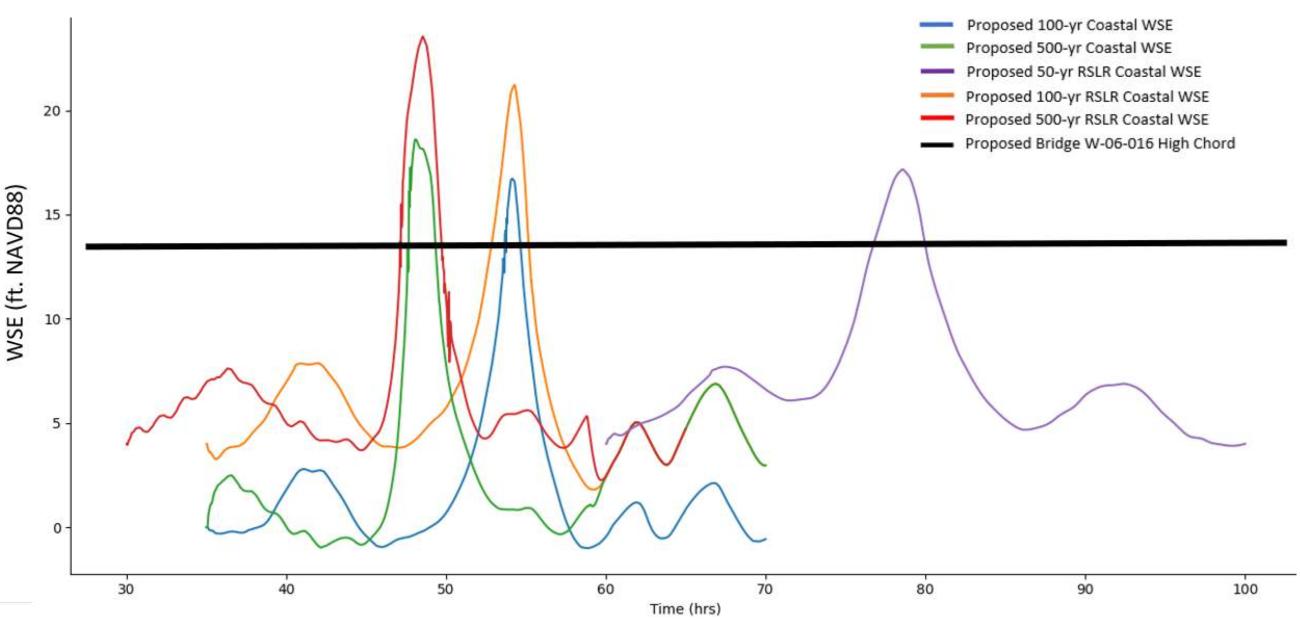

Figure 43. Coastal WSE at Proposed Bridge W-06-016

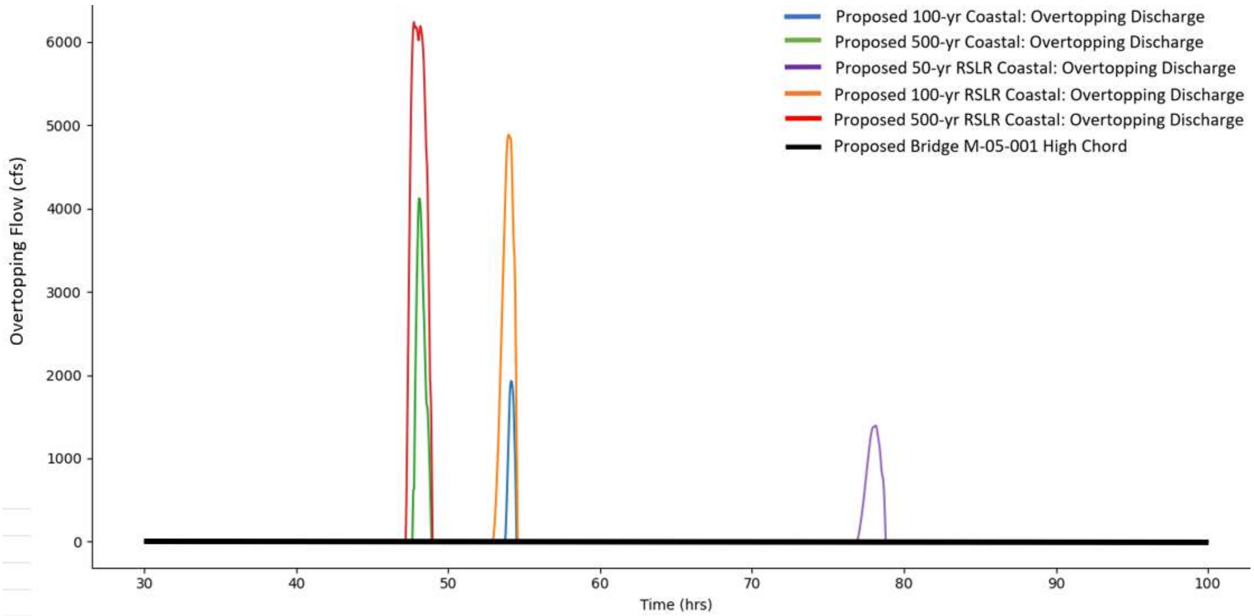

Figure 44. Coastal Flow Overtopping Proposed Bridge M-05-001

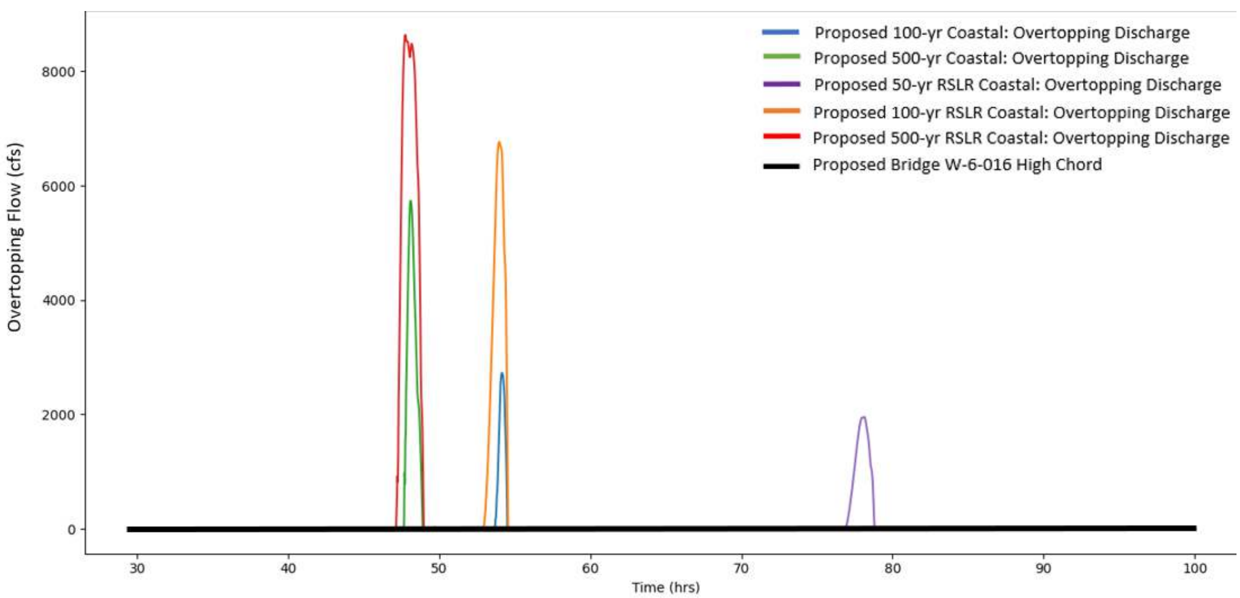

Figure 45. Coastal Flow Overtopping Proposed Bridge W-06-016

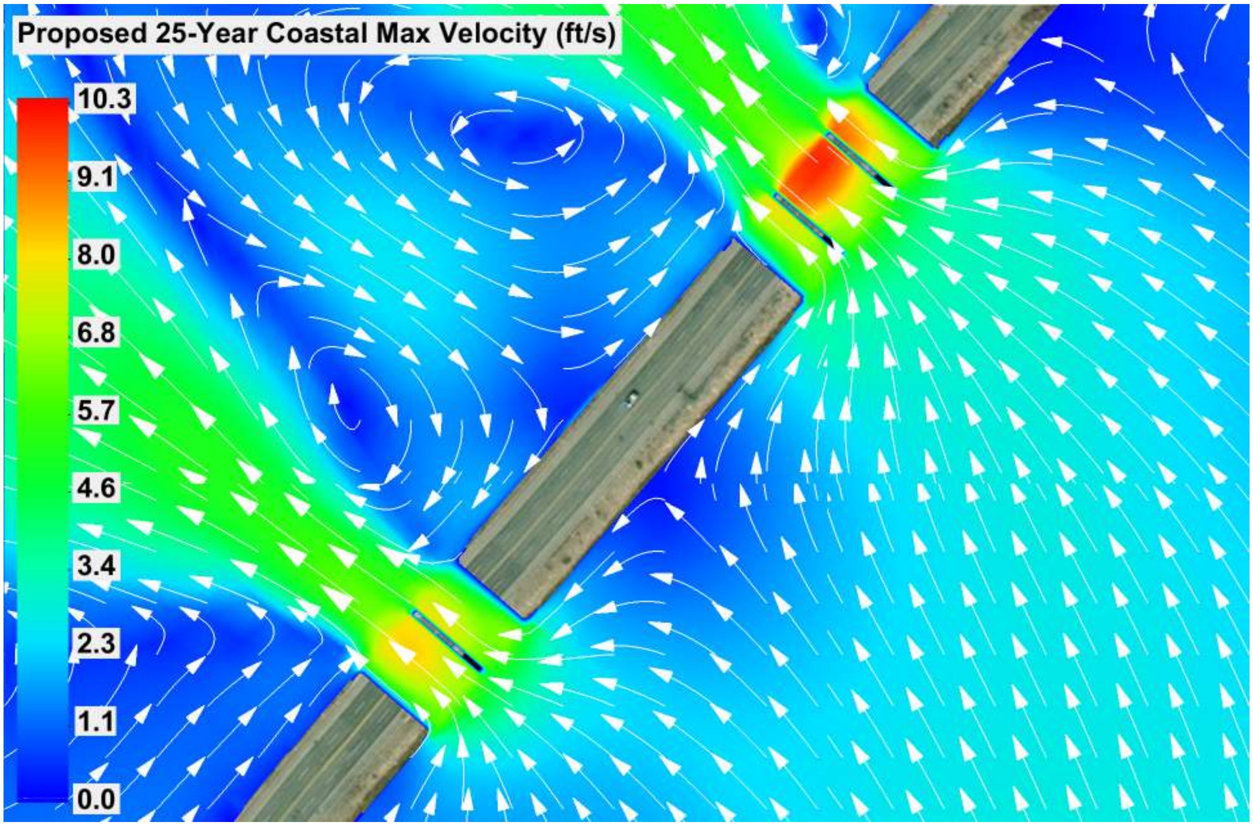

Figure 46. Proposed 25-Year Coastal Max Velocities

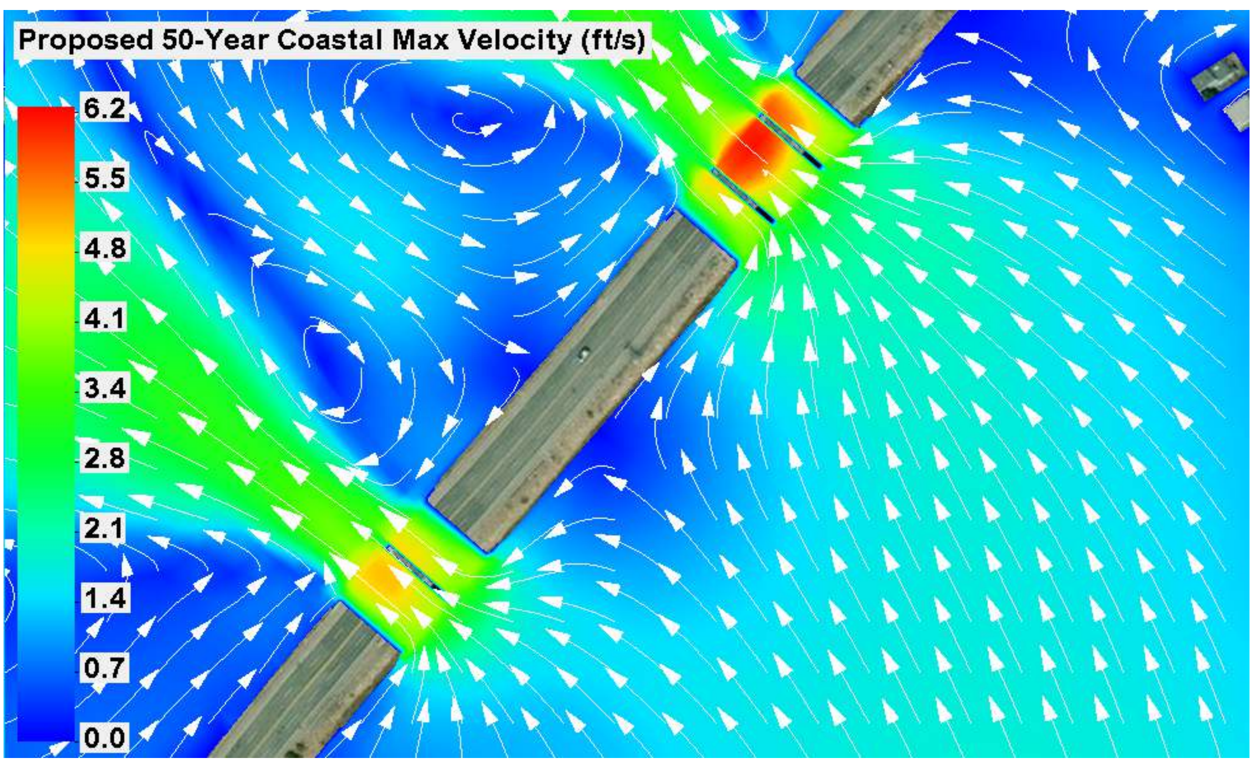

Figure 47. Proposed 50-Year Coastal Max Velocities

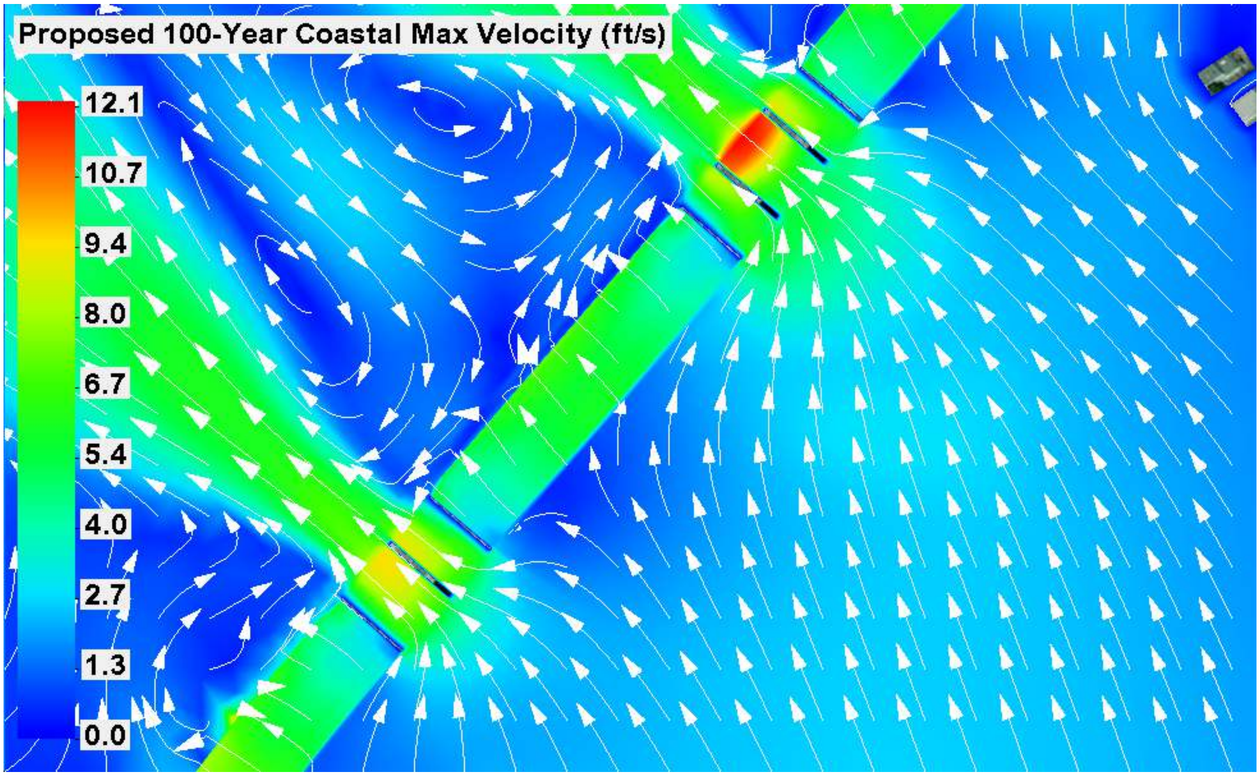

Figure 48. Proposed 100-Year Coastal Max Velocities

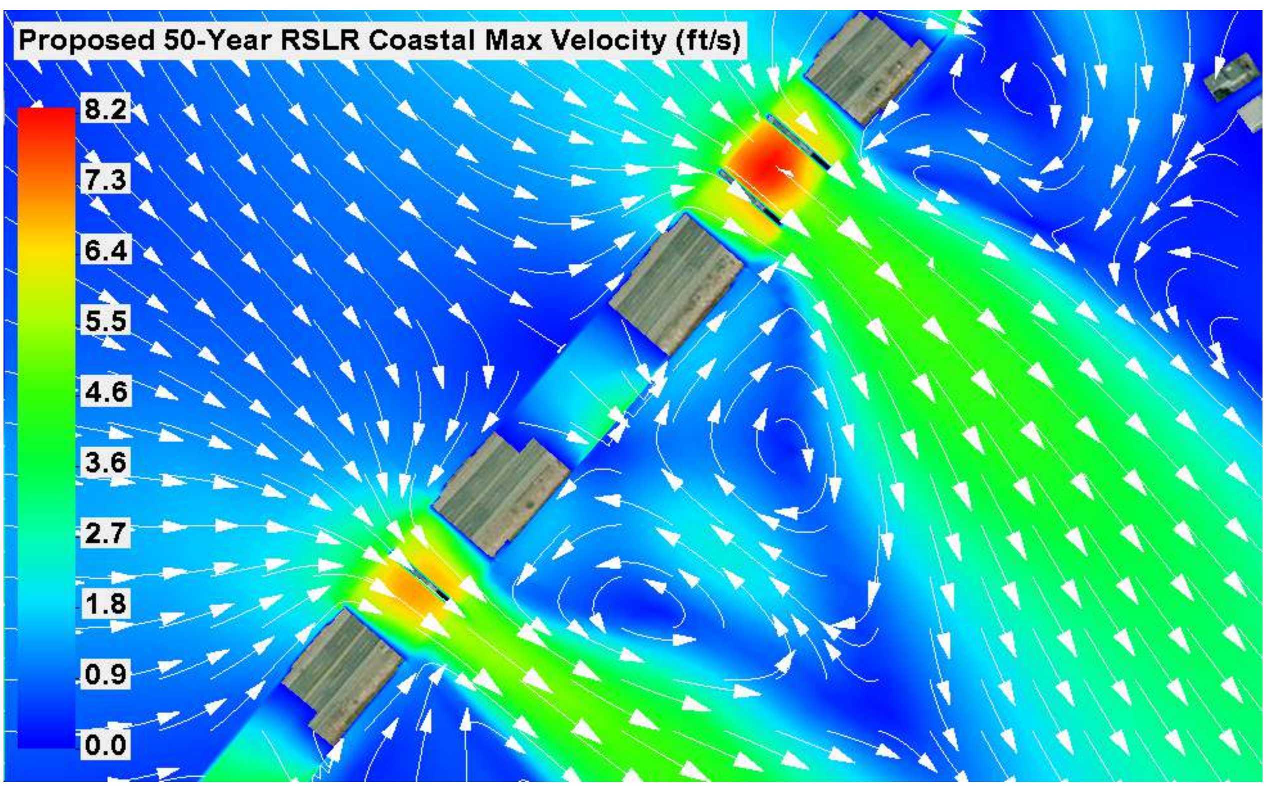

Figure 49. Proposed 50-Year RSLR Coastal Max Velocities

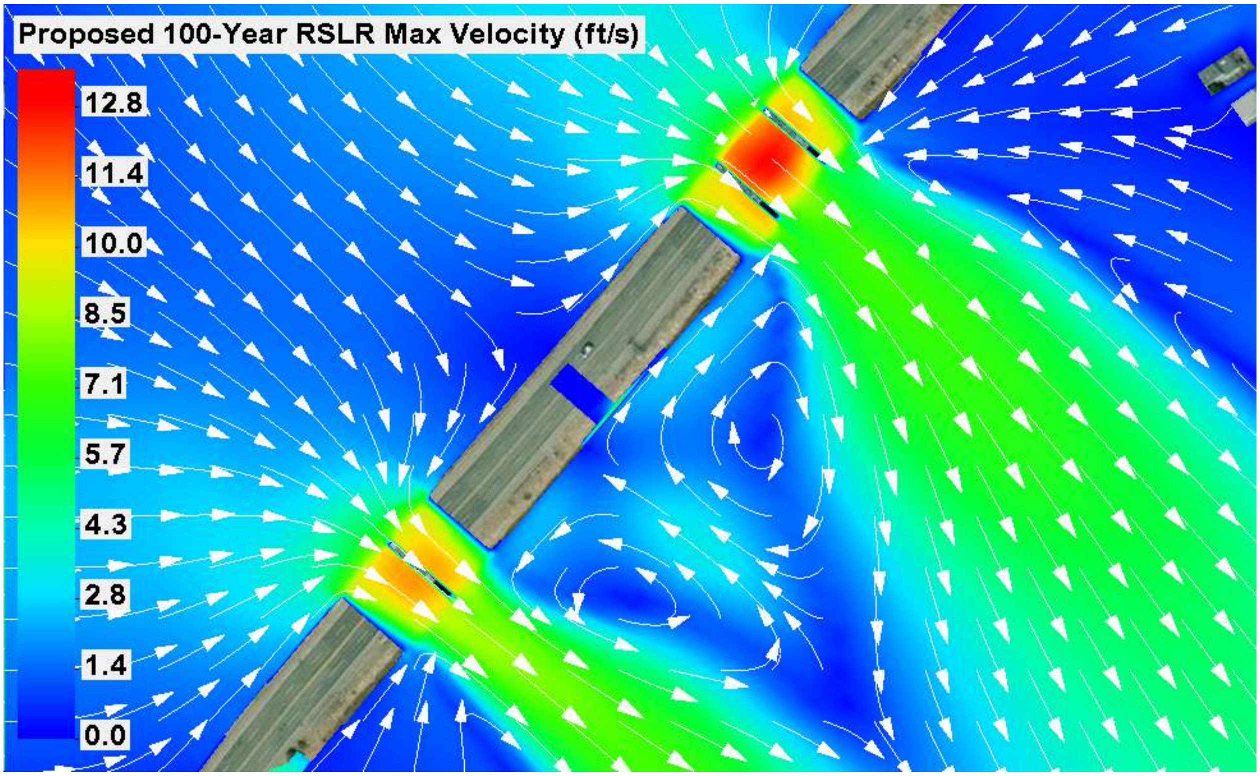

Figure 50. Proposed 100-Year RSLR Coastal Max Velocities

4.3.3 Spectral Wave Modeling

