

EB-1 TIDAL WETLANDS PROJECT

BALTIMORE, MARYLAND

HYDRODYNAMIC MODELING STUDY

Prepared for:



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1.0 INTRODUCTION

The following report produced by Moffatt & Nichol (M&N) is in support of the Middle Branch Resiliency Initiative (MBRI) EB-1 Project for GreenVest, LLC (GreenVest). The purpose of the study is to characterize the hydrodynamics and inform the engineering design of the project geometry.

The project is located along the eastern side of the mouth of the Patapsco River, Maryland immediately to the north of Reed Bird Island Park and one mile southwest of the Masonville Dredged Material Containment Facility (DMCF) in Baltimore Harbor. The existing shoreline at EB-1 is characterized by dense phragmites vegetation. South Hanover Street and Potee Street bound the site to the northeast and southwest, respectively.

The project involves the conversion of approximately 3.8-acres of existing non-tidal wetlands to tidal wetlands. The proposed tidal wetlands include low marsh, high marsh, supratidal, and upland vegetated areas. Vegetated channels will receive stormwater runoff from upland sources and lead runoff into the marsh interior.

A gravel shoreline is proposed along the outboard edge of the wetlands to provide protection from erosive forces associated with currents of the Patapsco River. Various inlets connecting the proposed tidal wetlands to the Patapsco River are incorporated to promote tidal flushing and provide shallow water habitat. The proposed habitat zones, proposed gravel shoreline, and existing elevation contours are shown in Figure 1-1.



Figure 1-1: MBRI EB-1 project area and existing elevation contours

Historic nautical charts of Baltimore Harbor were reviewed to understand the extent of the natural shoreline at the project site prior to human intervention. The nautical chart

from 1931 (Figure 1-2) suggest that wetland areas were present at the project site and have since receded. For comparative purposes, the present-day nautical chart at the project site is shown in Figure 1-3.

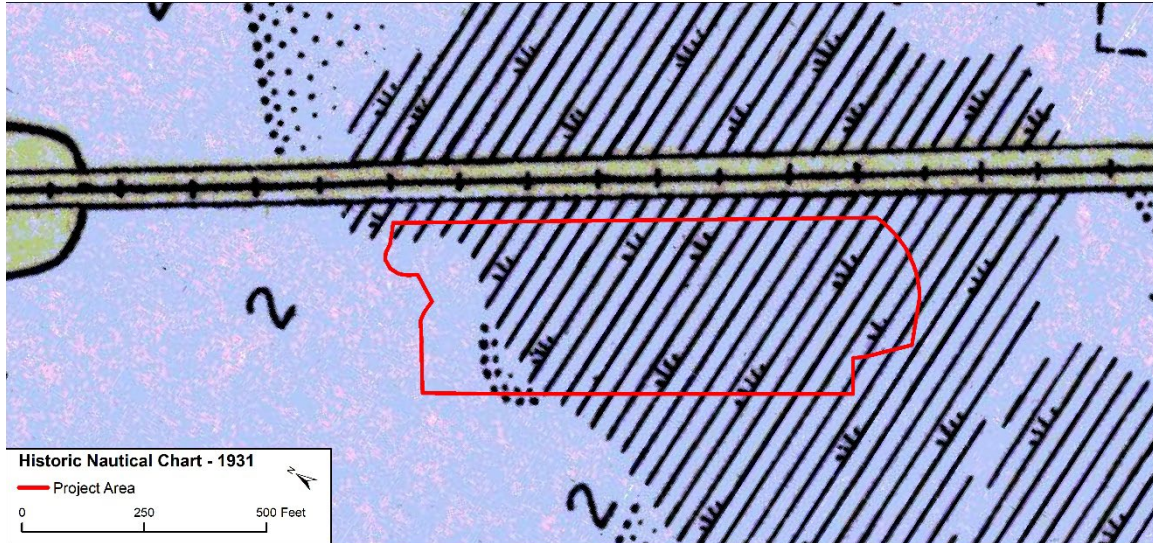


Figure 1-2: Historic (1931) nautical chart of Baltimore Harbor at project site

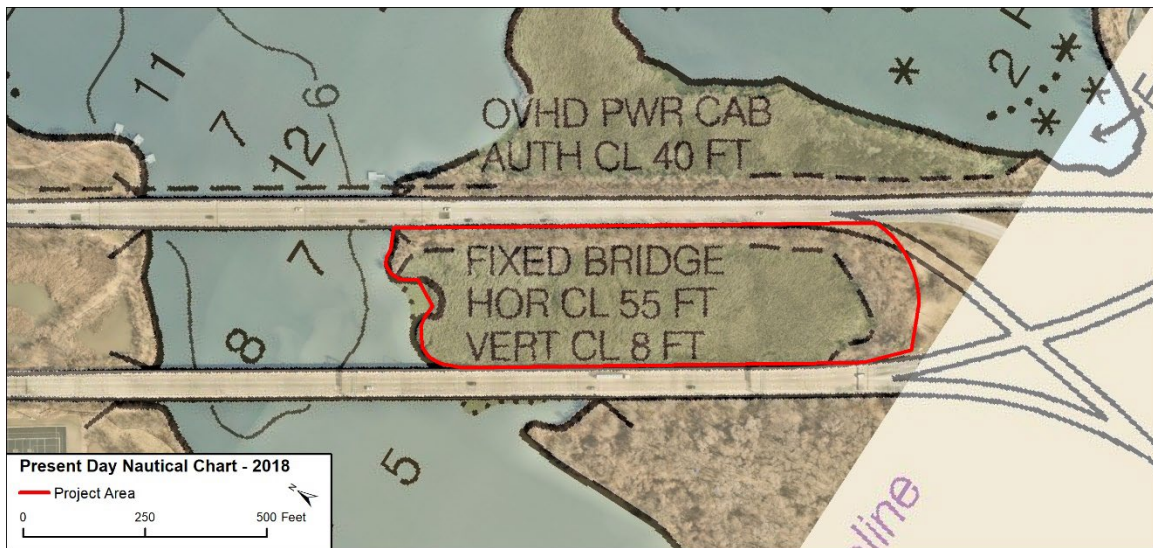


Figure 1-3: Present day (2018) nautical chart of Baltimore Harbor at project site

As part of the engineering services to GreenVest, M&N has developed a two-dimensional hydrodynamic (HD) tidal model of the upper Chesapeake Bay, Patapsco River, and Baltimore Harbor. The HD model is used 1) to assess the impacts of the project to the typical day-to-day hydrodynamics at the project location, and 2) to assess the effects of elevated flow conditions that may also be included in the basis for the project design.

2.0 METEROLOGICAL & OCEANOGRAPHIC CONDITIONS

2.1 Wind

Wind data measured at Baltimore/Washington International Thurgood Marshall Airport (KBWI) was analyzed to characterize the wind near the project site. The station used for this analysis is located approximately 6.3 miles from the project site where the anemometer is located approximately 33 ft above the ground surface, unobstructed for all directions. The period of record for this station is from 1945 until present, providing approximately 79 years of data. A wind rose for this data is shown in Figure 2-2.

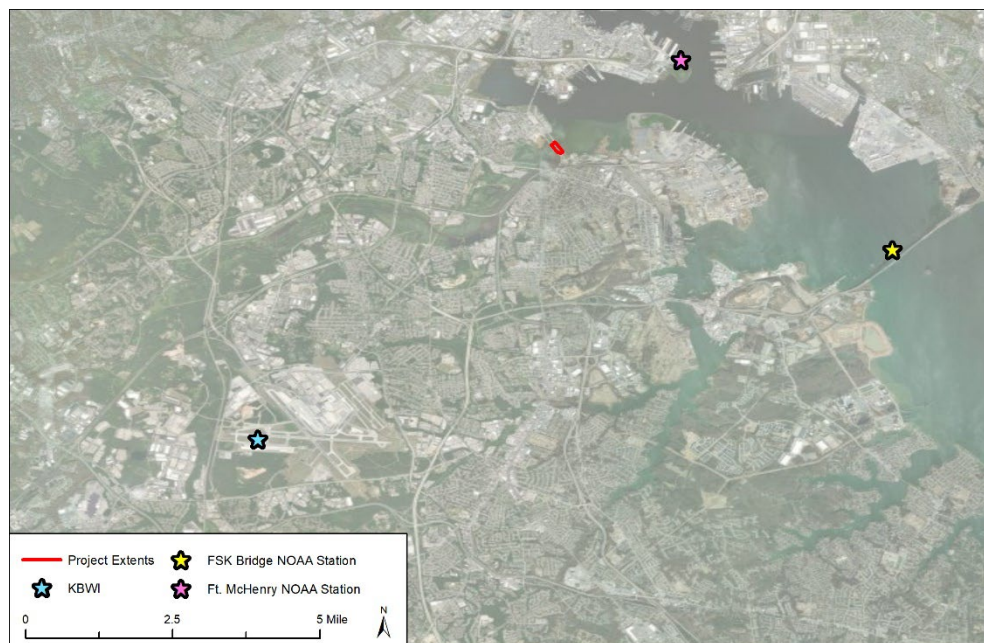


Figure 2-1: KBWI Airport, Francis Scott Key Bridge, and Fort McHenry

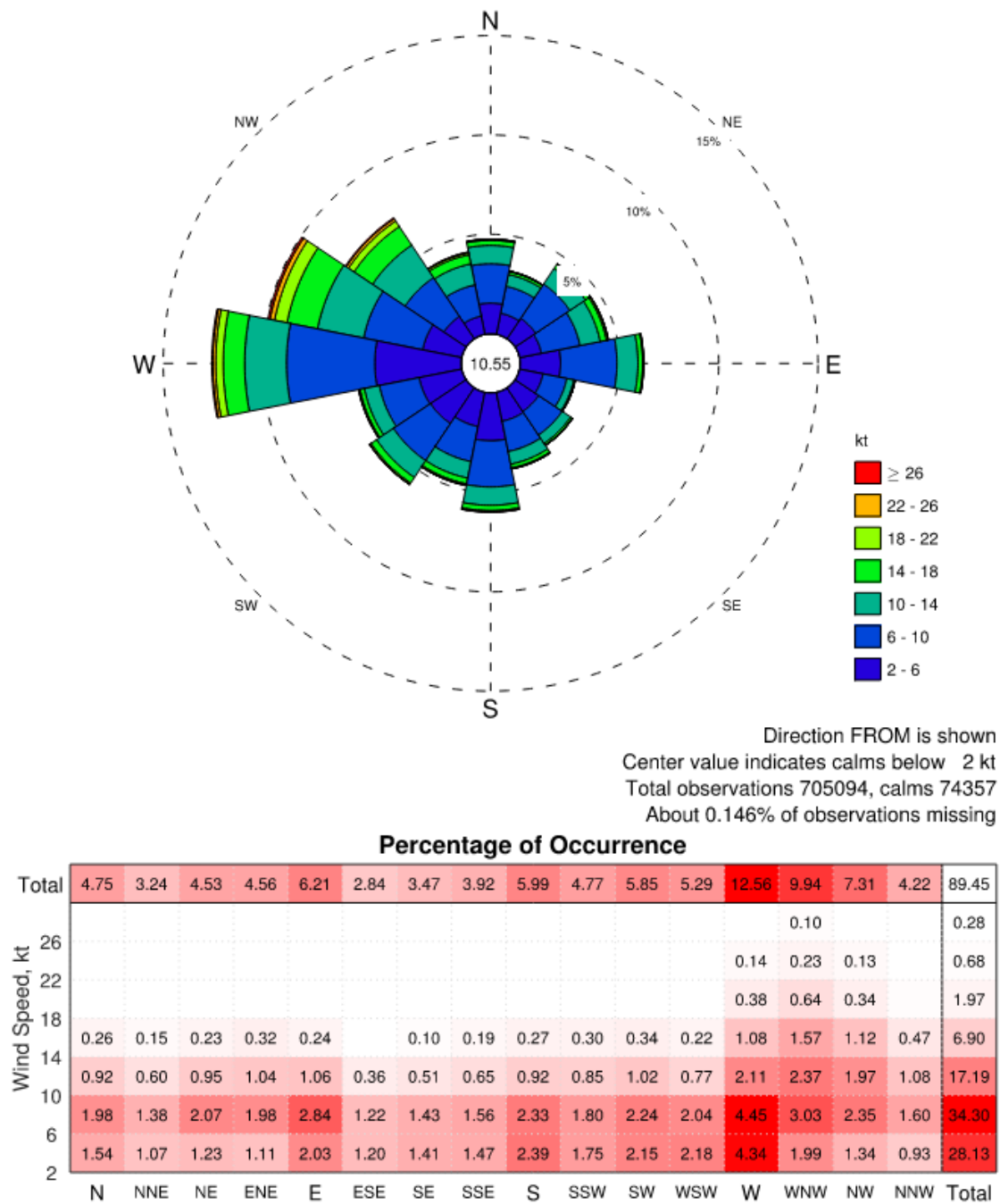


Figure 2-2: Wind Rose - Baltimore/Washington International Thurgood Marshall Airport (1945 – Present)

2.2 Water Levels

Water levels at the project site are dominated by astronomical and atmospheric tides. However, sea level rise may present a significant factor in the water level over longer periods of time.

2.2.1 Astronomical Tides

The effects of typical astronomical tides can be summarized by the tidal datums from the Fort McHenry tide gauge (NOAA Station 8574680; Table 2-1), located approximately two miles from the project site (Figure 2-1).

Table 2-1: Tidal Datums – NOAA Station 8574680 (Fort McHenry)

Tidal Datum	Tide Level [ft]		
	Relative to MLLW	Relative to MSL	Relative to NAVD88
Highest Astronomical Tide	2.22	1.41	1.38
Mean Higher-High Water	1.66	0.85	0.82
Mean High Water	1.37	0.56	0.53
North American Vertical Datum of 1988	0.84	0.03	0.00
Mean Sea Level	0.81	0.00	-0.03
Mean Low Water	0.22	-0.59	-0.62
Mean Lower-Low Water	0.00	-0.81	-0.84

2.2.2 Atmospheric Tides (Storm Surge)

In addition to astronomical forcing, local and regional atmospheric events affect water levels through fluctuations in wind and pressure. These effects are most pronounced during storm events, leading to what is commonly called “storm surge”. In Baltimore Harbor, the amount of storm surge is dependent on the wind direction during a storm event due to water being forced in or out of the harbor.

Historic water level measurements from NOAA Station 8574680 were evaluated with respect to the measured wind direction from KBWI. Wind data measured at KBWI was used in this analysis due to the much longer data collection period (1945 to present) as compared to wind data measured at the Francis Scott Key (FSK) Bridge (Figure 2-3).

The measured water levels exceed +5.0 ft NAVD88 (i.e., major flood in Baltimore [1]) for wind directions from east-southeast (ESE) to south-southwest (SSW). The largest water level recorded in Baltimore was +7.3 ft NAVD88 associated with Hurricane Isabel in 2003 where the concomitant wind direction was from the south-southeast (SSE).

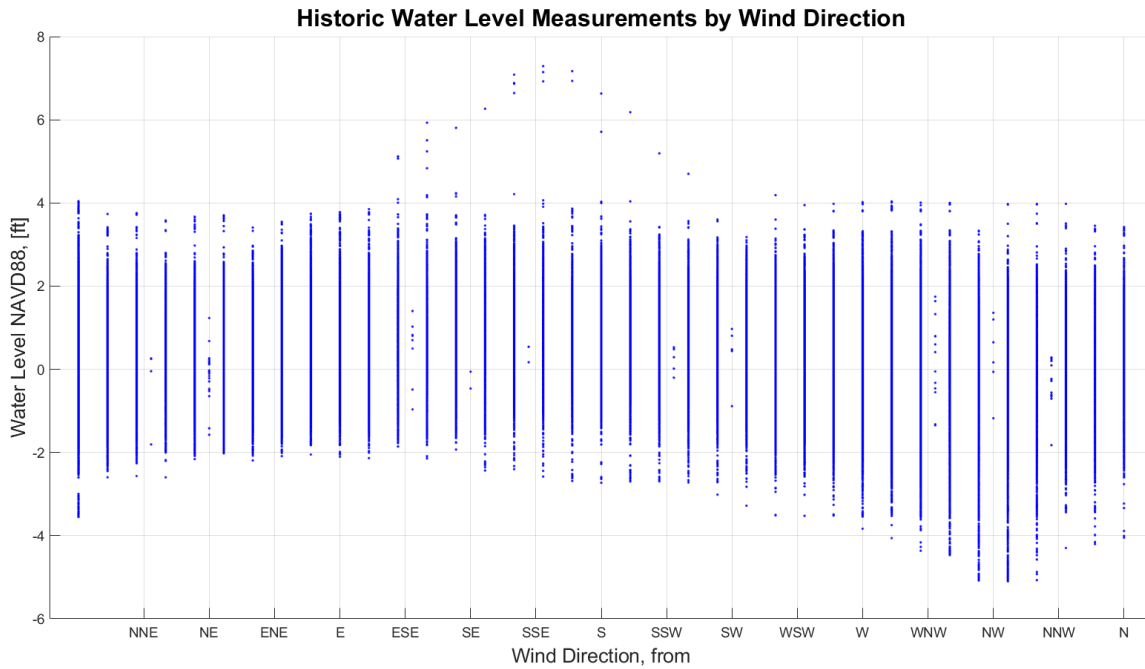


Figure 2-3: Historic Water Levels (NOAA Station 8574680) vs. Measured Wind Direction (KBWI)

2.2.3 Sea Level Rise

This section outlines sea level rise projections that were used for sensitivity analyses in the coastal modeling study. For clarity in illustrating the possible effects of sea level rise, the sensitivity analyses performed in this study use the benchmark projections reported in the Maryland state guidance.

The relative sea level rise (RSLR) projections used for this project are based on the recommendations made in *Sea-level rise: Projections for Maryland 2023* [1] and the probabilistic AR6 projections.

Figure 2-4 illustrates the range of projections for sea level rise relative to a baseline sea level between 1995 and 2014 (i.e., approximately 2005). The lines shown are the median estimate for each emissions scenario. It is important to note that out to about year 2060, there is minimal difference between the scenarios. All scenarios show +/-1 ft of sea level rise relative to today's level and is therefore likely to occur within the project design life. As a result, up to 2 ft of sea level rise is appropriate for sensitivity analysis.

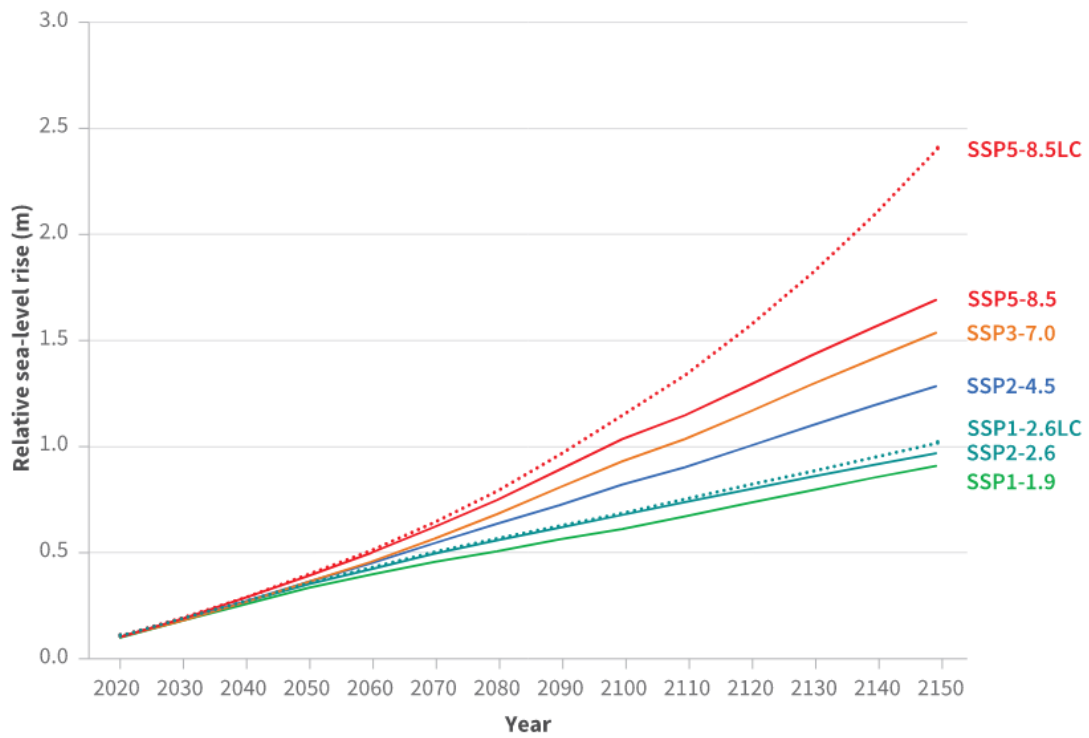


Figure 2-4: Sea Level Rise Projections

3.0 HYDRODYNAMIC (HD) MODEL

A regional two-dimensional HD MIKE 21 Flexible Mesh model was developed to simulate the hydrodynamic conditions experienced in Chesapeake Bay and Baltimore Harbor. MIKE 21 is a modeling system developed by DHI for oceanographic, coastal, and estuarine dynamics applications. The model can predict time dependent flow parameters such as free surface elevation and current velocity at each point in the computational domain based on the numerical solution of the two-dimensional shallow water equations. The model can be forced with either time-varying or constant boundary conditions that may include tides, river discharges, wind, and other parameters [2].

For this application the HD model utilizes an unstructured flexible mesh with variable spatial resolution (Section 3.1.1). A two-dimensional model was determined to be adequate for this modeling effort to simulate the flow at the project site.

Bathymetric, tidal, wind, and river discharge data used for model development and forcing were obtained from various public sources (Section 3.1.2 and Section 3.1.3). Additionally, a topographic and bathymetric survey of the project site was conducted in November 2023 by Mercado Consultants, Inc and was incorporated into the HD model.

3.1 HD Model Development

3.1.1 Model Domain and Computational Grid

The HD model domain includes the upper Chesapeake Bay from Annapolis, MD to Tolchester Beach, MD as well as the Patapsco River and Baltimore Harbor (Figure 3-1). The grid cell resolution varies throughout the model domain where the coarsest resolution is assigned in the Chesapeake Bay and the finest resolution is assigned near the project area (Figure 3-2). The maximum and minimum grid cell size near the project area is 25 m² and 0.3 m², respectively, fine enough to properly resolve the bathymetry, shoreline, and hydrodynamics near the project site. The model grid and mesh are shown in the right most panel of Figure 3-2 for reference.



Figure 3-1: HD model domain and boundary locations

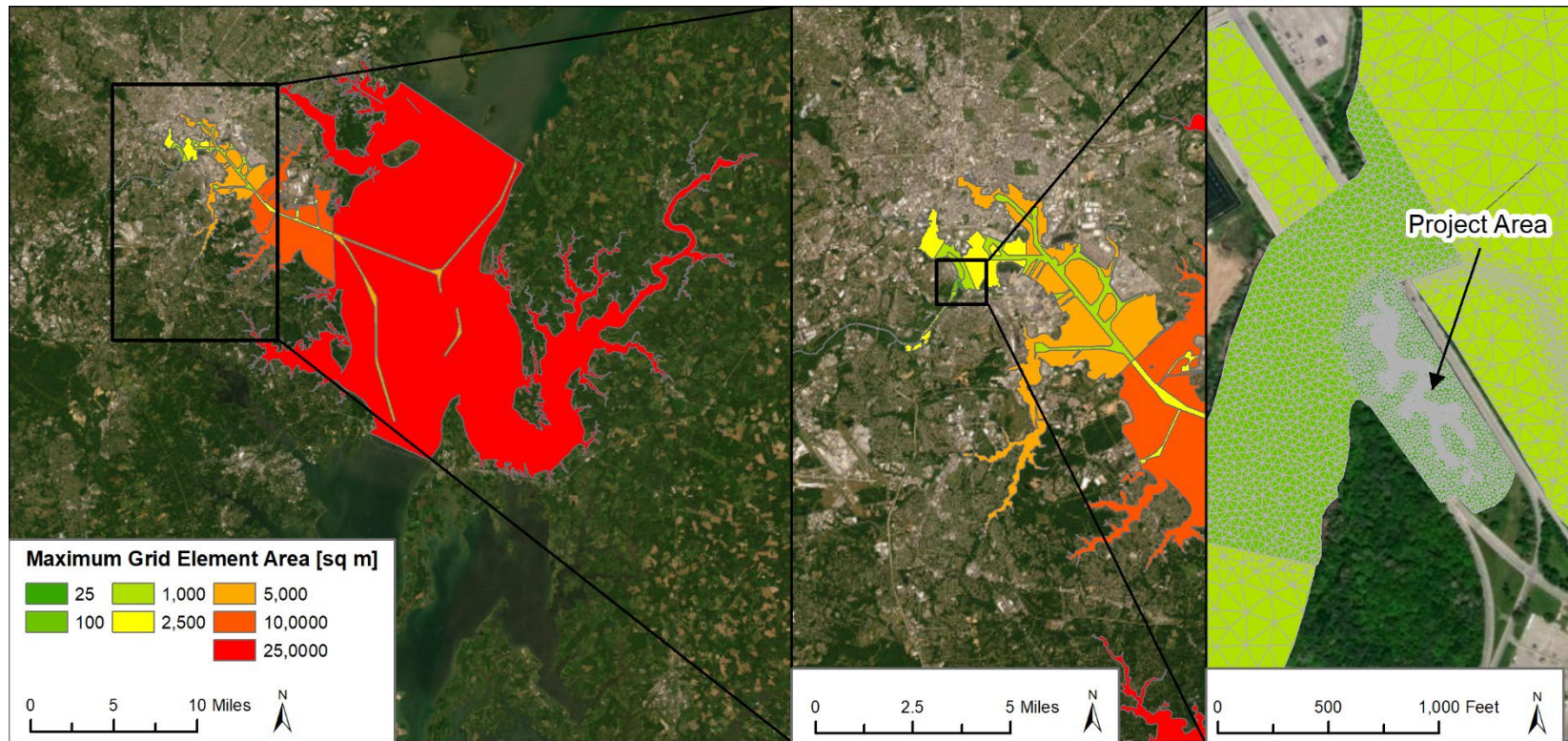


Figure 3-2: Model grid resolution

3.1.2 Shoreline and Bathymetric Data

The land boundary in the HD model was obtained from NOAA's National Geodetic Survey Continually Updated Shoreline Product (CUSP). The obtained shoreline had high resolution that was not necessary to model the hydrodynamics at the project site. Therefore, the shoreline was simplified based upon comparison to aerial imagery to follow the natural shoreline.

Bathymetry data was combined from five data sources, which are listed below. The datasets are listed in order of priority as data sets overlap in some areas.

- Bathymetric survey conducted by Mercado Consultants, Inc. in November 2023
- Bathymetric survey conducted by Mercado Consultants, Inc. in May 2023
- Bathymetric survey conducted by Mercado Consultants, Inc. in March 2022
- United States Army Corps of Engineers (USACE) survey data for the navigation channel.
- NOAA National Centers for Environmental Information (NCEI) Chesapeake Bay (M130) Bathymetric Digital Elevation Model

Before combining, the datasets were converted from their original horizontal reference system to UTM Zone 18N (NAD83). The vertical reference for all datasets were converted from their original reference datum to MLLW. The bathymetry used in the HD model after being interpolated to the model grid (Section 3.1.1) is shown in Figure 3-3.

