Project ID P026862

HYDRAULIC ANALYSIS
FOR THE
US-21 HARBOR RIVER BRIDGE, BEAUFORT COUNTY, S.C.
FMIS#: P026(862)

PREPARED BY

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Executive Summary

The South Carolina Department of Transportation (SCDOT) plans to replace the US-21 Bridge over Harbor River in Beaufort County, South Carolina. The SCDOT contracted with HDR, Inc. (HDR) to complete the field study, geotechnical investigations, alternatives analysis, conceptual level bridge and roadway designs, asbestos/lead paint surveys, environmental document, wetland jurisdictional determination, preliminary utility report, and preliminary hydraulic design in order to support the SCDOT with preparation of the design build package. Subsequently, HDR contracted INTERA Incorporated (INTERA) to assist in the hydraulic design. Specifically, INTERA was tasked with the bridge hydraulic modeling following SCDOT's guidelines to develop the 10-year water surface with wave height (wave crest elevations) and the 100- and 500-year return period design hydraulic parameters.

To that end, INTERA developed an ADCIRC model (hydraulics) and a SWAN model (waves) based on the existing RMA2 model (hydraulics). The new models include updated survey collected for this project. A limited model calibration compared NOAA predicted tides to simulated tides that produced a percent difference well within FEMA's guidelines. Model results provided the information to develop the 10-year wave crest elevation and 100- and 500-year hydraulic conditions. Table ES.1 presents the maximum significant wave crest elevation for the 10-year wave at 150 ft intervals along the existing bridge alignment. Table ES.2 summarizes the hydraulic data as required for SCDOT plan sets.

Table ES. 1 10-Year Wave Crest Elevation

Distance from the West Abutment (ft)	10-Year Wave Crest Elevation (ft-NAVD88)
0	+13.3
150	+13.5
300	+13.6
450	+13.6
600	+13.5
750	+13.5
900	+13.5
1050	+13.4
1200	+13.5
1350	+13.7
1500	+13.8

Distance from the West Abutment (ft)	10-Year Wave Crest Elevation (ft-NAVD88)
1650	+13.7
1800	+13.7
1950	+13.7
2100	+13.8
2250	+13.9
2400	+13.9
2550	+13.5
2700	+13.4
2850	+13.3

Table ES. 2 Hydrology Data for Tidal Bridges

Hydrology Data for Tidal Bridges: Vertical Datum is NAVD88			
Mean Higher High Tide Elevation	=	+2.97	ft.
Mean Lower Low Tide Elevation	=	-3.72	ft.
10-year Tidal Surge Height	=	+13.7*	ft. (includes wave height)
100-year Stillwater Height	=	+14.8	ft.
500-year Stillwater Height	=	+19.0	ft.
Maximum vel. within bridge	=	100-yr. Tidal Surge Velocity: 8.7 fps	500-yr. Tidal Surge Velocity: 9.6 fps

*Varies along the proposed bridge alignments

1.0 Introduction

The South Carolina Department of Transportation (SCDOT) plans to replace the US-21 Bridge over Harbor River in Beaufort County, South Carolina. The SCDOT contracted with HDR, Inc. (HDR) to complete the field study, geotechnical investigations, alternatives analysis, conceptual level bridge and roadway designs, asbestos/lead paint surveys, environmental document, wetland jurisdictional determination, preliminary utility report, and preliminary hydraulic design in order to support the SCDOT with preparation of the design build package. Subsequently, HDR contracted INTERA Incorporated (INTERA) to assist in the preliminary hydraulic design.

This preliminary hydraulic analysis combines the latest SCDOT and FHWA technical guidelines with hydraulic modeling and coastal engineering methodologies required by the nature of the study area. For the complex coastal hydrodynamics of the study area, the analysis will conform with the Level 3 analysis detailed in SCDOT's "Requirements for Hydraulic Design Studies" and with AASHTO's "Guide Specifications for Bridges Vulnerable to Coastal Storms."

The analysis will establish the 100-year maximum wave crest elevations along the bridge alignment by applying methodologies consistent with AASHTO's *Guide Specifications for Bridges Vulnerable to Coastal Storms* to determine the required clearances (low steel elevation). At the bridge ends, the minimum clearance is set at the 10-year surge plus wave height plus freeboard as discussed in the Hydraulic Requirements.

Following this introduction and a brief description of the study area (Chapter 2), this report addresses the tasks performed to calculate the wave crest elevations at the site associated with the 10-year return period event and 100- and 500-year hydraulic conditions. Chapter 3 describes the setup and application of the hydrodynamic and wave models. Chapter 3 also documents the results of model simulations.

2.0 Study Area

Determining the bridge low chord elevations requires the development of the design hydraulic and wave characteristics. Developing these characteristics requires detailed knowledge of the study area. Factors affecting the hydraulics and waves include the bridge's proximity and connection to the Atlantic Ocean, tidal fluctuations, hurricane storm surge, flows from rainfall runoff, and hurricane wind speeds. This chapter details these parameters at the bridge location.

2.1 Bridge Location

Figure 2.1 displays the bridge location. As the figure illustrates, the bridge crosses the Harbor River near St. Helena Sound. Harbor River connects St. Helena Sound and Port Royal Sound. Harbor River's proximity to and connection with St. Helena Sound and Port Royal Sound with numerous connecting creeks to the Atlantic Ocean provides simultaneous propagation of hurricane surge and waves to the bridge location. Topography in the area consists of grassy marshes and low barrier islands.

Figure 2.1 US-21 Bridge over Harbor River Location Map

2.2 Tidal Benchmarks

Figure 2.2 presents the location of the tidal benchmarks near the bridge. As the figure illustrates, the National Oceanic and Atmospheric Administration (NOAA) station 8668146 lies at the bridge. Table 2.1 presents tidal datums for the NOAA station 8668146. These values represent the 1983 – 2001 tidal epoch and were referenced to NOAA's Charleston (8665530) control tide station.

Figure 2.2 Tidal Benchmark Location Map

Table 2.1 NOAA Station 8668146 Tidal Benchmark Information

Tidal Datum Type	Elevation (ft, NAVD88)
Mean Higher High Water (MHHW)	+2.97
Mean High Water (MHW)	+2.58
Mean Tide Level	-0.46
Mean Sea Level	-0.31
Mean Low Water (MLW)	-3.51
Mean Lower Low Water (MLLW)	-3.72

2.3 Hurricane Storm Surge

Minimum bridge low steel elevation calculations are a function of the wave height and the maximum elevation of the storm surge associated with the 10-, 100-, and 500-year events. The Pooled Fund Study, Tidal Hydraulic Modeling for Bridges, User's Manual (Pooled Fund Study SPR-3 [22]., 1997), provides methods to develop these elevations as a function of time (hydrograph) offshore of the bridge area. Two-dimensional models provide a cost-effective method to propagate the storm surge to the bridge site. The surge hydrographs developed herein serve as the offshore boundary conditions for the two dimensional modeling.

The Pooled Fund Study provides a time dependent relationship (time series) between surge elevation and ADCIRC gage water surface elevation, hurricane forward speed, and radius to maximum winds. ADCIRC gage water surface elevations are provided at locations along the eastern seaboard. Figure 2.3 presents the locations of the ADCIRC gages closest to the bridge site. As the figure illustrates, the closest gage to the bridge is number 417. NOAA Technical Report NWS 38, Hurricane Climatology for the Atlantic and Gulf Coasts of the United States, provides the hurricane parameters — hurricane forward speed and radius to maximum winds — necessary for determining the shape of the hydrograph. Combining the elevations of the storm surge time series with a mean tidal range produces a realistic hydrograph. The mean tide range from the most seaward NOAA station 8668498 (Fripps Inlet) was aligned with the hurricane hydrograph so that the mid-rising tide coincided with the maximum storm surge. Figure 2.4 presents the 10-, 100-, and 500-year hurricane hydrographs.

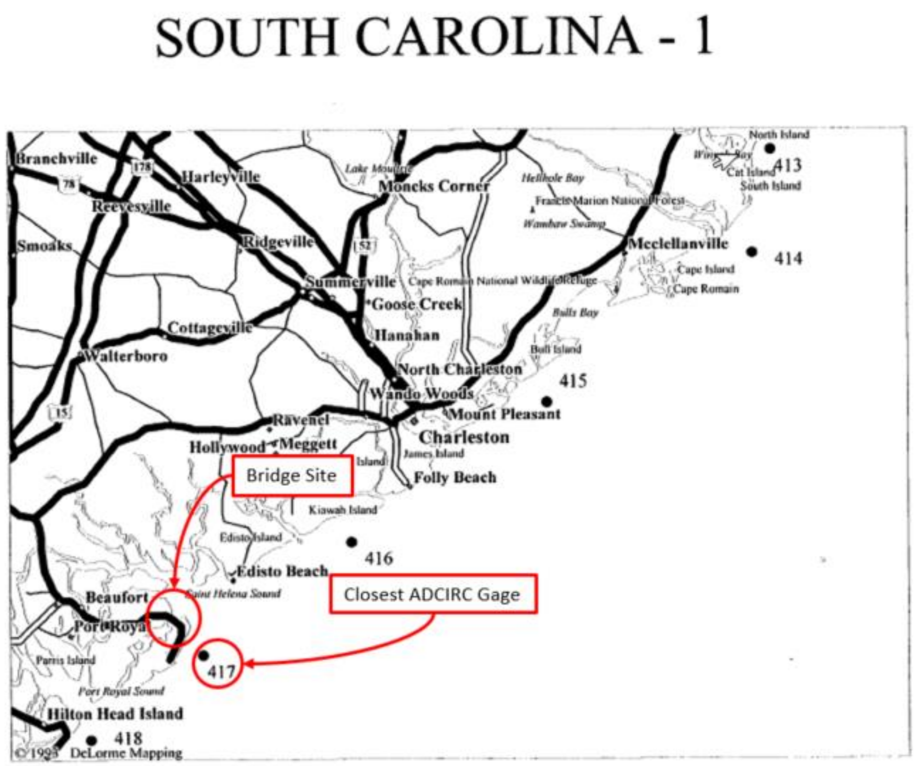

Figure 2.3 Pooled Fund Study ADCIRC Gage Locations near the Bridge

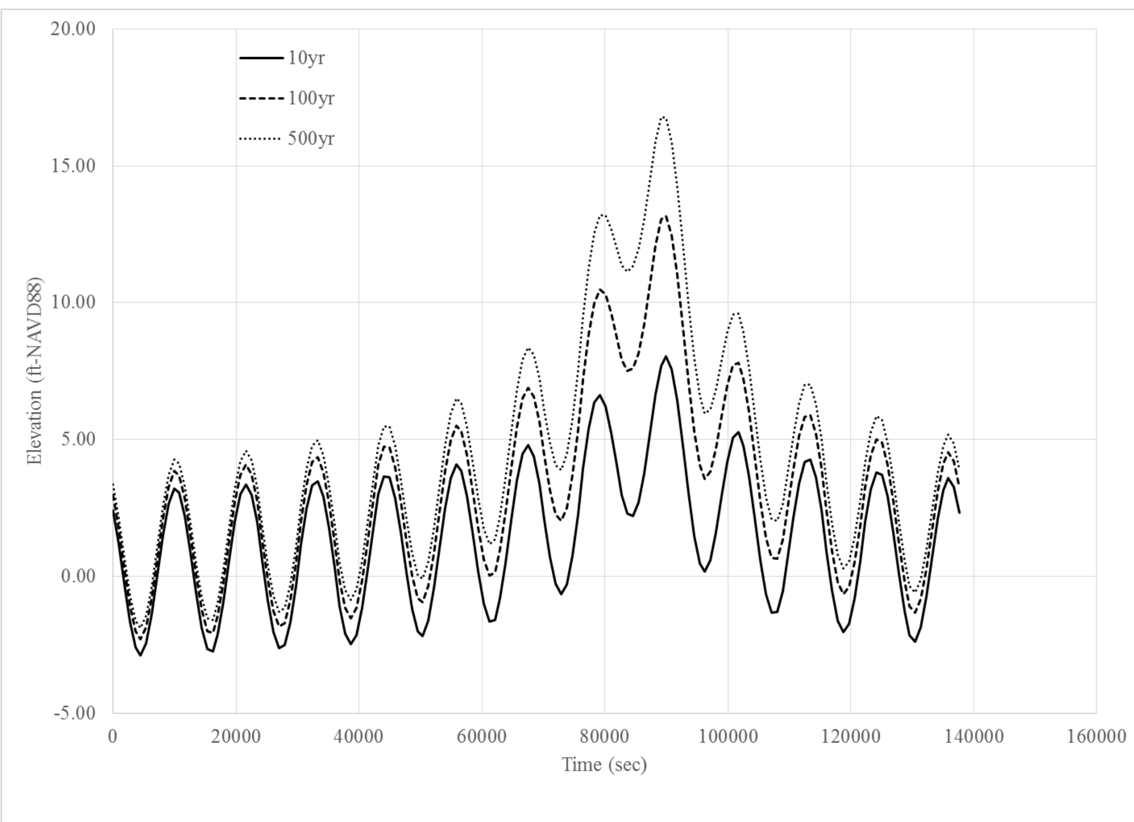

Figure 2.4 Synthetic Hurricane Hydrographs

2.4 Riverine Flows

Four major rivers discharge into the waterways adjacent to the Harbor River: the Coosaw, Morgan, Broad, and Beaufort Rivers. Although the proximity of the bridge crossing ensures the effects of hurricane storm surge dominate the hydraulics, rainfall runoff can affect the storm surge. As a result, some discharge due to rainfall runoff is necessary to accurately develop the hydraulic conditions at the bridge crossing. That said, it is unlikely the two extreme events will occur simultaneously. At this location, the runoff associated with a hurricane would be significantly delayed by the time required for overland flow, resulting in a peak runoff occurring long after the surge peak. However, there will be some flow in these rivers affecting water surface elevations in the Harbor River during hurricane storm surge events. To account for these effects, model boundary conditions will include constant flows equal to the mean annual (2.33-year) flood (the dominant bank full flood). Table 2.2 presents the flows developed from USGS regression equations for the lower coastal plain of South Carolina (Guimaraes and Bohman, 1998). For the Coosaw and Broad Rivers, drainage areas were obtained from USGS South Carolina water resources. For the Beaufort and Morgan Rivers, drainage basins are delineated by the highest contours defining the boundaries between rivers (Hull, 2000).