A spectral wave numerical model was developed in the MIKE21 Spectral Wave (SW) Flexible Mesh (FM) software to assess potential wave conditions at the proposed bridge during present and future sea level conditions. MIKE21 HD FM is a state-of-the-art commercial software distributed by Danish Hydraulics Institute (DHI) that simulates the growth, decay and transformation of wind-generated waves and swell in offshore and coastal areas. This model is 2D in the sense that it simulates hydrodynamics using vertically depth averaged equations.

4.3.3.1 Horizontal and Vertical Datum

Spectral wave modeling was conducted in a horizontal datum of NAVD83 (2011) State Plane, Massachusetts, in units of meters, and a vertical datum of NAVD88, in units of meters.

4.3.3.2 Elevation Data

Elevation data for the spectral wave model is discussed in Section 3.1.

4.3.3.3 Model Domain

During usual water level conditions, the Marion-Wareham bridge is shielded from wind waves generated in Buzzards Bay. However, the peninsula of land that blocks wave action is low in elevation and would be inundated during 100-year return period surge events.

Figure 51 shows the approximate topographic contours associated with the 100-year and 100-year with RSLR storm events, estimated using FEMA's 100-year AEP value.

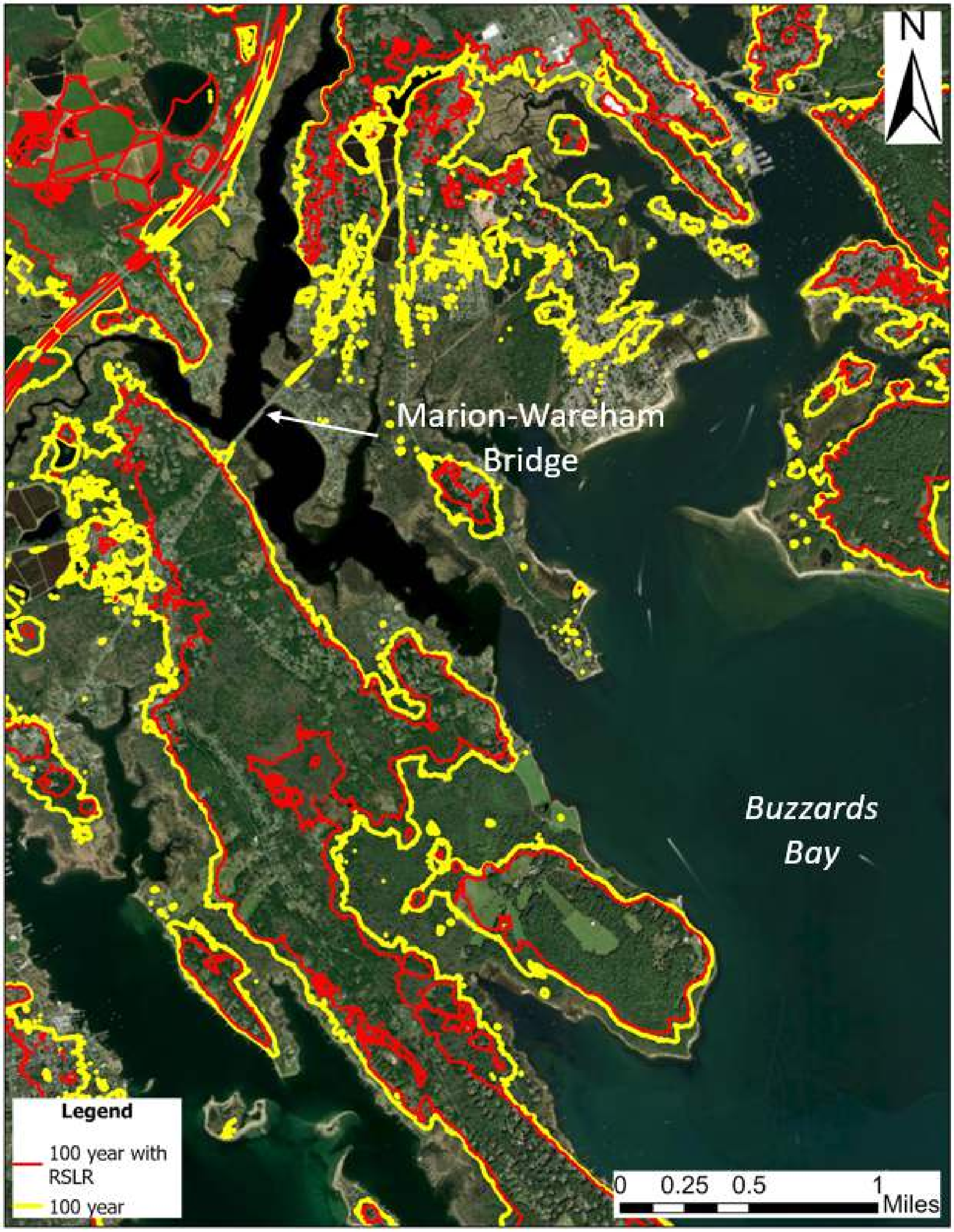

Figure 51. Approximate Topographic Contours for the 100-year Surge with and without RLSR.

Based on the approximate extents of flooding during the return period water levels there is potential for waves created within Buzzards Bay to reach the bridge. The hydrodynamic model grid was adjusted to capture the longest fetch (approximately 8 miles from east southeast) anticipated to impact the project site. Portions of the grid that would not factor into wave propagation towards the project site were removed to reduce computational time. Overall resolution of the grid was kept the same as described in Section 4.3.2.3. Figure 52 shows the adjusted grid used for the SW model.