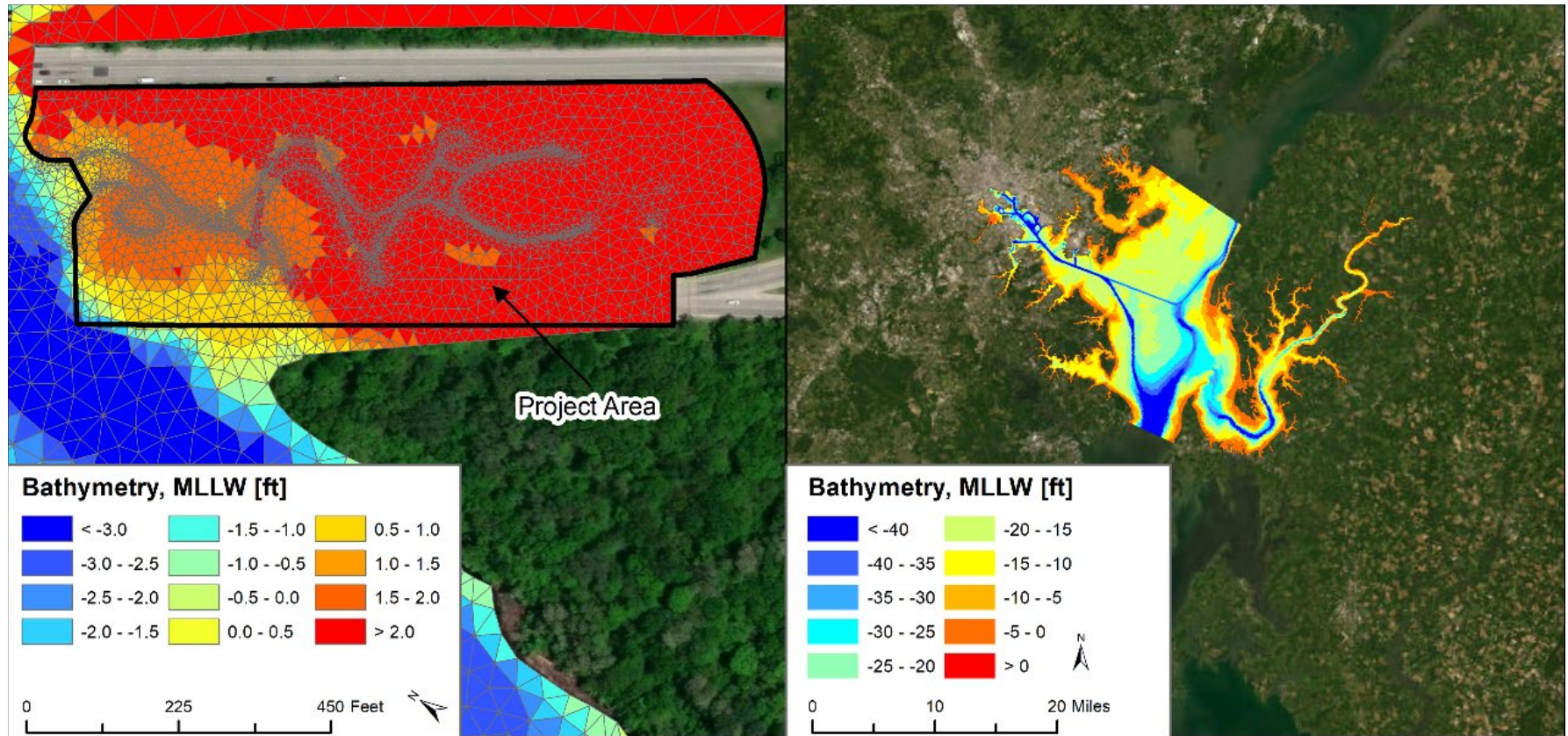


Figure 3-3: Model bathymetry (without-project)

3.1.3 Environmental Conditions

Tides, wind, and river discharge data were incorporated into the HD model and were applied to the model as boundary conditions. Bed roughness was included in the model and is discussed further in Section 3.2. The following subsections describe the applied environmental conditions in further detail.

The HD model has two tidal boundary conditions (Annapolis and Tolchester Beach) and two discharge boundary conditions (Patapsco River and Gwynns Falls). The locations of the boundary conditions are shown in Figure 3-1.

Tidal Boundary Conditions

Time series of measured water levels from the NOAA CO-OPS Station 8575512 and 8573364 were applied to the Annapolis and Tolchester Beach boundaries, respectively. The water level was specified in the HD model to vary in time and constant along the boundary.

Current velocities at the Annapolis and Tolchester Beach boundaries were extracted from the Eastcoast 2001 tidal database [3] and applied as a Flather type condition [4].

Wind

Temporally varying wind data obtained from Baltimore/Washington International Thurgood Marshall Airport (KBWI) was applied to the whole model domain. The wind data provided wind direction and two-minute averaged wind speeds at 170 ft above local MSL at six minute increments. Before being used in the HD model, the wind data was converted to a 33 ft (10 m) reference height using the formulation described in ISO 19901-1:2015(E) [5] and converted to an hour average using formulation described in the Coastal Engineering Manual (CEM) [6].

River Discharge

Time series of river discharge measured from USGS Stations 01589035 and 01589352 were applied to the Patapsco River and Gwynns Falls boundaries, respectively. These two stations were chosen as they would have a significant effect on the local hydrodynamics at the project site.

3.2 HD Validation

3.2.1 Validation Approach

The HD model was previously calibrated by tuning the bed roughness values and assessing model performance by comparing results to verified water level data reported at NOAA CO-OPS Station 8574680 (Ft. McHenry; Baltimore, MD). For this application, the HD model was then validated by comparing modeled water levels measured at the Ft. McHenry tide gauge and current speeds measured by M&N using an Aanderaa RCM-9 current meter in the Middle Branch area of Baltimore Harbor during the time period April 20, 2005 – July 16, 2005 as part of the Masonville DMCF Environmental Assessment [7]. The location of the NOAA CO-OPS station and deployed Aanderaa RCM-9 are shown in Figure 3-4.

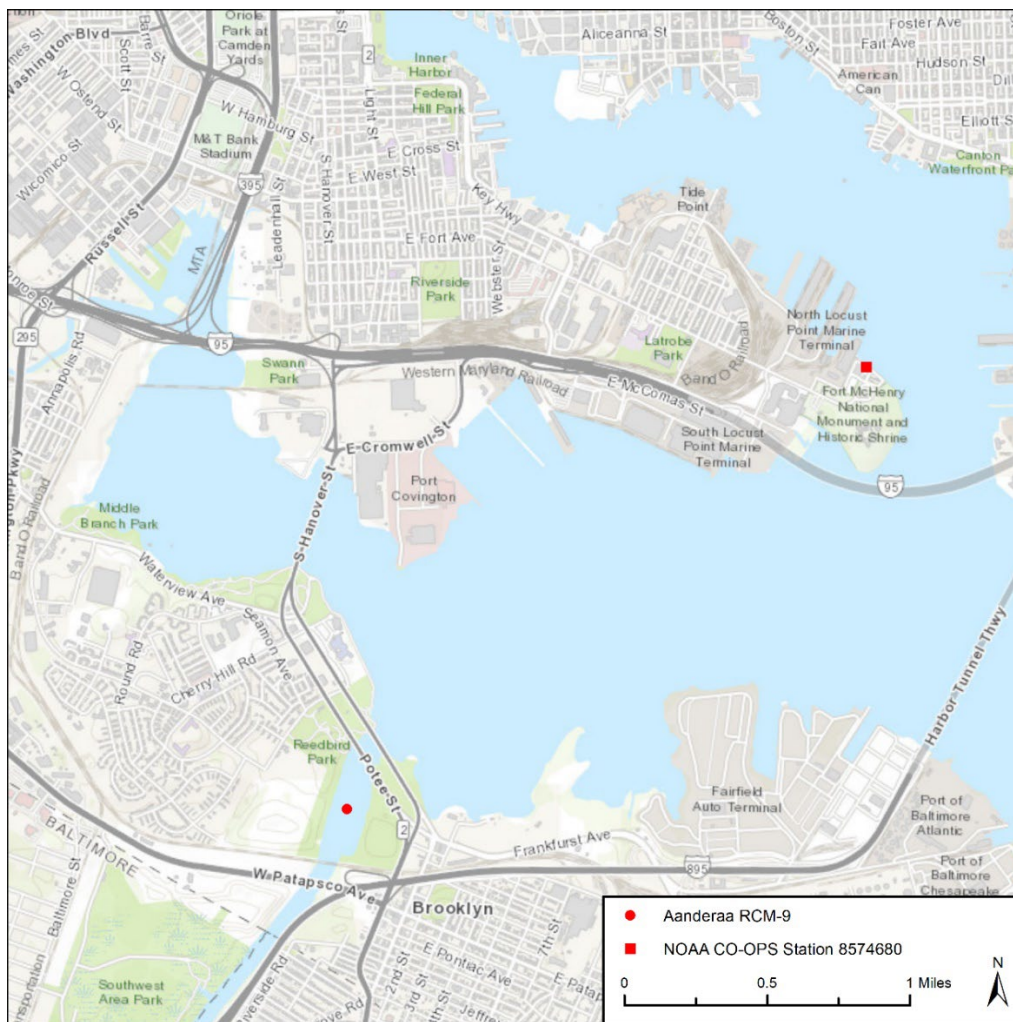


Figure 3-4: NOAA CO-OPS Station 8574680 (Ft. McHenry) and deployed Aanderaa RCM-9 locations

Simulation time periods of April 29, 2021 to June 1, 2021 and April 18, 2005 to May 21, 2005 were used for model validation of water levels and current speeds, respectively. The water level validation time period was chosen to capture a spring-neap tidal cycle while the current speed validation time period was based on when the Aanderaa RCM-9 was deployed while also capturing a spring-neap tidal cycle. The first two days of the validation time period were allocated for model spin-up and not included in the analysis.

Various statistics were calculated to quantify the HD model performance as compared to the measured data by the NOAA CO-OPS station and the Aanderaa RCM-9. Model results were extracted at locations close to the NOAA CO-OPS station and Aanderaa RCM-9 deployment location. The statistics calculated include root mean square error (ϵ_{rms}), bias (MAE), correlation coefficient (R), and time lag (ΔT).

In the following description of the statistics, x and y represent the measured and calculated data, respectively. The overbar in the formulations below denote sample mean.

Root Mean Square Error (ϵ_{rms}):

$$\epsilon_{rms} = \sqrt{\overline{(x - y)^2}}$$

Bias:

$$\text{Bias} = \overline{|y - x|}$$

Correlation coefficient (R):

$$R = \frac{\text{cov}(x,y)}{\sigma_x \sigma_y}; \text{ where } \text{cov} \text{ is the covariance and } \sigma \text{ is the standard deviation.}$$

The time lag (ΔT) was estimated by computing the cross-correlation function between x and y . The value for ΔT was taken at the maximum correlation between x and y . In this application, a positive ΔT indicates y lags x .

The model performance statistics were compared to suggested guidance for coastal and estuarine modeling [8]. For this application, the performance metrics for near the head of an estuary were used based on the geographic location of Baltimore Harbor and are given in Table 3-1.

Table 3-1: Suggested model performance criteria

Performance Metric	Physical Parameter	Suggested Criteria
RMSE	Water Level	< 1.0 ft or < 10 – 15% of measured water level
	Current Speed	< 0.3 ft/s or < 10 – 20% of measured peak current speed
Bias (MAE)	Water Level	< 20% of measured water level
	Current Speed	< 10 – 15% of measured peak current speed
Correlation Coefficient (R)	Water Level	> 0.95
	Current Speed	> 0.90 – 0.95
Lag	Water Level	< 25 minutes of measurement
	Current Speed	

Source: J. J. Williams and L. S. Esteves, "Guidance on Setup, Calibration, and Validation of Hydrodynamic, Wave, and Sediment Models for Shelf Seas and Estuaries," *Advances in Civil Engineering*, 2017.

3.2.2 Validation Results

The bed roughness was parameterized using the Manning number (M), which is the inverse of the commonly used Manning's n coefficient. Previous modeling efforts by M&N showed the application of a linearly varying roughness inversely proportional to the still water depth was the most successful calibration method. The calibrated HD model described herein utilizes the aforementioned spatially varying bed roughness where M (n) varies between 33 (0.03) and 100 (0.01) for water depths less than 50 ft. Water depths greater than 50 ft were set to $M = 100$ ($n = 0.01$). The area in the HD model of the Patapsco River from the domain boundary (Figure 3-1) to where the river mouth feeds into Baltimore Harbor was set to $M = 33$ ($n = 0.03$). The bed roughness values used in the model near the project site generally range between $M = 35 - 36$ ($n = 0.028 - 0.029$) for without project conditions. The with project roughness values are discussed in Section 3.2.2.

Water Level Validation Results

Time series of the modeled and measured water level using the calibrated bed roughness values are shown in Figure 3-5. The error associated with the modeled water levels were evaluated by methods described in Section 3.2.1 and are given in Table 3-2.

The HD model reproduces the water surface elevations experienced in the Baltimore Harbor to an acceptable degree of accuracy as suggested by the performance criteria given in Table 3-1 and is therefore determined as validated for this application.

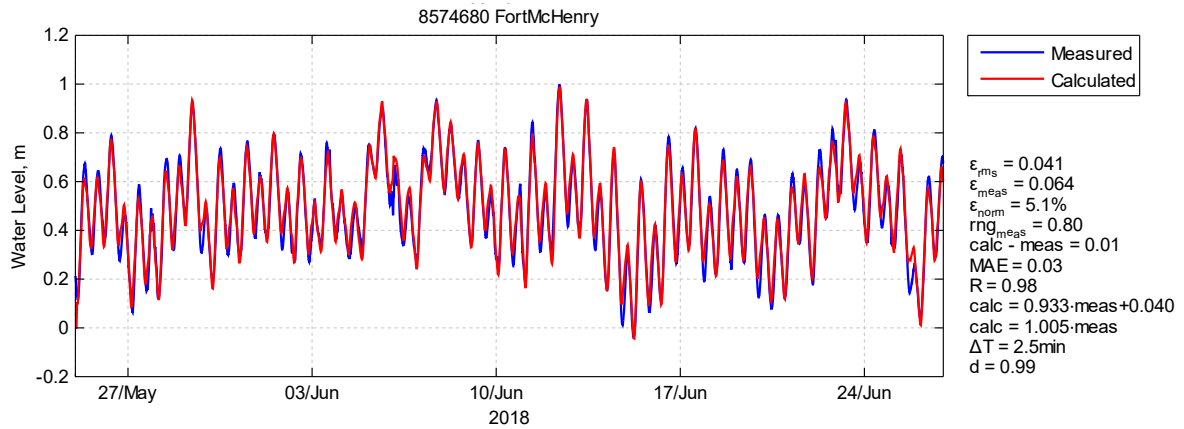


Figure 3-5: HD model water level validation results (Calibration statistics in meters)

Table 3-2: HD model calibration performance – Water level (NOAA CO-OPS Station 8574680)

Performance Metric	Metric Value
RMSE [ft]	0.13
Bias [ft]	0.10
R	0.98
ΔT [min]	2.5

Current Speed Validation Results

The HD model was validated by comparing modeled current speeds to measured current speeds by an Aanderaa RCM-9 at the location shown in Figure 3-4 (red circle). The results of the HD model current speed validation are shown in Figure 3-6 where the measured data was smoothed using a moving average to filter out background noise associated with data collection. The error associated with the modeled current speeds resulting from the validated HD model is shown in Table 3-3.

The HD model reproduces the current speeds experienced in near the project site to an acceptable degree of accuracy as suggested by the performance criteria given in Table 3-1. It should be noted that the correlation coefficient (R) did not meet the suggested criteria of $> 0.90 - 0.95$. This could be due to the relatively large amount of noise in the modeled current speed signal or the lack of reliable bathymetry in the measurement location that would significantly affect the hydrodynamics. Aside from the correlation coefficient, the other metrics meet the suggested criteria very well and therefore the model is determined as validated for this application.

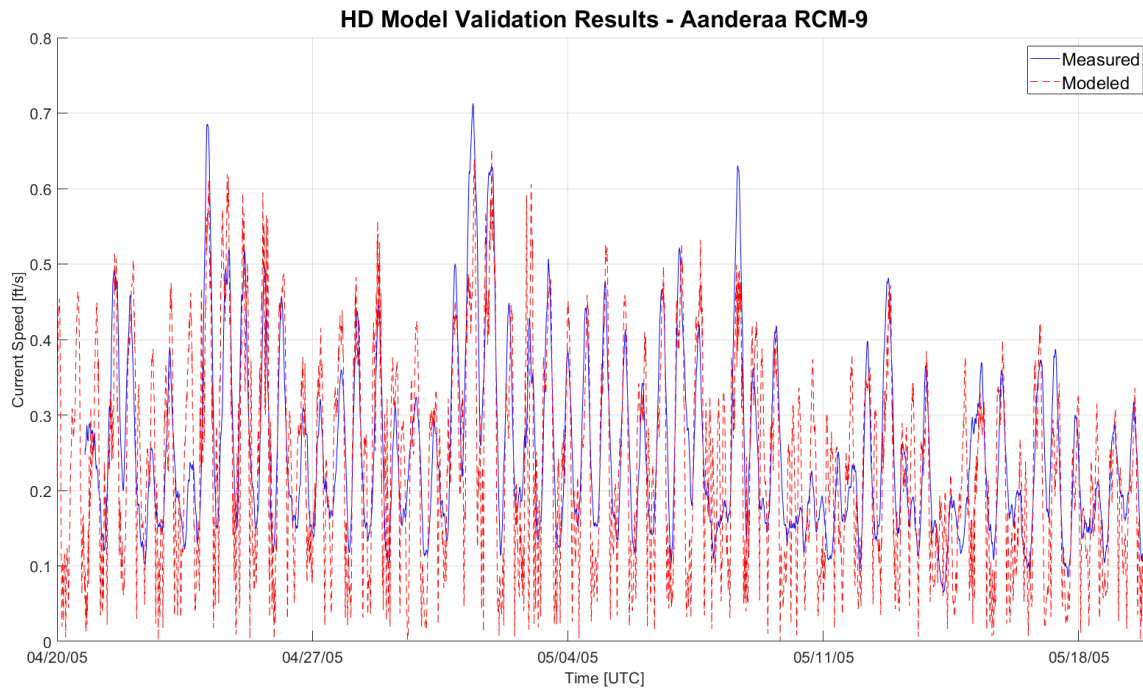


Figure 3-6: HD model current speed validation results

Table 3-3: HD model validation performance – Current speed (Aanderaa RCM-9)

Performance Metric	Metric Value
RMSE [ft/s] (% of peak measurement)	0.09 (12%)
Bias [ft/s] (% of peak measurement)	0.08 (10%)
R	0.77
ΔT [min]	12

3.3 HD Production Simulations

3.3.1 Production Simulation Overview

A simulation time period of May 25, 2018 to June 27, 2018 was used for production simulations to capture a spring-neap tidal cycle and to also capture high and intermediate flow events measured at the Patapsco River and Gwynns Falls. The first two days of the simulation time period were allocated for model spin-up and not included in the analysis. Utilizing this simulation time period, five production HD model simulations were run and evaluated to assess the effects of the project geometry on the local hydrodynamics, including a sensitivity simulation assessing the effect of increasing water level within the project footprint. Production simulations include:

- Existing conditions (i.e, without project geometry) including 2- and 10-year flow events

- Existing conditions with elevated tail water (+2 ft) including 2- and 10-year flow events
- Existing conditions with 100-year return period flow event
- Proposed project geometry including 2- and 10-year flow events
- Proposed project geometry with 100-year return period flow event

The high and intermediate flow events correspond to 10- and 2-year return period (RP) based on historical data measured by USGS gauges 01589035 (Patapsco River) and 01589352 (Gwynns Falls). An extreme value analysis of the Patapsco River USGS gauge is shown in Figure 3-7 and tabulated in Table 3-4. A hydrograph measured by the USGS gauges during the production simulation time period is shown in Figure 3-8.

A 100-year return period was analyzed to assess the effect of an extreme flow event that could occur during the design life of this project. To simulate the 100-year return period flow event, discharge measurements from the Patapsco River and Gwynns Falls were multiplied by a respective scale factor. The scale factor was determined by scaling the extreme value analysis 10-year return period discharge rate to the 100-year return period discharge rate. The Patapsco River and Gwynns Falls boundary conditions were scaled by factors of 1.81 and 1.58, respectively. A hydrograph displaying the 100-year scaled return period in respect to the 10-year return period of the Patapsco River is included as Figure 3-9.

Wind forcing was applied to the model during the simulation time period as discussed in Section 3.1.3.

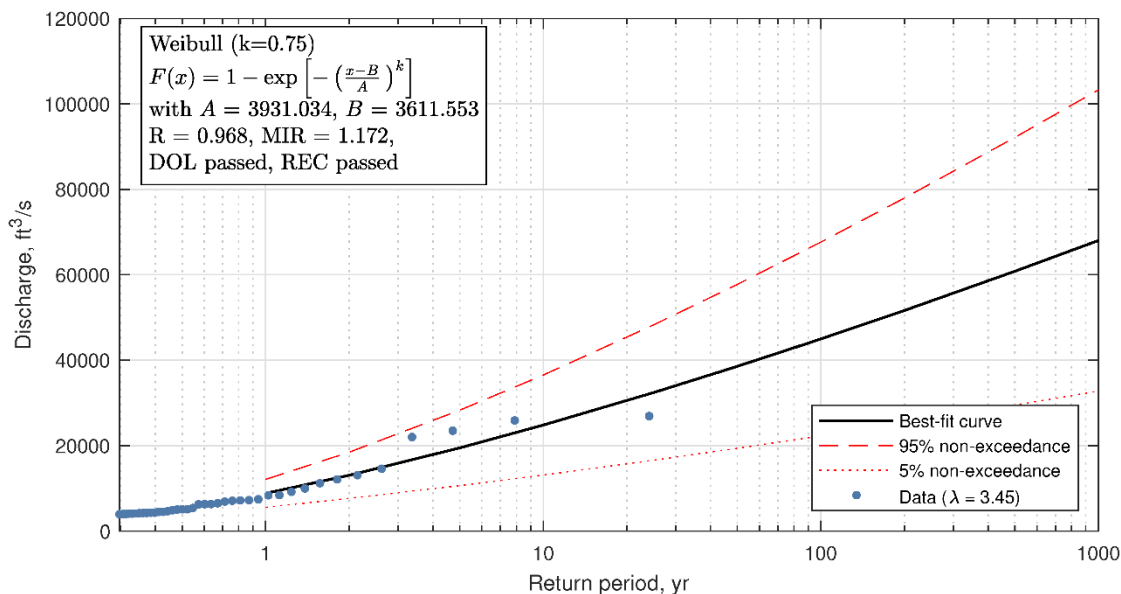


Figure 3-7: Extreme value analysis – Patapsco River (USGS Gauge 01589035) discharge

Table 3-4: Extreme value analysis – Patapsco River (USGS Gauge 01589035) discharge

Return Period [yr]	5% Non-Exceedance [ft³/s]	Best Fit [ft³/s]	95% Non-Exceedance [ft³/s]
2	7,650	13,060	18,470
10	13,110	24,815	36,525
25	16,590	32,435	48,280
50	19,380	38,565	57,750
100	22,290	44,975	67,655

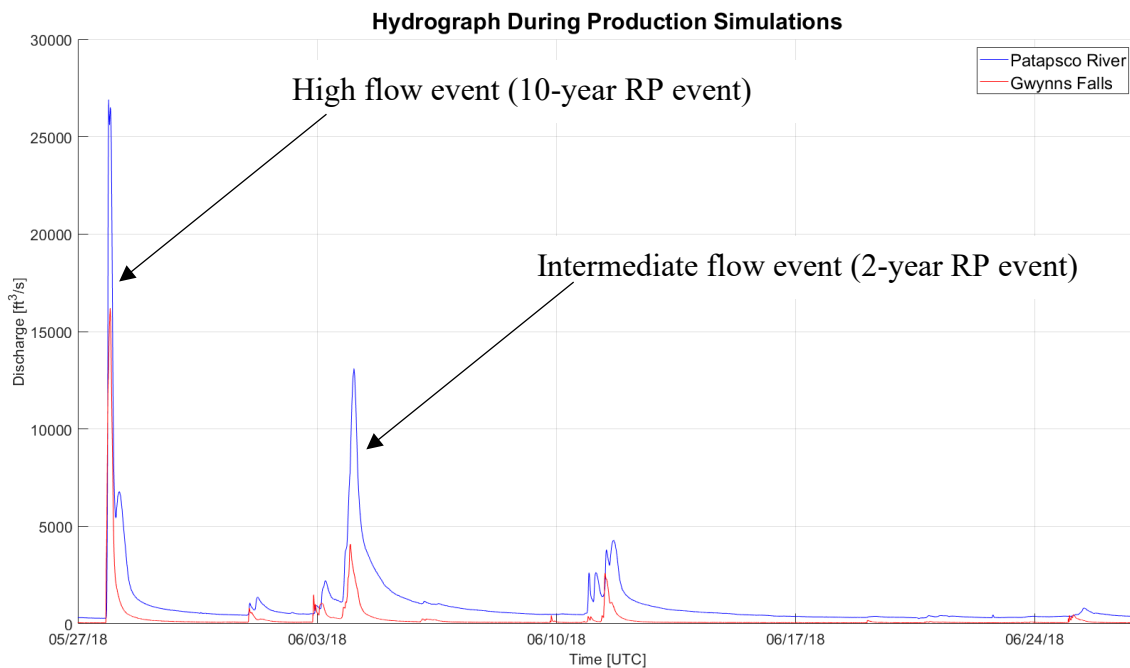


Figure 3-8: Hydrograph of Patapsco River and Gwynns Falls during production simulations

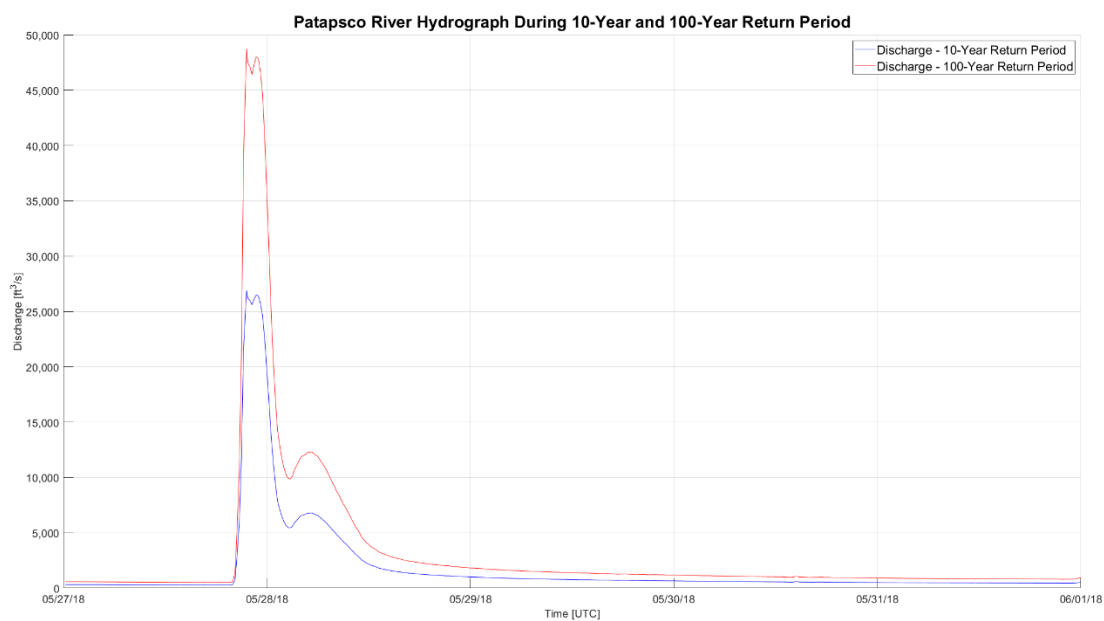


Figure 3-9: Hydrograph of Patapsco River during 10-year and 100-year return period flow event

3.3.2 Project Geometry

The project geometry and high-resolution grid incorporated into the HD model is shown in Figure 3-10. The HD model resolution within the project geometry is controlled with a maximum grid resolution of 25 m², fine enough to resolve the hydrodynamics throughout the project area. Characteristics of the project geometry that are included in the HD model are:

- Gravel shoreline placed with a 1:6 offshore slope to an elevation of +1.5 feet NAVD88.
- Supratidal marsh vegetation zones, represented with elevations ranging from +1.0 – +2.5 ft NAVD88.
- High marsh vegetation zones, represented with elevations ranging from +0.5 – +1.0 ft NAVD88.
- Low marsh vegetation zones, represented with elevations ranging from 0.0 – +0.5 NAVD88.
- Shallow open water channels, represented with elevations ranging from -0.8 – 0.0 ft NAVD88.