Table 2.2 Drainage Areas and Flood Discharges

River	Drainage Area (mi^2)	Mean Annual Flood Discharge (cfs)
Coosaw	1,364.5	5,703.4
Broad	761.3	3,957.0
Beaufort	47.2	693.6
Morgan	30.8	530.2

2.5 Hurricane Wind Speeds

Wave height is a key element in the determination of wave loading on bridge superstructures. Waves are generated at the air-sea interface by the action of surface wind stress. The heights of the waves are determined by wind speed, water depth, and the distance the wind blows over the water body (fetch). Wind speed, based on probability, is determined by historic events. The AASHTO *Guide Specification for Bridges Vulnerable to Coastal Storms* (AASHTO, 2008) recommends obtaining wind speeds from the ASCE Standard 7-05 (ASCE, 2005) to develop wave heights. The ASCE Standard 7-05 provides maps of the 50-year 3-second gust wind speed and conversion factors for the 100-year value. From the ASCE Standard 7-05 map, the 50-year 3-second gust wind speed is 130 mph, which converts to a 100-year 3-second gust of 139.1 mph. The 10-year 3-second gust is extrapolated from the 50- and 100-year values. For model input, this wind speed is adjusted to the 10-minute sustained wind speed using the conversion factor of 1.38 (Harper et al, 2008). Table 2.3 summarizes the wind speeds. The 10-minute sustained wind speed provides the wind boundary condition for the wave model presented in the following chapter.

Table 2.3 Summary of Wind Speeds

Return Period	3-Second Gust (mph)	10-Minute Sustained (mph)
10	108.9	78.9
50	130.0	94.2
100	139.1	100.8

2.6 Proposed Bridge Alignments

Figure 2.5 through Figure 2.9 present the five proposed bridge alignments. In the figures, the green line represents the existing alignment and the blue line represents the proposed alignment. These alignments provide the locations to evaluate the hydraulic and wave conditions presented in the following chapter.