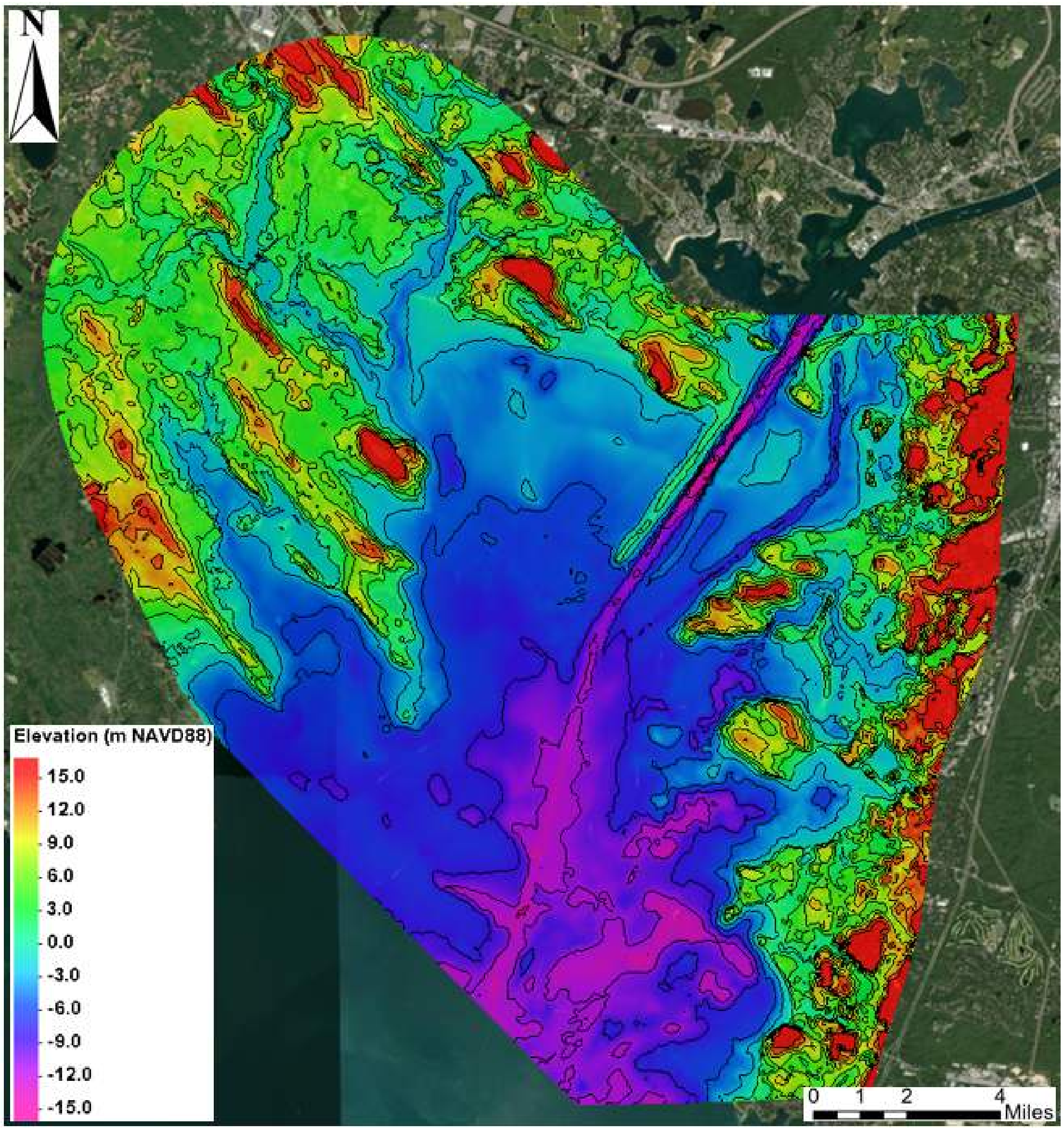

Figure 52. MIKE21 SW Model Grid Extents

4.3.3.4 Model Forcing Conditions

This modeling effort was primarily focused on waves generated locally by wind within Buzzards Bay and the Weweantic River. Due to the topography of the area and distance from the Atlantic Ocean, waves generated in the open ocean are unlikely to reach the project site and were not modeled.

4.3.3.4.1 Wind

The 100-year wind speed were extracted from the ASCE Hazard Tool which displays wind data from ASCE/SEI 7-22. This source of wind data provides the most up-to-date and coordinated loading provisions for general structural design. The windspeeds reported by ASCE are 3-second gust values measured 33 feet above ground level. To be appropriately applied in the model, the windspeeds were converted to 45-minute sustained winds using USACE guidance (USACE, 1984). Note that these windspeeds also needed to be converted from overland to overwater windspeeds. Table 14 summarizes the conversion of the 100-year windspeeds. The model applied this wind speed as a spatially and temporally uniform wind field at 10-degree increments from 70 to 200 degrees.

4.3.3.4.2 Water Level

Water levels in the model were set constant for the respective runs. SW modeling was performed for the 100-year water surface elevation with and without RSLR. Table 22 shows the WSEs applied in the wave model.

Table 21. MIKE21 SW Model Windspeeds

Case	Overland 3 sec Gust (mph)	Openwater Factor	Overwater 3 sec Gust (mph)	45 min Sustain Windspeed (mph)
25-year	85	1.1	94	63
100-year	104	1.1	115	78

Table 22. WSEs applied in MIKE21 SW Modeling

Case	WSE (ft) NAVD88
25-year	+9.6
100-year	+16.0
RSLR 100 year	+21.0

4.3.3.5 Model Results

The significant wave heights were extracted on the downstream side of each bridge bent. Table 23 compares the significant wave height between model runs. The largest wave heights originated from the southeast at 120 degrees. The controlling wave heights and their corresponding peak period and wave direction for each case are shown in Table 23. Model results at each bridge bent were used in hydraulic loading calculations as discussed in Section 4.5.

Table 23. Comparison Between Models Runs

Cases	Significant Wave Height (ft)	Peak Period (s)	Wave Direction (deg)
25-year	2.5	2.8	130
100-year	4.3	3.3	120
RSLR 100-year	5.5	3.8	120

4.4 Maximum Water Surface Elevation

Following AASHTO guidelines, the wave crest elevation was calculated by assuming 70% of the maximum wave height lies above the maximum surge elevation. AASHTO also approximates that the maximum wave height is the significant wave height (H_s) times a factor of 1.8. The results for the maximum wave crest elevations for the different conditions is shown in Table 24.

Table 24. Maximum Wave Crest Elevation for All Scenarios

Scenario	Still Water Level (ft NAVD88)[1]	H_s (ft)[2]	Maximum Wave Height (ft)	Maximum Wave Crest Elevation (ft, NAVD88)
25-year	+9.6	2.5	4.5	+12.8
100-year	+16.0	4.3	7.7	+21.4
100-year w/ RSLR	+21.0	5.5	9.9	+27.9
[1] Still water level taken from the MIKE21 HD Model results.				
[2] H_s taken from the MIKE21 SW model results.				

4.5 Hydraulic Loadings

Hydraulic loadings from waves and currents were computed for the proposed bridge designs at each of the 3 bents. The typical sections for bridge loading analysis are shown in Figure 53 through Figure 56. The bridge loading analysis followed AASHTO Guidelines for Bridges Vulnerable to Coastal Storms (BVCS) (AASHTO, 2008). BVCS provides an approach for evaluating loadings on the superstructure and the substructure, both of which were computed for this assessment. For Weweantic River, all hydraulic conditions were perpendicular to the bridge, so longitudinal loads were not included in the analysis. Table 25 summarizes the hydraulic input in the model runs.

Table 25. Hydraulic Conditions Applied in Loading Analysis

Loading Case	Still Water Level (ft NAVD88)	Hs (ft)	Tp (s)	Current Velocity* (ft/s)
25-year	+9.6	2.5	2.8	7.8
100-year w/ RSLR	+21.0	5.5	3.8	7.5

** Direction is considered perpendicular to the bridge in line with wave action*

The complete hydraulic loadings are provided in Appendix F. A summary of the maximum forces is provided in Table 26. The analysis provides two superstructure loading conditions: (1) the maximum vertical force with the accompanying horizontal force and (2) the maximum horizontal force and the accompanying vertical force. Each of these conditions should be evaluated independently. All loadings account for a 1.75 wave load factor as recommended in BVCS.

Table 26. Summary of Hydraulic Loading Calculations

	25-yr Force	100-yr w/ RSLR Force
Superstructure Max. Vertical Force	9.1 kips/ft	66.1 kips/ft
Superstructure Max. Horizontal Force	1.2 kips/ft	5.8 kips/ft
Substructure Max. Force	320.3 kips	307.1 kips