As described in Section 3.2.2, the calibrated roughness values applied to the HD model is spatially varying and linearly proportional to the still water depth for depths below 50 ft. The roughness values used in the with project HD production simulations generally range between $M = 33 - 34$ ($n = 0.029 - 0.030$).

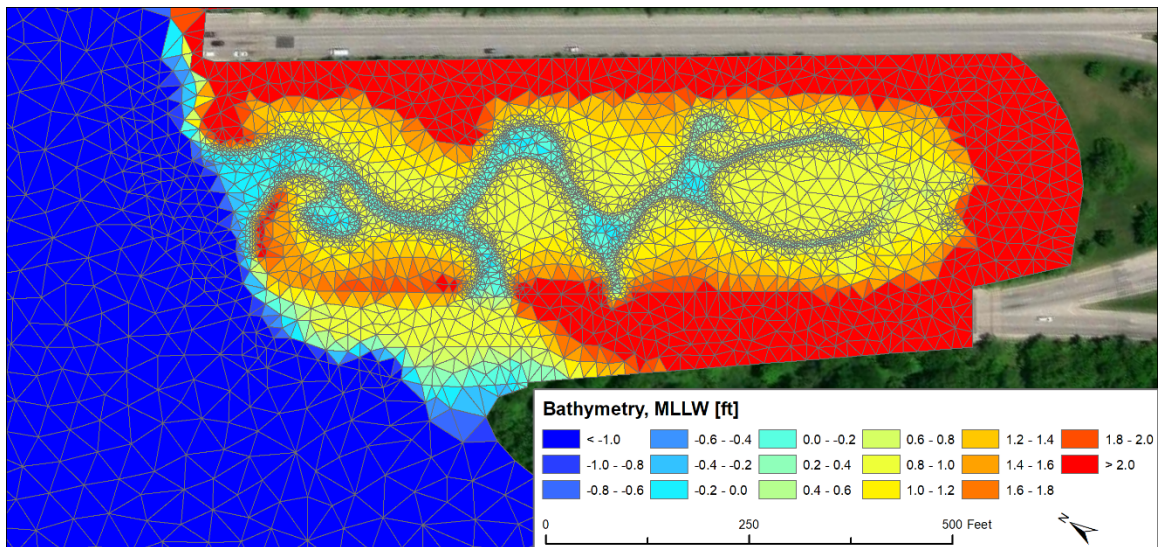


Figure 3-10: Project geometry applied to model grid

3.3.3 Production Simulation Results

Changes in local hydrodynamics at the project site are evaluated by comparing the with and without project production simulations during six unique hydrodynamic conditions:

- 1) High tide (a relatively high condition identified, likely including some atmospheric effects),
- 2) Low tide (a little lower than MLLW, representing a relatively typical low water tidal condition),
- 3) An intermediate river discharge event (2-year RP),
- 4) A high flow river discharge event (10-year RP),
- 5) An extreme flow event (100-year RP), and
- 6) A high tail water (+2 ft) flow event (primarily reporting during low tide).

The high tail water scenario (Condition No. 6) was assessed during low tide when velocities are anticipated to be greater than at high tide. It is assumed that as surface level decreases, velocities increase due to flow restriction within the channels. As a result, the low tide was assessed as a sensitivity analysis and provides a more conservative comparison than assessing during high tide.

The modeled water level immediately outside of the project area for these conditions are given in Table 3-5 for reference. The evaluated hydrodynamic conditions provide useful insight on the expected hydrodynamics for water levels ranging from approximately -1.0 to +6.6 ft NAVD88.

Table 3-5: Modeled water level at project site for evaluated flow/tide conditions

Hydrodynamic Condition	Modeled Water Level, NAVD88 [ft]
Low Tide	-0.98
High Tide	+2.41
Intermediate Flow Event (2-yr RP)	+3.33
High Flow Event (10-yr RP)	+3.92
Extreme Flow Event (100-yr RP)	+6.55
Elevated Tail Water at Peak Low Tide (+2 ft)	+1.02

Production simulations show that the flow from the Patapsco River typically enters the proposed wetlands through both north and south inlets on the western boundary of the project side during flood tide and flows into the southern inlet and exits through the primary inlet on the north during ebb tide. An example of the with project flow pattern is shown in Figure 3-11. The flow pattern with and without the project geometry during the evaluated hydrodynamic conditions are shown in Appendix A.

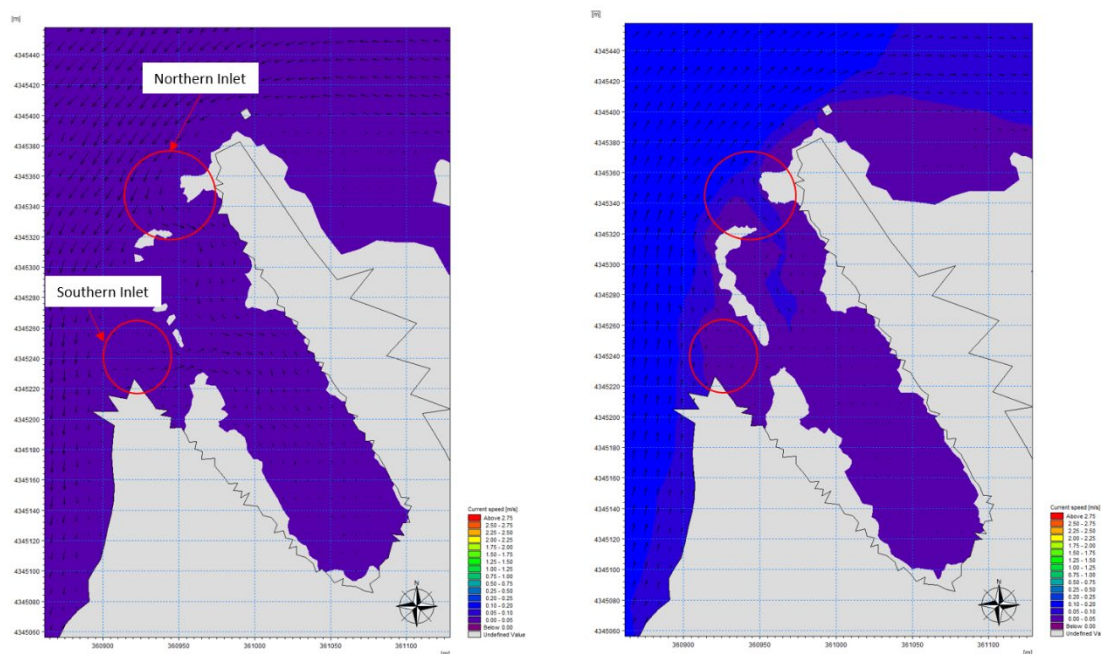


Figure 3-11: Flood (left) and ebb (right) tide flow patterns within project footprint

The interior low marsh regions of the project footprint begin to accumulate water when the water level exceeds approximately +0.31 ft NAVD88 and become completely inundated when the water level increases to approximately MHW (+0.53 ft NAVD88). As the tidal cycle progresses and water level increases above MHW, the high marsh regions begin to experience inundation. The high marsh regions are completely inundated at approximately +0.96 ft NAVD88 and are shown to drain from the wetlands into Baltimore Harbor with the falling tide. Some persistent pools are observed in the southeastern portion of the project footprint. Water remains within the shallow open water channels throughout the inundation cycle. It should be noted that the hydrodynamic model does not account for infiltration or evaporation that could occur and therefore lower the retained water. An example of this inundation cycle was modeled during the time period of June 13, 2018 04:12 – 14:30 UTC and is shown in Figure 3-12.

The with and without-project production simulations were compared at nine points within the project area (Figure 3-13 and Figure 3-14). Time series at these nine points for the with and without-project production simulations are shown in The modeled hydrodynamics during five hydrodynamic conditions and at the nine model comparison points are summarized in Table 3-6 – Table 3-8.

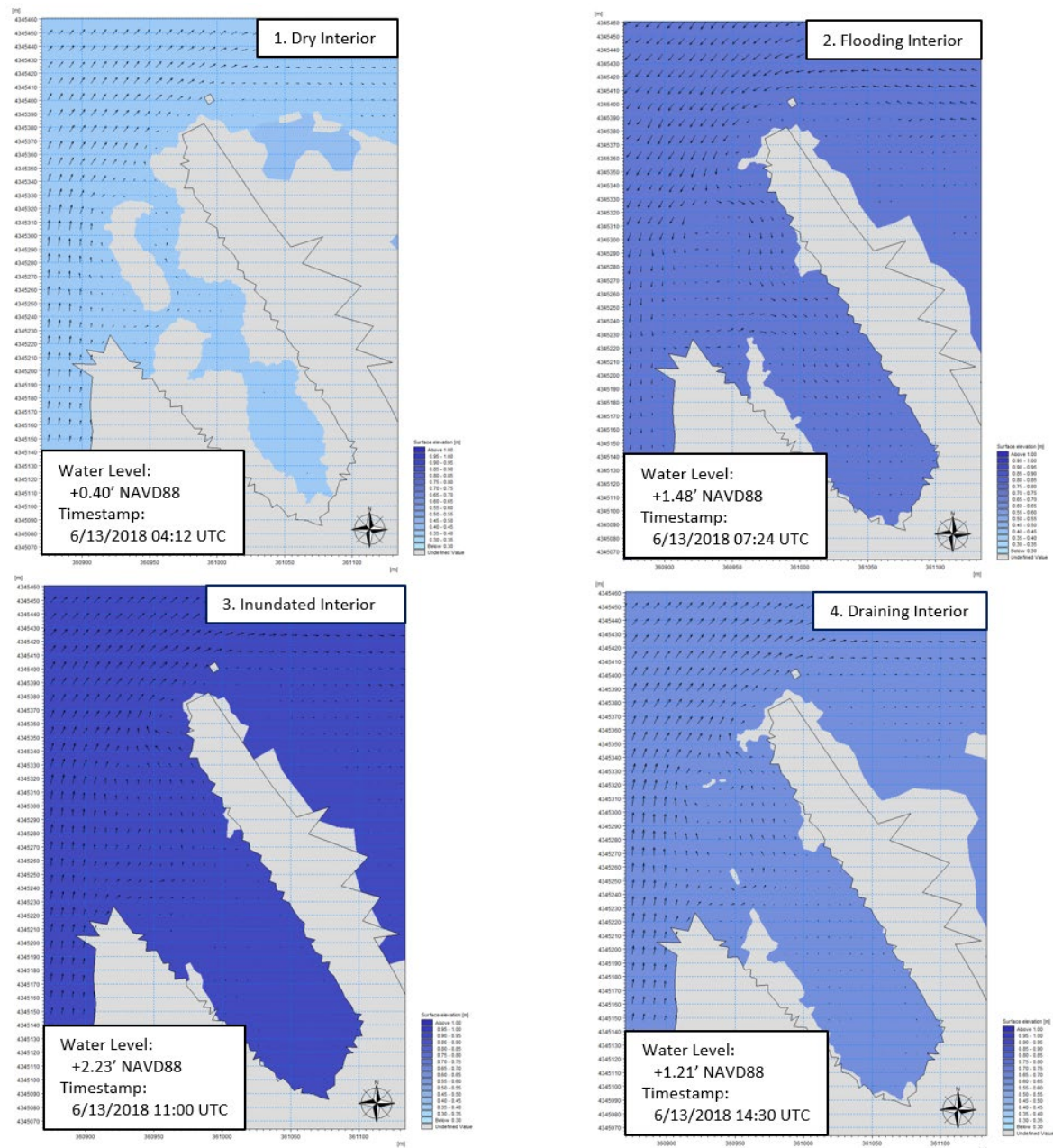


Figure 3-12: Modeled inundation cycle within project footprint

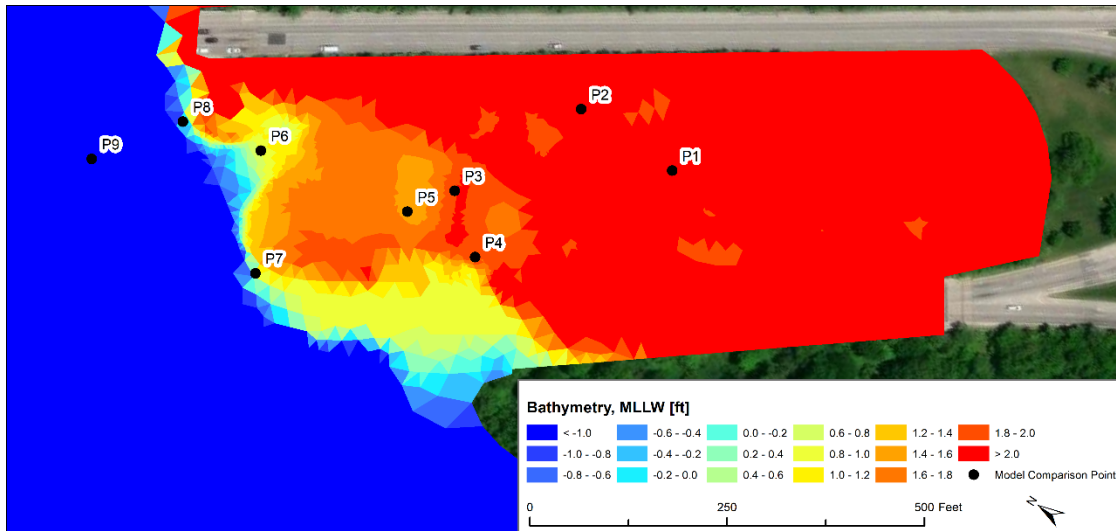


Figure 3-13: Model comparison points (without-project geometry shown for reference)

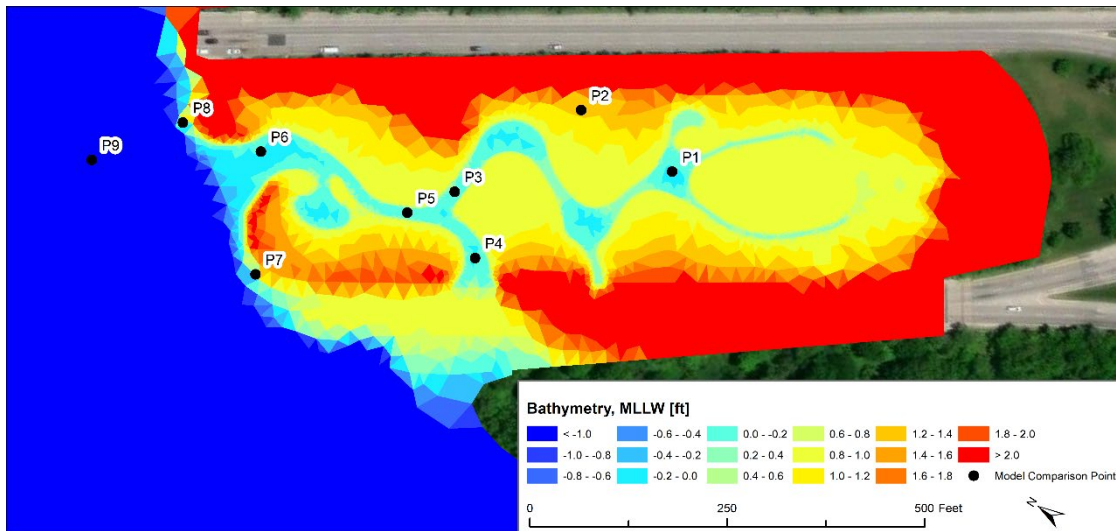


Figure 3-14: Model comparison points (with-project geometry shown for reference)

The elevated tail water conditions were compared to the existing project conditions over the modeled time period to assess the effect of increased water level on the project hydrodynamics. The elevated water level results in decreased current speeds within the Patapsco River, evident at Point 9. This trend is further observed in the time series of the eight other chosen hydrodynamic extraction points, particularly at Point 4 and 6 (Appendix B). This decrease is concurrent with the general assumption that as available flow area increases and flow restriction decreases, flow velocity decreases. The increase in water level shows the project footprint, including the high marsh zones, as becoming inundated. This is expected, as the +2 ft water level results in a near-project water level of +1.02 ft NAVD88 at peak low-tide. This elevation exceeds the high marsh zone elevation. From

this sensitivity analysis, it is concluded that increased water levels with otherwise consistent flow conditions will result in decreased flow velocities through the project footprint.

Modeled water levels indicate that the interior portion of the wetlands provides drainage within the project area. Point 5 shows a decrease in water level between the proposed wetlands and existing conditions during peak low tide conditions. It is observed that at this point, pools of water form with the existing conditions. The addition of the shallow open water channels in the proposed wetlands allows for water to flow from these pools during peak low tide conditions, resulting in an overall decrease in water level (Figure 3-15). During the 10-year return period, the exterior points of the project geometry (Points 7, 8, and 9), model slightly greater water levels, ranging from +0.1 – 0.4 ft. This indicates a changing flow pattern that funnels water into the proposed wetland and around the proposed gravel shoreline during the elevated flow condition. Remaining points within the high marsh regions and shallow open water channels (Points 1 – 4 and Point 6) indicate relatively unchanged water levels between existing and proposed conditions.

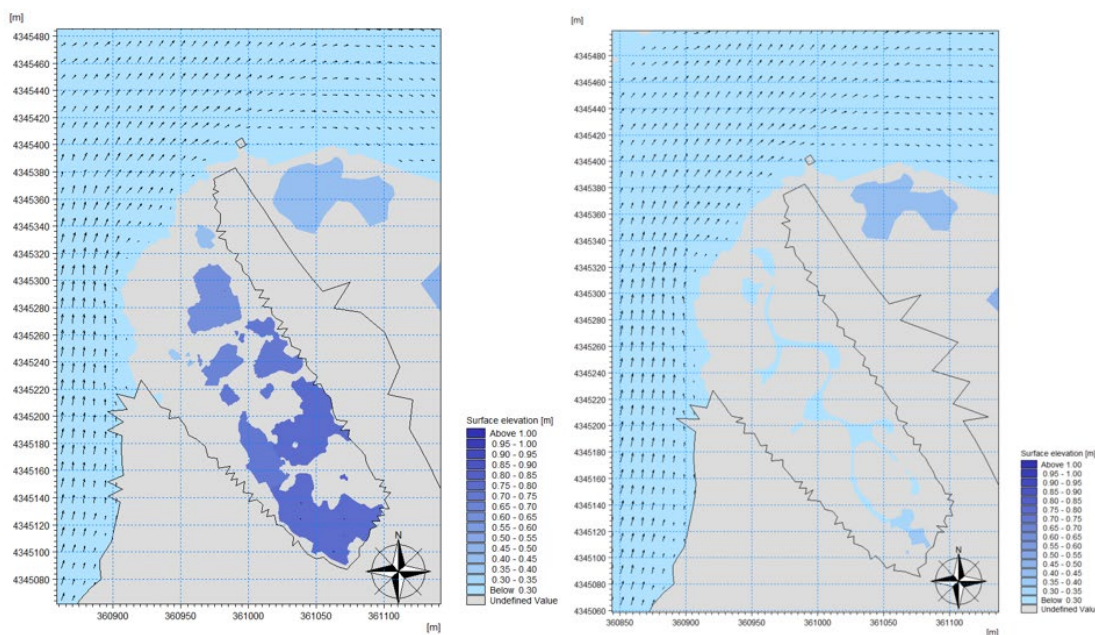


Figure 3-15: Comparison of water levels for without (left) and with (right) project production simulations during peak low tide.

Multiple points within the project geometry indicate significant changes in current speed between the existing and proposed conditions. Point 2, within the high marsh zone of the project geometry, shows a 0.3 ft/s increase in current speed due to the proposed project geometry during an extreme flow event (100-yr RP). During the extreme flow event, the water level exceeds the elevation of the high marsh elevation. It is important to note that

the other four modeled hydrodynamic conditions show either no change or a decrease in current speed at this location due to the duration of the corresponding flow event. The two inlet locations, Point 4 and Point 6, showed the greatest increase in current speed during the intermediate flow event of 0.6 and 0.8 ft/s, respectively. This increase in current speed can be attributed to water flowing through the shallow open water channel where higher ground elevations are present under existing conditions and therefore do not experience typical tidal flow. Due to this significant increase in current speed, there is the potential for sediment motion at this location within the entrance to the channel. Additional significant changes include decreased velocities adjacent to the southern gravel shoreline (Point 7) and interior shallow open water channel (Point 5). The decrease at these locations is observed for the intermediate, high, and extreme flow events. The northern gravel shoreline (Point 8) experiences a significant decrease in current speed of 1.5 ft/s during the high flow event; however, small velocity increases are observed in the intermediate and extreme flow events.

Current direction within the project geometry remains within 90 degrees absolute change for all hydrodynamic conditions at all points except for Point 2 and Point 5. Point 2, located within the high marsh region, shows a near 180-degree change in current direction during the intermediate and extreme flow event; however, negligible changes are observed during the remaining hydrodynamic conditions. The change in current direction at Point 5, located within the shallow open water channel, during the peak low-tide can likely be attributed to the drainage and flow occurring within the project. As discussed above, the existing conditions result in pooled water during peak low tide at various locations within the interior project bounds. As water flows within the project geometry at this location, a change in flow direction and pattern is expected. Remaining regions within the model show small or negligible changes in current direction.

Table 3-6: Modeled Water Level During Hydrodynamic Conditions at Model Comparison Points

Modeled Water Level [ft NAVD88]									
Hydrodynamic Condition	Point 1			Point 2			Point 3		
	Without-project	With-project	Difference*	Without-project	With-project	Difference*	Without-project	With-project	Difference*
Peak Low Tide	-	-0.3	-	1.6	-	-	-	-0.5	-
Peak High Tide	2.4	2.4	0.0	2.4	2.4	0.0	2.4	2.4	0.0
Intermediate Flow Event	2.0	2.0	0.0	2.0	2.0	0.0	2.0	2.0	0.0
High Flow Event	3.6	3.6	0.0	3.6	3.6	0.0	3.6	3.6	0.0
Extreme Flow Event	6.5	6.5	0.0	6.5	6.5	0.0	6.5	6.5	0.0
Hydrodynamic Condition	Point 4			Point 5			Point 6		
	Without-project	With-project	Difference*	Without-project	With-project	Difference*	Without-project	With-project	Difference*
Peak Low Tide	-	-0.5	-	1.2	-0.5	-1.7	-	-	-
Peak High Tide	2.4	2.4	0.0	2.4	2.4	0.0	2.4	2.4	0.0
Intermediate Flow Event	2.0	2.0	0.0	2.0	1.9	-0.1	1.5	1.7	0.2
High Flow Event	3.6	3.6	0.0	3.6	3.7	0.1	3.3	3.4	0.1
Extreme Flow Event	6.5	6.5	0.0	6.5	6.5	0.0	6.3	6.4	0.1
Hydrodynamic Condition	Point 7			Point 8			Point 9		
	Without-project	With-project	Difference*	Without-project	With-project	Difference*	Without-project	With-project	Difference*
Peak Low Tide	-	-	-	-	-	-	-1.0	-1.0	0.0
Peak High Tide	2.4	2.4	0.0	2.4	2.4	0.0	2.4	2.4	0.0
Intermediate Flow Event	1.4	1.4	0.0	1.2	1.2	0.0	1.4	1.4	0.0
High Flow Event	2.8	3.0	0.2	1.0	1.4	0.4	2.5	2.6	0.1
Extreme Flow Event	5.9	5.9	0.0	4.7	4.6	-0.1	5.4	5.3	-0.1
* Difference is computed as with-project minus without-project modeled current speeds									

Table 3-7: Modeled Current Speed During Hydrodynamic Conditions at Model Comparison Points

Modeled Current Speed [ft/s]									
Hydrodynamic Condition	Point 1			Point 2			Point 3		
	Without-project	With-project	Difference*	Without-project	With-project	Difference*	Without-project	With-project	Difference*
Peak Low Tide	-	0.0	-	0.0	-	-	-	0.0	-
Peak High Tide	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.0
Intermediate Flow Event	0.0	0.0	0.0	0.1	0.0	-0.1	1.0	0.8	-0.2
High Flow Event	0.2	0.1	-0.1	0.4	0.3	-0.1	1.2	0.5	-0.7
Extreme Flow Event	0.1	0.2	0.1	0.1	0.4	0.3	0.1	0.2	0.1
Hydrodynamic Condition	Point 4			Point 5			Point 6		
	Without-project	With-project	Difference*	Without-project	With-project	Difference*	Without-project	With-project	Difference*
Peak Low Tide	--	0.0	--	0.0	0.0	0.0	--	--	--
Peak High Tide	0.1	0.1	0.0	0.1	0.1	0.0	0.1	0.2	0.1
Intermediate Flow Event	1.0	1.6	0.6	1.5	1.0	-0.5	2.3	3.1	0.8
High Flow Event	0.6	0.5	-0.1	2.8	2.1	-0.7	4.0	3.9	-0.1
Extreme Flow Event	0.2	0.3	0.1	2.5	1.8	-0.7	5.1	4.8	-0.3
Hydrodynamic Condition	Point 7			Point 8			Point 9		
	Without-project	With-project	Difference*	Without-project	With-project	Difference*	Without-project	With-project	Difference*
Peak Low Tide	-	-	-	-	-	-	0.7	0.6	
Peak High Tide	0.2	0.2	0.0	0.2	0.2	0.0	0.3	0.3	0.0
Intermediate Flow Event	4.8	3.8	-1.0	2.9	3.0	0.1	5.5	5.4	-0.1
High Flow Event	7.9	6.8	-1.1	8.5	7.0	-1.5	8.8	8.7	-0.1
Extreme Flow Event	9.0	8.3	-0.7	10.7	10.8	0.1	10.1	10.2	0.1
* Difference is computed as with-project minus without-project modeled current speeds									

Table 3-8: Modeled Current Direction During Hydrodynamic Conditions at Model Comparison Points

Modeled Current Direction [degN]									
Hydrodynamic Condition	Point 1			Point 2			Point 3		
	Without-project	With-project	Difference*	Without-project	With-project	Difference*	Without-project	With-project	Difference*
Peak Low Tide	-	96	-	51	-	-	-	98	-
Peak High Tide	152	165	13	180	171	-9	18	21	3
Intermediate Flow Event	17	62	45	345	159	-186	12	19	7
High Flow Event	200	219	19	157	162	5	48	53	5
Extreme Flow Event	274	221	-53	350	167	-183	54	46	-8
Hydrodynamic Condition	Point 4			Point 5			Point 6		
	Without-project	With-project	Difference*	Without-project	With-project	Difference*	Without-project	With-project	Difference*
Peak Low Tide	-	227	-	114	346	232	-	-	-
Peak High Tide	74	58	-16	25	352	327	329	318	-11
Intermediate Flow Event	85	60	-25	29	356	327	316	306	-10
High Flow Event	72	50	-22	38	24	-14	332	331	-1
Extreme Flow Event	35	18	-17	34	23	-11	359	356	-3
Hydrodynamic Condition	Point 7			Point 8			Point 9		
	Without-project	With-project	Difference*	Without-project	With-project	Difference*	Without-project	With-project	Difference*
Peak Low Tide	-	-	-	-	-	-	39	40	1
Peak High Tide	3	358	355	22	12	-10	38	38	0
Intermediate Flow Event	355	341	-14	24	28	4	36	36	0
High Flow Event	8	4	-4	358	353	-5	30	30	0
Extreme Flow Event	16	14	-2	10	5	-5	28	28	0
* Difference is computed as with-project minus without-project modeled current speeds									

4.0 SUMMARY

The HD model was validated to measured water level data from NOAA CO-OPS Station 8574680 (Ft. McHenry) and to data measured by M&N near the project site using an Aanderaa RCM-9 current meter. Following validation, production simulations of existing (i.e., without-project) and with-project conditions were conducted to evaluate the changes in the local hydrodynamics. The production simulations covered the time period May 25, 2018 to June 27, 2018 to capture a spring-neap tidal cycle and to also capture a high (10-year return period) and intermediate (2-yr return period) flow event measured at the Patapsco River and Gwynns Falls. Additional simulations included a 100-year return period flow event for both existing and with-project conditions, and a sensitivity analysis assessing an elevated tail water for the with-project conditions.