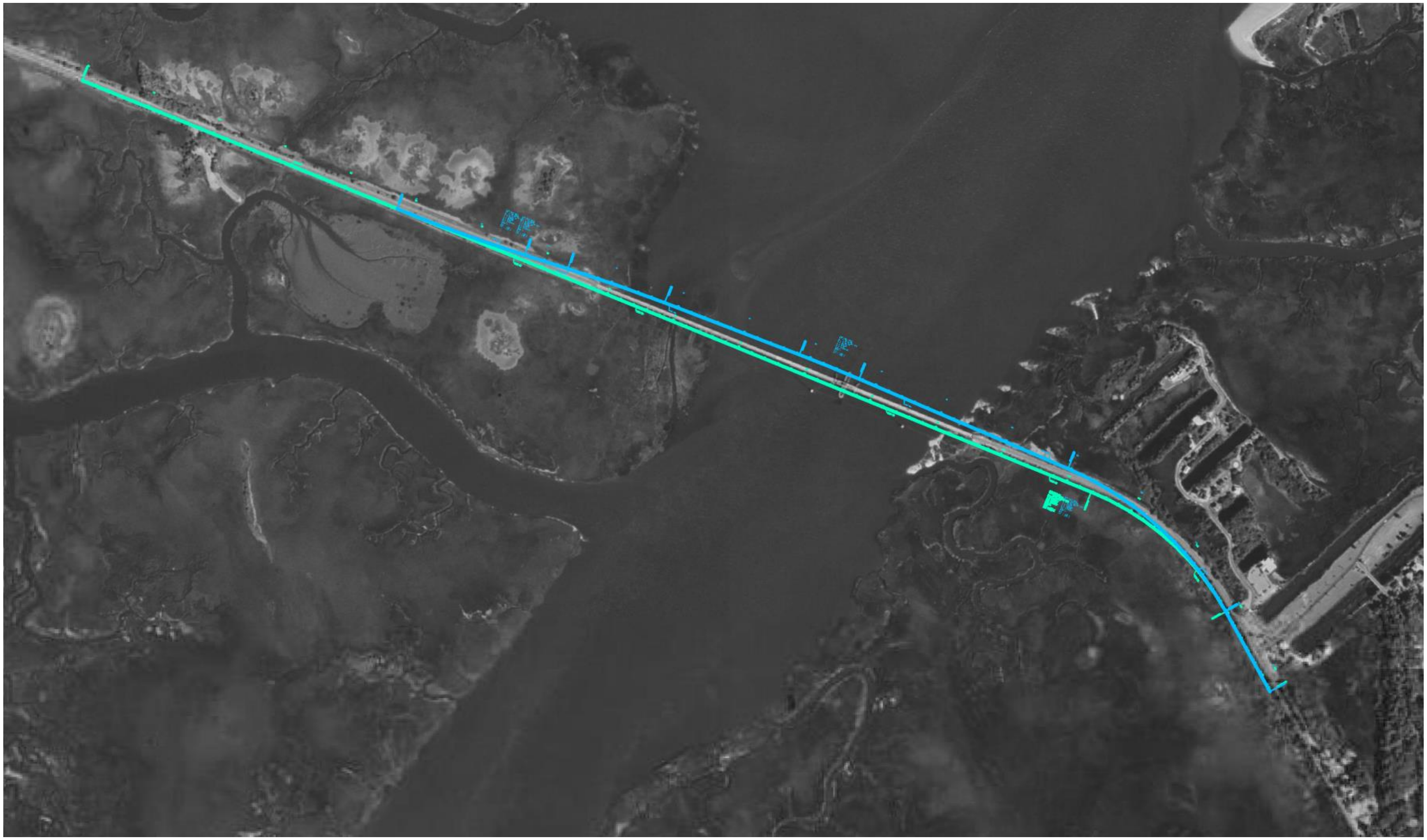

Figure 2.5 Alternative 1 Bridge Alignment

Figure 2.6 Alternative 1B Bridge Alignment

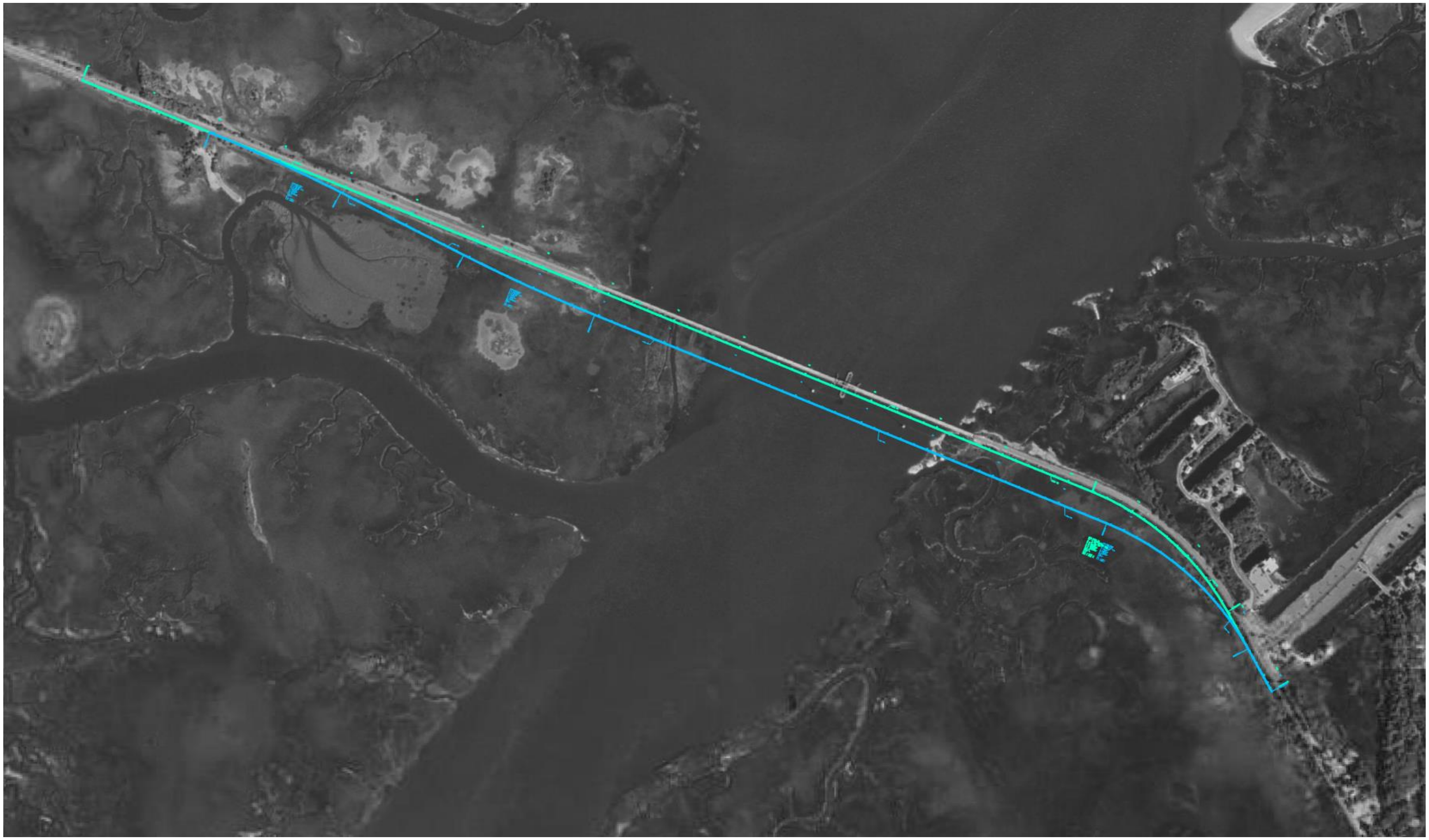

Figure 2.7 Alternative 2A Bridge Alignment

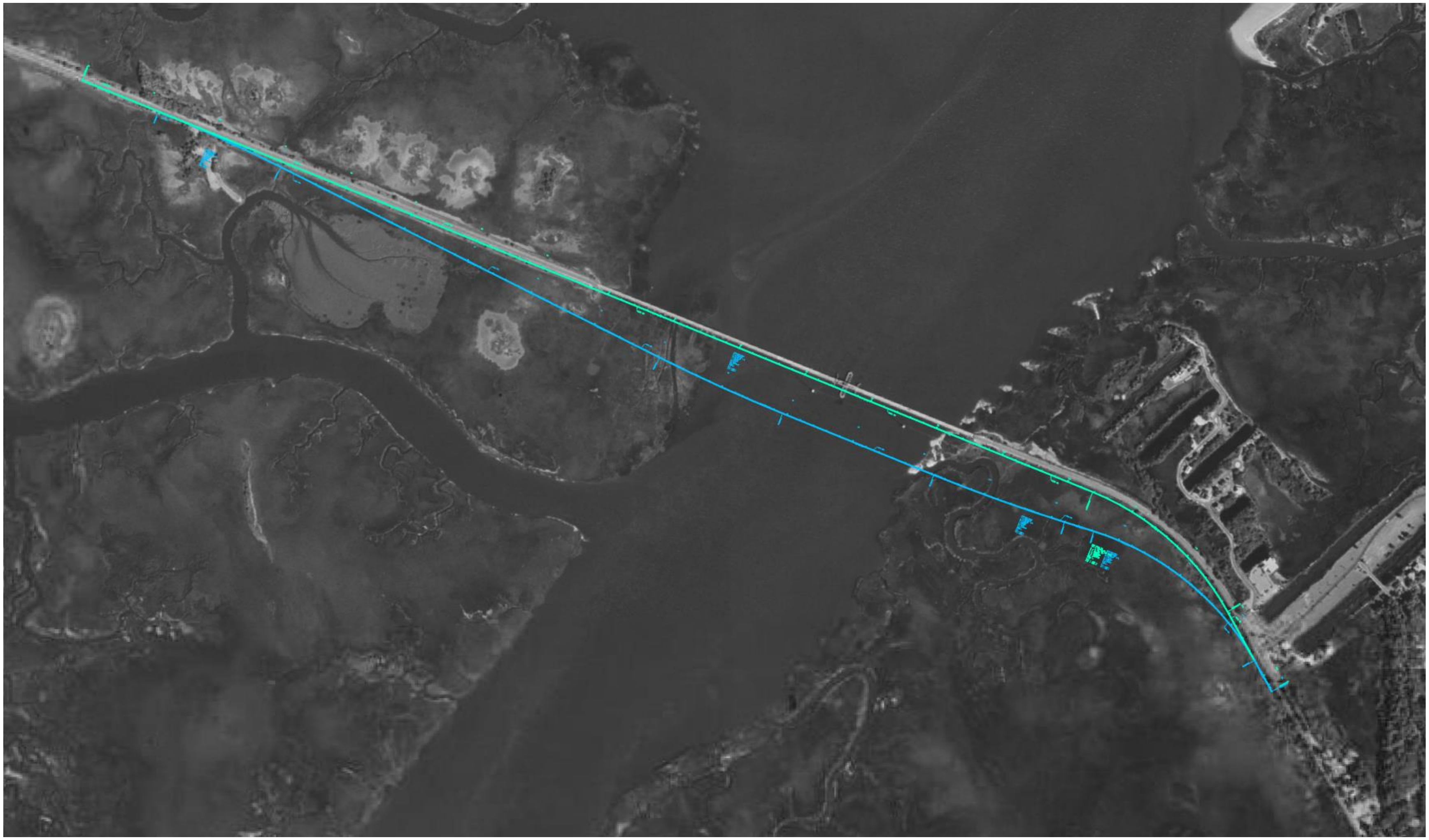

Figure 2.8 Alternative 2B Bridge Alignment

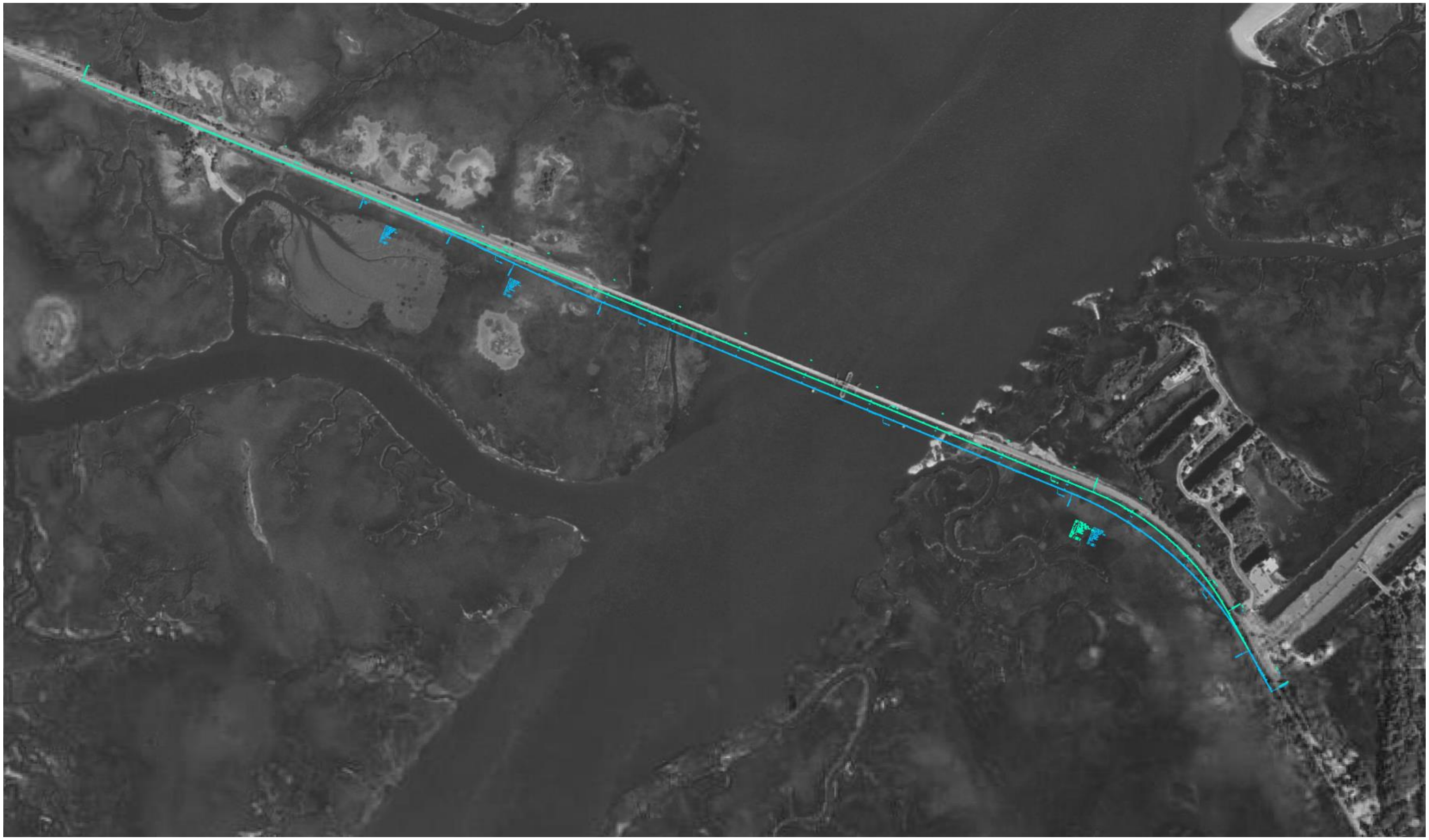

Figure 2.9 Alternative 3 Bridge Alignment

3.0 Wave and Hydraulic Modeling

Determining the low steel elevation requires knowledge of specific wave and hydraulic parameters — wave height and surge elevation. Determining these parameters requires a detailed wave and hydraulic analysis of the study area. The complex nature of the water bodies of interest dictates the application of time-dependent, two-dimensional wave and hydrodynamic numerical models to simulate hurricane conditions in the vicinity of the bridge.

Given the complexities of the interconnected water bodies, the ADvanced CIRCulation Model for Ocean, Coastal, and Estuarine Waters (ADCIRC) is best suited for modeling the hydraulic conditions. ADCIRC is a numerical model developed specifically for generating long duration hydrodynamic circulation along shelves, coasts, and within estuaries. The intent of the model is to produce numerical simulations for very large computational domains in a unified and systematic manner. The collaboration of many researchers have led to the development of the ADCIRC model including investigators at the University of Notre Dame (J. J. Westerink), the University of North Carolina at Chapel Hill (R.A. Luettich), the University of Texas at Austin (M.F. Wheeler and C. Dawson), the University of Oklahoma (R. Kolar), the State of Texas (Jurji), and the Waterways Experiment Station (N. Scheffner) (Luettich and Westerink, 2000).

ADCIRC is a robust computer program for solving the equations of motion for a moving fluid on a rotating earth. The equation formulation includes applying the traditional hydrostatic pressure and Boussinesq approximations and discretizing the equations in space via the finite element (FE) method and in time via the finite difference (FD) method. The ADCIRC program includes both a two-dimensional depth integrated (2DDI) mode and a three-dimensional (3D) mode. For both, the model solves for water surface elevation via the depth-integrated continuity equation in Generalized Wave-Continuity Equation (GWCE) form. The model solves for water flow velocity via either the 2DDI or 3D momentum equations. These equations retain all the nonlinear terms. ADCIRC includes solution capabilities in either a Cartesian or a spherical coordinate system.

Possible boundary conditions for the model include specified elevation (harmonic tidal constituents or time series); specified boundary normal flow (harmonic tidal constituents or time series); zero boundary normal flow; slip or no slip conditions for velocity; external barrier overflow out of the domain; internal barrier overflow between sections of the domain; surface stress (wind and/or wave radiation stress); atmospheric pressure; or outward radiation of waves (Sommerfield condition). ADCIRC can be forced with: elevation boundary conditions; normal flow boundary conditions; surface stress (wind) boundary conditions; tidal potential; or an earth load/self-attraction tide. This bridge hydraulic study application included constant normal flow boundary conditions (at the rivers), a time-dependent elevation boundary condition at the offshore mesh boundary, and closed boundary conditions at the land boundaries.

The program SWAN (Simulating WAves Nearshore) provides the tool to simulate wave heights and periods. SWAN, developed at the Delft University of Technology in the Netherlands, is a one-, and two-dimensional numerical model for estimating wave parameters in coastal areas, lakes and estuaries from given wind, bathymetric, and current conditions. The model is based on the wave action balance equation with sources and sinks (Holthuijsen et al., 2003). The wave propagation processes represented in SWAN include propagation through geographic space, refraction due to spatial variations in bottom and current, shoaling due to spatial variations in bottom and current, blocking and reflections by opposing currents, and transmission through, blockage by or reflection from obstacles. Wave generation and dissipation processes represented in SWAN include generation by wind, dissipation by white-capping, dissipation by depth-induced wave breaking, dissipation by bottom friction, and wave-wave interactions (quadruplets and triads). The model contains both stationary and non-stationary operational modes formulated for Cartesian, curvilinear, or spherical coordinate systems.

The inputs to the SWAN model include the bathymetric/topographic mesh, water surface elevations, and wind speed direction. For this application, the ADCIRC model's mesh and 100-year water surface elevation output provided the inputs for SWAN. Model boundary forcing conditions included a constant 100-year wind speed from ASCE Standard 7-05 (ASCE, 2005), applied at 10 degree increments from 0 to 360 degrees.

The following sections describe the surge and wave models development (set up), including a limited calibration of the ADCIRC model, for the study area, and applications for design storm events. A summary section provides minimum low chord elevations, based on the model application results, for various bridge design alternatives.

3.1 Model Development

Generation of the ADCIRC/SWAN mesh — presented in Figure 3.1— considered aerial photographs, an existing RMA2 mesh (Hull, 2000), and topographic maps. In the figure, the entire mesh is presented to the left with a magnified inset image to right. The mesh includes the entire Harbor River and extends approximately 75 miles along the coast and 45 miles offshore to the 110 foot contour. As the figure illustrates, the mesh includes high resolution at the bridge crossing. In total, the final mesh contains more than 31,000 nodes, with approximately 65% defining the bathymetry and topography of the US-21 Bridge, Harbor River, and adjacent islands and creeks. Node spacing ranges from approximately 60 feet at the bridge, to approximately 18,000 feet in the Atlantic Ocean.

Figure 3.2 presents the mesh nodal elevations with the entire mesh presented to the left and a zoomed image to the right. The nodal elevations define the topography and bathymetry of the mesh domain. Table 3.1 presents the sources along with the coverage type, data type, responsible agency, and the vertical and horizontal datums. These data were converted to a common datum (NAVD88) and interpolated to the mesh.

Figure 3.3 illustrates the location and type of boundary conditions applied. For the calibration run, tidal constituents were applied at the ocean boundary with no flows in the rivers. For the design runs, the hurricane storm surge hydrographs described in Chapter 2.0 were applied at the ocean boundaries and the mean annual discharges were applied at the rivers boundaries.