**Loadings in table include a 1.75 load factor as recommended in AASHTO BVCS (2008)*

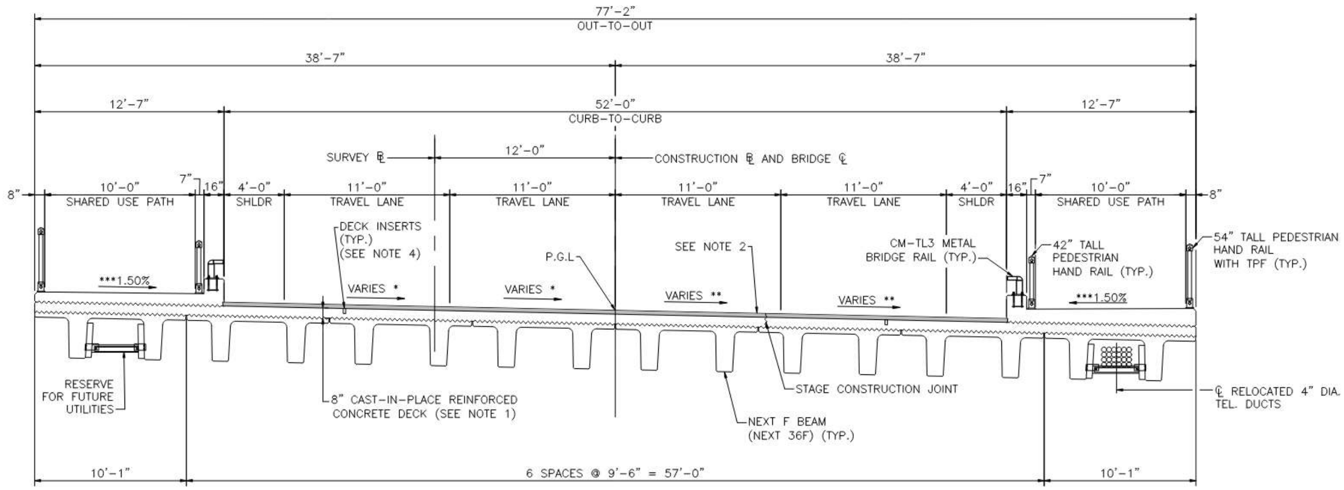

Figure 53. Superstructure Design for Bent 1 Applied for Hydraulic Analysis

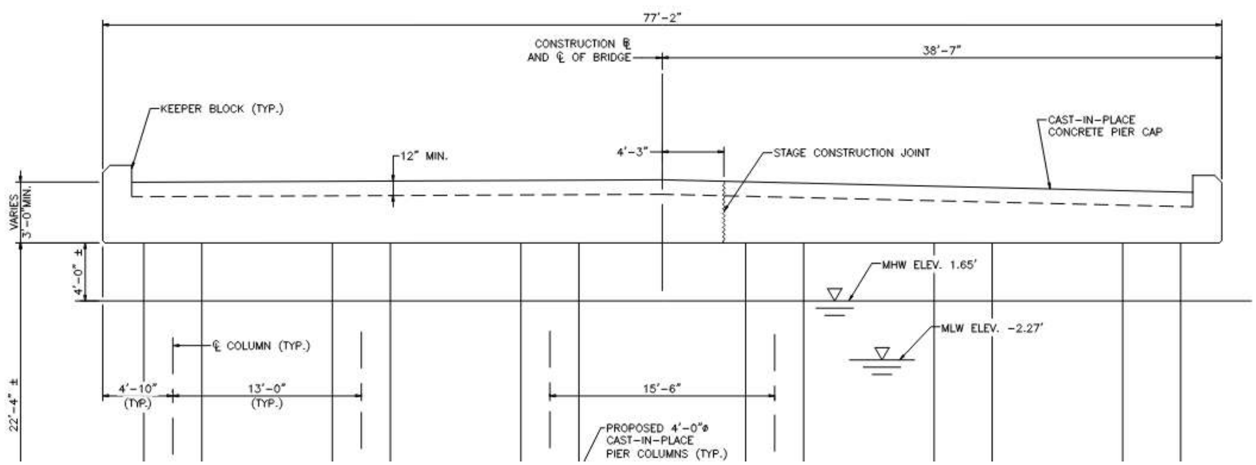

Figure 54. Pile Bent Elevation for Bent 1

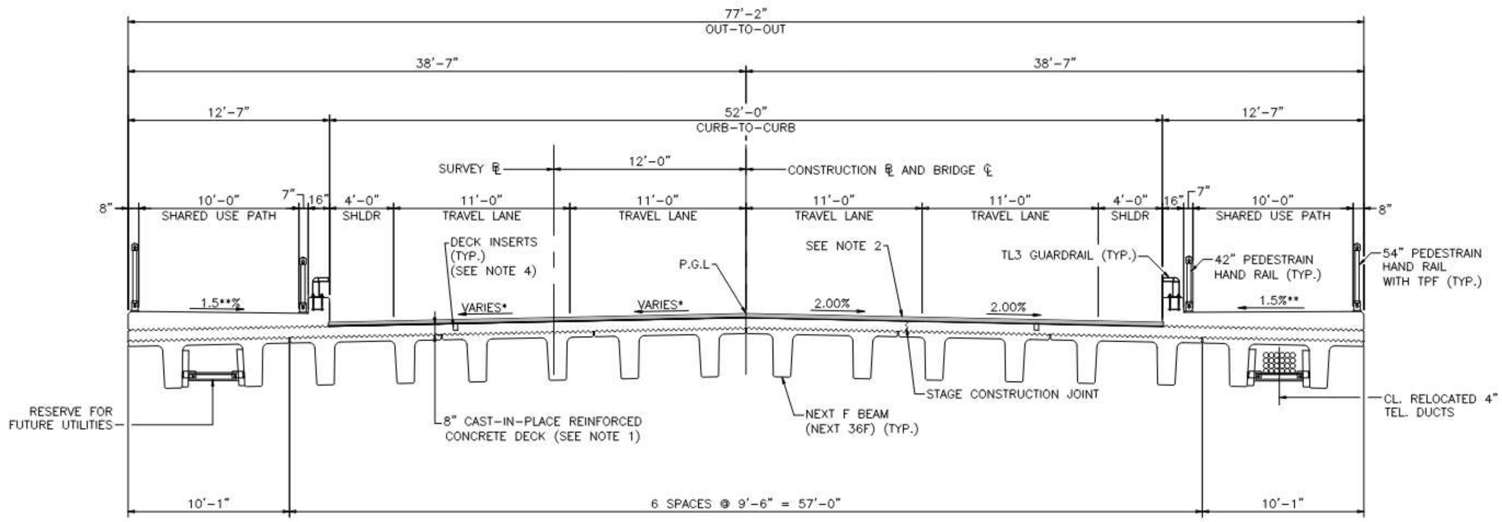

Figure 55. Superstructure Design for Bent 2 and 3 Applied for Hydraulic Analysis

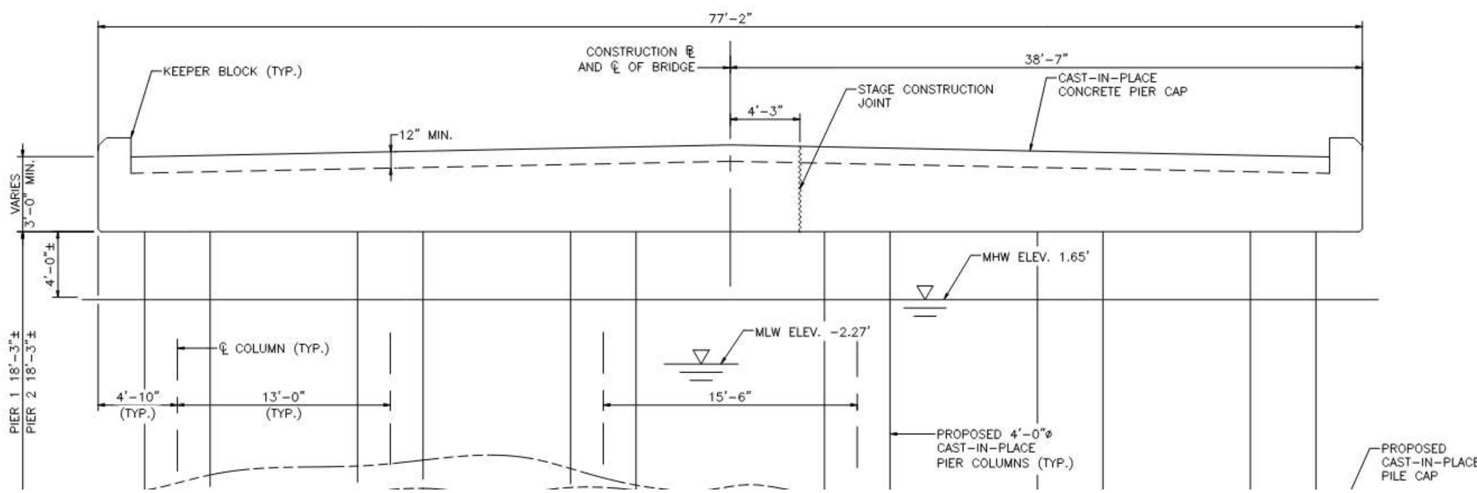

Figure 56. Pile Bent Elevation for Bent 2 and 3

5 Scour Analysis

A scour analysis for the proposed structures was performed in accordance with the guidelines set forth by the FHWA through their publication, Hydraulic Engineering Circular Number 18, *Evaluating Scour at Bridges, Fifth Edition* (HEC-18). Using the parameters provided by SRH-2D hydraulic model and MIKE21, total scour of the proposed bridges was estimated for the 50-year and 100-year riverine floods, 25-year, 50-year, and 100-year coastal floods, and the 50-year and 100-year coastal floods with RSLR. Scour was analyzed for the 25-year coastal flood event due to flow velocities exceeded those of the 50-year coastal flood. This comparison was conducted to determine if the scour is greater for the 25-year flood compared to the 50-year flood, given that the 25-year flood produced higher velocities. It was determined that the 25-year coastal flood event did produce higher scour estimations.

The results of the scour calculations based on this methodology do not account for the bed hardening from previous scour countermeasures and are therefore highly conservative. The following section describe the different components of scour which were analyzed to determine the maximum scour prism for each bridge.

SRH-2D's bridge scour coverage was used to export variables used to calculate scour. The bank arcs at the proposed bridges are located near the existing abutments will be cut off.

Determination of which timestep to obtain data from the numerical model is paramount when an unsteady flow model, such as the coastal model, is used. These large surge events were evaluated to determine when the parameters should be extracted for the calculations. Scour was estimated for three different conditions: maximum velocity before the bridges were in pressure flow, once the bridges were in pressure flow but before overtopping of the structures, and maximum WSE. The timestep chosen was the one that produced the maximum velocity at each of the bridges. The resulting scour estimations from this condition is detailed below.

5.1 Historical Scour

As previously mentioned, these bridges have a history of scour issues, and there are currently scour holes upstream and downstream of both bridges. Bridge plans of the previous bridges at this location were reviewed. This included plans from 1901, 1929, and 1956. It was concluded that the lengths of the bridges have not changed much over time. There has also been a hardening of the channel under the bridge as riprap is called out in multiple plans. Review of the bridge inspection reports reveal the bridges are scour critical.

MassDOT provided a bathymetrical survey for the project area showing the bed of the channel has scour holes upstream and downstream of both bridges, with the more severe of the two being W-06-016. On the Sippican modeled reach, the average bed elevation is approximately -5.8 feet upstream and -8.6 feet downstream. The scour hole upstream of Bridge M-05-001 = W-06-013 has a maximum depth of approximately -20.0 feet, and the downstream scour hole is approximately -18.7 feet. This equates to approximately 10 to 14 feet of scour.

On the Weweantic modeled reach, the average bed elevation is approximately -5.8 feet upstream and -10.2 feet downstream. The scour hole upstream of Bridge M W-06-013 has a maximum depth of approximately -31.3 feet, and the downstream scour hole is approximately -28.2 feet. This equates to approximately 22 to 26 feet of scour.

Figure 57 shows where bathymetric profiles were extracted from the elevation data for the surveyed reach of the Sippican (RED) and Weweantic (BLUE) Rivers. Figure 58 shows the elevation profile for the Sippican River. Note the scour holes located upstream and downstream of Bridge M-05-001 = W-06-013. Figure 59 shows the elevation profile for the Weweantic River. Note the scour holes located upstream and downstream of Bridge W-06-016. The rise at the bridges in both profile plots are likely the hardening of the channel previously mentioned. Riprap placed years ago appears to have hardened the bed underneath the bridges while contraction scour has eroded the surrounding areas.