The HD model was evaluated during six hydrodynamic conditions: 1) a representative typical high tide, 2) a representative typical low tide, 3) an intermediate (2-year RP) flow event, 4) a high flow event (10-year RP), 5) an extreme flow event (100-year RP) and 6) with project high tail water (+2 ft) flow event. The interior wetlands became inundated during high tide events where the water level is above approximately +0.53 ft NAVD88. As water level increases above MHW, the high marsh elevations become wet and are inundated at approximately +0.96 ft NAVD88.

The simulations show an alteration in the flow field where water is directed into the proposed wetlands from the Patapsco River, generating a current flowing from south to north. The majority of water circulation within the project geometry occurs near the mouth of the proposed wetlands, adjacent to the Patapsco River. The current flow field with and without the project geometry during the evaluated hydrodynamic conditions are shown in Appendix A.

Time series of the modeled hydrodynamics at nine points within the proposed wetlands are shown in Appendix B and reported in Table 3-6 – Table 3-8 during the six evaluated hydrodynamic conditions. During low tide and high tide, the maximum modeled current speed was 0.9 ft/s at Point 9, located within the Patapsco River. The remaining points within the project geometry do not show current speeds exceeding 0.3 ft/sec during low and high tide events. Modeled current speeds at higher flow events range from 0.0 to 10.0 ft/sec, with the greatest current speeds observed along the Patapsco River (Points 7, 8, and 9). This range of current speeds indicates a potential for sediment movement within the project footprint during elevated flow events. The greatest changes between the existing geometry and propose wetlands in current speed were observed within the open water channels near the entrance and northern high marsh regions, which be attributed to increased flow area where water cannot currently flow during typical tidal conditions.

The modeled current speeds at Points 7 and 8 are adjacent to the location of the proposed gravel shoreline and should be used to determine the appropriate gravel/stone size for a stable edge using respected standards or formulations (e.g., Coastal Engineering Manual). The modeled current speeds during the extreme flow event (8.3 – 10.9 ft/s) may provide a

good basis for sizing the gravel as this would ensure stability during a 100-yr flow event that could occur during the project life.

The modeled water levels are shown to decrease within the interior channels during low flow events due to draining and tidal cycling through the wetlands. A decrease of -1.7 ft was observed at Point 5, which is located within a ponding region of the existing geometry. With the exception of Point 5, water levels did not vary by more than 0.5 ft between the existing and with project simulations.

5.0 CONCLUSIONS

A two-dimensional regional hydrodynamic (HD) model was developed to model the local hydrodynamics to the incorporation of the EB-1 Project. Key conclusions of the project include:

- Along the Patapsco River and proposed shoreline, peak velocities with the project geometry range between 0.0 and 0.6 ft/sec during typical low and high tides (Points 7, 8, and 9). These current speeds can be used to assess the need for scour protection along the entrance to the proposed wetlands and gravel shoreline. During elevated flow conditions, current speeds increase to values up to 10.9 ft/sec.
- At the entrance of the two main inlets for the project, current speeds range between 0.0-0.2 ft/sec during typical low and high tides (Points 4 and 6). During elevated flow conditions, current speeds increase to between 0.3-4.8 ft/sec.
- Further within the shallow open water channels, current speeds are relatively similar to the inlet velocities during representative high and low tides (Points 3 and 5). The narrower channel (Point 5) experiences greater current speeds during elevated flow conditions.
- The more eastern and interior channel (Point 1) did not have current speeds exceeding 0.2 ft/sec for any of the hydrodynamic conditions modeled. Current speeds within the eastern interior channel were *de minimis* during typical low and high tides.
- The interior low marsh regions of the project footprint begin to accumulate water when the water level exceeds approximately +0.31 ft NAVD88 and become completely inundated when the water level increases to approximately +0.53 ft NAVD88. The high marsh regions begin to accumulate water beyond this point, and eventually become inundated at approximately +0.96 ft NAVD88. The interior project footprint drains into the Patapsco River with the falling tide; however, persistent water remains within the shallow open water channels.
- Limitations of the study include, but are not limited to:
 - The model does not take into consideration evaporation or absorption that could occur and therefore lower the retained water.
 - The 100-year return period was modeled by scaling extreme value analysis for discharge boundary conditions, however data from a 100-year flow event was not captured during the production simulation.
 - The model does not take into consideration storm runoff and additional upland sources that may increase the amount of retained water within the project footprint.

- The model does not take into account the roughness of established wetlands plants or any structural elements (e.g., woody vents).

6.0 REFERENCES

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- [8] J. J. Williams and L. S. Esteves, "Guidance on Setup, Calibration, and Validation of Hydrodynamic, Wave, and Sediment Models for Shelf Seas and Estuaries," *Advances in Civil Engineering*, 2017.

APPENDIX A

HYDRODYNAMIC MODEL PLAN VIEWS

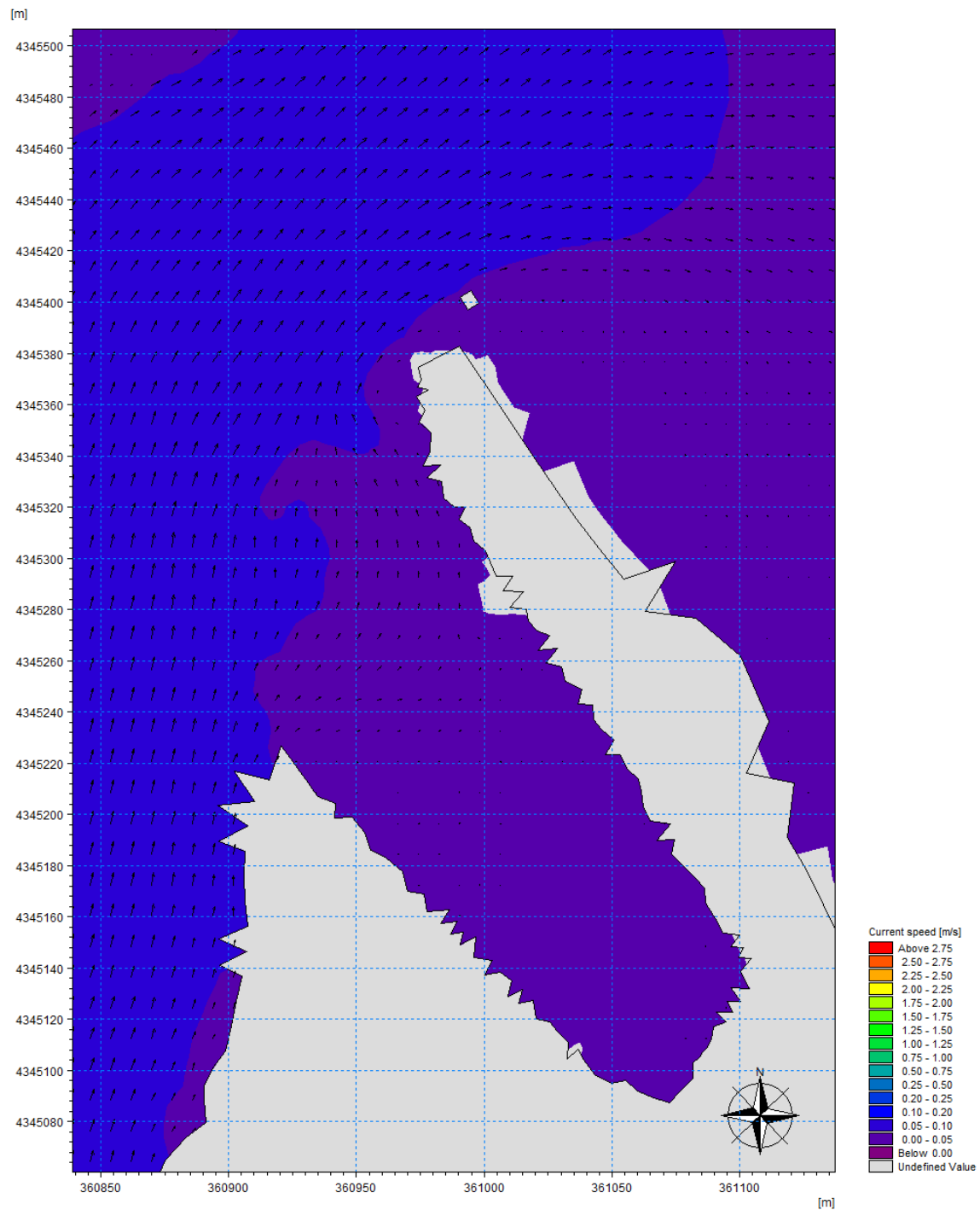


Figure A-1: Water current speed and direction during day-to-day peak high tide without the project geometry included (June 12, 2018 09:42)

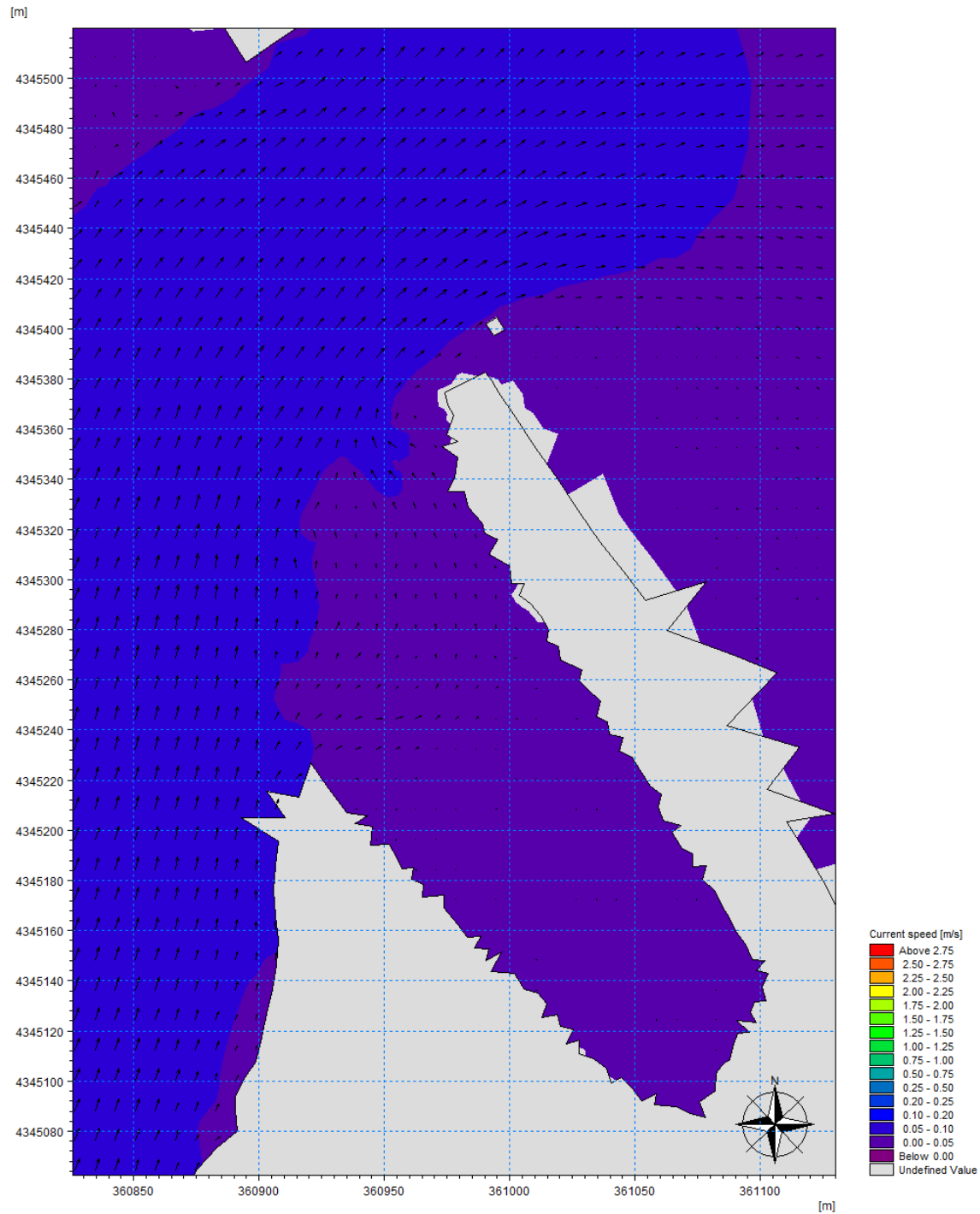


Figure A-2: Water current speed and direction during day-to-day peak high tide with the project geometry included (June 12, 2018 09:42)

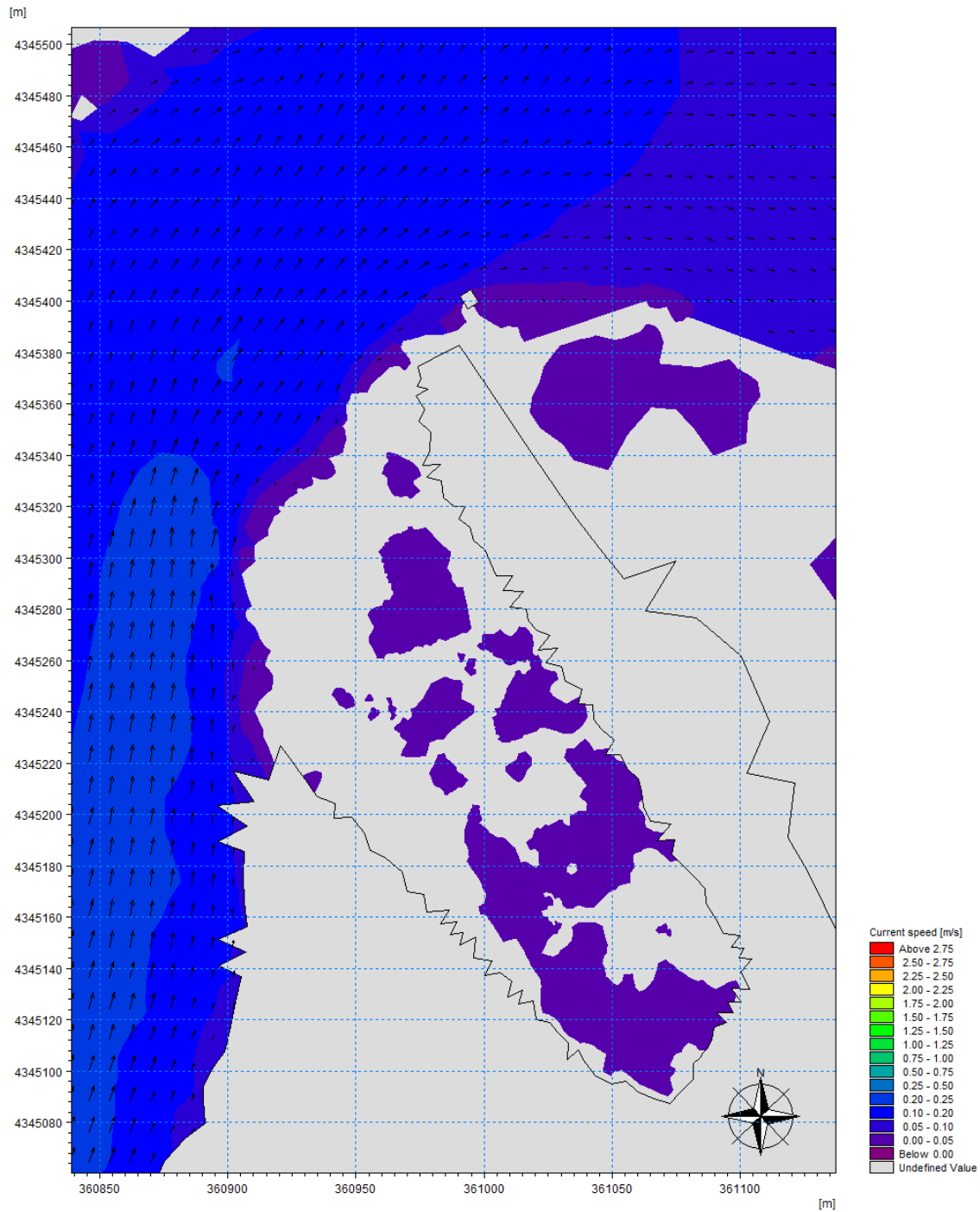


Figure A-3: Water current speed and direction during day-to-day peak low tide without the project geometry included (June 15, 2018 05:42)

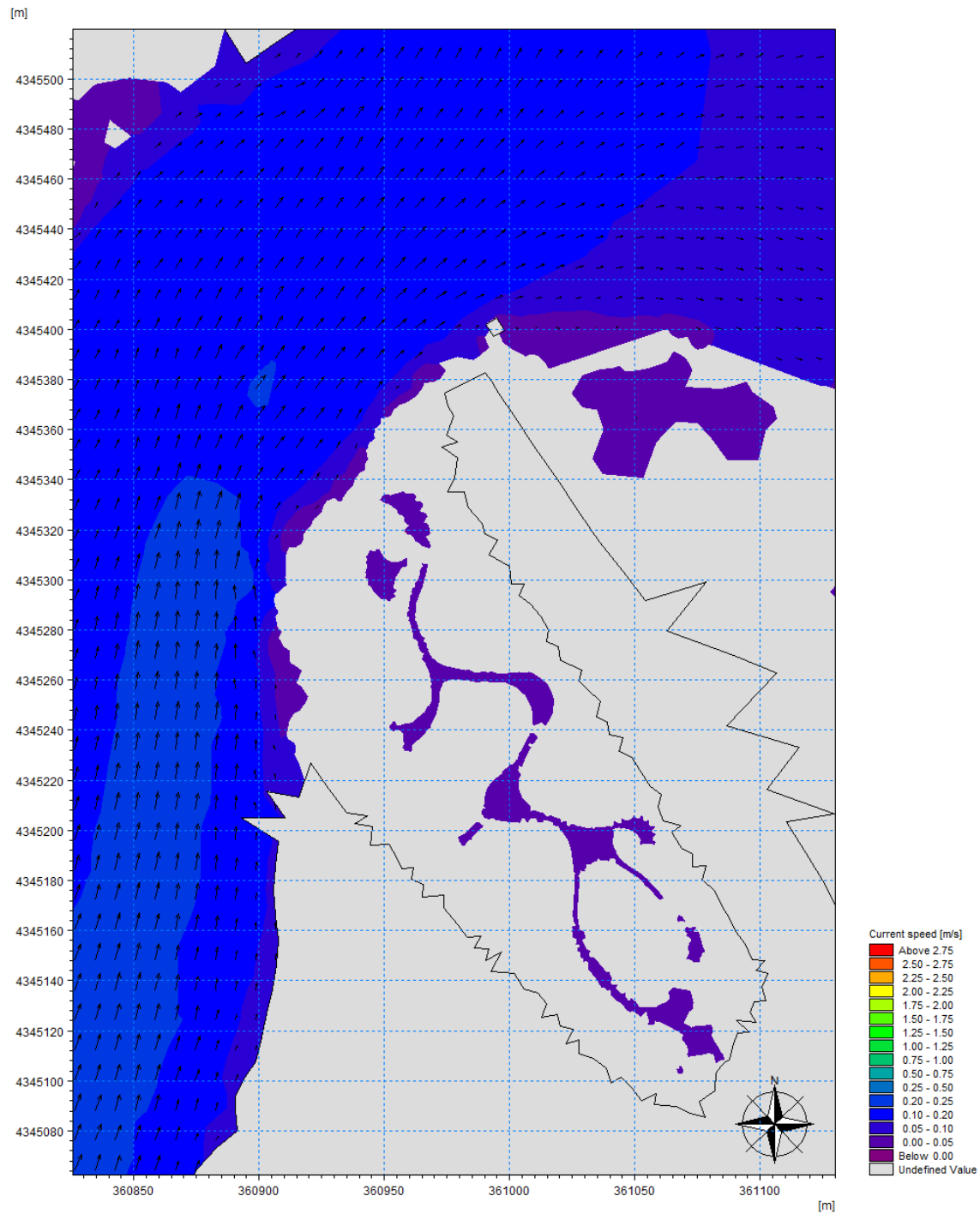


Figure A-4: Water current speed and direction during day-to-day peak low tide with the project geometry included (June 15, 2018 05:42)

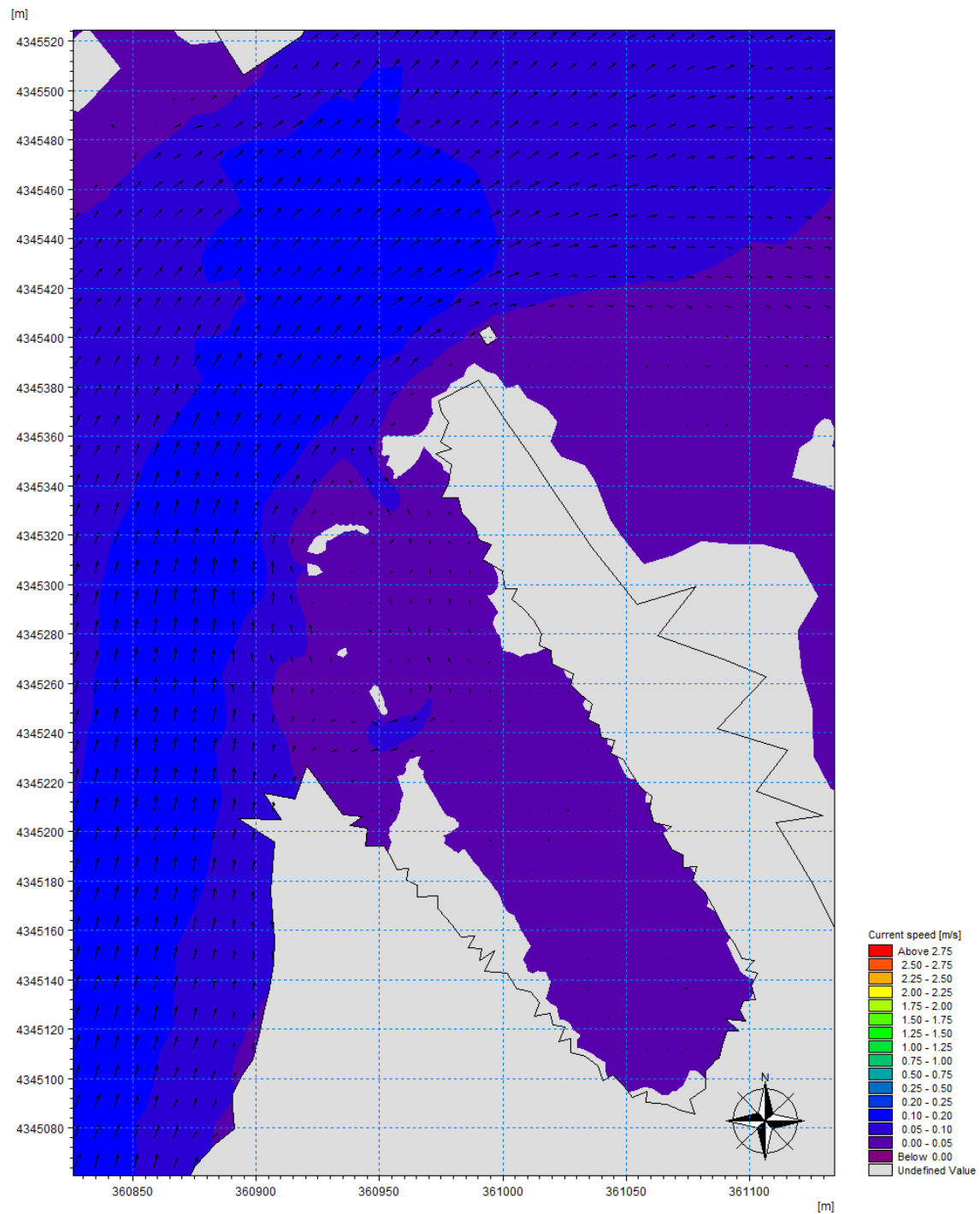


Figure A-5: Water current speed and direction during day-to-day peak low tide and elevated tail water with the project geometry included (June 15, 2018 05:42)

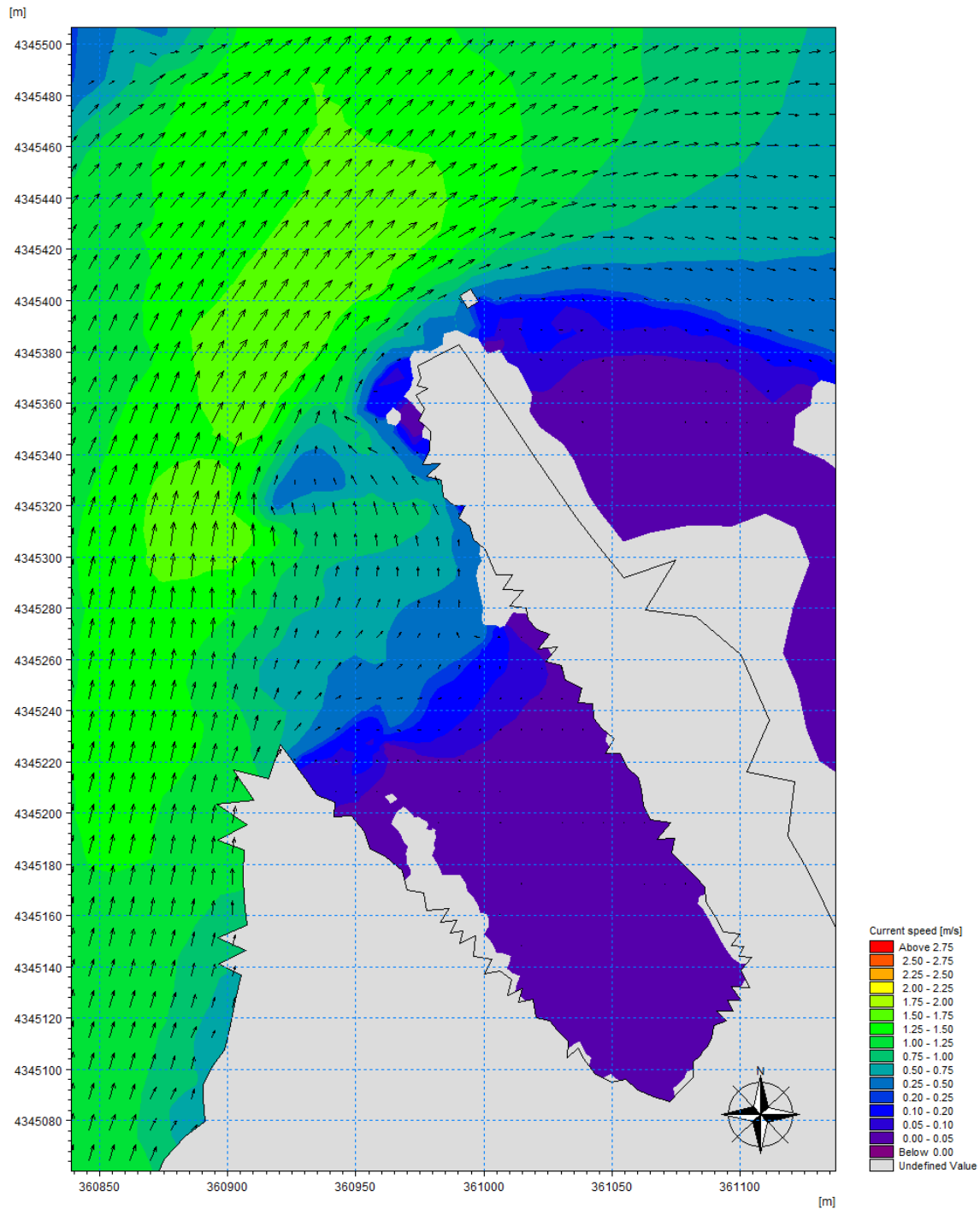


Figure A-6: Water current speed and direction during 2-year return period flow event without the project geometry included (June 4, 2018 03:24)