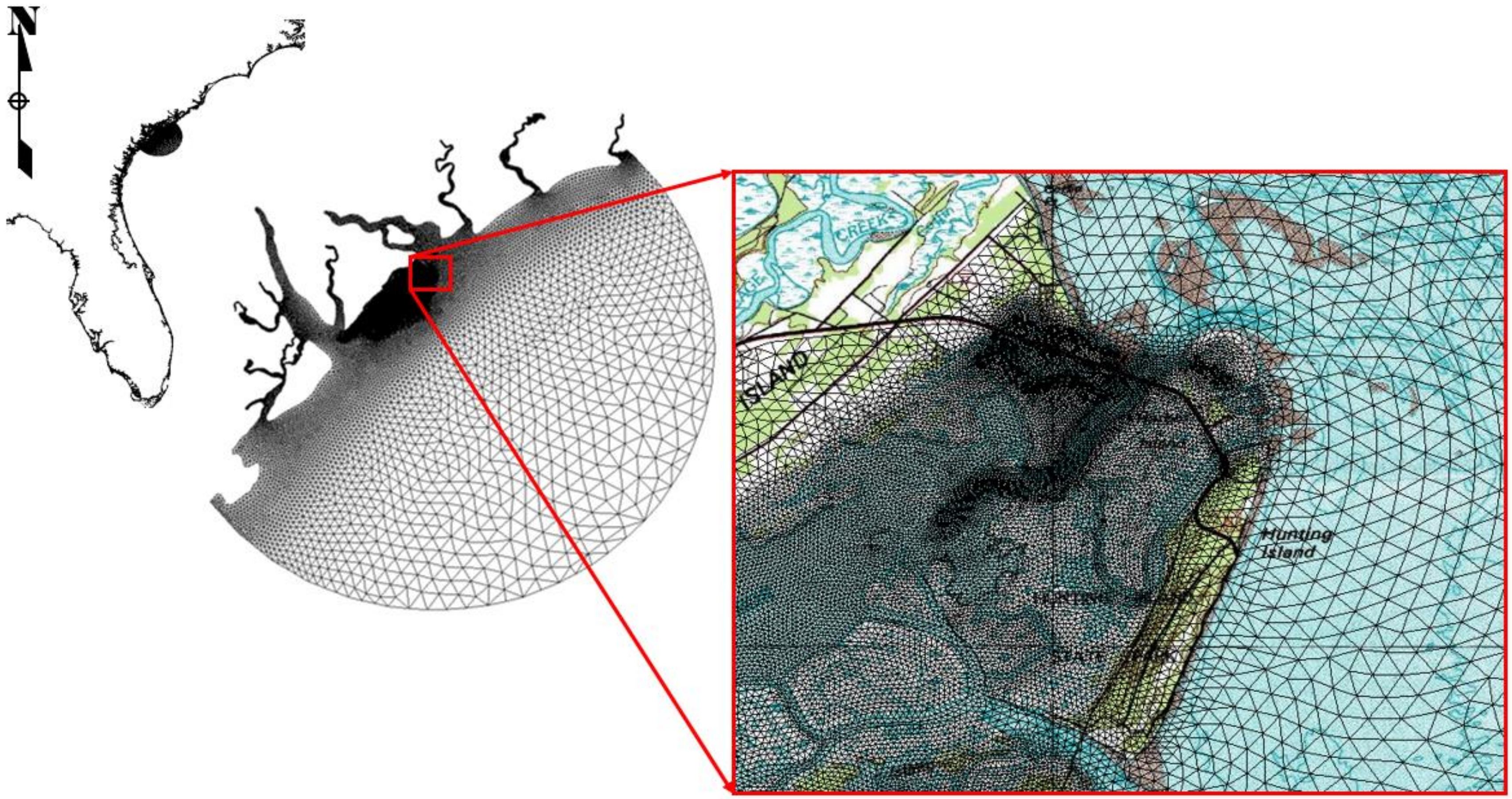

Figure 3.1 Model Mesh

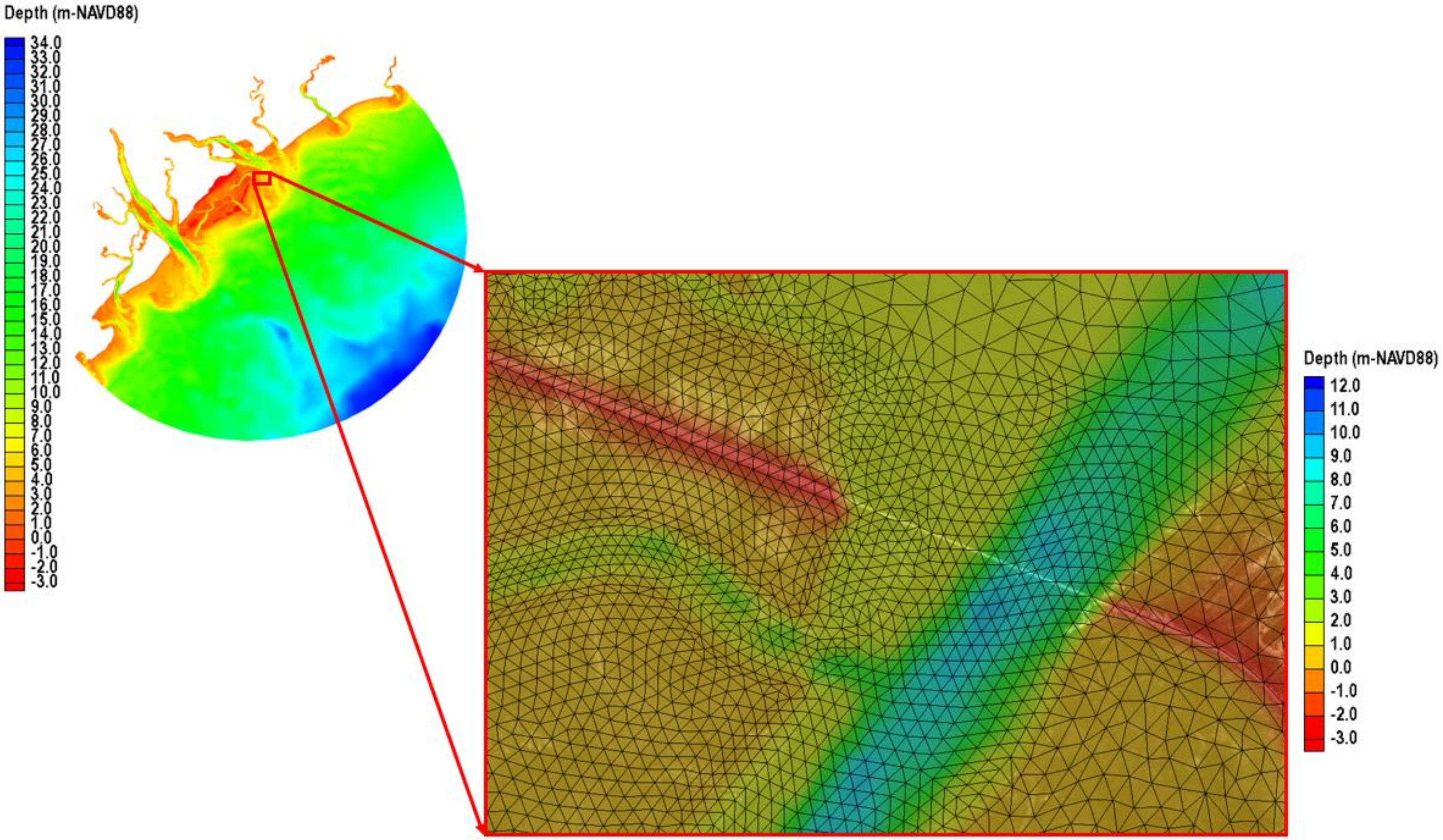

Figure 3.2 Model Bathymetry

Table 3.1 Elevation Data Sources and Datums

Coverage Type	Data Type	Agency	Source	Datum	
				Vertical	Horizontal
Bathymetry	Hydrographic Survey	GEL Engineering LLC	XYZ Text File	ft-NAVD88	.
Topography/ Bathymetry	3 arc-second U.S. Coastal Relief Model (CRM)	NOAA/NGDC	https://www.ngdc.noaa.gov/mgg/coastal/ crm.html	m-MSL	World Geographic (Latitude-Longitude)
Topography/ Bathymetry	Mesh Scatter Point	Taylor Engineering Inc.	Existing RMA2 Model applied in a previous SCDOT study for the bridge	ft-NGVD	State Plane NAD 83(US) South Carolina.

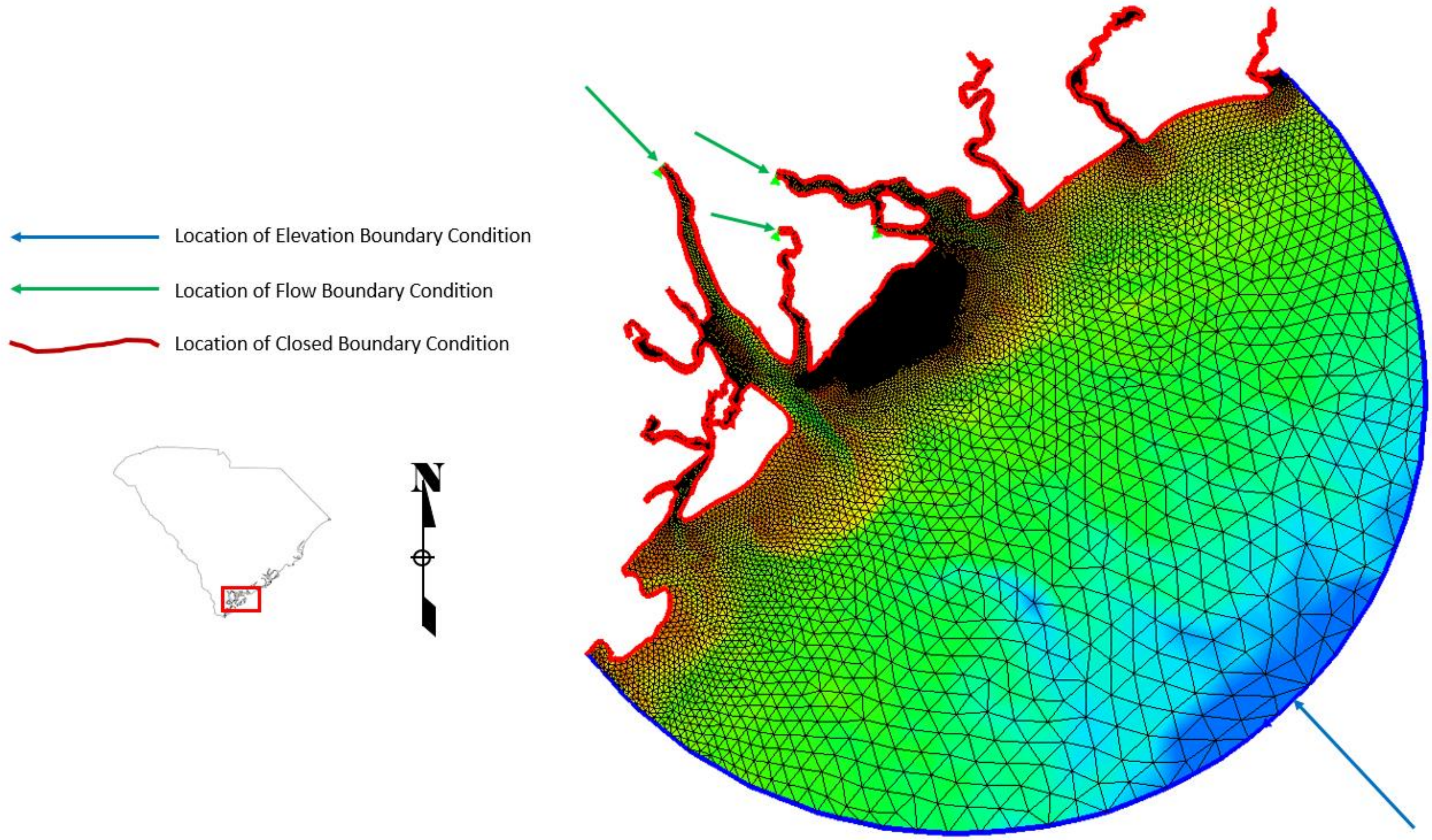

Figure 3.3 ADCIRC Model Boundary Condition Locations

3.1.1 Model Calibration

Model calibration involves an iterative process of adjusting model parameters until the model results at a set location matches measured values within acceptable limits. Calibrating ADCIRC involves the adjustment of model friction and lateral eddy viscosity until simulated water surface elevations match measured values within acceptable limits. FEMA (2007) defines this limit as 10% or less for tidal calibrations.

Calibration of the model applies the following error estimations as a quantitative method to judge their ability to reproduce measured events. The first equation provides an estimate of the mean error, E, the average of the deviation between calculated (simulated) and measured values.

$$E = \frac{\sum_{i=1}^{N} (\chi_c - \chi_m)_i}{N} \qquad (3.1)$$

where χ_c is the calculated (simulated) value, χ_m is the measured value, and N is the total number of data points. A positive value for the mean error would indicate that the model overestimates the event, while a negative value would indicate the model underestimates the event.

The root-mean square error, E_{rms}, given by the following equation, indicates the absolute error of the comparison. The variables remain the same as indicated above.

$$E_{rms} = \sqrt{\frac{\sum_{i=1}^{N} \left(\chi_c - \chi_m\right)_i^2}{N}} \qquad (3.2)$$

The final error estimator, E_{pct}, is the percent error. This variable gives an indication of the degree to which the calculated values misrepresent the measured values. Percent error, defined in terms of RMS error, is given as

$$E_{pct} = \frac{E_{rms}}{R} \qquad (3.3)$$

where R is a representative range of the variable χ.

For the measured values, this limited ADCIRC calibration applied predicted water surface elevation data from the National Oceanographic and Atmospheric Administration (NOAA) gage (8668146) located at the bridge. At that location, the gage provides predictions of water surface elevation (WSE) at an approximate six hour interval.

Figure 3.4 presents time series of the results of the ADCIRC model calibration. In the figure, the horizontal axis is time, the vertical axis is water surface elevation (ft-MSL), the black circles represent the water surface elevation predicted at the NOAA gage, and the black line represents the simulated water surface elevation. The results of the calibration, presented in Figure 3.4, illustrate the model’s accuracy. The ADCIRC simulated WSE agrees well with the predicted data from NOAA. Table 3.2 presents a summary of the results for the calibration. From the table, the mean error is 6 inches and RMS error is 6 inches yielding a percent error of 6.4%. Using FEMA's criteria, the model is acceptably calibrated (< 10% percent error).

Table 3.2 Summary of the Calibration Results

Mean Error (ft)	RMS Error (ft)	Percent Error
0.5	0.5	6.4%

As noted above, calibrating ADCIRC involves the adjustment of model friction and lateral eddy viscosity until model simulated water surface elevations match measured (in this case NOAA predicted) values within acceptable limits. For this application of ADCIRC, friction was applied as a spatially variable Manning's Friction Factor (*n*). Table 3.3 presents the final values of Manning's Friction Factor. These factors, along with a global lateral eddy viscosity of 2.0, were applied to calibrate the model.