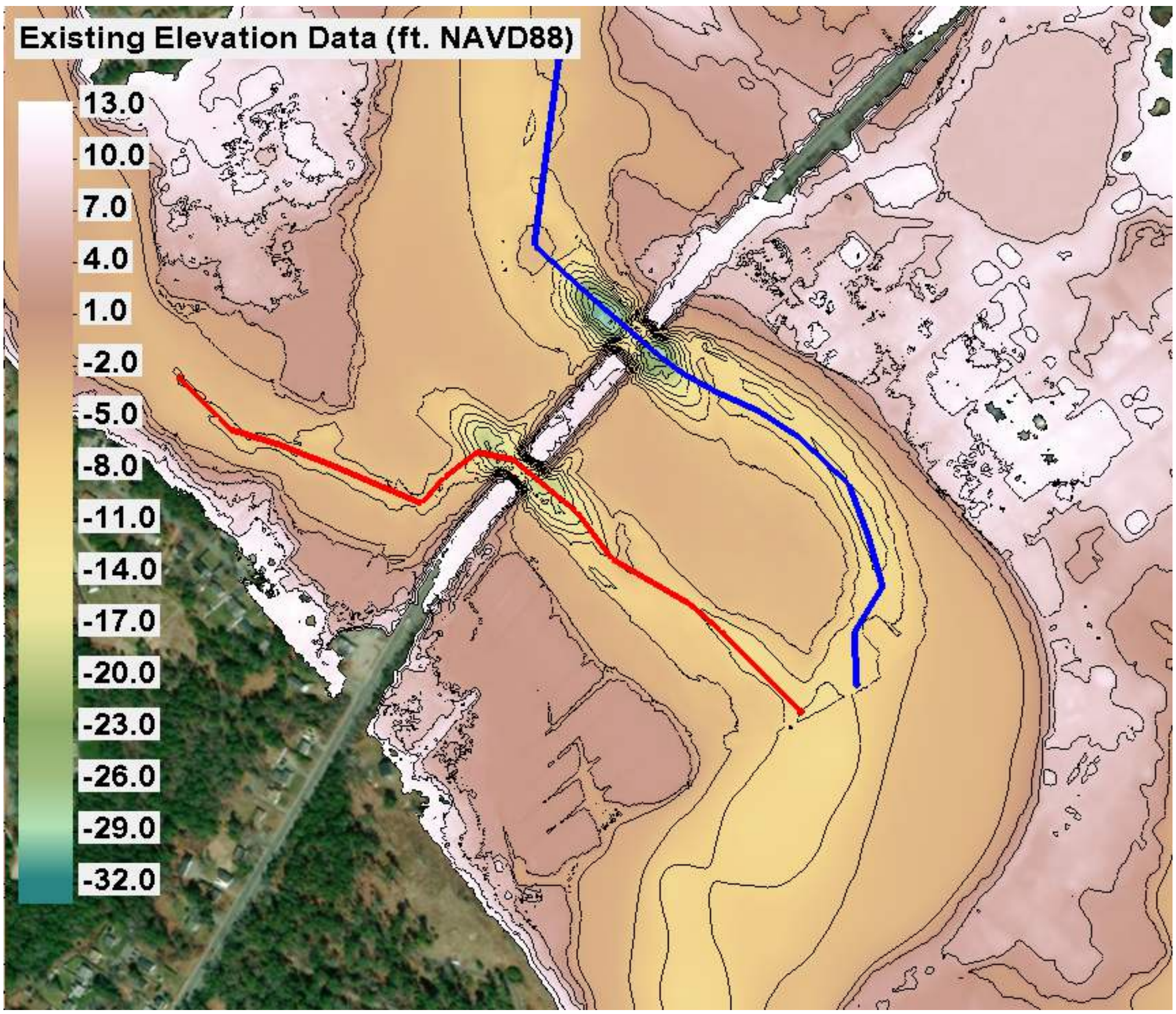

Figure 57. Location of Bathymetric Profiles

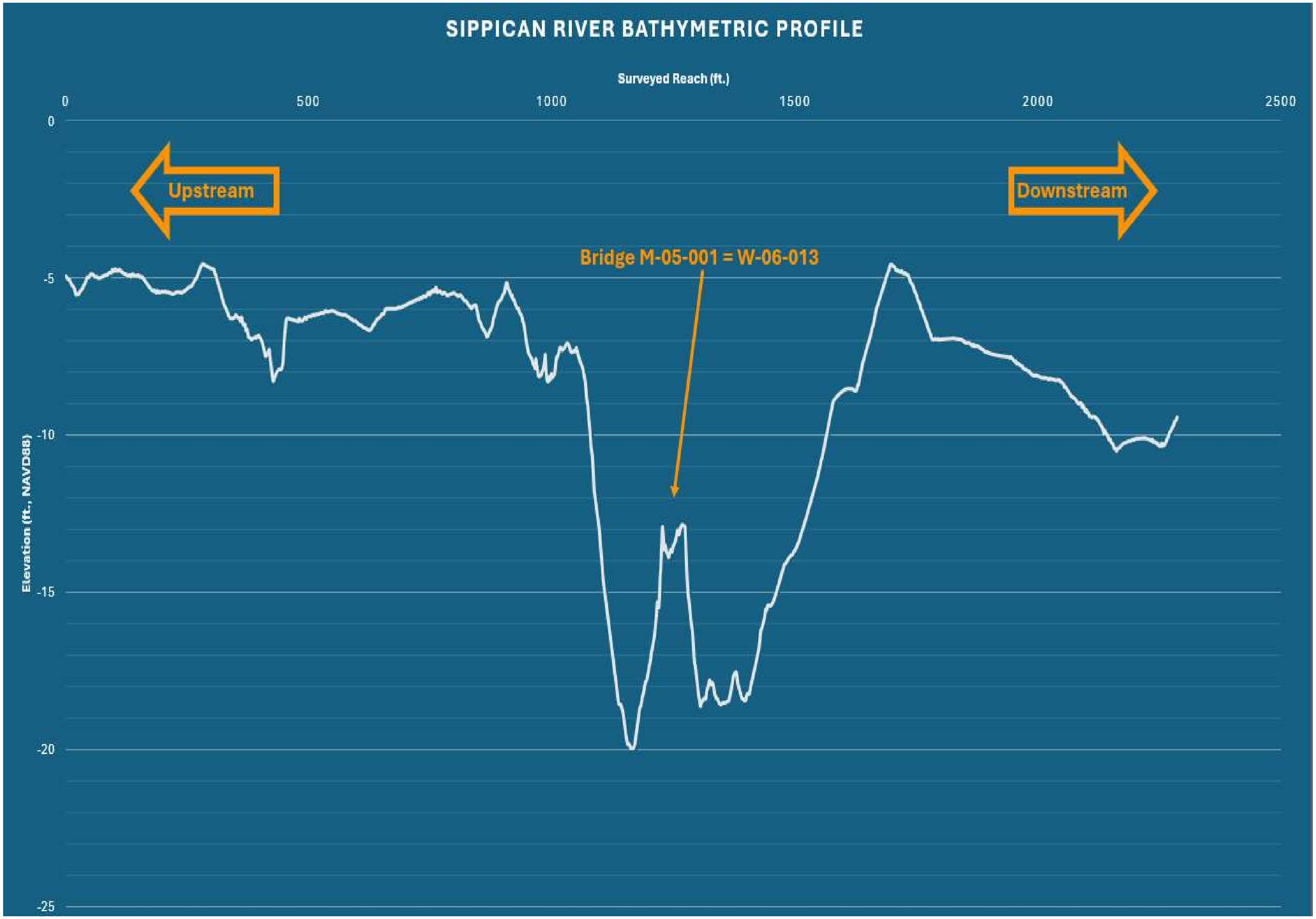

Figure 58. Sippican River Surveyed Reach

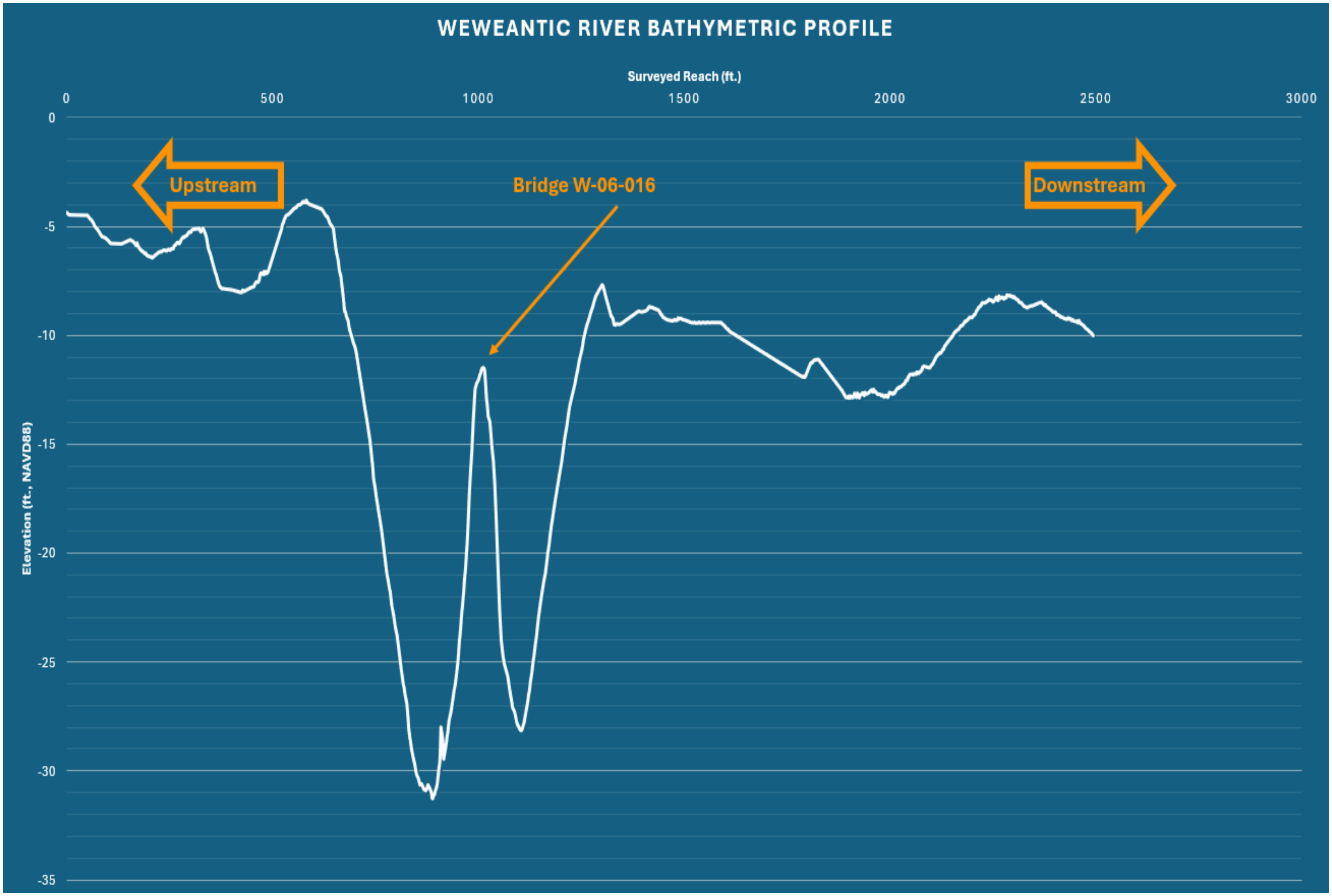

Figure 59. Weweantic River Surveyed Reach

5.2 Long-term Degradation

Due to expedition of this analysis and lack of data, long-term degradation at the bridge sites was not considered.