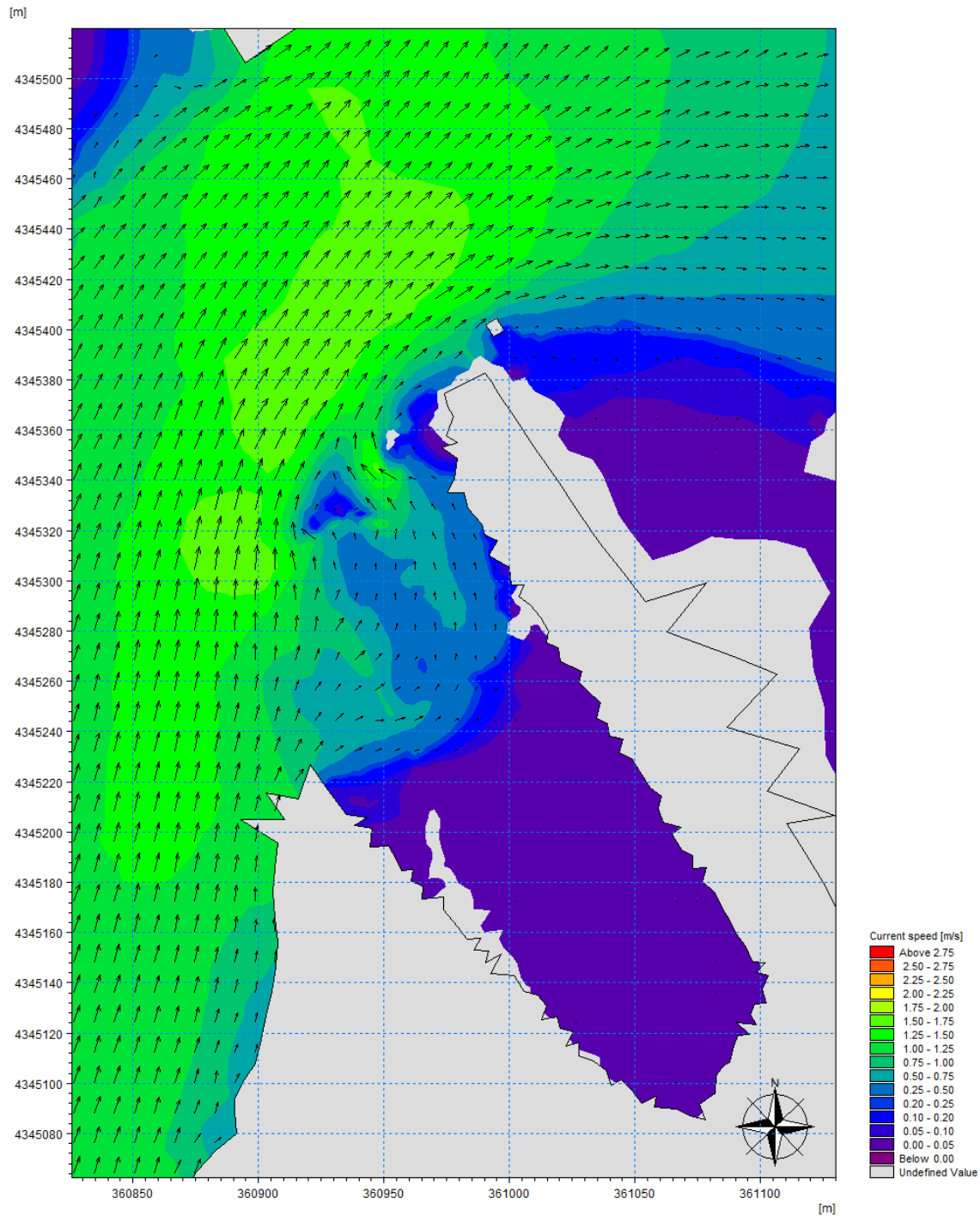


Figure A-7: Water current speed and direction during 2-year return period flow event with the project geometry included (June 4, 2018 03:24)

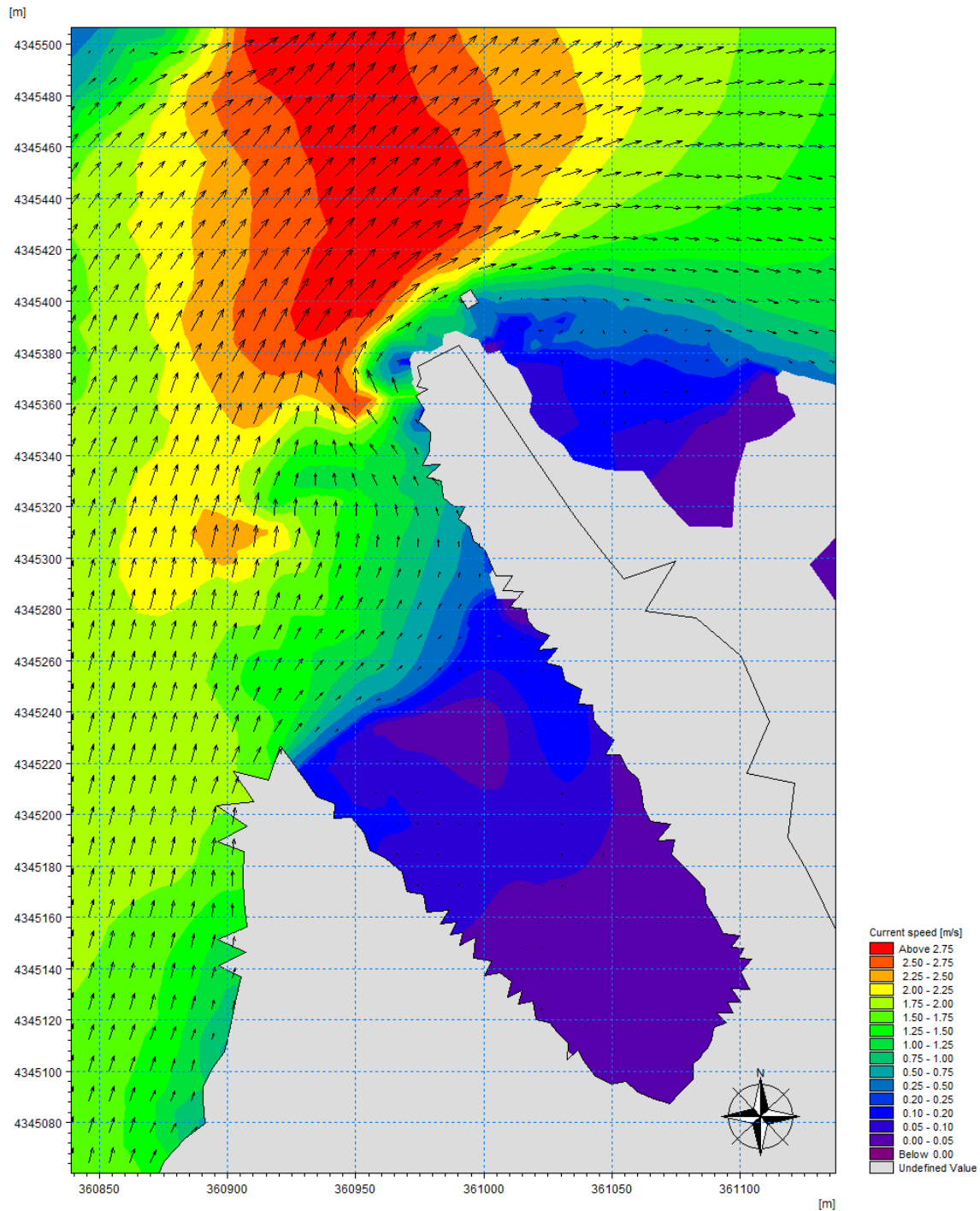


Figure A-8: Water current speed and direction during 10-year return period flow event without the project geometry included (May 5, 2018 23:54)

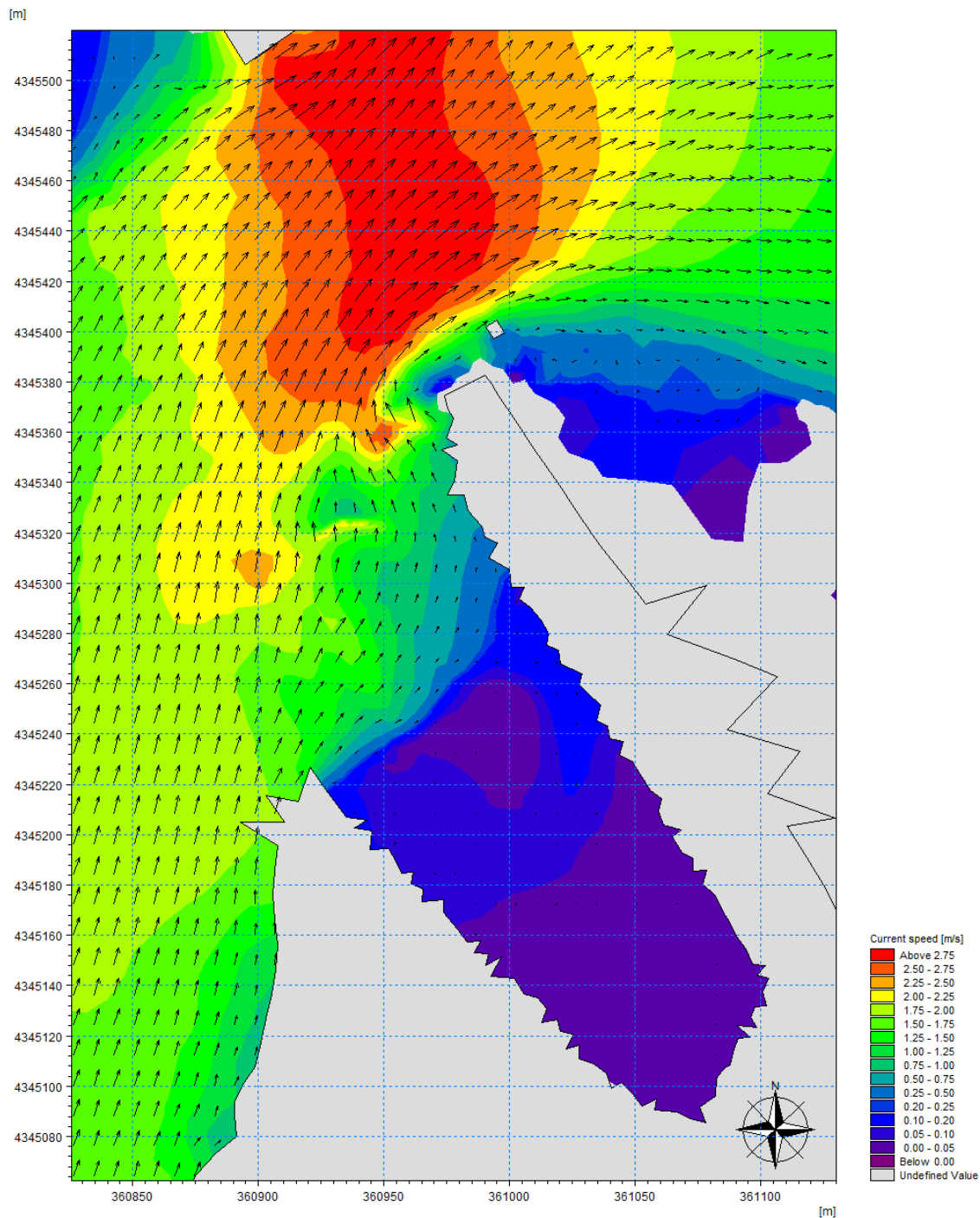


Figure A-9: Water current speed and direction during 10-year return period flow event with the project geometry included (May 5, 2018 23:54)

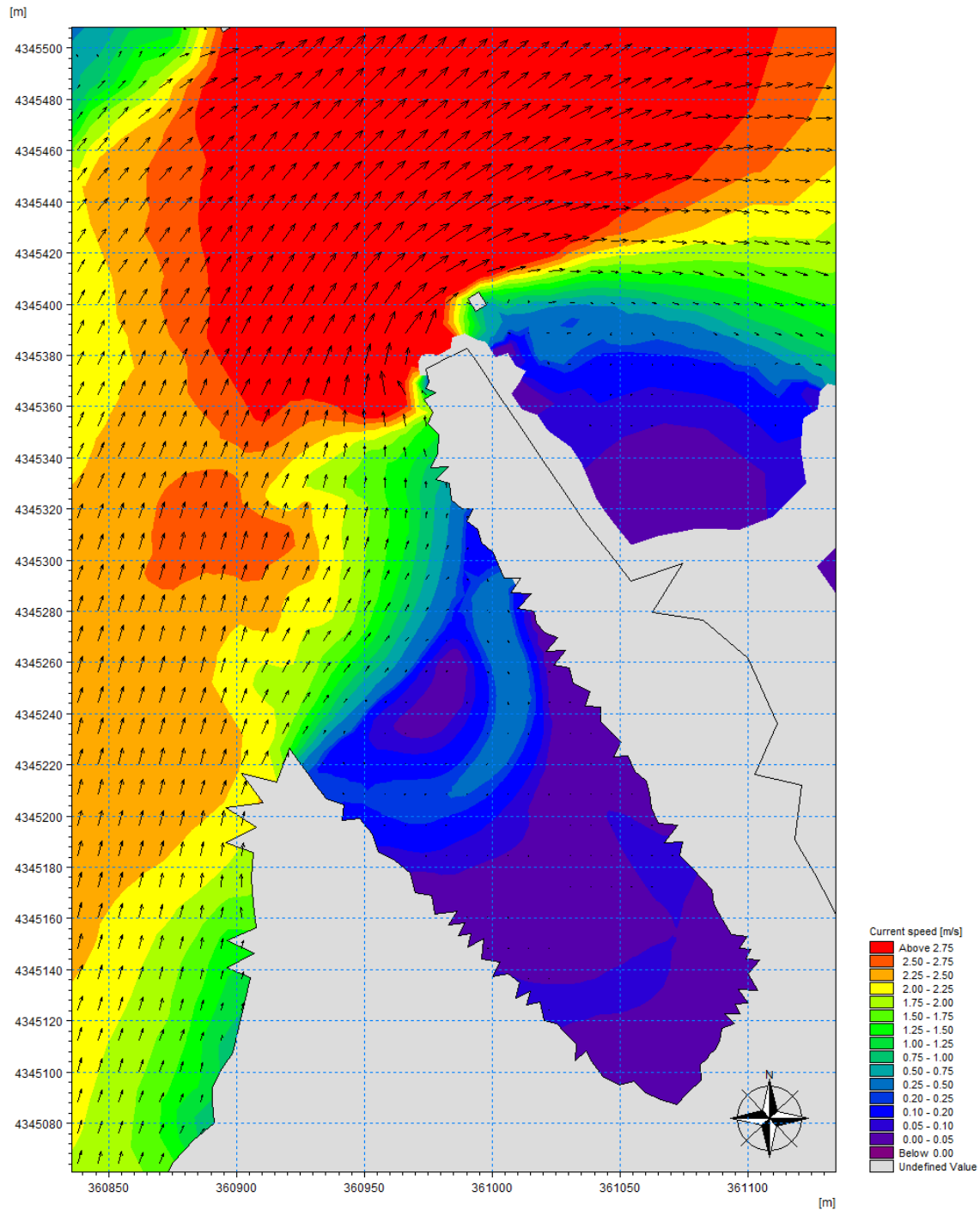


Figure A-10: Water current speed and direction during 100-year return period flow event without the project geometry included.

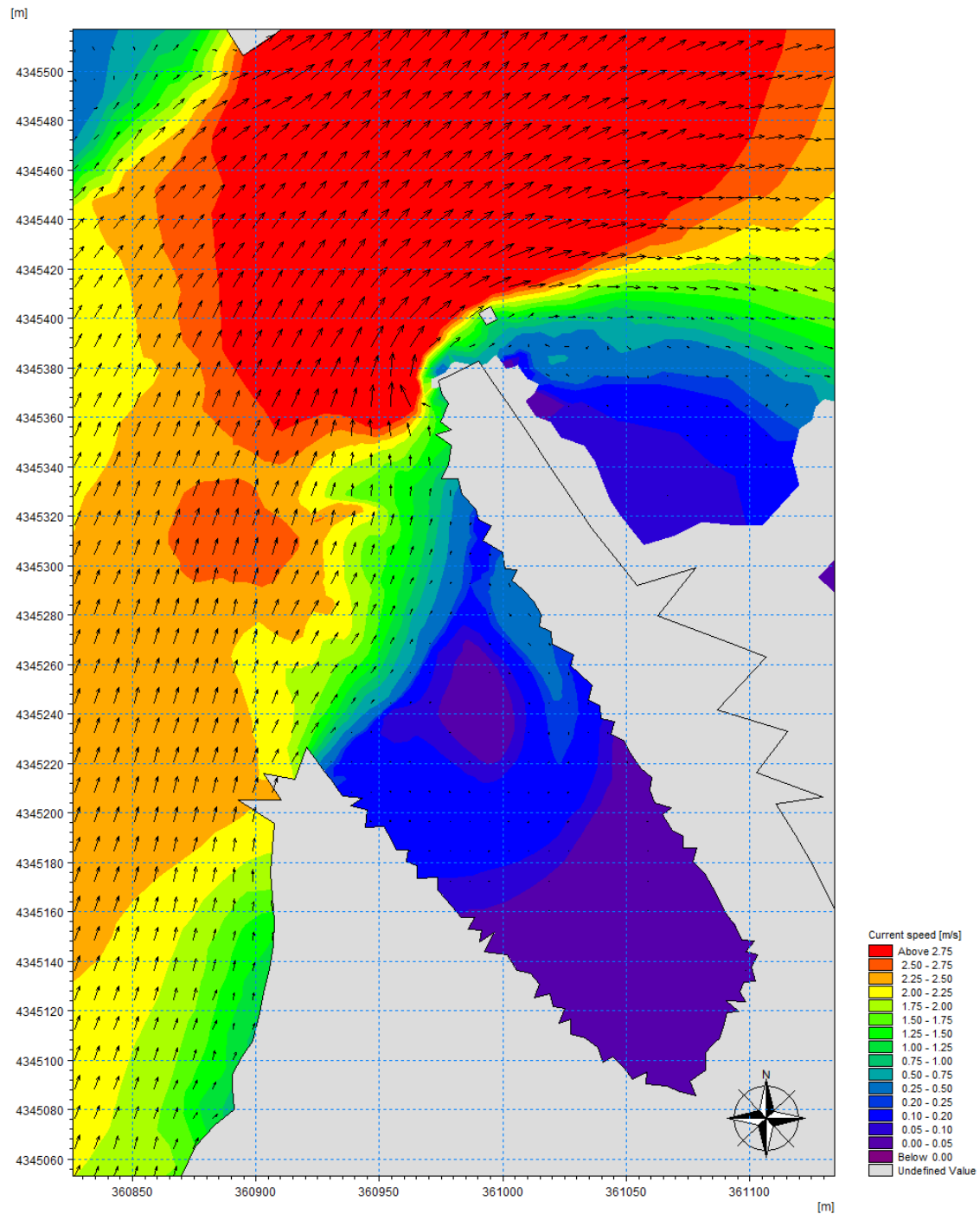


Figure A-11: Water current speed and direction during 100-year return period flow event with the project geometry included.

APPENDIX B

HYDRODYNAMIC TIME SERIES

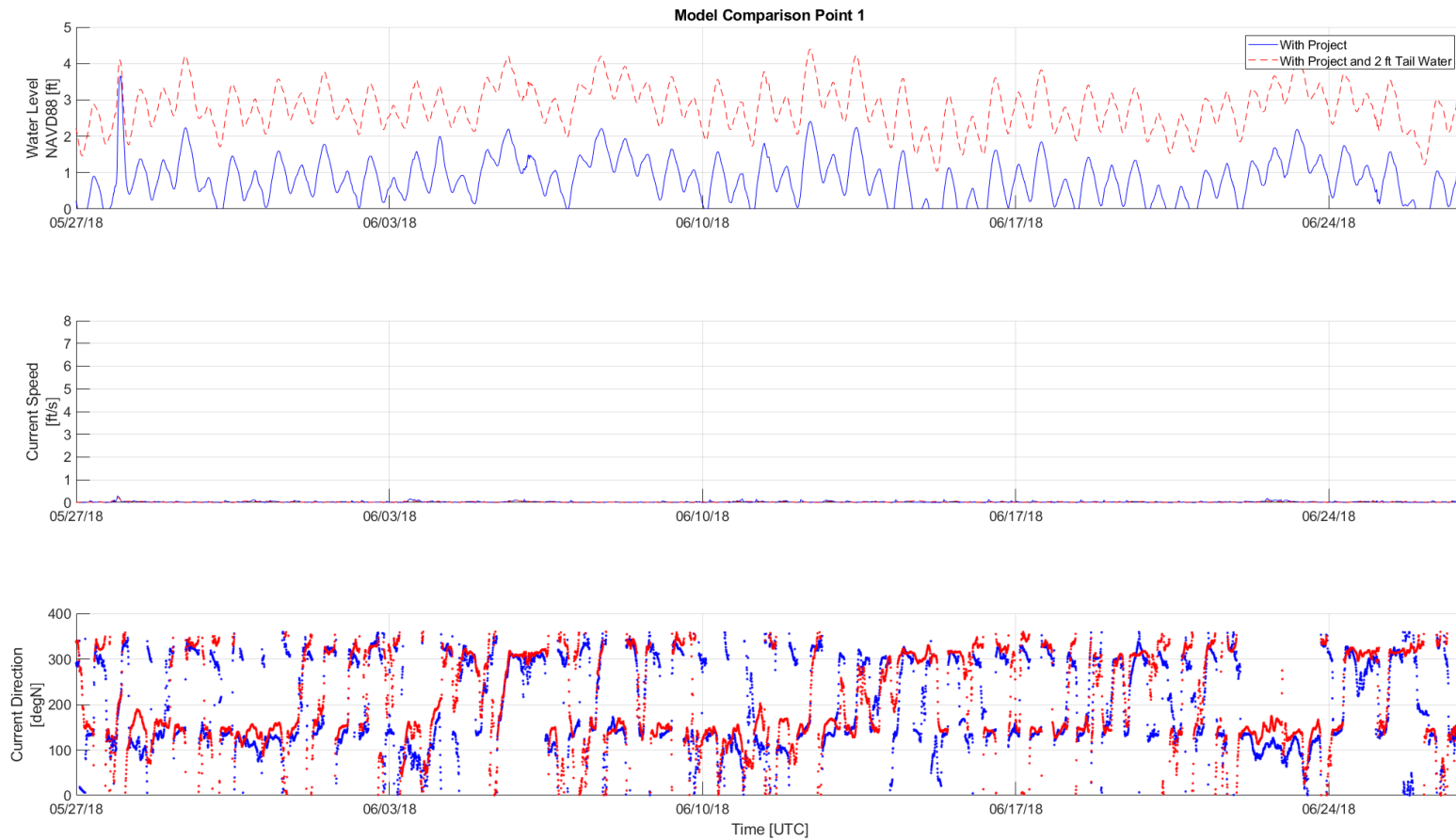


Figure B-1: Modeled with project (blue) and with project and elevated tail water (red) project hydrodynamics at model comparison point 1

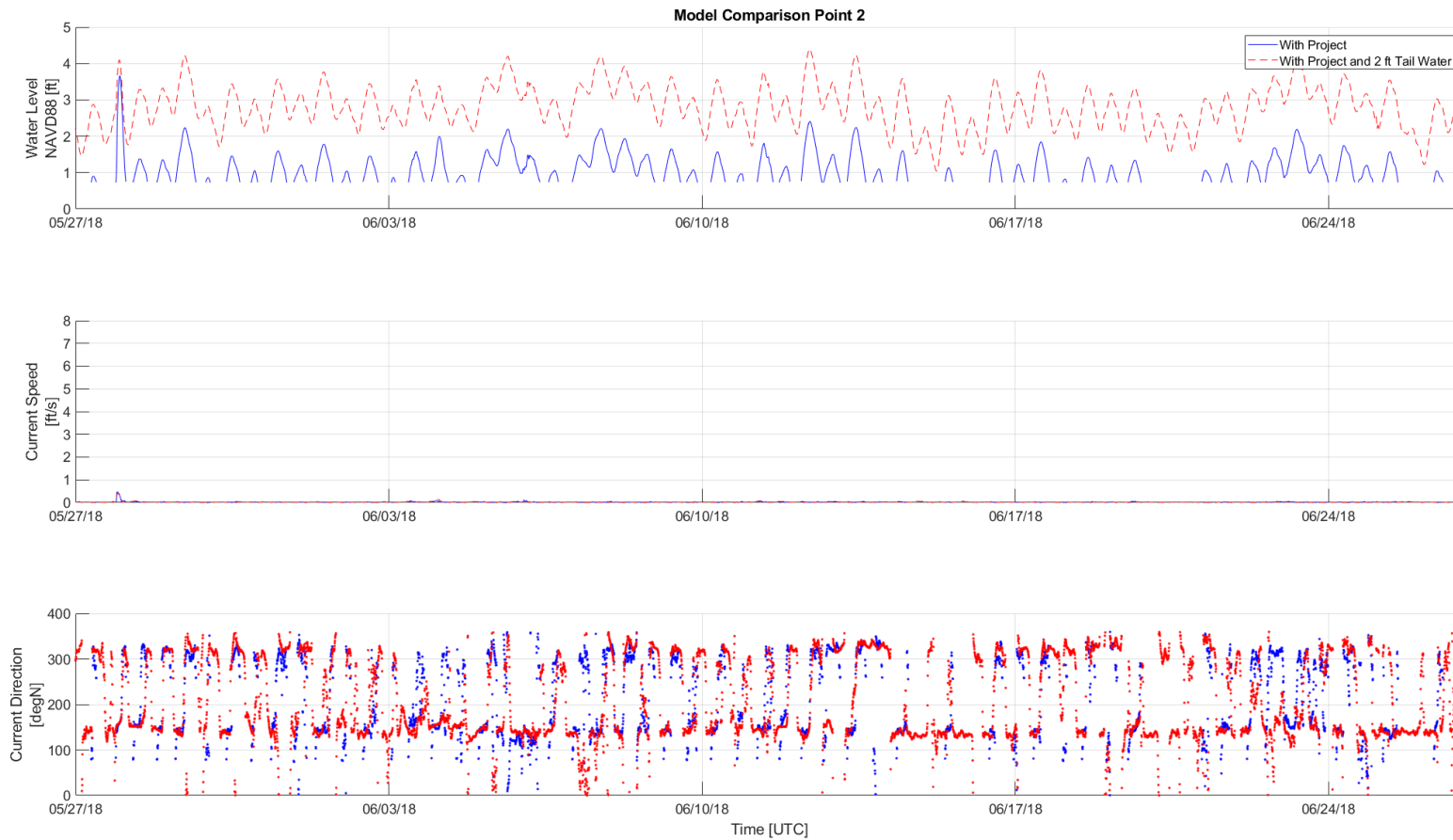


Figure B-2: Modeled with project (blue) and with project and elevated tail water (red) project hydrodynamics at model comparison point 2