Table 3.3 Manning's Friction Factors

Area	Manning's Friction Factor (*n*)
Waterways	0.017
Upland	0.2

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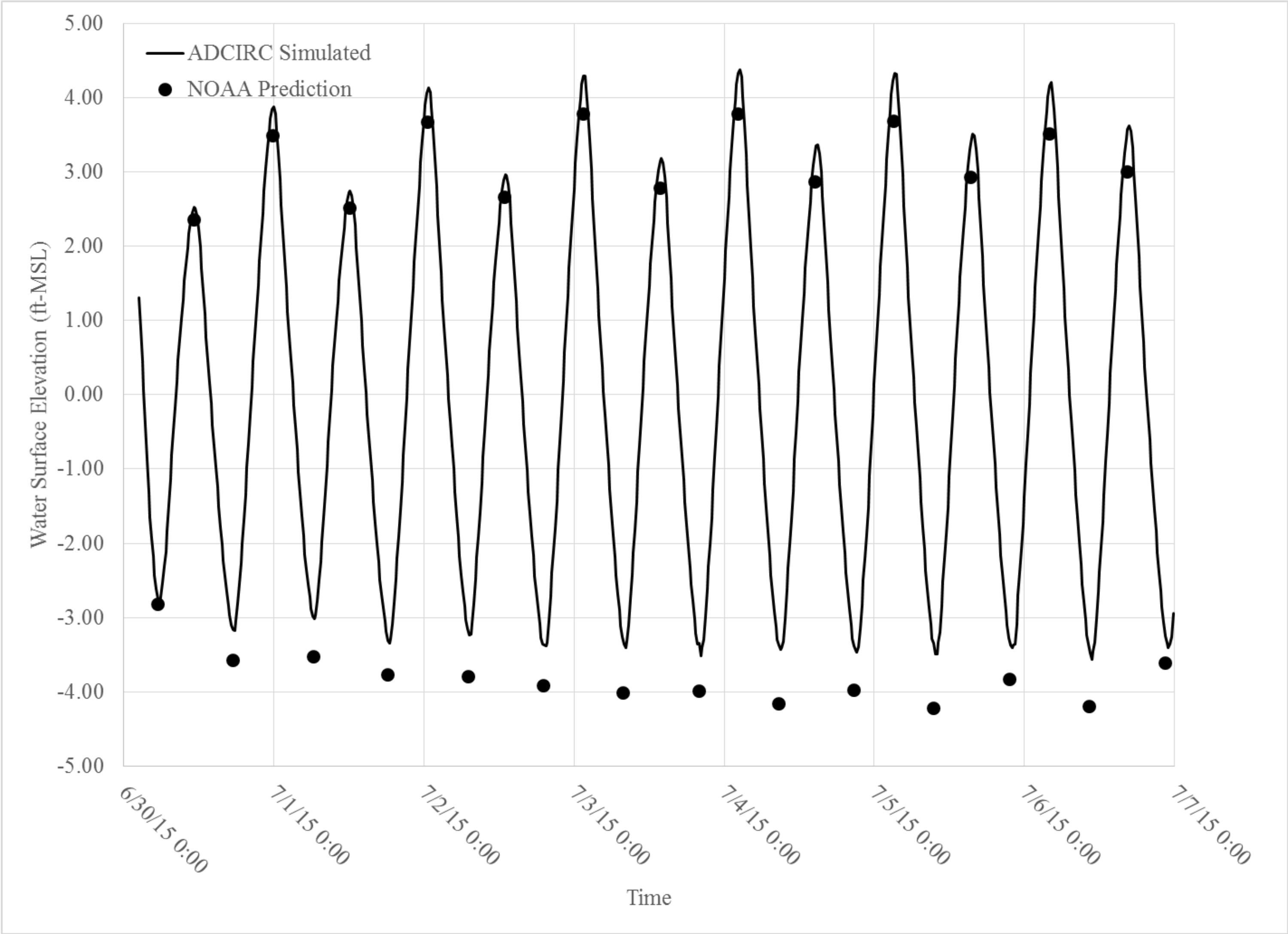

Figure 3.4 Tide Calibration Comparison Plot

3.2 Storm Surge Simulations

Once calibrated, the model was applied to simulate the design conditions at the bridge. The hydrographs described in Chapter 2.0 provided the boundary conditions for the simulations presented below. The results provide the wave and hydraulic parameters necessary to calculate low chord elevations.

Figure 3.5 presents a time series plot of the still water elevation (i.e. no contribution from waves) for the 10-, 100-, and 500-year storm surge events at the bridge. In the figures, the horizontal axis is time in hours, the vertical axis is water surface elevation in feet (NAVD88), and the 10-, 100-, and 500-year values are represented by the blue, red, and black lines. As these figures illustrate, water surface elevations during the 10-, 100-, and 500-year events reach +8.4ft-NAVD88, +14.8 ft-NAVD88, and +19.0 ft-NAVD88. These values provide the water surface elevations (along with wave model results) to determine low steel elevations and provide input for the wave model.

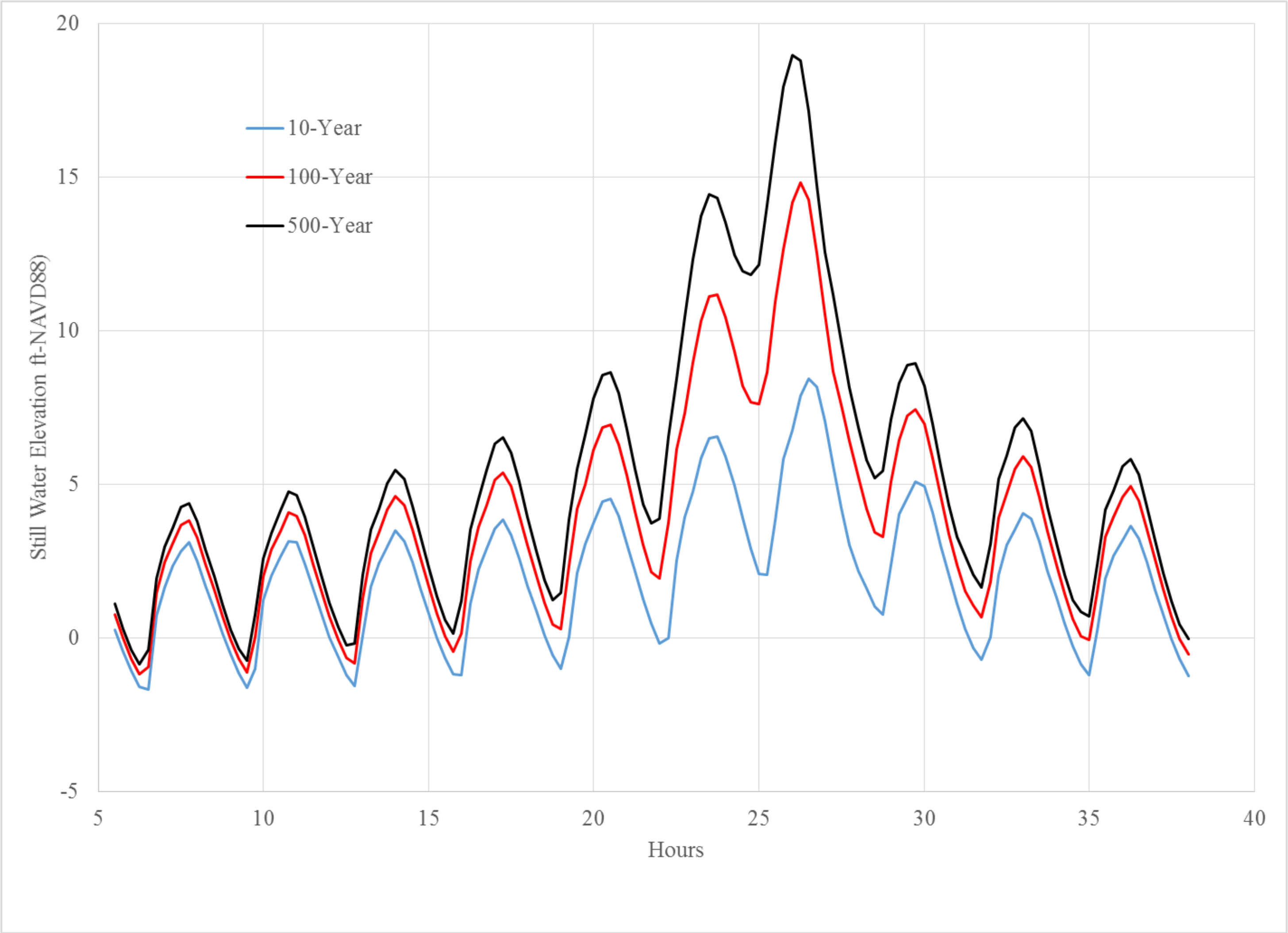

Figure 3.5 Time Series of the Simulated 10-, 100-, and 500-Year Still Water Elevation at the Bridge (i.e. no contribution from waves)

3.3 Wave Simulations

This section develops the 10- and 100-year wave heights. The 10- and 100-year wave heights provide the required clearance elevation (low steel) and the wave heights required by FHWA for wave force calculations (for future analysis). The inputs to the SWAN model include the bathymetric/topographic mesh, water surface elevations, and wind speed magnitude and direction. For this application, SWAN employs the mesh and 10- and 100-year water surface elevations from the ADCIRC model. The model boundary conditions include the 10- and 100-year wind speed from ASCE Standard 7-05 (ASCE, 2005) as described in Chapter 2.0. Figure 3.6 and Figure 3.7 illustrate the bridge's exposure to potential fetch directions across Saint Helena Sound and Harbor River. As Figure 3.6 demonstrates, the bridge is exposed to waves generated by winds from the north-northwest (340^{o}) to the east-northeast (70^{o}) — directions that correspond to the wind directions during hurricane landfall (i.e. counterclockwise rotation) near this location. The fetch directions presented in Figure 3.7 — south southwest (200^{o}) to west southwest (240^{o}) — represent directions that correspond to the wind directions after hurricane landfall. As such, SWAN boundary conditions consider the 10- and 100-year wind speeds applied from 340^{o} to 70^{o} at 10 degree increments and from 200^{o} to 240^{o} at 10 degree increments.