5.3 Contraction Scour

Contraction scour occurs as a wide flow area is constricted to a narrower, confined flow area. As the flow enters this confined area, velocities increase, causing scour in the constricted area. There are two types of contraction scour: Horizontal and Vertical. Horizontal contraction scour can also be split into two categories: Clear-Water and Live-Bed. Depending on which event is being considered, the appropriate type of contraction scour was calculated for that event.

The maximum velocity used in the calculations occurred either before or after pressure flow occurred. Therefore, only horizontal contraction scour is estimated in this report.

HEC-18 describes four conditions in which contraction scour can occur at bridges depending on if there is overbank flow or relief bridges. However, the main factor of these condition is whether there is active movement of sediment in the flow, i.e. if the condition is Clear-Water (CW) or Live-Bed (LB).

The Clear-Water condition is defined as flow upstream of the bridge is not actively transporting sediment, hence the water is clear. This occurs when the flow is not actively transporting sediment from the upstream reach to the downstream reach, or the sediment that is being transported is mostly in suspension at a lower capacity than the allowed by the flow. This condition is most common in overbank situations and relief bridges.

The Live-Bed condition is defined as flow upstream of the bridge is actively transporting sediment, hence the stream bed is actively changing. This occurs when the velocity of the flow is great enough to dislodge and/or move sediment from the upstream reach and through the bridge section. This condition is most common in the main channel of a watercourse.

Contraction scour depths may be limited by armoring, which may be the case for these two sites. If it is concluded the bed is armored, HEC-18 recommends calculating both Live-Bed and Clear-Water conditions and taking the smaller of the two values.

It should be noted that the soil samples obtained from the site have an average D_{50} of 0.40. This size was used in the scour calculations.

Horizontal contraction scour depths at each proposed bridge for the 50-year and 100-year riverine floods, 25-year, 50-year, 100-year coastal floods, and 50-year and 100-year coastal floods with RSLR are shown in Table 27.

Table 27. Contraction Scour

Contraction Scour Depth (ft)	Proposed Bridge M-05-001	Proposed Bridge W06-016
50-Year Riverine	0	0
100-Year Riverine	0	0
25-Year Coastal	24.13	23.68
50-Year Coastal	13.33	18.53
100-Year Coastal	24.05	16.97
50-Year RSLR Coastal	15.22	23.68
100-Year RSLR Coastal	20.40	26.77

5.4 Local Pier Scour

Local pier scour was estimated on each pier for the proposed bridge using the procedures outlined in Chapter 7 of HEC-18. This method is appropriate for both clear-water and live-bed scour conditions. It should be noted again that the HEC-18 pier scour equation does not consider the hardening of the stream bed beneath the bridges, and is therefore, likely conservative.

The substructure of the intermediate bents of the proposed bridges consists of piers above the pile cap supported by drilled socket piles. This configuration would normally be considered a complex pier. However, complex pier scour equations in HEC-18 were developed for a group of piles with multiple columns, whereas the proposed Marion-Wareham bridges consist of a single column of piles. Therefore, it was determined that simple pier (multiple columns skewed to flow) scour equations (Section 7.6 of HEC-18) would be used to estimate the local pier scour at the intermediate bents of the bridges.

Instead of using the diameter of the piers (4 feet) in the scour calculations, the diameter of the piles (5 feet) was used in the scour calculations. This is considered a more conservative approach.

Local pier scour depths at each pier of the bridge for 50-year and 100-year riverine floods, 25-year, 50-year, and 100-year coastal flood, and 50-year and 100-year coastal flood with RSLR are shown in Table 28.

Table 28. Local Pier Scour

Local Pier Scour Depth (ft)	Proposed Bridge M-05-001	Proposed Bridge W06-016	
	Pier 1	Pier 1	Pier 2
50-Year Riverine	4.59	10.19	4.69
100-Year Riverine	4.30	10.70	4.88
25-Year Coastal	14.82	17.60	15.80
50-Year Coastal	13.59	15.52	15.48
100-Year Coastal	8.40	18.70	7.92
50-Year RSLR Coastal	8.18	15.90	10.36
100-Year RSLR Coastal	11.53	19.83	13.18

5.5 Local Abutment Scour

Chapter 8 of HEC-18 outlines various methodologies for estimating local scour at abutments. The NCHRP 24-20 methodology is the preferred methodology and was utilized to estimate the local abutment scour of the bridge. FHWA has recently announced that the other methods tend to be overly conservative and should no longer be used to estimate abutment scour depth.

The NCHRP 24-20 abutment scour equations were used to estimate the scour at the abutments. However, it should be noted that there are no abutment scour equations in current HEC-18 methodologies. Pressure flow is considered an extreme case, and an abutment scour countermeasure is recommended.

The NCHRP 24-20 method predicts the total scour at the abutment, i.e., both the contraction scour and the localized abutment scour components combined. Either clear-water or live-bed NCHRP 24-20 abutment scour equations are used based on the appropriate conditions.

The NCHRP 24-20 abutment scour method consider three different scour condition types. Types a and b consider the proximity of the abutment to the channel, while type c is the scour condition resulting from failure of the roadway embankment approach. For this analysis, only types a and b were considered. More specifically, HEC-18 describes type a scour as occurring when the abutment is in or close to the main channel, and type b scouring as occurring when the abutment is set back from the main channel. Evaluating the cross-sectional opening at the bridges shows there is no sign of a floodplain evident. Therefore, the abutment is close to the channel, and type a abutment scour calculations were used.

The proposed bridges are designed with vertical wall abutments. Therefore, the NCHRP 24-20 scour amplification factor figure for vertical/wingwall abutments was used to estimate the abutment scour. The abutment arcs in the SRH-2D bridge scour coverage, which determine where the variables from the equation are obtained, are located near the permanent abutment sheeting.

The vertical wall abutments did not get wet during the riverine floods evaluated; therefore, there is no abutment scour for the 50-year and 100-year riverine events evaluated.

Local abutment scour values for the 50-year and 100-year riverine flood, 25-year, 50-year, and 100-year coastal flood, and the 50-year and 100-year coastal flood with RSLR are shown in Table 29.

Table 29. Local Abutment Scour

NCHRP Abutment Scour Depth (ft)	Proposed Bridge M-05-001		Proposed Bridge W06-016	
	West Abutment	East Abutment	West Abutment	East Abutment
50-Year Riverine	0	0	0	0
100-Year Riverine	0	0	0	0
25-Year Coastal	42.93	43.30	45.91	45.88
50-Year Coastal	31.75	32.05	44.07	44.07
100-Year Coastal	40.43	40.69	37.80	37.63
50-Year RSLR Coastal	30.81	31.12	45.73	45.70
100-Year RSLR Coastal	39.23	39.18	52.99	52.91

5.6 Wave Scour

Wave scour is typically not a controlling factor in unless subject directly to waves from the open ocean. Due to this reason and the expedition of this analysis, wave scour was not calculated for this project.

5.7 Total Scour

The summation of long-term degradation, local scour, and contraction scour provides a total scour depth estimation. For this site, long-term degradation was deemed negligible, and is therefore not considered in this summation. After evaluating and comparing the scour values based on the riverine conditions and the coastal surge events with and without RSLR, it was determined that the coastal events generate more scour than the riverine flows. Table 30 and Table 31 show the total scour depths calculated at proposed Bridge M-05-001 and proposed Bridge W-06-016, respectively.

Table 30. Proposed Bridge M-05-001 Total Scour Depths

Proposed Bridge M-05-001	50-Year Riverine Total Scour Depth (ft)	100-Year Riverine Total Scour Depth (ft)	25-Year Coastal Total Scour Depth (ft)	50-Year Coastal Total Scour Depth (ft)	100-Year Coastal Total Scour Depth (ft)	50-Year Coastal RSLR Total Scour Depth (ft)	100-Year Coastal RSLR Total Scour Depth (ft)
West Abutment	0	0	42.93	31.75	40.43	30.81	39.23
Pile 1	4.59	4.30	38.95	26.92	32.45	23.40	31.93
East Abutment	0	0	43.30	32.05	40.69	31.12	39.18

Table 31. Proposed Bridge W-06-016 Total Scour Depths

Proposed Bridge W-06-016	50-Year Riverine Total Scour Depth (ft)	100-Year Riverine Total Scour Depth (ft)	25-Year Coastal Total Scour Depth (ft)	50-Year Coastal Total Scour Depth (ft)	100-Year Coastal Total Scour Depth (ft)	50-Year Coastal RSLR Total Scour Depth (ft)	100-Year Coastal RSLR Total Scour Depth (ft)
West Abutment	0	0	45.91	44.07	37.80	45.73	52.99
Pile 1	10.19	10.70	41.28	34.05	35.66	39.58	46.60
Pile 2	4.69	4.88	39.48	34.02	24.89	34.04	39.95
East Abutment	0	0	45.88	44.07	37.63	45.70	52.91

5.8 Scour Summary

The 25-year coastal flood produces more total scour than the 50-year coastal flood and the 100-year coastal flood. The 25-year coastal surge begins at a low tide while the other events begin at a mid to high tide. As a result, the 25-year event creates less overland flow and therefore constricts the flow, increasing the velocities. The lower water level in the 25-year event is estimated to create a deeper scour hole than a higher water level with the same velocity.