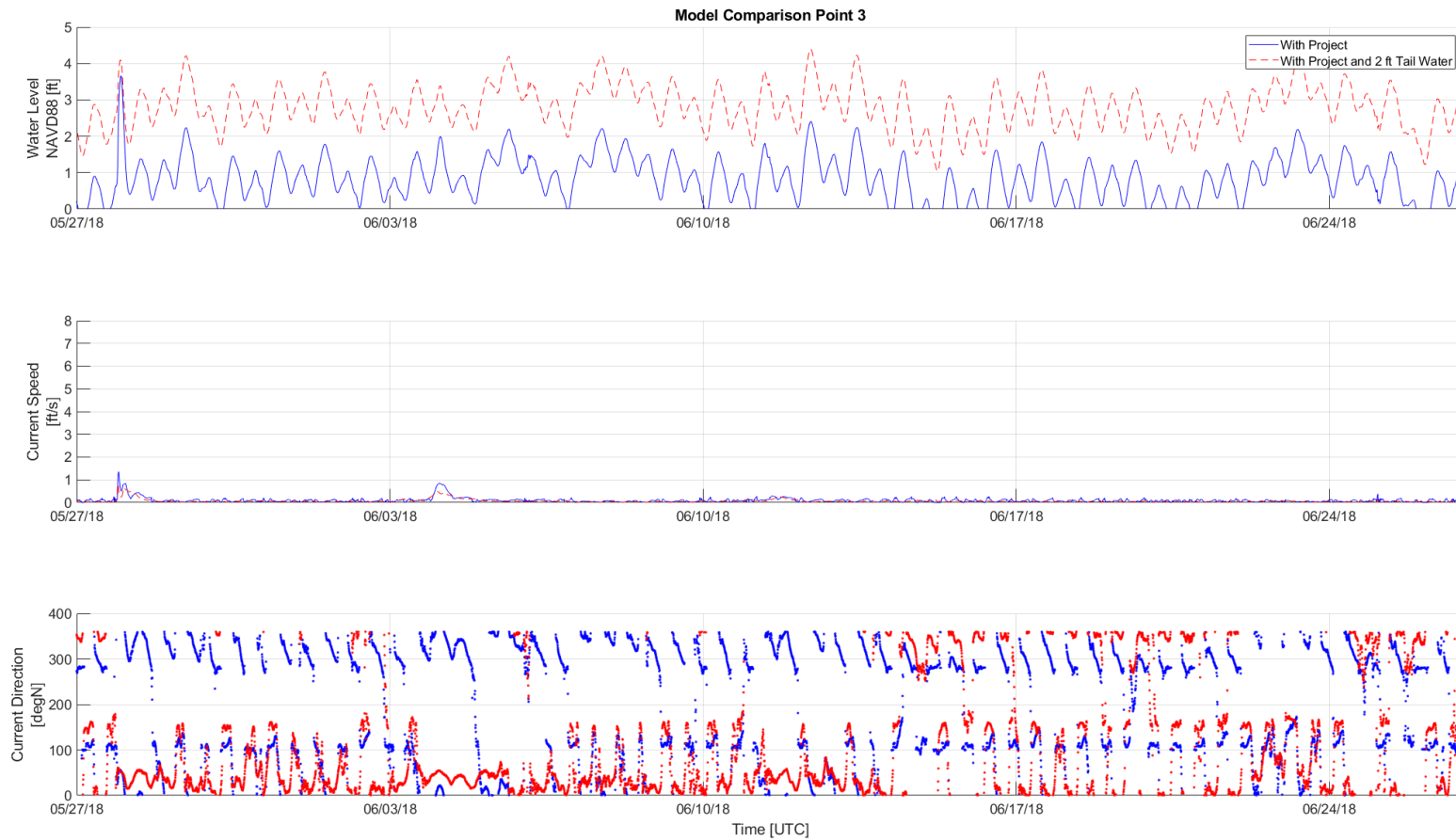


Figure B-3: Modeled with project (blue) and with project and elevated tail water (red) project hydrodynamics at model comparison point 3

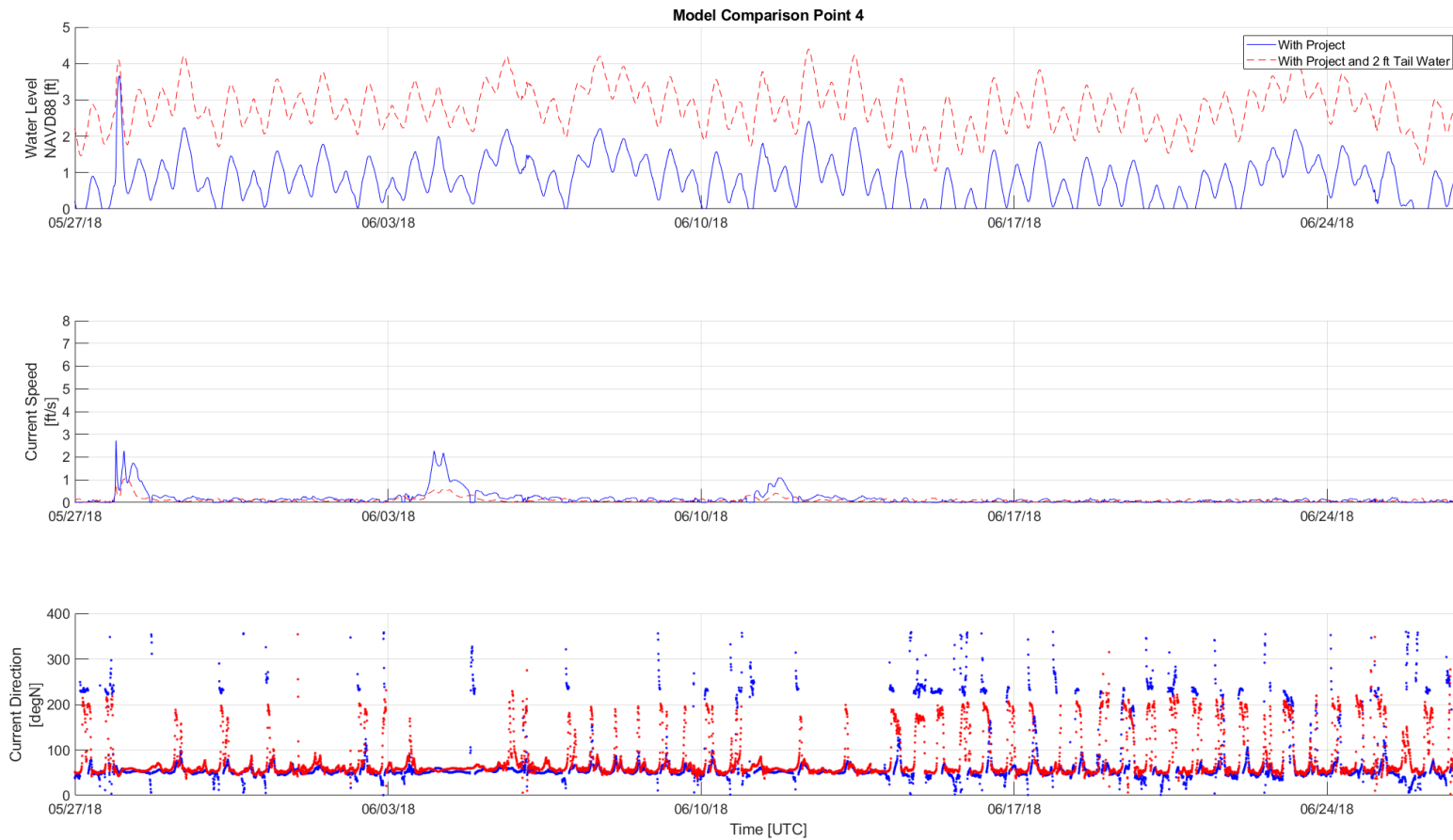


Figure B-4: Modeled with project (blue) and with project and elevated tail water (red) project hydrodynamics at model comparison point 4

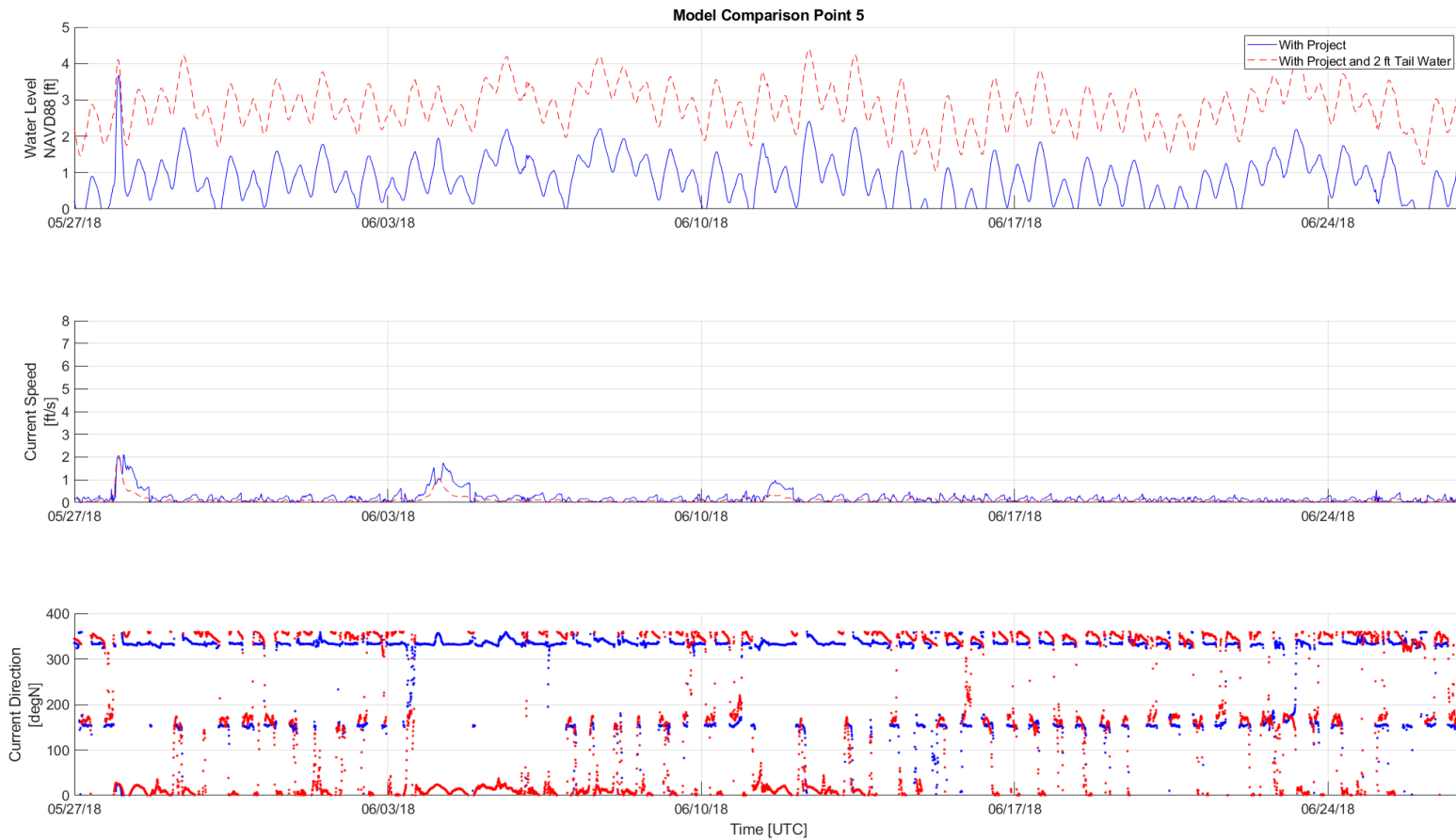


Figure B-5: Modeled with project (blue) and with project and elevated tail water (red) project hydrodynamics at model comparison point 5

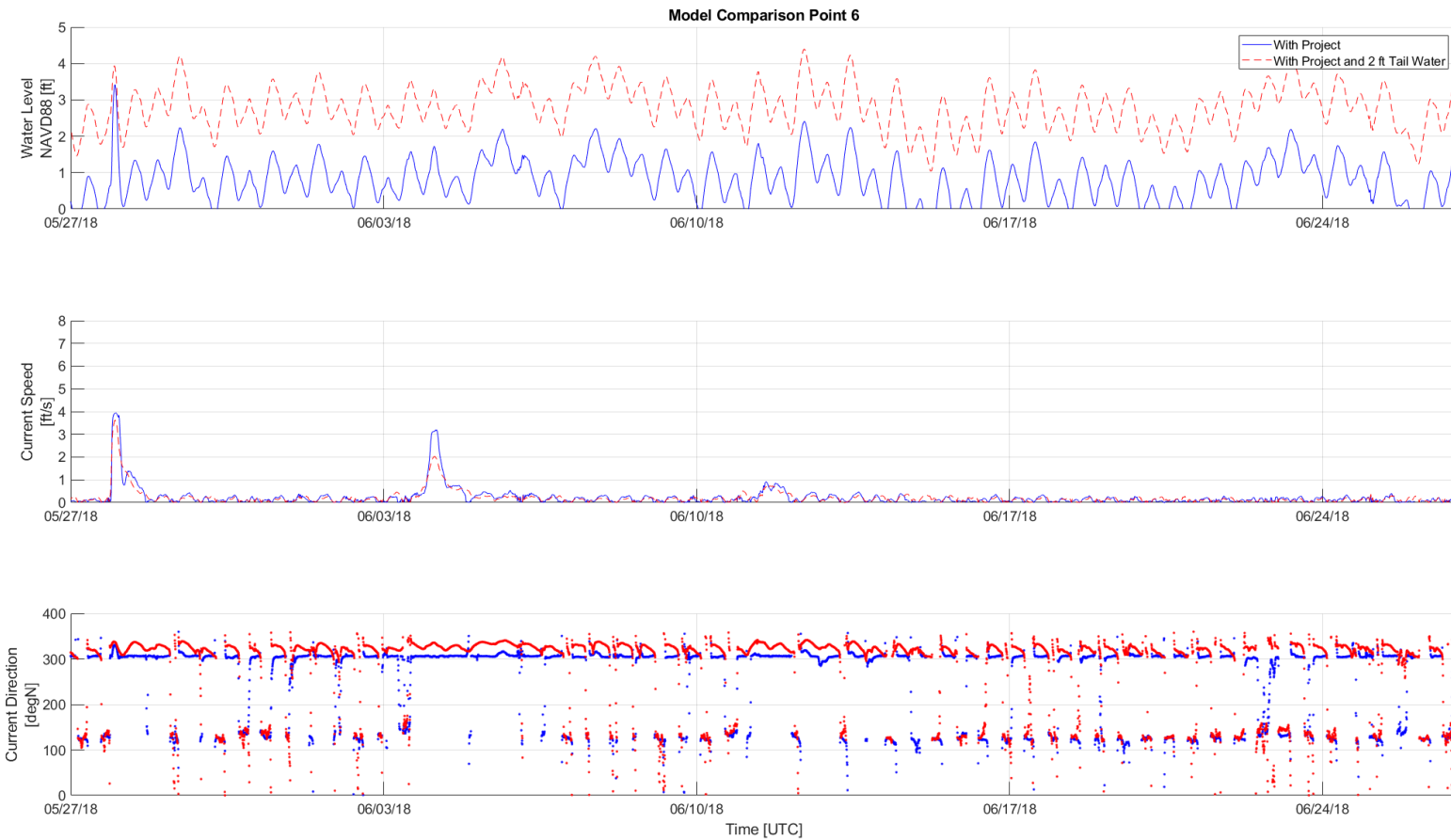


Figure B-6: Modeled with project (blue) and with project and elevated tail water (red) project hydrodynamics at model comparison point 6

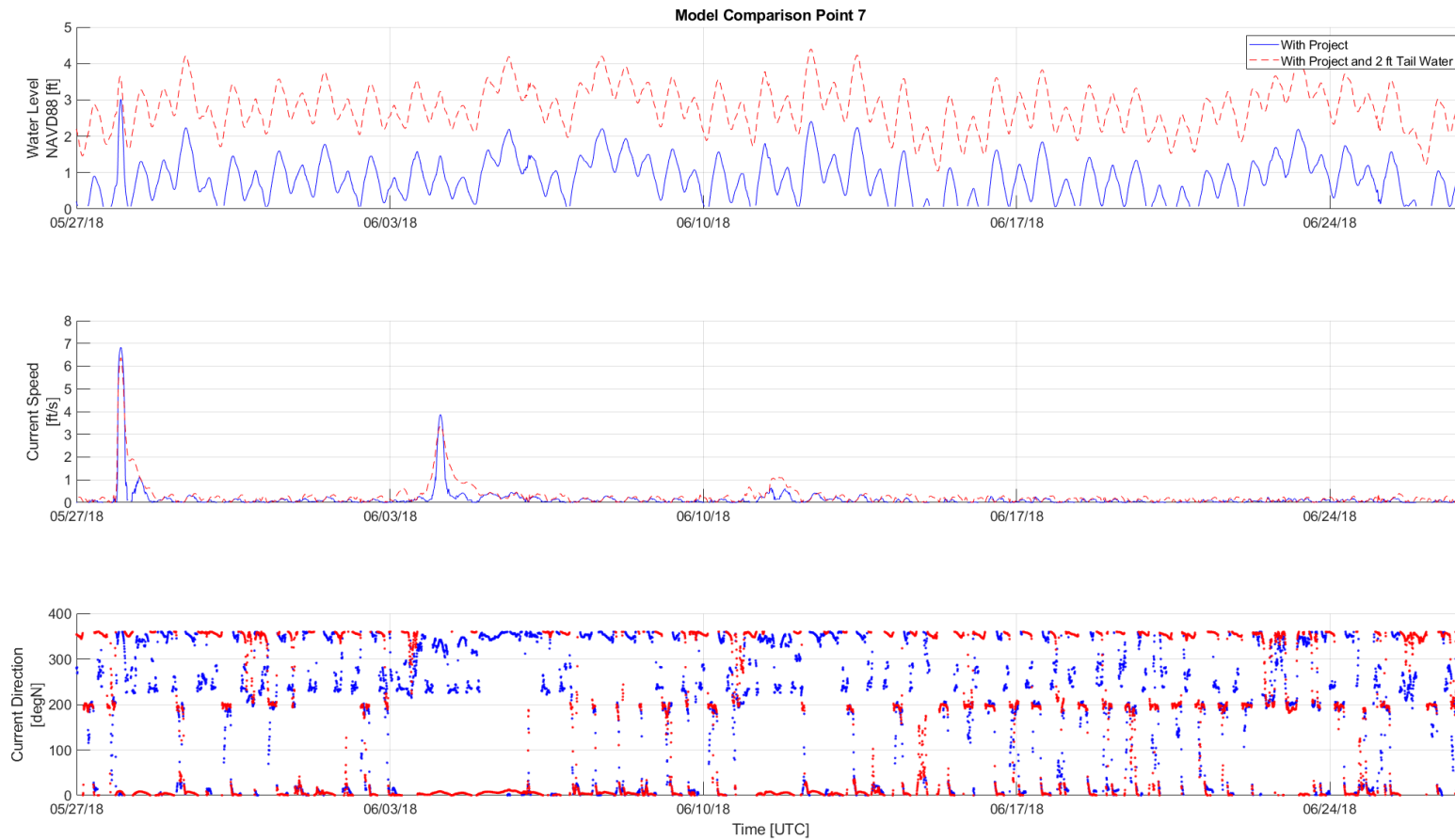


Figure B-7: Modeled with project (blue) and with project and elevated tail water (red) project hydrodynamics at model comparison point 7

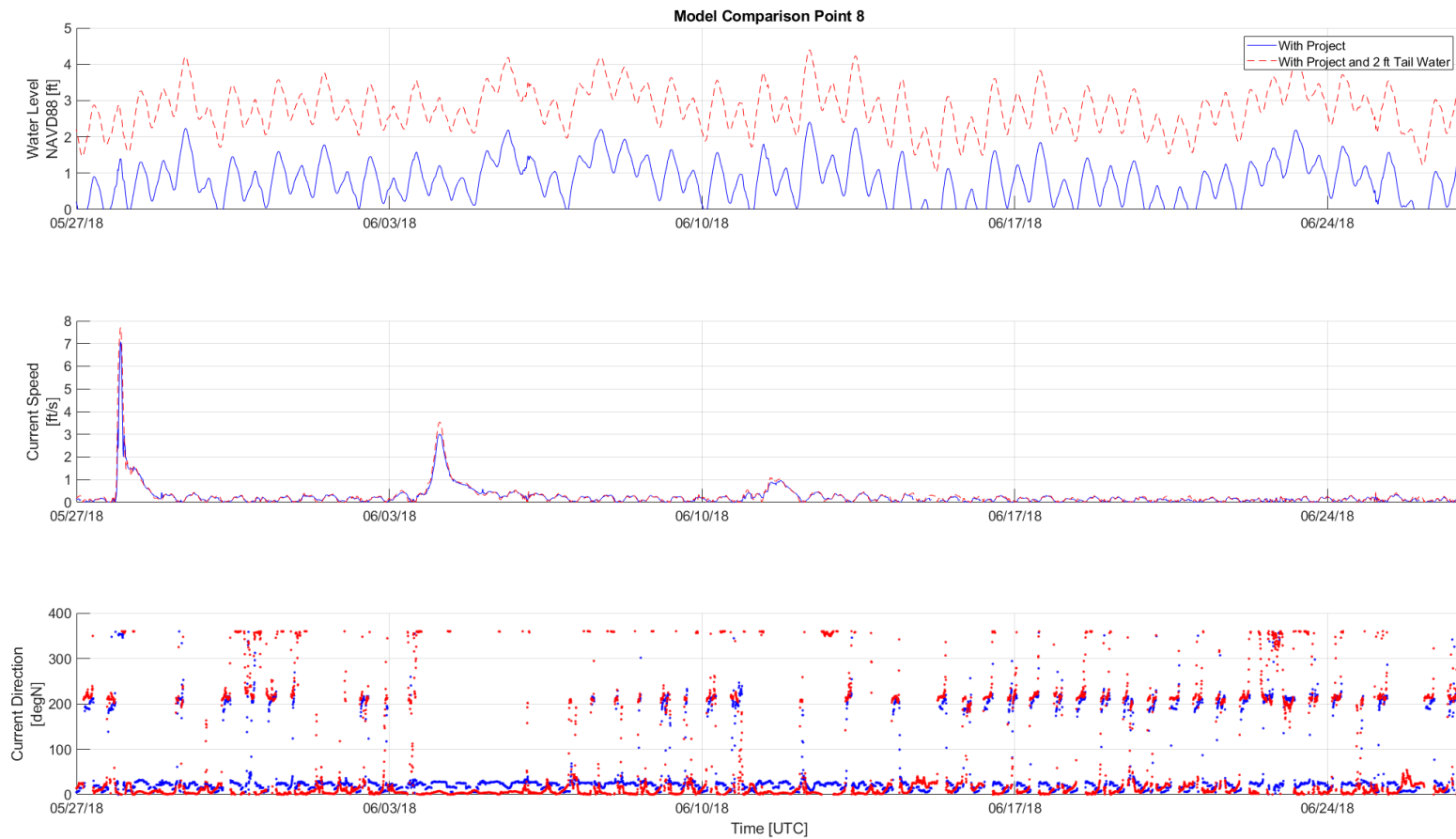


Figure B-8: Modeled with project (blue) and with project and elevated tail water (red) project hydrodynamics at model comparison point 8

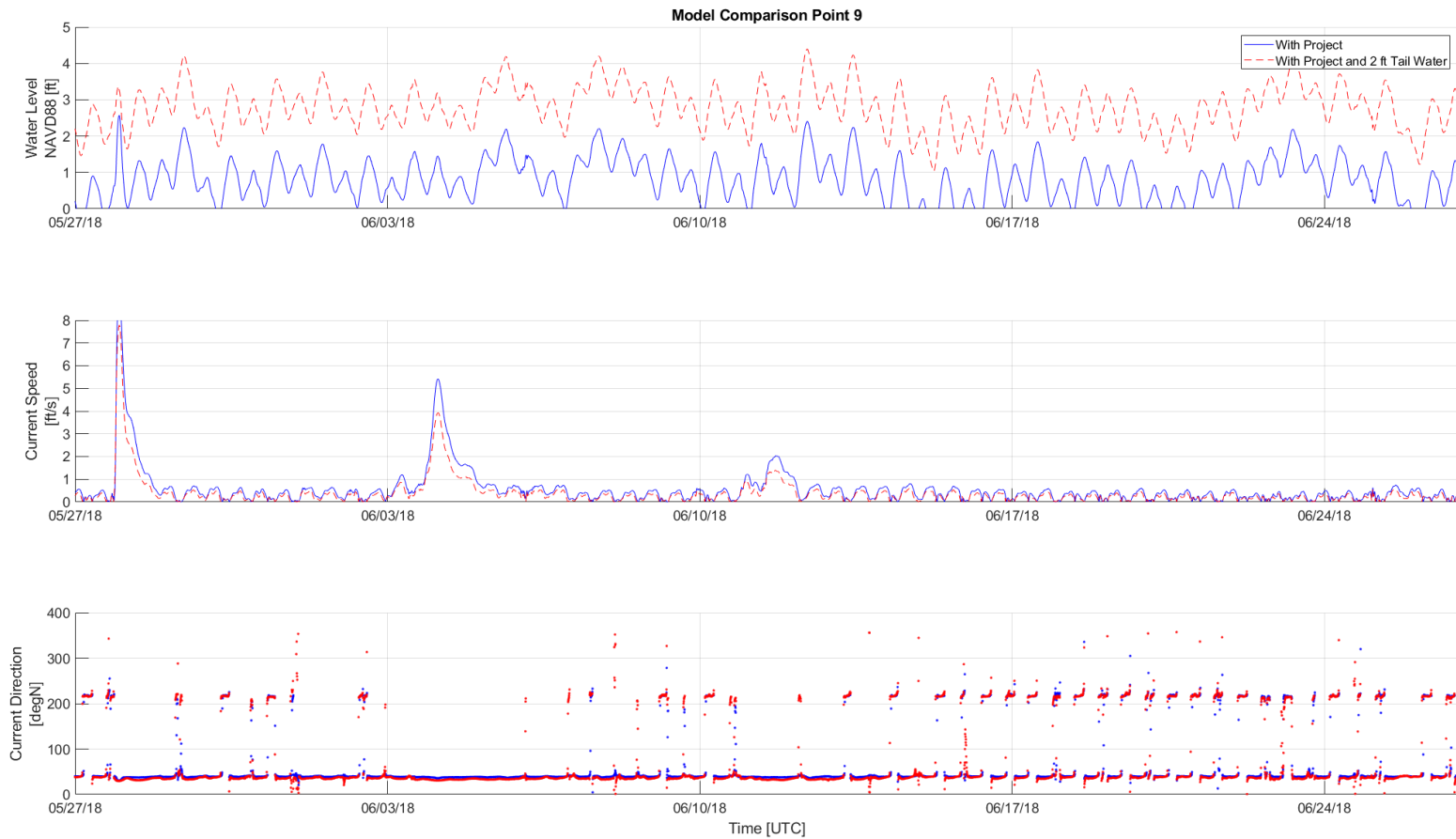


Figure B-9: Modeled with project (blue) and with project and elevated tail water (red) project hydrodynamics at model comparison point 9