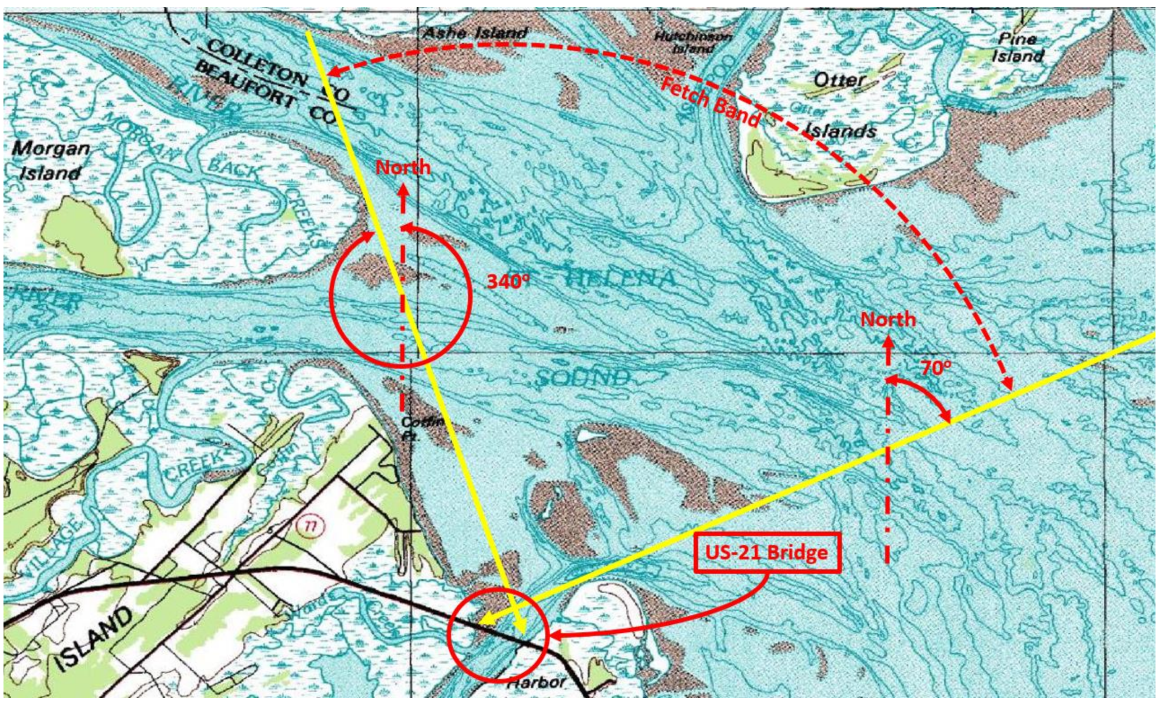

Figure 3.6 Helena Sound Fetch Directions

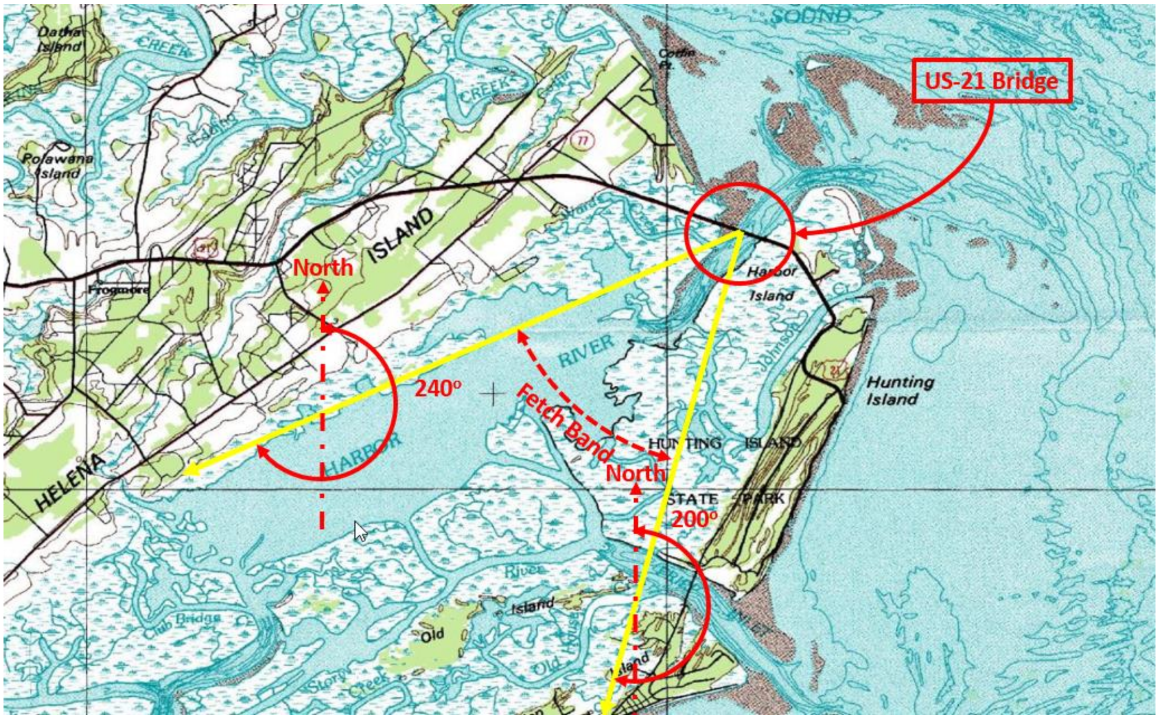

Figure 3.7 Harbor River Fetch Directions

Figure 3.8 through Figure 3.11 present contours of the largest significant wave heights (the average highest one third of the waves) occurring for the 10- and 100-year wave model simulations for the 340° and 30° wind directions. In the figures, blue contours represent the largest waves and red contours the smallest waves. In general, the two wind directions presented the figures (340° and 30°) represent the wind directions that produce the largest waves. As the figures illustrate, within the bridge opening those waves vary from 3 ft to more than 5 ft for the 10-year wind speed and from 4.5 ft to more than 8 ft for the 100-year wind speeds.

Figure 3.12 and Figure 3.13 present profiles of significant wave height for all of the model wind directions along the existing bridge alignment. For both the 10- and 100-year conditions, the largest waves occur near the channel. Table 3.4 summarizes the results for each return period. As expected, the largest waves — 5.3 ft for the 10-year and 8.2 ft for the 100-year — occur in the deepest section of the channel.

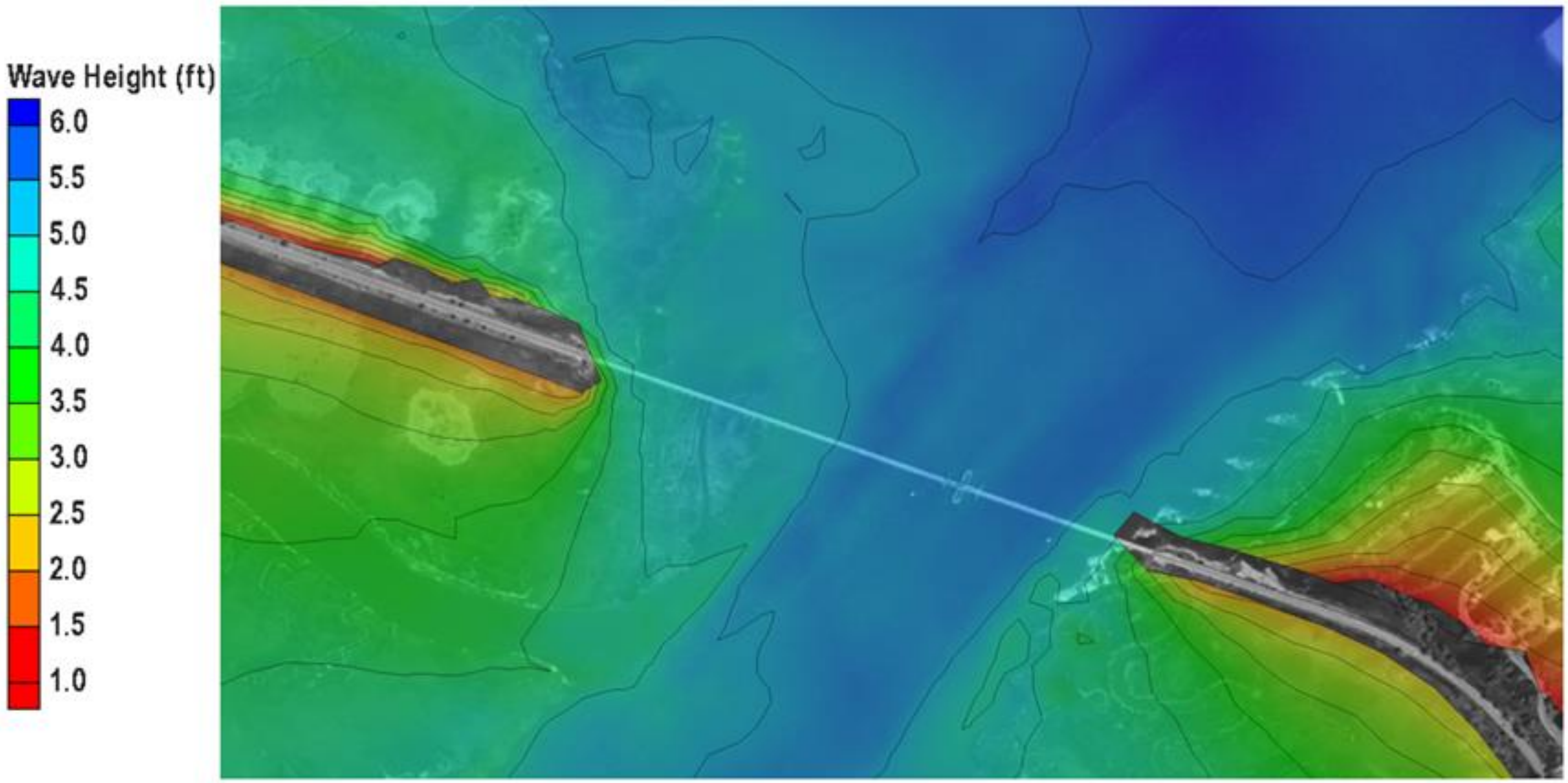

Figure 3.8 Contours of Significant Wave Height for the 10-year Wind Speed from 340°

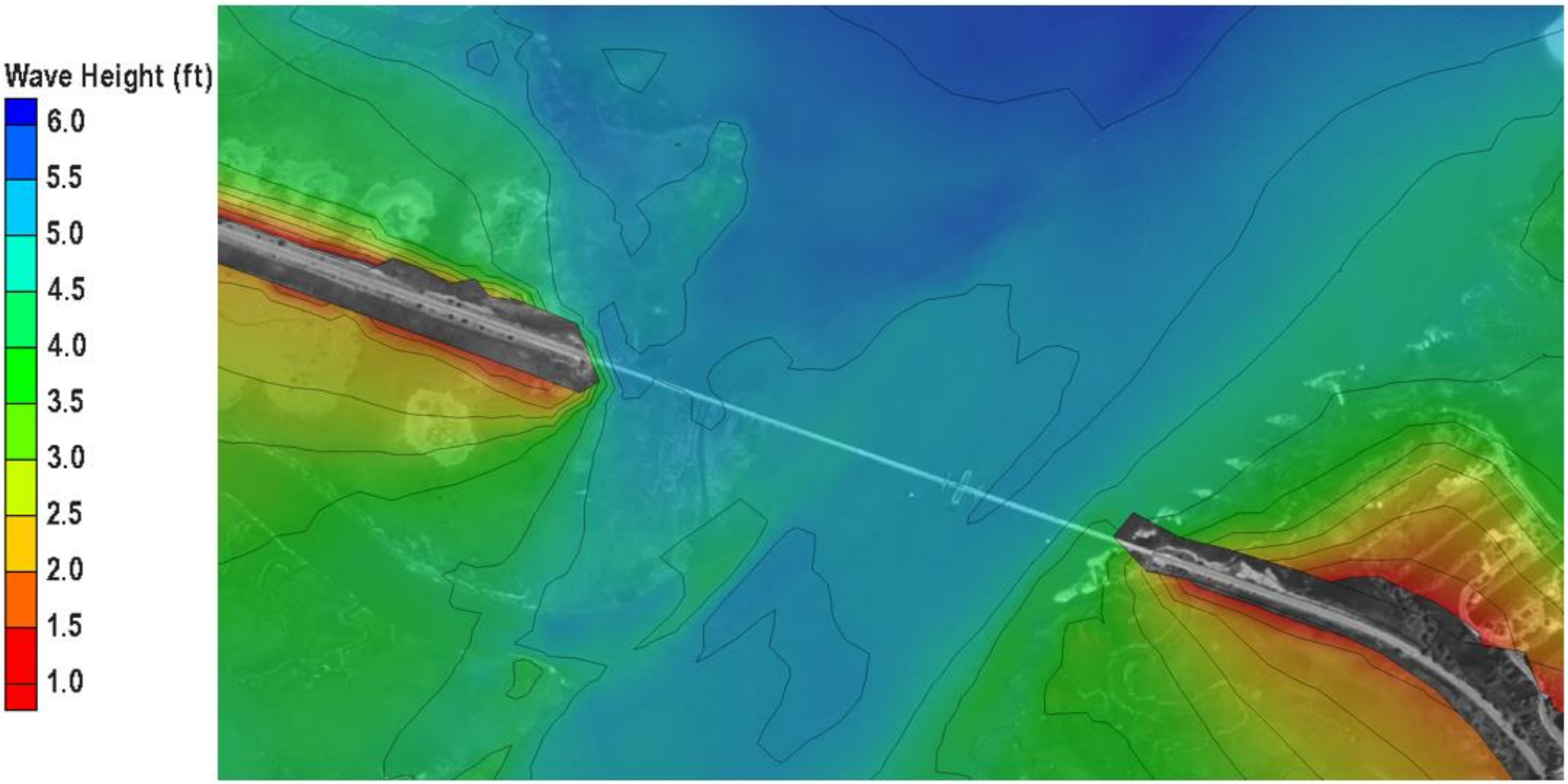

Figure 3.9 Contours of Significant Wave Height for the 10-year Wind Speed from 30°