Since the more recurring 25-year coastal flood produces scour depths that exceed the 50-year coastal scour design event, the 25-year coastal flood will effectively serve as the scour design event. The scour check surge events for Bridge M-05-001 were determined to be the coastal events without RSLR since their scour depth estimations were greater, and therefore controlled than the coastal events with RSLR. However, the scour check event for Bridge W-06-016 are the coastal events with RSLR since their estimated scour depths govern over the coastal events without RSLR.

It is recommended that the piles of the bridges are constructed to be deeper than the scour depth estimations of the scour design and check flood events.

The top-width of a local scour hole ranges from 2.0 to 5.6 times the depth of the local scour, depending on site characteristics. HEC-18 suggests using 4.0 times the depth of the of the scour hole to represent the top-width for practical purposes. This equates to a scour having 2H:1V side-slopes when plotted. When all scour is plotted, there may be overlapping of these holes. This results in indeterminate scour, which may possibly result in deeper scour than the independent estimates at one or the other. Therefore, HEC-18 recommends evaluating for this overlap, and if it exists, the design should be reevaluated. When the scour holes are plotted, the pier and the abutment scour holes do overlap.

The scour elevations should be determined by subtracting the total scour depth from the minimum ground elevation at each bent.

Table 32 and Table 33 display the maximum scour value component calculated from each flood and the design total scour depth for Bridge M-05-001 and Bridge W-06-016, respectively.

Table 32. Proposed Bridge M-05-001 Scour Summary

Return Frequency	Contraction Scour (ft)	Maximum Local Pier Scour (ft)	Long Term Degradation (ft)	Maximum Abutment Scour (ft)	Design Total Abutment Scour (ft)	Design Total Pier Scour (ft)
Riverine 50-Year	0	4.59	NA	0	0	4.59
Riverine 100-Year	0	4.30	NA	0	0	4.30
Coastal 25-Year	24.13	14.82	NA	43.30	43.30	38.95
Coastal 50-Year	13.33	13.59	NA	32.05	32.05	26.92
Coastal 100-Year	24.05	8.40	NA	40.69	40.69	32.45
Coastal 50-Year RSLR	15.22	8.18	NA	31.12	31.12	23.40
Coastal 100-Year RSLR	20.40	11.53	NA	39.23	39.23	31.93

NA – Not Applicable

Table 33. Proposed Bridge W-06-016 Scour Summary

Return Frequency	Contraction Scour (ft)	Maximum Local Pier Scour (ft)	Long Term Degradation (ft)	Maximum Abutment Scour (ft)	Design Total Abutment Scour (ft)	Design Total Pier Scour (ft)
Riverine 50-Year	0	10.19	NA	0	0	10.19
Riverine 100-Year	0	10.70	NA	0	0	10.70
Coastal 25-Year	23.68	17.60	NA	45.91	45.91	41.28
Coastal 50-Year	18.53	15.52	NA	44.07	44.07	34.05
Coastal 100-Year	16.97	18.70	NA	37.80	37.80	35.66
Coastal 50-Year RSLR	23.68	15.90	NA	45.73	45.73	39.58
Coastal 100-Year RSLR	26.77	19.83	NA	52.99	52.99	46.60

NA – Not Applicable

6 Scour Countermeasure Recommendations

Since pressure flow occurs during the coastal storms evaluated, an abutment countermeasure is required.

According to the MassDOT manual, the scour countermeasure for the abutments should be designed for the scour check flood. For scour countermeasure design purposes, 100-year coastal surge without RSLR was used to design the abutment scour countermeasures at both bridges. When using variables from the 100-year coastal surge with RSLR, the countermeasure design would be overconservative and very costly.

The MassDOT Manual states that for narrow bridge openings and where the scour check flood elevation exceeds the bridge's low chord elevation, riprap should be placed through the full width of the channel, from abutment to abutment. The WSE produced from the 100-year coastal surge check scour event exceeds the low chord of both of the bridges; therefore, the entire width of the channel should be riprapped for both bridges.

The scour countermeasure design at the bridge abutments for the scour check flood event for proposed Bridge M-05-013 and proposed Bridge W-06-016 are displayed in Table 38 and Table 39, respectively.

Table 34. Proposed Bridge M-05-001 Scour Countermeasure Design at the Bridge Abutments for the Scour Check Flood Event

Proposed Bridge M-05-001	Class #	Riprap Thickness (ft)	Riprap Extend from Abutment (ft)	Riprap Extend along u/s and d/s of the Embankment (ft)
Proposed Alternative 1	IV	2.5	25	25

Table 35. Proposed Bridge W-06-016 Scour Countermeasure Design at the Bridge Abutments for the Scour Check Flood Event

Proposed Bridge W-06-016	Class #	Riprap Thickness (ft)	Riprap Extend from Abutment (ft)	Riprap Extend along u/s and d/s of the Embankment (ft)
Proposed Alternative 1	V	3	25	25

The gradation requirements for the scour countermeasure design for Bridge M-05-001 and Bridge W-06-016 are displayed in Table 36 and Table 37, respectively.

Table 36. Bridge M-05-001 Gradation Requirements for Riprap

Class	% of Rock Equal or Smaller by Count, Dx	Range of Intermediate Dimensions, inches	Range of Rock Mass, pounds
IV	100	30.0	2,200
IV	85	19.5 – 23.0	600 – 1,000
IV	50	14.5 – 17.5	240 – 420
IV	15	9.2 – 13.0	62 - 180

Table 37. Bridge W-06-016 Gradation Requirements for Riprap

Class	% of Rock Equal or Smaller by Count, Dx	Range of Intermediate Dimensions, inches	Range of Rock Mass, pounds
V	100	36.0	3,800
V	85	23.5 – 27.5	1,050 – 1,750
V	50	17.0 – 20.5	410 – 720
V	15	11.0 – 15.5	110 - 310

It is recommended that a geotextile filter layer is placed underneath the riprap. The riprap at the abutments should be sloped at a minimum of 2H:1V and the entire channel width should be riprapped underneath the bridges.

HEC-23 states that the rock riprap thickness should be increased by 50% when it is placed under water to provide for the uncertainties associated with this type of placement and that if an abutment encroaches into the main channel, then a riprap toe or key should be considered.

7 Conclusions and Recommendations

The scour analysis for the proposed design of Bridge M-05-001=W-06-013 and Bridge W-06-016 was performed using guidance set forth by FHWA. Observing the scour holes currently present at the bridges and knowing these equations have safety factors embedded into them, the scour estimations provided in this report were deemed to be realistic for the events they represent.

The conclusion of this analysis is the hardening of the bed appears to be currently preventing scour under the bridge to the magnitude it is observed upstream and downstream. However, there is no way to know how long this hardening will last. If this hardened layer, or a portion of it which could cause a cascading affect, was somehow removed, the resulting scour underneath the bridges would likely be deeper than existing scour holes upstream and downstream of the bridges.

Therefore, this scour investigation of the proposed bridge designs recommends the proposed bent piles should be placed either to a depth below the scour and check-scour envelope, securely embedded into bedrock, or have scour countermeasures designed, installed, maintained, and checked after every large storm event.

Table 38. Proposed Bridge M-05-001 Hydraulic Design Data

Proposed Bridge M-05-001 Hydraulic Design Data Estimated from SRH-2D Outputs		
Drainage Area	89.5	sq. miles
Design Flood Discharge	12,523	cfs
Design Flood Annual Chance (Return Frequency)	25-Year	Coastal (without RSLR)
Design Flood Velocity	8.29 (Flood)	feet/sec
Design Flood Elevation	9.84	feet (NAVD88)
Base (100-Year) Flood Data		
Base Flood Elevation (no RSLR)	16.75	feet (NAVD88)
Base Flood Elevation (RSLR)	21.37	feet (NAVD88)
Design Flood Scour Data		
Scour Design Flood Annual Chance (Return Frequency)	25-Year (scour depths exceed the 50-Year)	Coastal
Design Flood Abutment Scour Depth	43.30	feet
Design Flood Pier Scour Depth	38.95	feet
Check Flood Scour Data		
Scour Check Flood Annual Chance (Return Frequency)	100-year	Coastal
Check Flood Abutment Scour Depth	40.69	feet
Check Flood Pier Scour Depth	32.45	feet
Flood of Record		
Discharge	N/A	cfs
Frequency	N/A	year
Maximum Elevation	N/A	feet (NAVD88)
Date	N/A	
History of Ice Floes	N/A	
Evidence of Scour & Erosion	NBIS inspection records used Item 113 = 3 field review indicates action is required to protect exposed foundations from effects of additional erosion and corrosion.	

Table 39. Proposed Bridge W-06-016 Hydraulic Design Data

Proposed Bridge W-06-016 Hydraulic Design Data Estimated from SRH-2D Outputs		
Drainage Area	89.50	sq. miles
Design Flood Discharge	20,320	cfs
Design Flood Annual Chance (Return Frequency)	25-year	Coastal (without RSLR)
Design Flood Velocity*	10.27 (Flood)	feet/sec
Design Flood Elevation*	9.84	feet (NAVD88)
Base (100-Year) Flood Data		
Base Flood Elevation (no RSLR)	16.77	feet (NAVD88)
Base Flood Elevation (RSLR)	21.34	feet (NAVD88)
Design Flood Scour Data		
Scour Design Flood Annual Chance (Return Frequency)	25-Year (scour depths exceed the 50-Year)	Coastal
Design Flood Abutment Scour Depth	45.91	feet
Design Flood Pier Scour Depth	41.28	feet
Check Flood Scour Data		
Scour Check Flood Annual Chance (Return Frequency)	100-year RSLR	Coastal
Check Flood Abutment Scour Depth	52.99	feet
Check Flood Pier Scour Depth	46.60	feet
Flood of Record		
Discharge	N/A	cfs
Frequency	N/A	year
Maximum Elevation	N/A	feet (NAVD88)
Date	N/A	
History of Ice Floes	N/A	
Evidence of Scour & Erosion	NBIS inspection records used Item 113 = 3 field review indicates action is required to protect exposed foundations from effects of additional erosion and corrosion.	

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