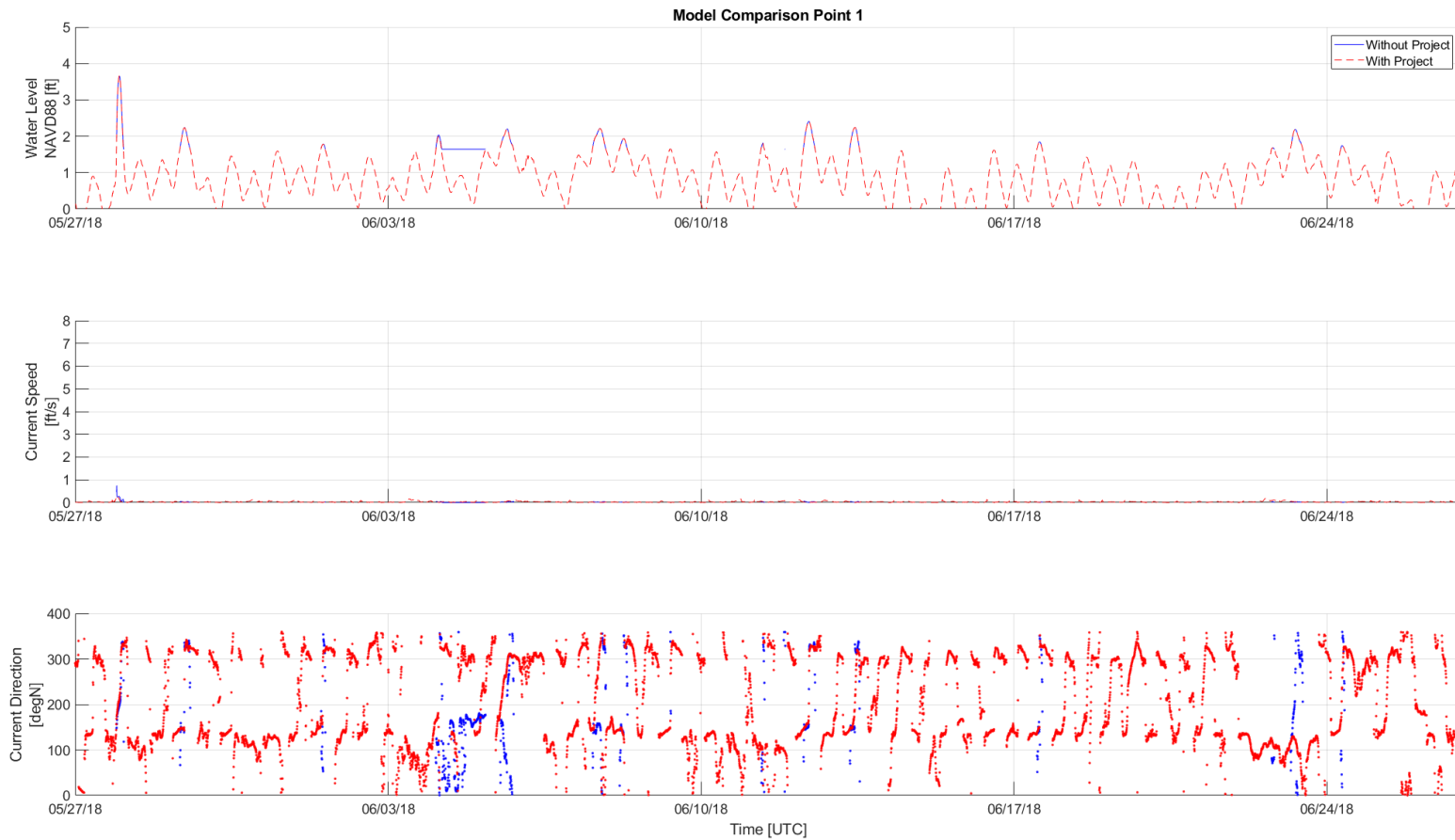


Figure B-10: Modeled with project (red) and without project (blue) project hydrodynamics at model comparison point 1

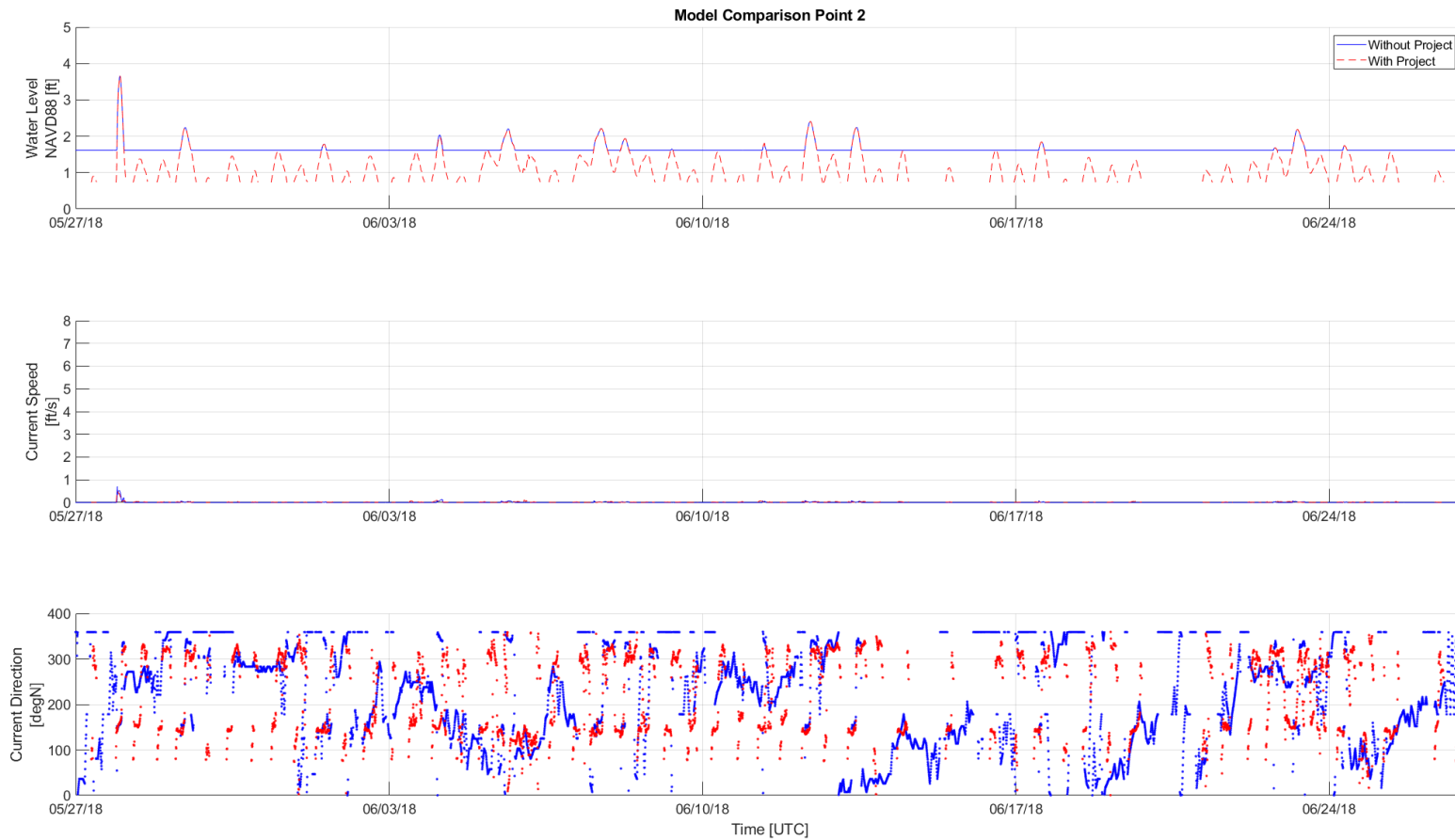


Figure B-11: Modeled with project (red) and without project (blue) project hydrodynamics at model comparison point 2

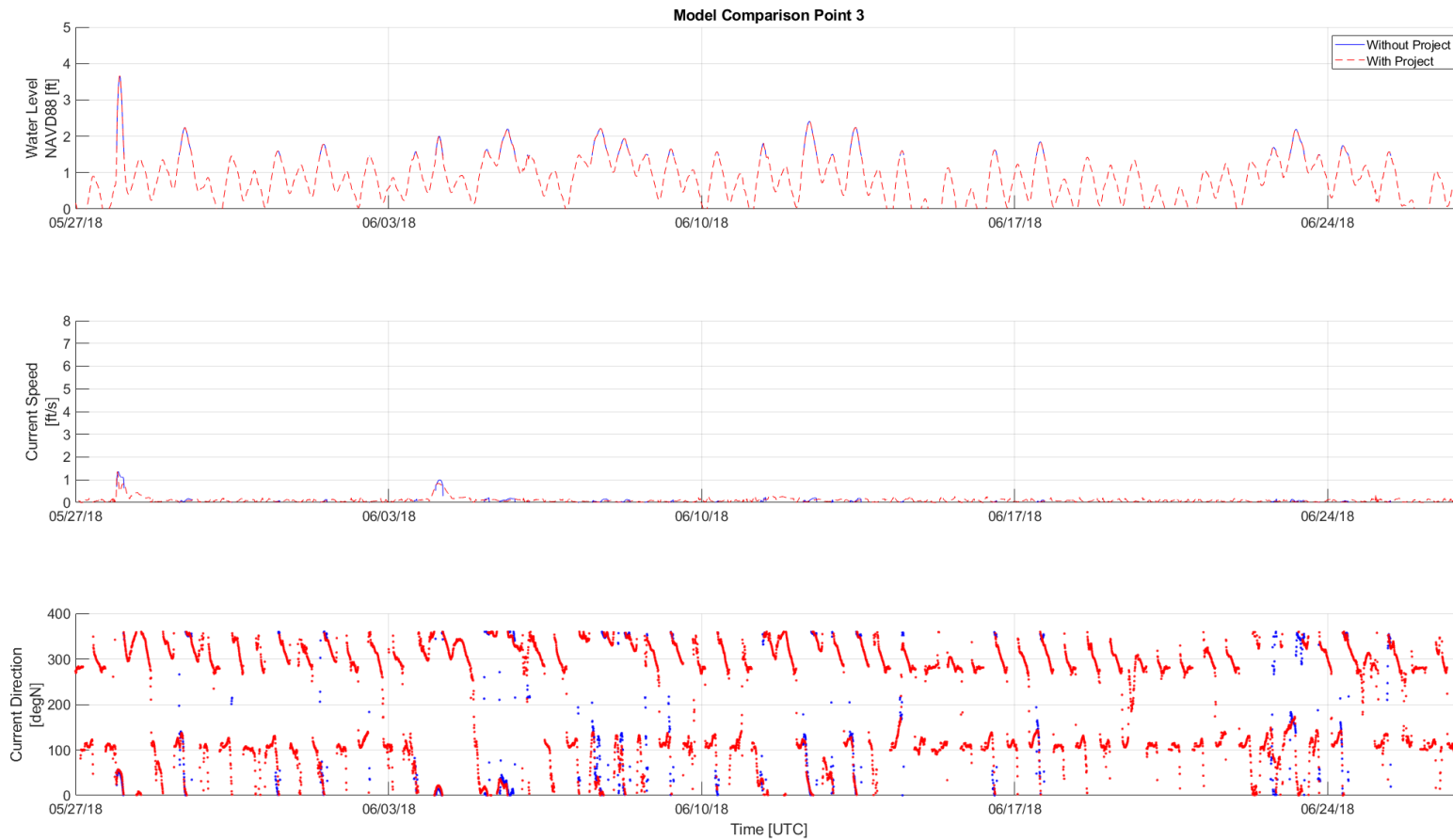


Figure B-12: Modeled with project (red) and without project (blue) project hydrodynamics at model comparison point 3

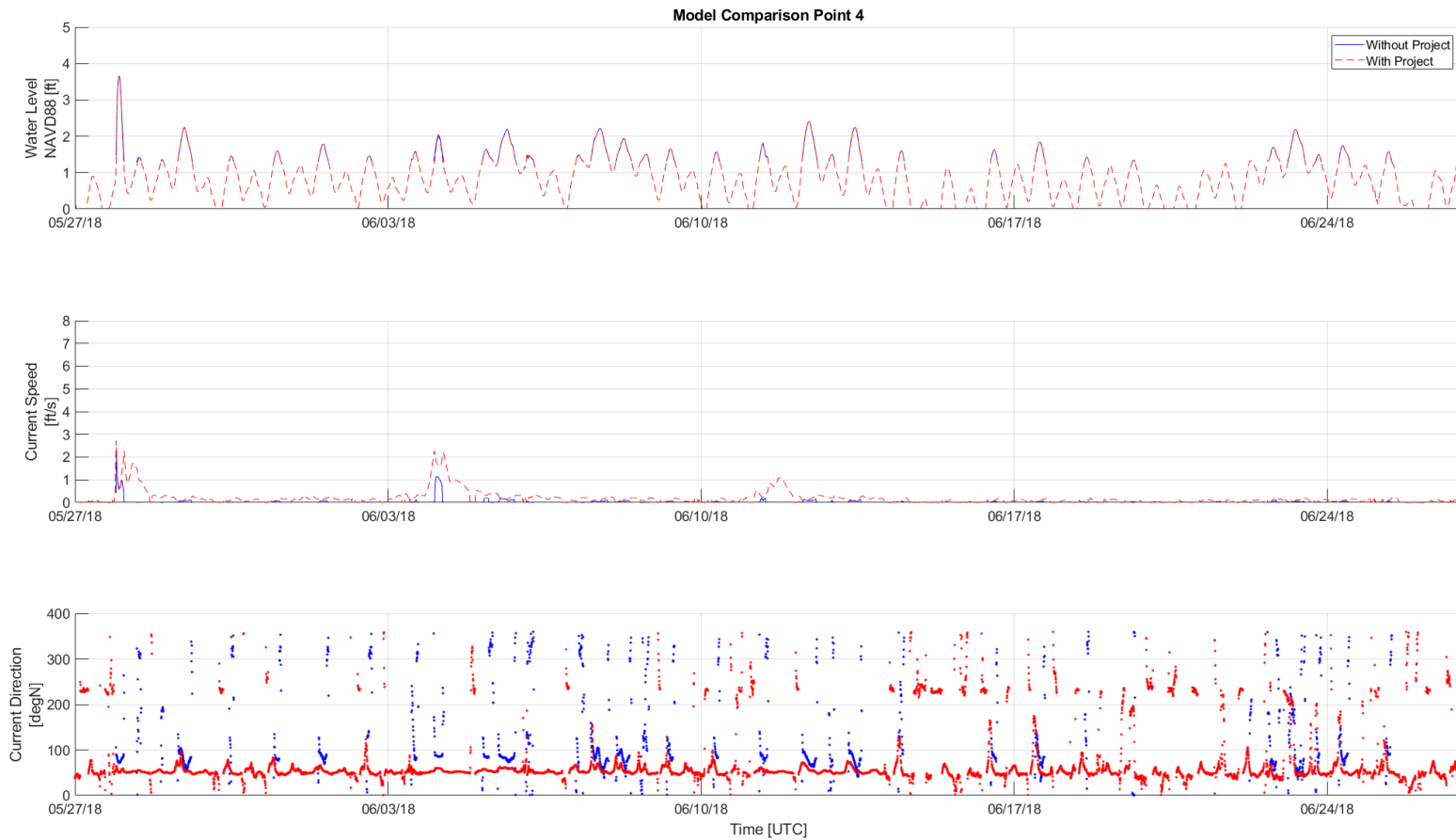


Figure B-13: Modeled with project (red) and without project (blue) project hydrodynamics at model comparison point 4

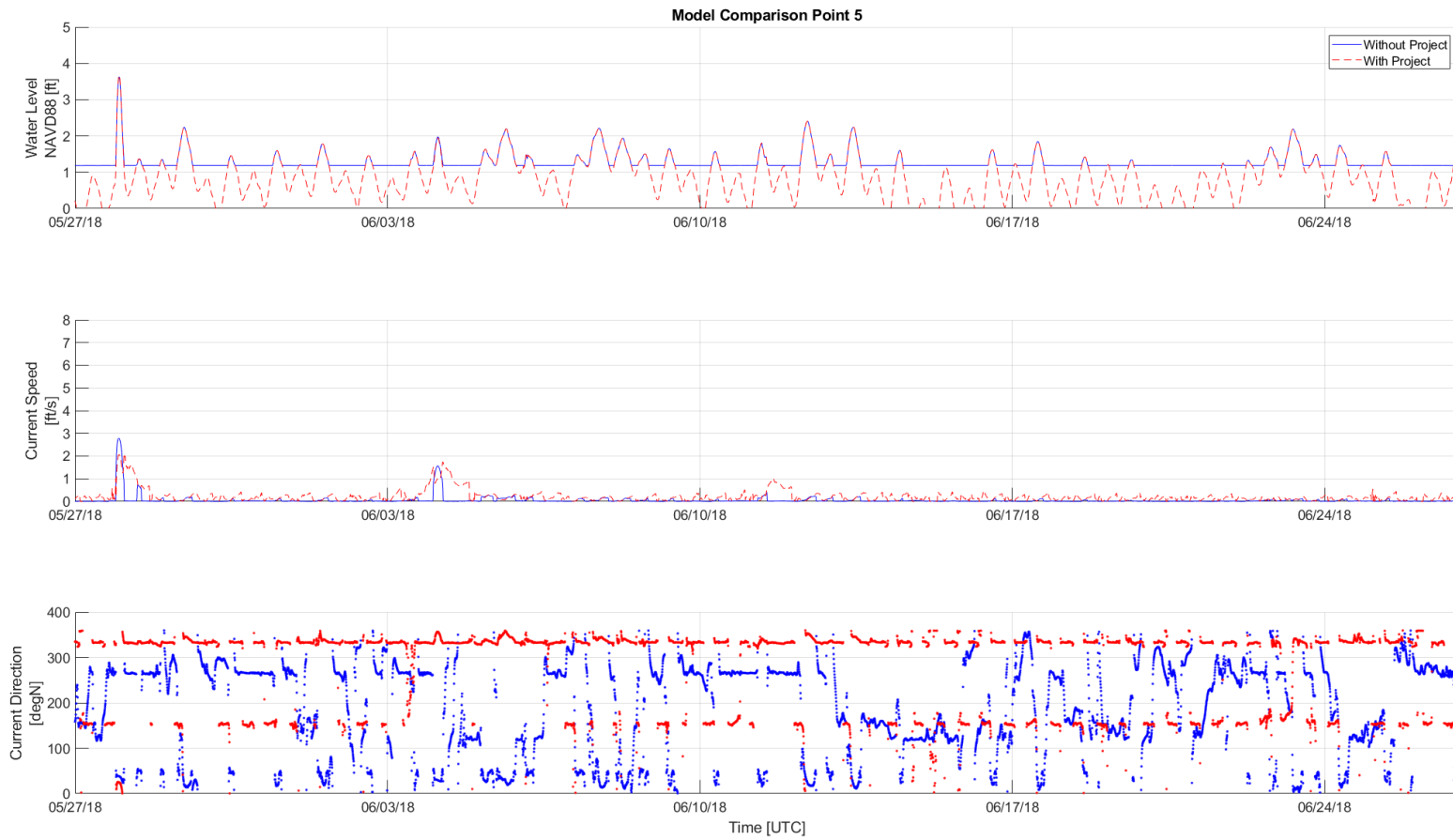


Figure B-14: Modeled with project (red) and without project (blue) project hydrodynamics at model comparison point 5

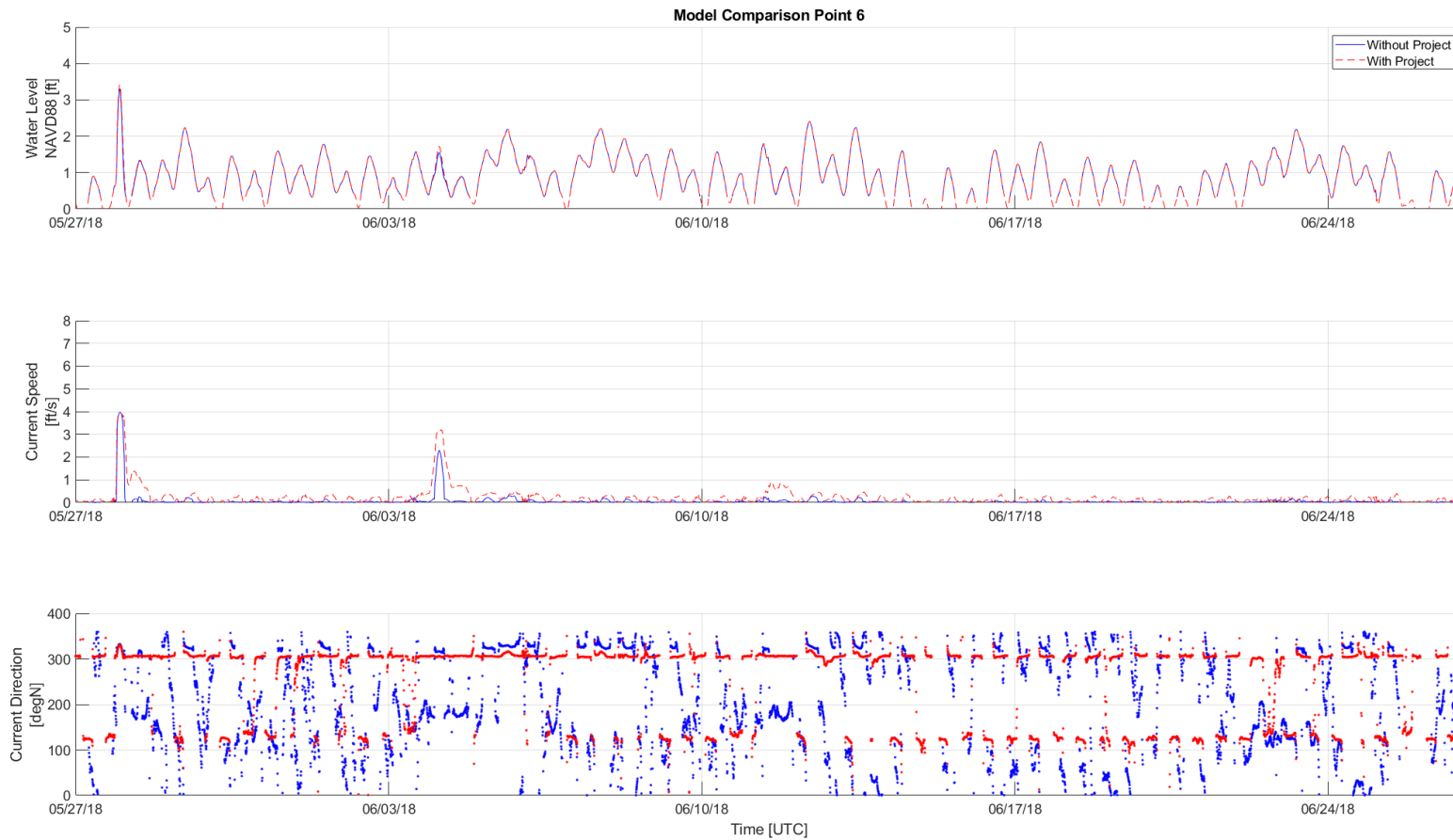


Figure B-15: Modeled with project (red) and without project (blue) project hydrodynamics at model comparison point 6

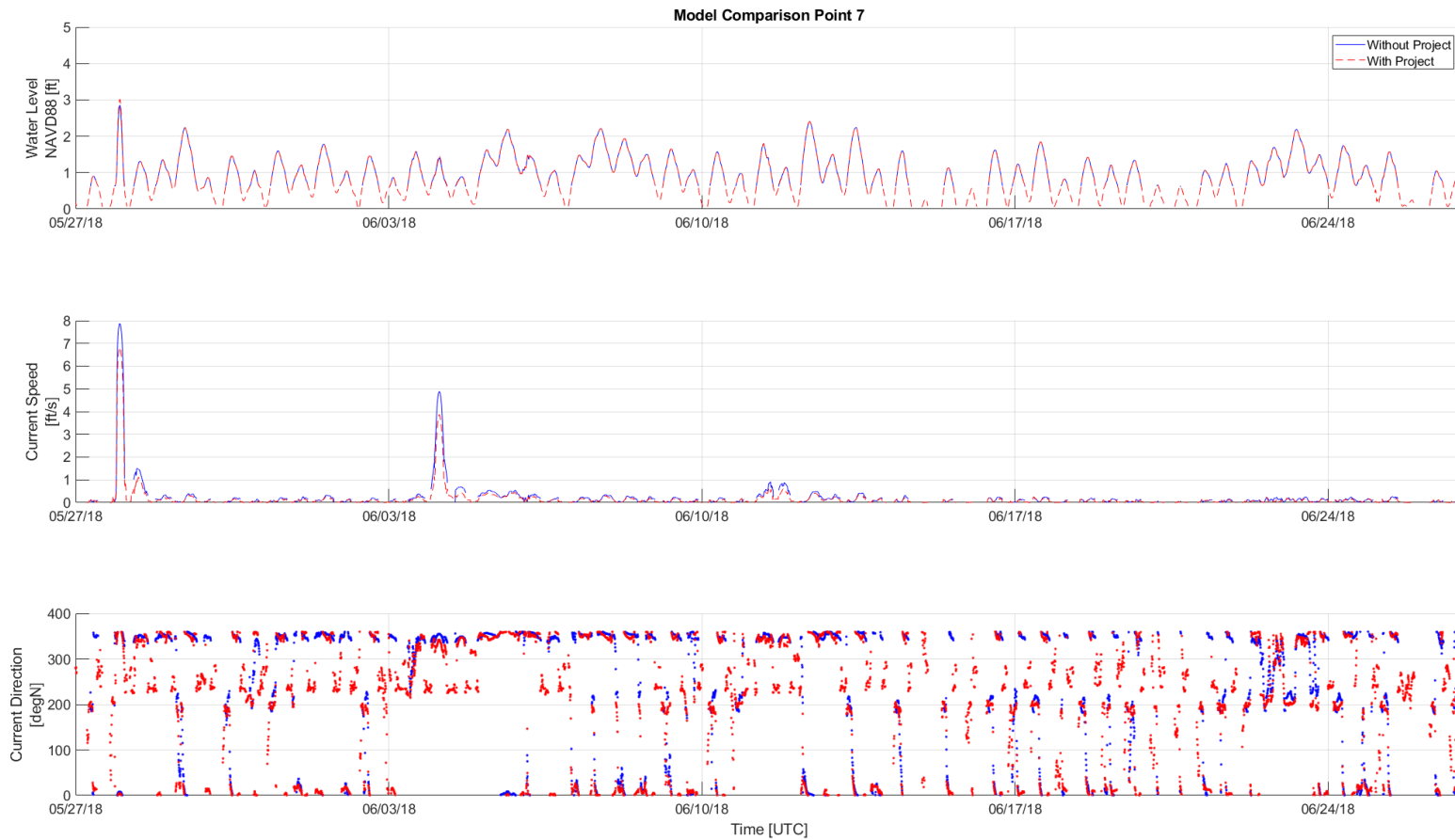


Figure B-16: Modeled with project (red) and without project (blue) project hydrodynamics at model comparison point 7