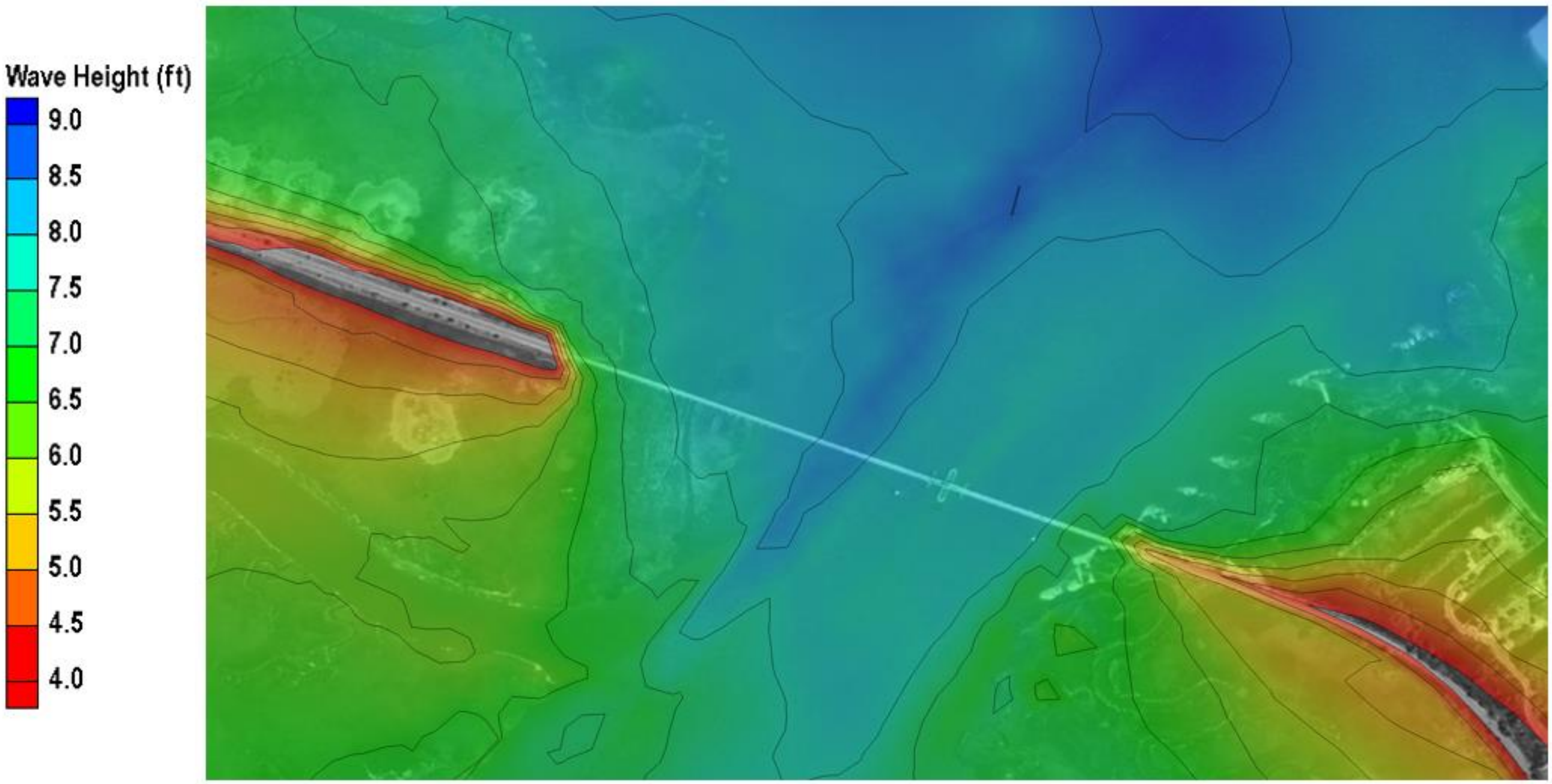

Figure 3.10 Contours of Significant Wave Height for the 100-year Wind Speed from 340°

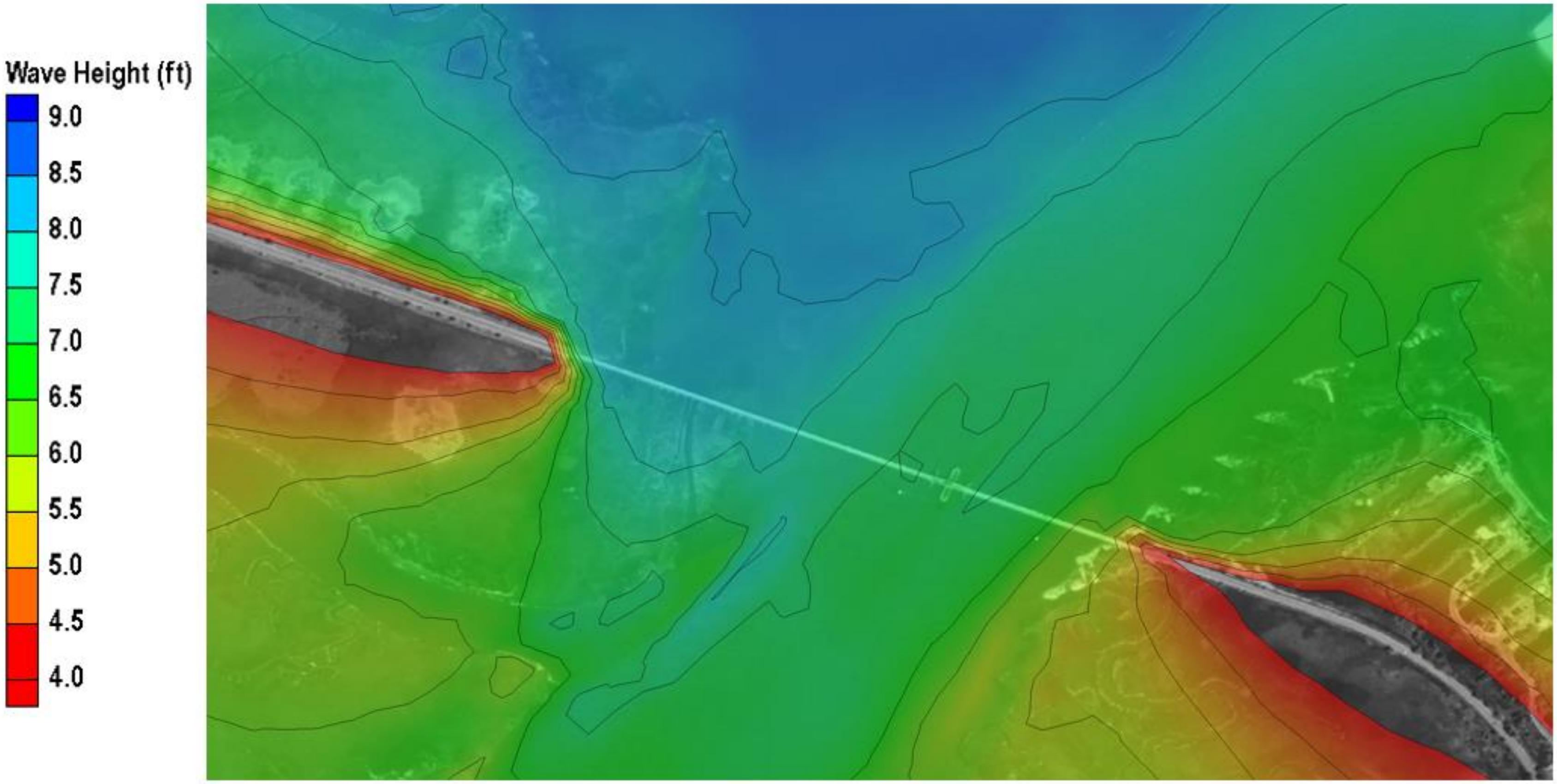

Figure 3.11 Contours of Significant Wave Height for the 100-year Wind Speed from 30°

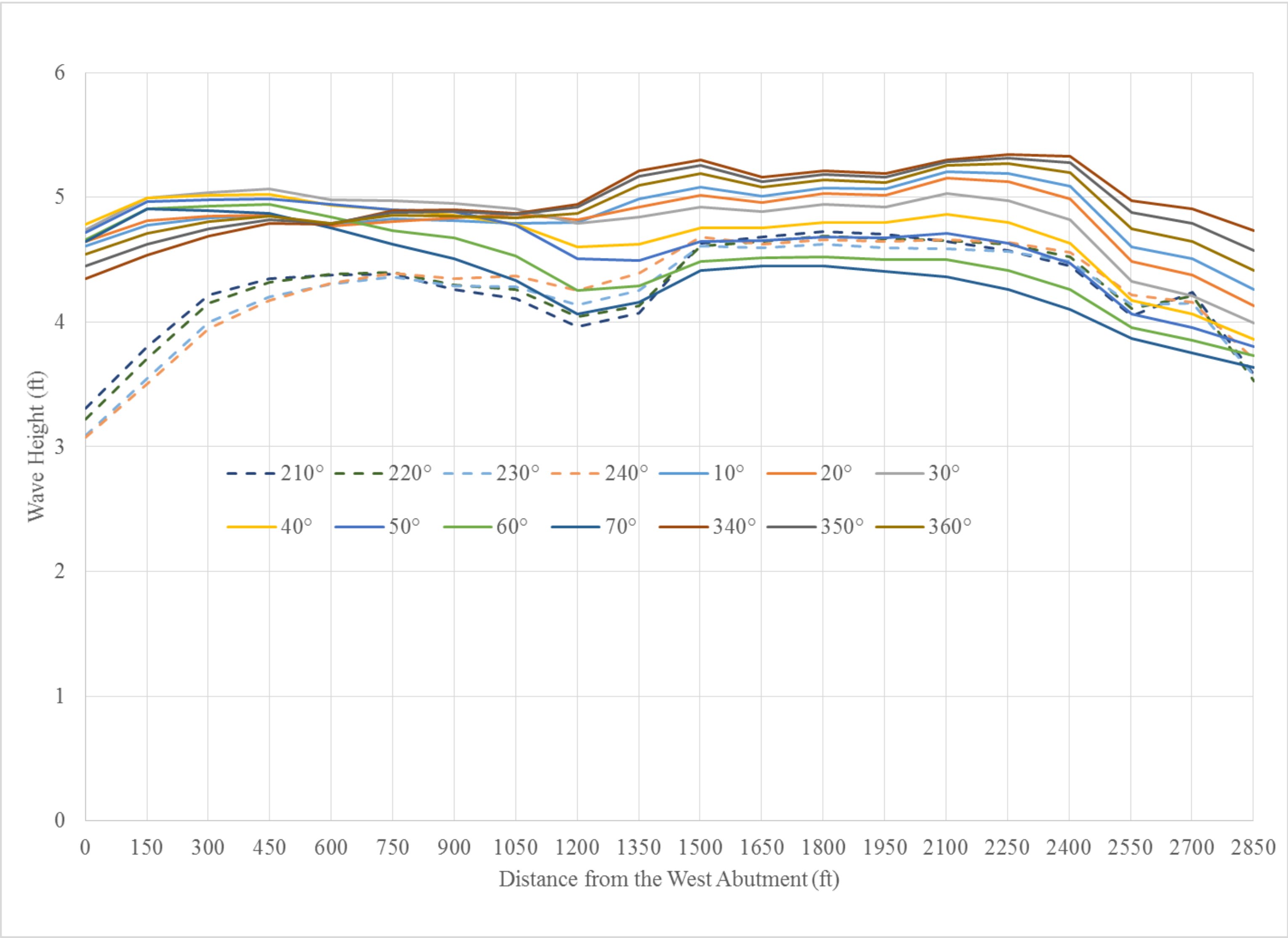

Figure 3.12 10-year Maximum Significant Wave Heights along the Existing Bridge Alignment

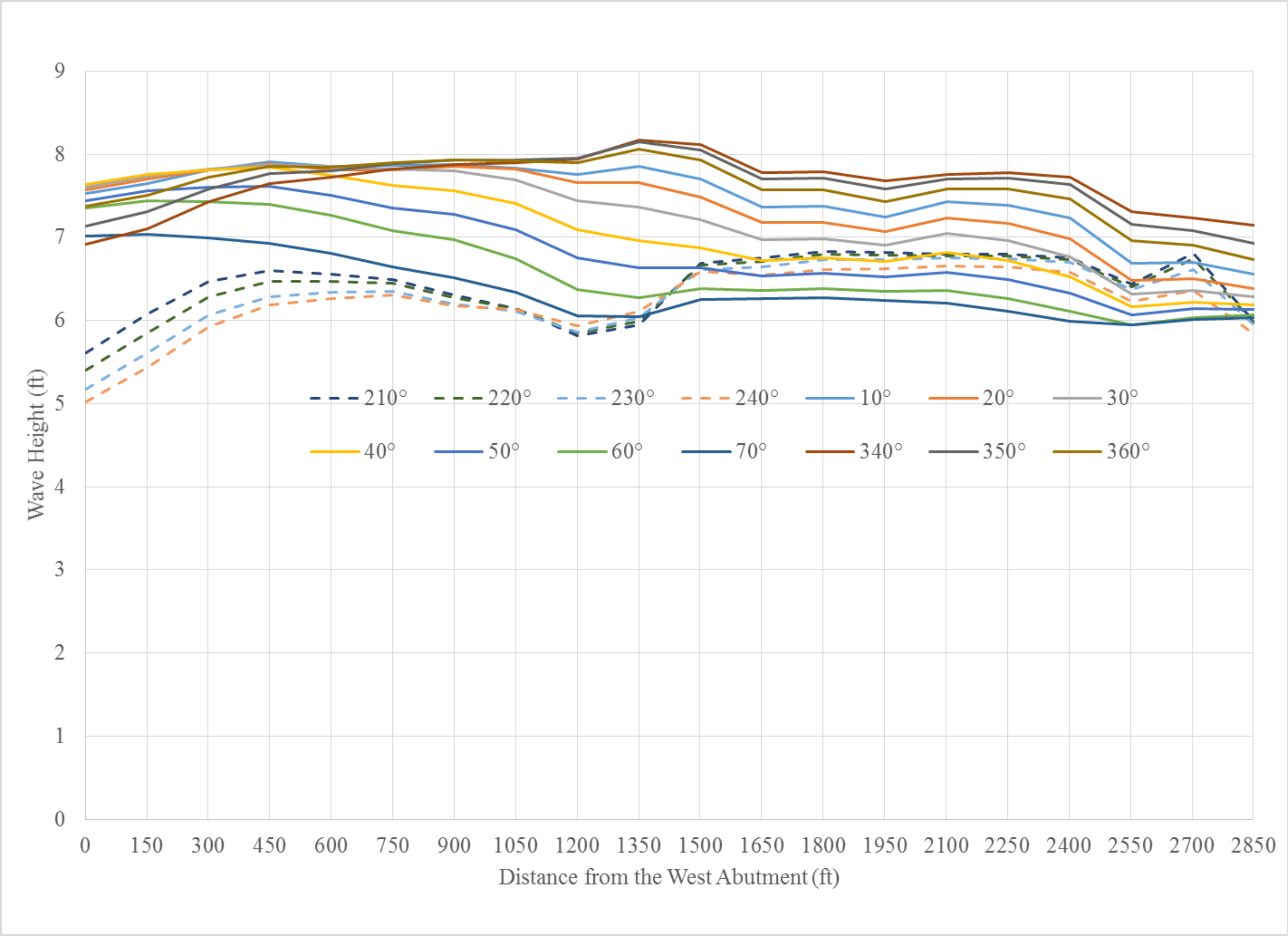

Figure 3.13 100-year Maximum Significant Wave Heights along the Existing Bridge Alignment

Table 3.4 10- and 100-year Maximum Significant Wave Heights

Distance from the West Abutment (ft)	Maximum Significant Wave Height (ft)	
	10-Year	100-Year
0	4.8	7.6
150	5.0	7.8
300	5.0	7.8
450	5.1	7.9
600	5.0	7.9
750	5.0	7.9
900	4.9	7.9
1050	4.9	7.9
1200	4.9	7.9
1350	5.2	8.2
1500	5.3	8.1
1650	5.2	7.8
1800	5.2	7.8
1950	5.2	7.7
2100	5.3	7.8
2250	5.3	7.8
2400	5.3	7.7
2550	5.0	7.3
2700	4.9	7.2
2850	4.7	7.1

3.4 Summary

The results of the hydraulic and wave modeling provide the information necessary to determine the low steel elevation — 2 feet above the 10-year water surface elevation with wave height (wave crest elevation). Table 3.5 through Table 3.9 present the 10-year wave crest elevations along the 5 alternative alignments. In the tables, the wave crest elevations and low steel elevations are presented at 100-ft intervals from the noted station. Wave heights in the wave crest elevation calculations are the 10% highest wave height (1.27*significant wave height) with 80% of the wave height above still water. Low steel elevations are 2 ft above the wave crest elevation. As the tables document, wave crest elevations are lower at the beginning and end of each alignment — with wave crest elevations as low as +9.9 ft-NAVD88 and corresponding low steel elevations as low as +11.9 ft-NAVD88.

Table 3.10 summarizes the *hydrology data for tidal bridges* calculated during this study. This summary is a requirement for SCDOT plan sets.

Table 3.5 10-year Wave Crest Elevations Alternative 1

Distance from 54+00.00 (ft)	Wave Crest Elevation (ft-NAVD88)	Low Steel Elevation (ft-NAVD88)
69+00.00	+11.8	+13.8
70+00.00	+13.0	+15.0
71+00.00	+13.2	+15.2
72+00.00	+13.2	+15.2
73+00.00	+13.3	+15.3
74+00.00	+13.3	+15.3
75+00.00	+13.3	+15.3
76+00.00	+13.3	+15.3
77+00.00	+13.3	+15.3
78+00.00	+13.3	+15.3
79+00.00	+13.2	+15.2
80+00.00	+13.2	+15.2
81+00.00	+13.2	+15.2
82+00.00	+13.4	+15.4
83+00.00	+13.7	+15.7
84+00.00	+13.8	+15.8
85+00.00	+13.7	+15.7
86+00.00	+13.7	+15.7
87+00.00	+13.7	+15.7
88+00.00	+13.7	+15.7
89+00.00	+13.7	+15.7
90+00.00	+13.8	+15.8
91+00.00	+13.8	+15.8
92+00.00	+13.8	+15.8
93+00.00	+13.8	+15.8
94+00.00	+13.4	+15.4
95+00.00	+13.0	+15.0

Table 3.6 10-year Wave Crest Elevations Alternative 1B

Distance from 54+00.00 (ft)	Wave Crest Elevation (ft-NAVD88)	Low Steel Elevation (ft-NAVD88)
69+00.00	+11.8	+13.8
70+00.00	+13.1	+15.1
71+00.00	+13.2	+15.2
72+00.00	+13.2	+15.2
73+00.00	+13.2	+15.2
74+00.00	+13.3	+15.3
75+00.00	+13.3	+15.3
76+00.00	+13.3	+15.3
77+00.00	+13.3	+15.3
78+00.00	+13.2	+15.2
79+00.00	+13.2	+15.2
80+00.00	+13.2	+15.2
81+00.00	+13.3	+15.3
82+00.00	+13.5	+15.5
83+00.00	+13.7	+15.7
84+00.00	+13.7	+15.7
85+00.00	+13.7	+15.7
86+00.00	+13.8	+15.8
87+00.00	+13.7	+15.7
88+00.00	+13.7	+15.7
89+00.00	+13.8	+15.8
90+00.00	+13.8	+15.8
91+00.00	+13.8	+15.8
92+00.00	+13.8	+15.8
93+00.00	+13.6	+15.6
94+00.00	+13.0	+15.0
95+00.00	+13.0	+15.0

Table 3.7 10-year Wave Crest Elevations Alternative 2A

Distance from 39+70.42 (ft)	Wave Crest Elevation (ft-NAVD88)	Low Steel Elevation (ft-NAVD88)
64+70.40	+10.0	+12.0
65+70.40	+10.0	+12.0
66+70.40	+10.0	+12.0
67+70.40	+9.9	+11.9
68+70.40	+9.9	+11.9
69+70.40	+10.4	+12.4
70+70.40	+12.5	+14.5
71+70.40	+13.0	+15.0
72+70.40	+13.1	+15.1
73+70.40	+13.1	+15.1
74+70.40	+13.2	+15.2
75+70.40	+13.2	+15.2
76+70.40	+13.2	+15.2
77+70.40	+13.1	+15.1
78+70.40	+13.1	+15.1
79+70.40	+13.1	+15.1
80+70.40	+13.1	+15.1
81+70.40	+13.4	+15.4
82+70.40	+13.6	+15.6
83+70.40	+13.6	+15.6
84+70.40	+13.6	+15.6
85+70.40	+13.7	+15.7
86+70.40	+13.7	+15.7
87+70.40	+13.7	+15.7
88+70.40	+13.7	+15.7
89+70.40	+13.7	+15.7
90+70.40	+13.7	+15.7
91+70.40	+13.7	+15.7
92+70.40	+13.5	+15.5
93+70.40	+13.1	+15.1
94+70.40	+13.0	+15.0
95+70.40	+12.8	+14.8
96+70.40	+12.4	+14.4
97+70.40	+12.2	+14.2

Distance from 39+70.42 (ft)	Wave Crest Elevation (ft-NAVD88)	Low Steel Elevation (ft-NAVD88)
98+70.40	+12.1	+14.1
99+70.40	+12.3	+14.3
100+70.40	+12.4	+14.4
101+70.40	+12.4	+14.4
102+70.40	+12.4	+14.4
103+70.40	+12.4	+14.4
104+70.40	+12.4	+14.4
105+70.40	+12.4	+14.4
106+70.40	+12.3	+14.3
107+70.40	+12.3	+14.3
108+70.40	+12.1	+14.1
109+70.40	+11.9	+13.9
110+70.40	+11.9	+13.9
111+70.40	+11.8	+13.8
112+70.40	+11.6	+13.6
113+70.40	+11.4	+13.4
114+70.40	+11.3	+13.3
115+70.40	+11.2	+13.2

Table 3.8 10-year Wave Crest Elevations Alternative 2B

Distance from 35+77.57 (ft)	Wave Crest Elevation (ft-NAVD88)	Low Steel Elevation (ft-NAVD88)
59+77.57	+10.1	+12.1
60+77.57	+10.1	+12.1
61+77.57	+10.3	+12.3
62+77.57	+10.4	+12.4
63+77.57	+10.4	+12.4
64+77.57	+10.2	+12.2
65+77.57	+10.2	+12.2
66+77.57	+10.2	+12.2
67+77.57	+10.1	+12.1
68+77.57	+10.1	+12.1
69+77.57	+11.8	+13.8
70+77.57	+12.6	+14.6
71+77.57	+13.0	+15.0
72+77.57	+13.0	+15.0
73+77.57	+13.1	+15.1
74+77.57	+13.1	+15.1
75+77.57	+13.2	+15.2
76+77.57	+13.1	+15.1
77+77.57	+13.0	+15.0
78+77.57	+13.0	+15.0
79+77.57	+13.0	+15.0
80+77.57	+13.1	+15.1
81+77.57	+13.5	+15.5
82+77.57	+13.6	+15.6
83+77.57	+13.5	+15.5
84+77.57	+13.6	+15.6
85+77.57	+13.6	+15.6
86+77.57	+13.6	+15.6
87+77.57	+13.6	+15.6
88+77.57	+13.7	+15.7
89+77.57	+13.7	+15.7
90+77.57	+13.7	+15.7
91+77.57	+13.6	+15.6

Distance from 35+77.57 (ft)	Wave Crest Elevation (ft-NAVD88)	Low Steel Elevation (ft-NAVD88)
92+77.57	+13.2	+15.2
93+77.57	+12.9	+14.9
94+77.57	+12.9	+14.9
95+77.57	+12.8	+14.8
96+77.57	+12.6	+14.6
97+77.57	+12.6	+14.6
98+77.57	+12.6	+14.6
99+77.57	+12.6	+14.6
100+77.57	+12.6	+14.6
101+77.57	+12.5	+14.5
102+77.57	+12.5	+14.5
103+77.57	+12.5	+14.5
104+77.57	+12.5	+14.5
105+77.57	+12.5	+14.5
106+77.57	+12.4	+14.4
107+77.57	+12.4	+14.4
108+77.57	+12.4	+14.4
109+77.57	+12.4	+14.4
110+77.57	+12.4	+14.4
111+77.57	+12.1	+14.1
112+77.57	+11.8	+13.8
113+77.57	+11.7	+13.7
114+77.57	+11.5	+13.5
115+77.57	+11.4	+13.4
116+77.57	+11.2	+13.2

Table 3.9 10-year Wave Crest Elevations Alternative 3

Distance from 51+46.44 (ft)	Wave Crest Elevation (ft-NAVD88)	Low Steel Elevation (ft-NAVD88)
70+46.44	+12.5	+14.5
71+46.44	+13.2	+15.2
72+46.44	+13.1	+15.1
73+46.44	+13.2	+15.2
74+46.44	+13.2	+15.2
75+46.44	+13.3	+15.3
76+46.44	+13.2	+15.2
77+46.44	+13.2	+15.2
78+46.44	+13.0	+15.0
79+46.44	+13.1	+15.1
80+46.44	+13.2	+15.2
81+46.44	+13.4	+15.4
82+46.44	+13.6	+15.6
83+46.44	+13.7	+15.7
84+46.44	+13.7	+15.7
85+46.44	+13.7	+15.7
86+46.44	+13.8	+15.8
87+46.44	+13.7	+15.7
88+46.44	+13.7	+15.7
89+46.44	+13.8	+15.8
90+46.44	+13.8	+15.8
91+46.44	+13.7	+15.7
92+46.44	+13.6	+15.6
93+46.44	+13.2	+15.2
94+46.44	+13.0	+15.0
95+46.44	+12.6	+14.6
96+46.44	+11.9	+13.9
97+46.44	+11.4	+13.4
98+46.44	+11.5	+13.5
99+46.44	+11.7	+13.7
100+46.44	+11.7	+13.7
101+46.44	+11.7	+13.7
102+46.44	+11.7	+13.7
103+46.44	+11.7	+13.7
104+46.44	+11.7	+13.7
105+46.44	+11.7	+13.7
106+46.44	+11.6	+13.6
107+46.44	+11.6	+13.6
108+46.44	+11.5	+13.5
109+46.44	+11.5	+13.5
110+46.44	+11.4	+13.4
111+46.44	+11.4	+13.4
112+46.44	+11.2	+13.2
113+46.44	+11.2	+13.2
114+46.44	+11.0	+13.0

Table 3.10 Hydrology Data for Tidal Bridges

Hydrology Data for Tidal Bridges: Vertical Datum is NAVD88			
Mean Higher High Tide Elevation	=	+2.97	ft.
Mean Lower Low Tide Elevation	=	-3.72	ft.
10-year Tidal Surge Height	=	+13.7*	ft. (includes wave height)
100-year Stillwater Height	=	+14.8	ft.
500-year Stillwater Height	=	+19.0	ft.
Maximum vel. within bridge	=	100-yr. Tidal Surge Velocity: 8.7 fps	500-yr. Tidal Surge Velocity: 9.6 fps

*Varies along the proposed bridge alignments

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Appendix A
Site Photographs

Figure 1 Upstream Face of the Bridge Looking West

Figure 2 Downstream Face of the Bridge Looking West

Figure 3 Upstream View

Figure 4 Downstream View