Figure B-17: Modeled with project (red) and without project (blue) project hydrodynamics at model comparison point 8

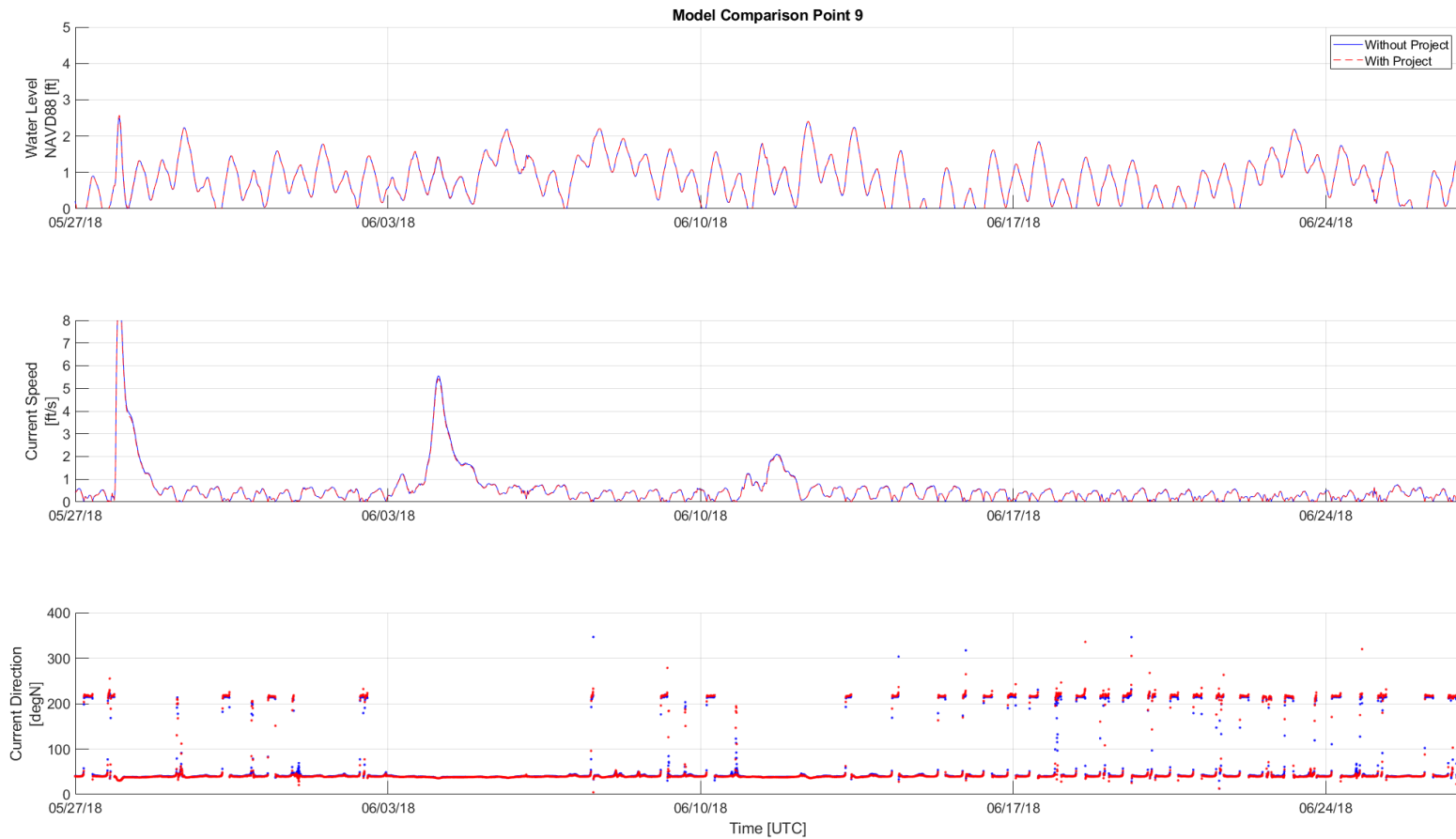


Figure B-18: Modeled with project (red) and without project (blue) project hydrodynamics at model comparison point 9

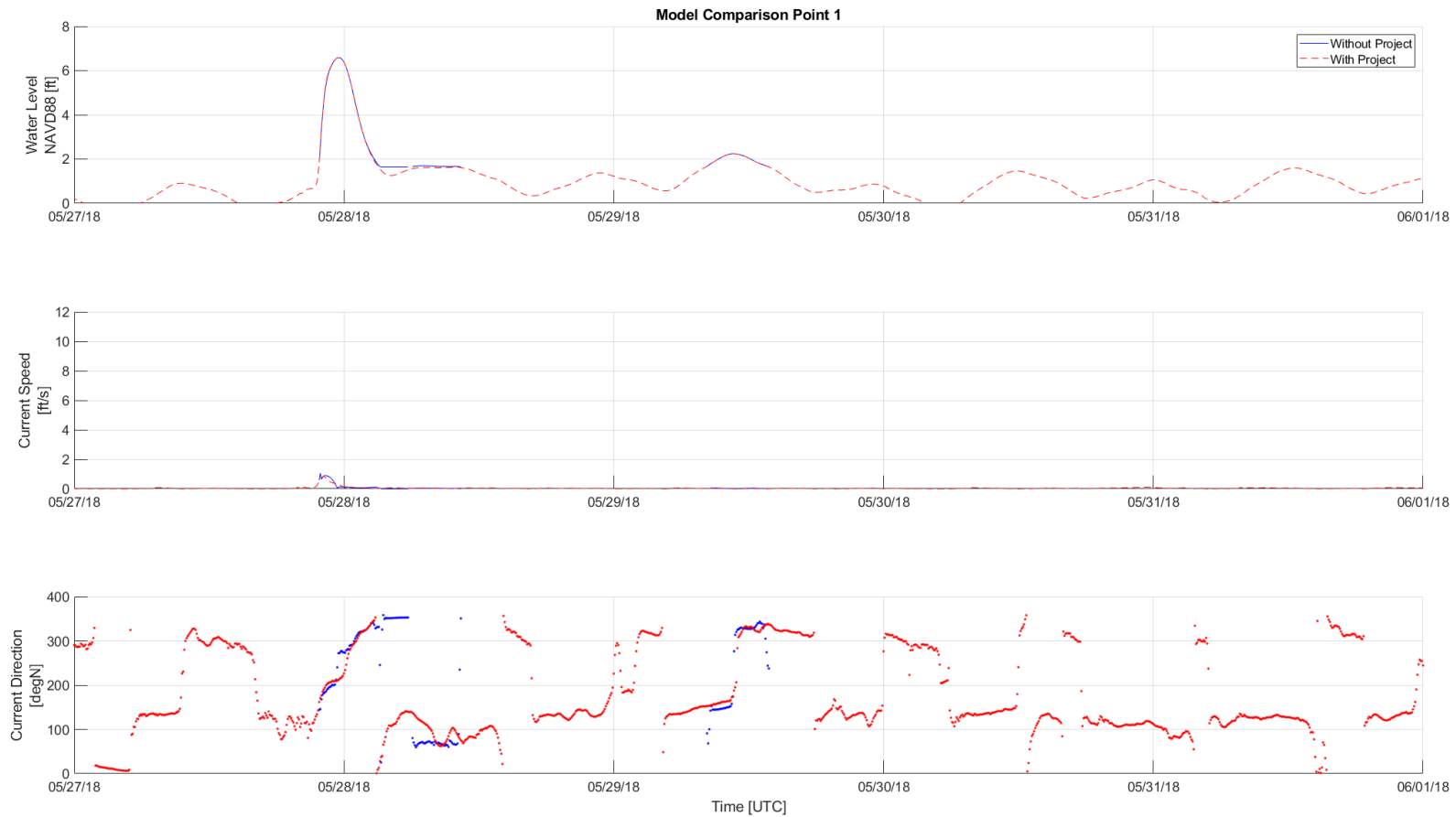


Figure B-19: Modeled with project (red) and without project (blue) project hydrodynamics during 100-year RP at model comparison point 1

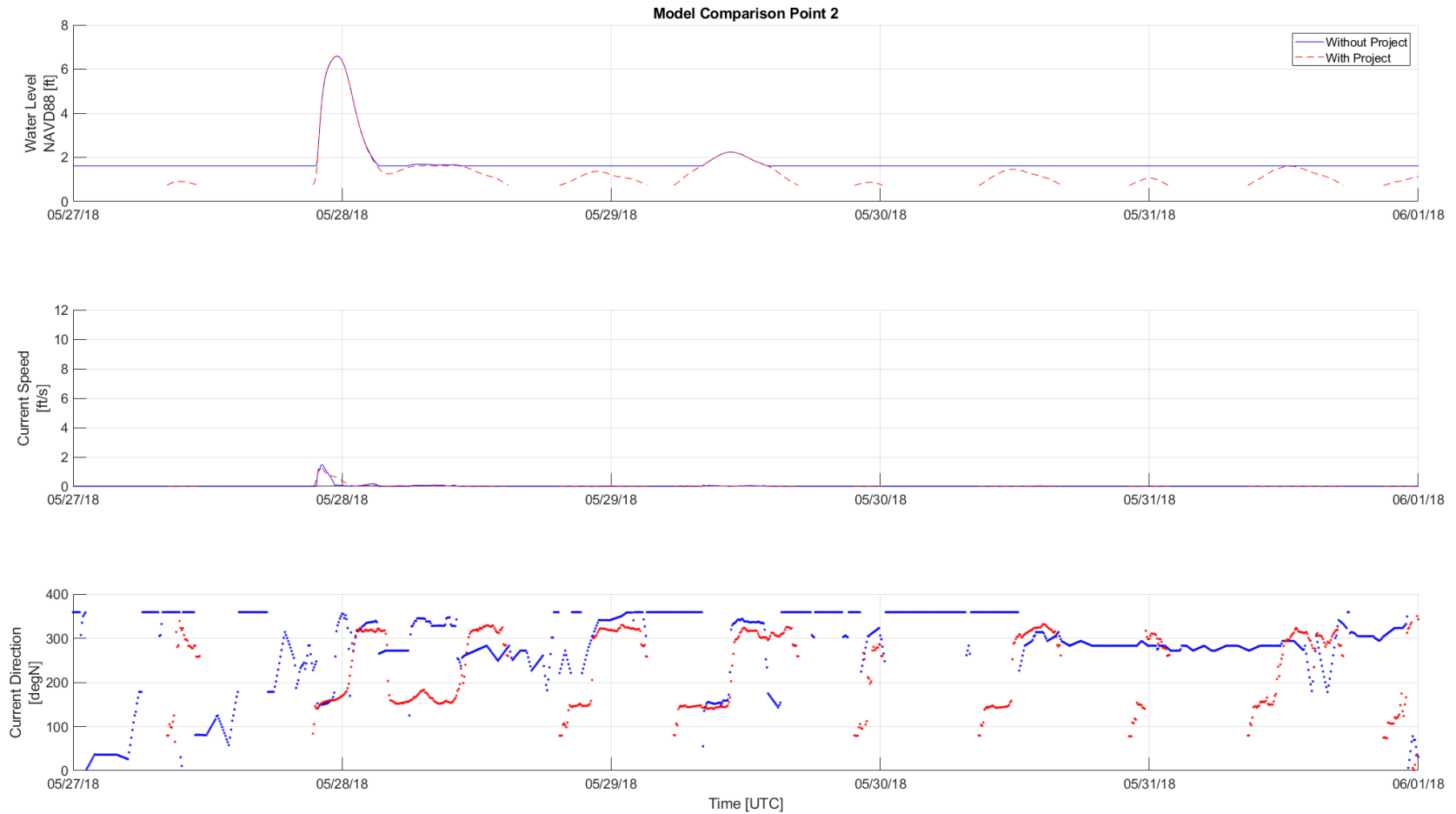


Figure B-20: Modeled with project (red) and without project (blue) project hydrodynamics during 100-year RP at model comparison point 2



Figure B-21: Modeled with project (red) and without project (blue) project hydrodynamics during 100-year RP at model comparison point 3

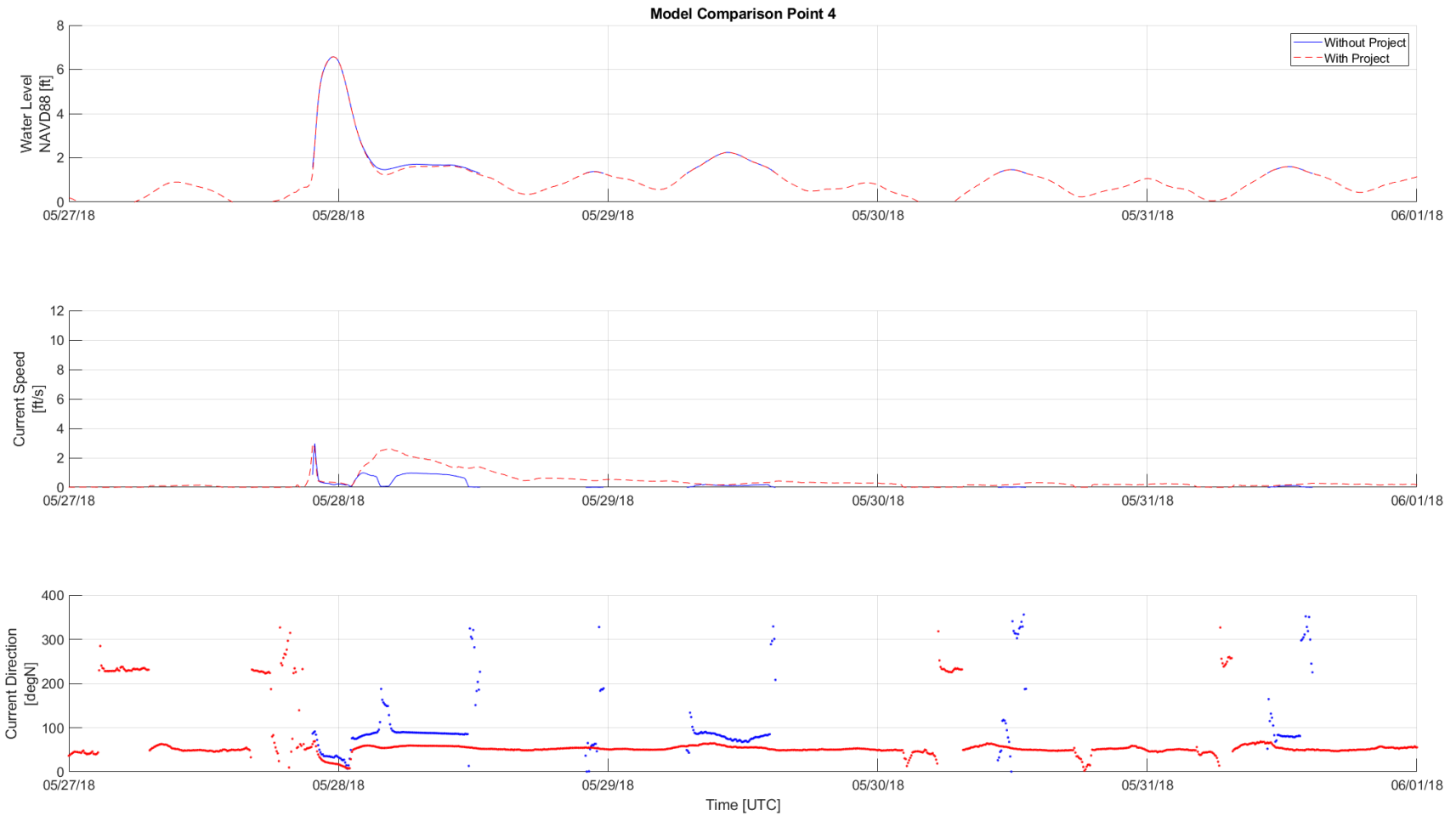


Figure B-22: Modeled with project (red) and without project (blue) project hydrodynamics during 100-year RP at model comparison point 4

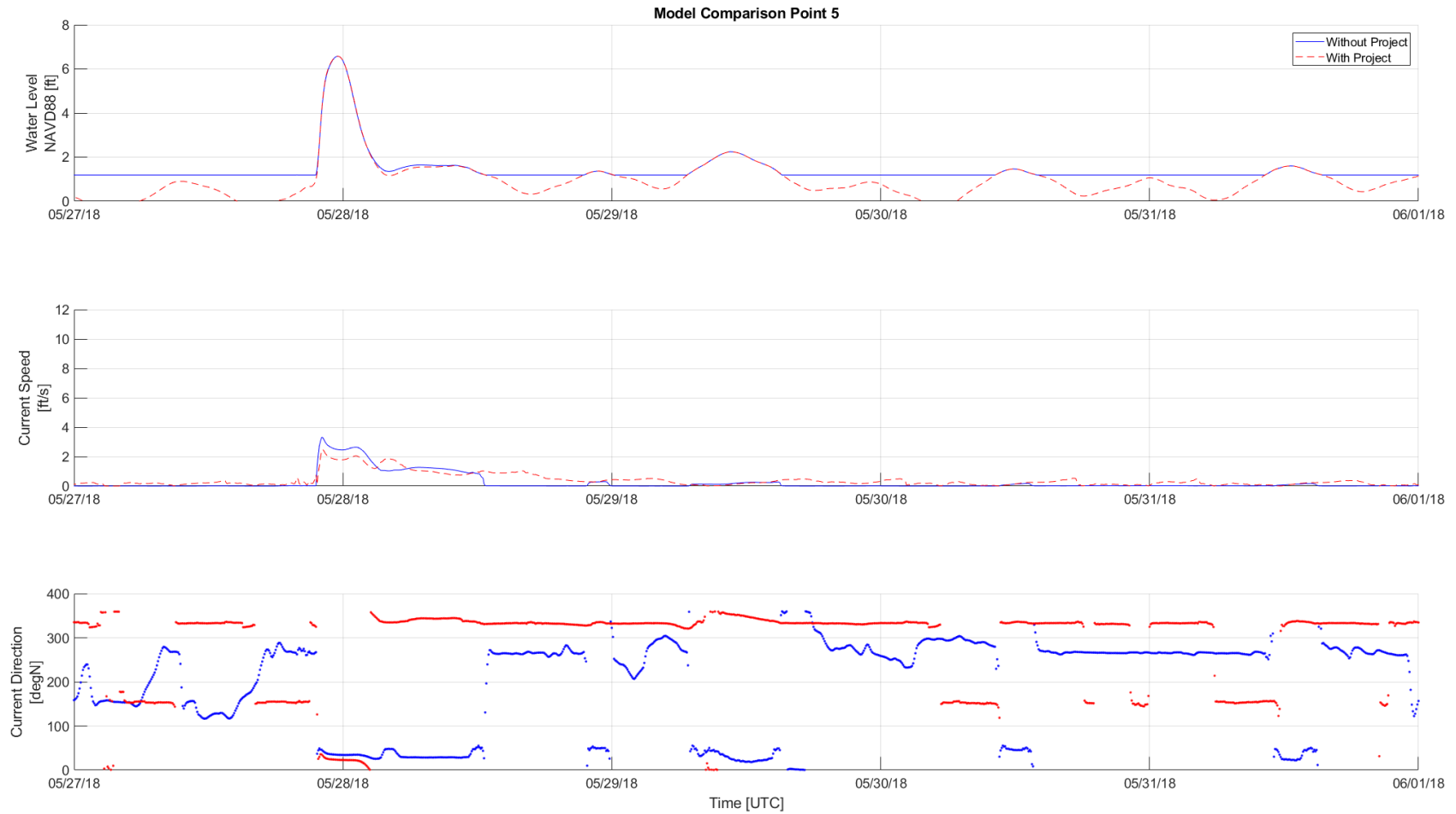


Figure B-23: Modeled with project (red) and without project (blue) project hydrodynamics during 100-year RP at model comparison point 5

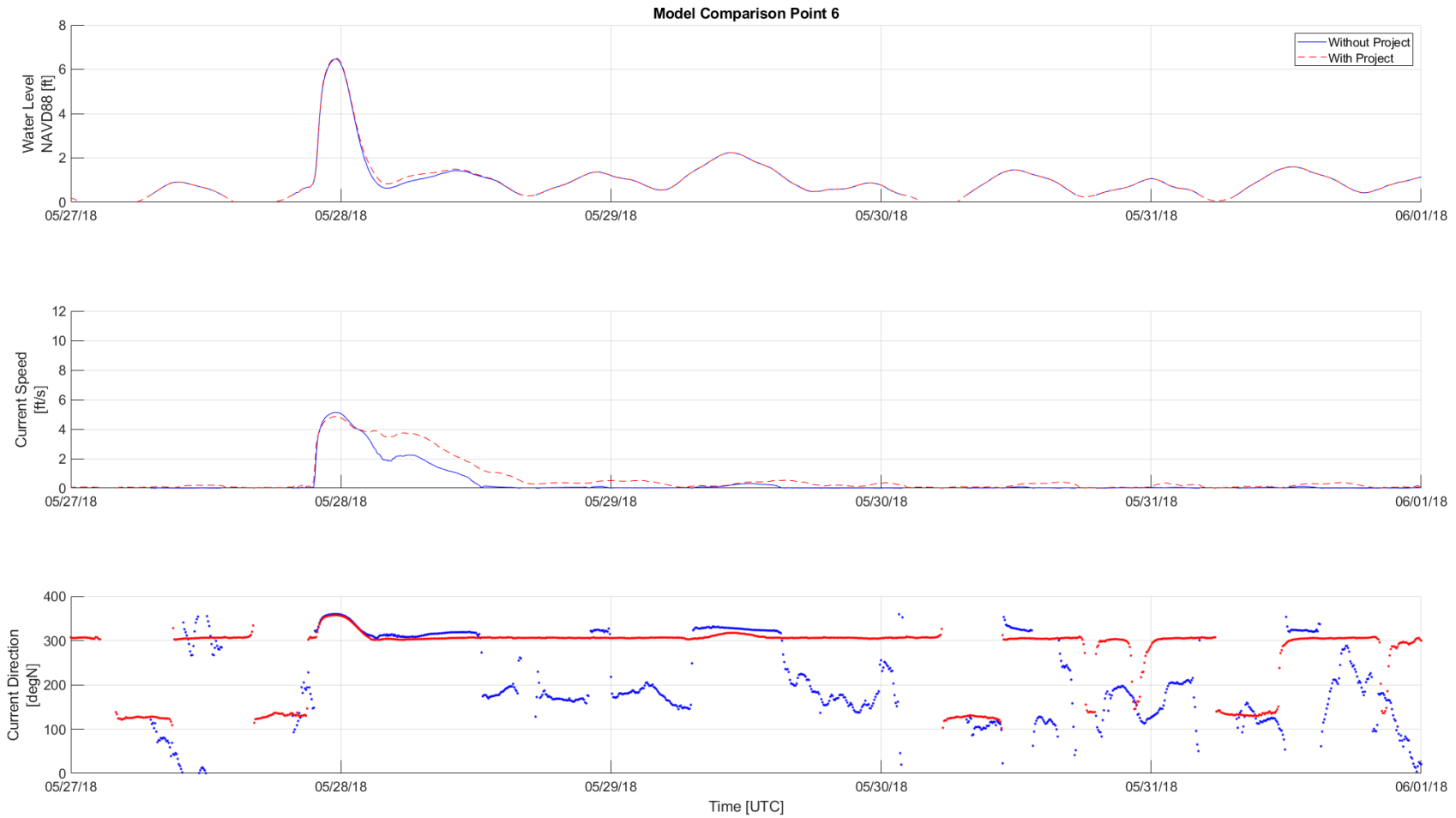


Figure B-24: Modeled with project (red) and without project (blue) project hydrodynamics during 100-year RP at model comparison point 6



Figure B-25: Modeled with project (red) and without project (blue) project hydrodynamics during 100-year RP at model comparison point 7

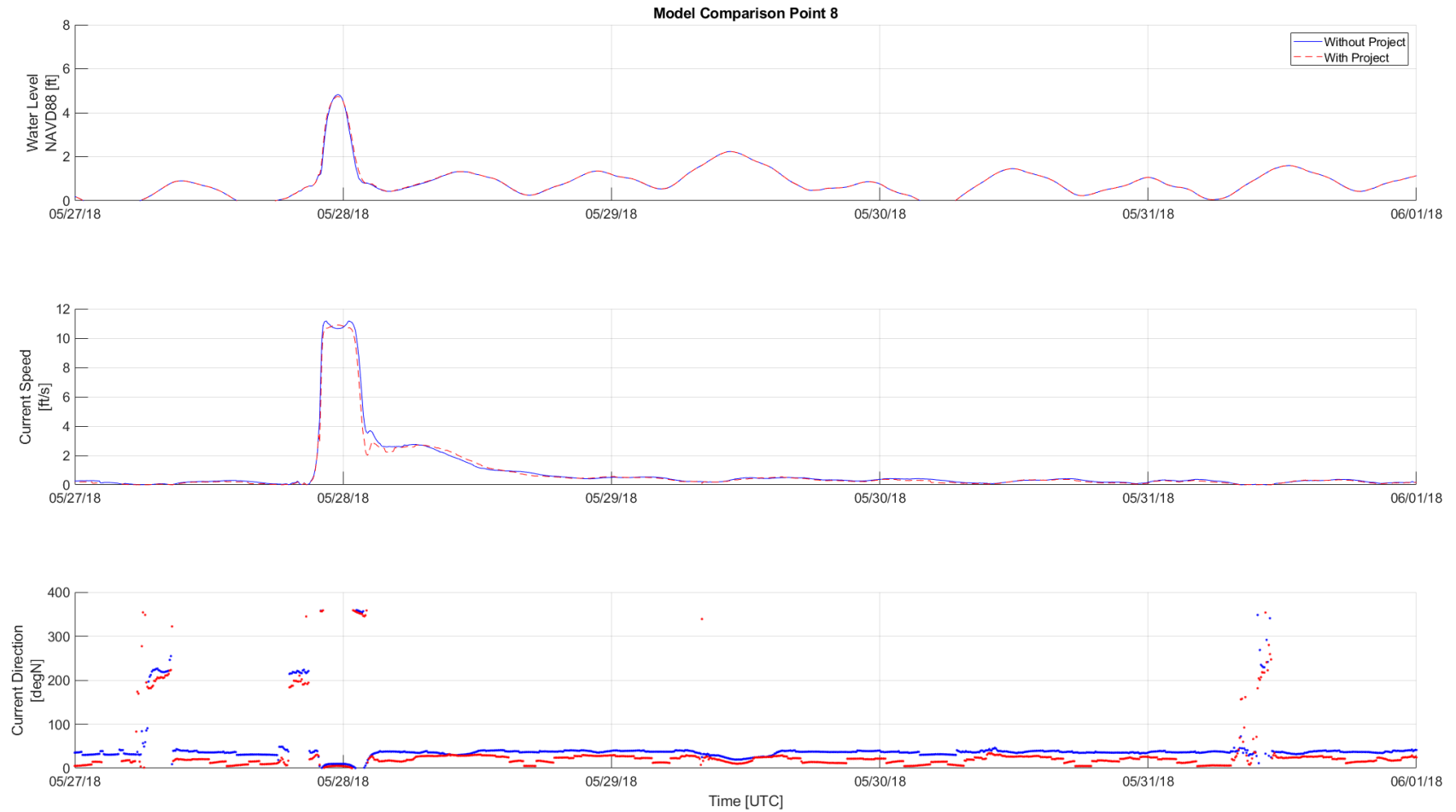


Figure B-26: Modeled with project (red) and without project (blue) project hydrodynamics during 100-year RP at model comparison point 8

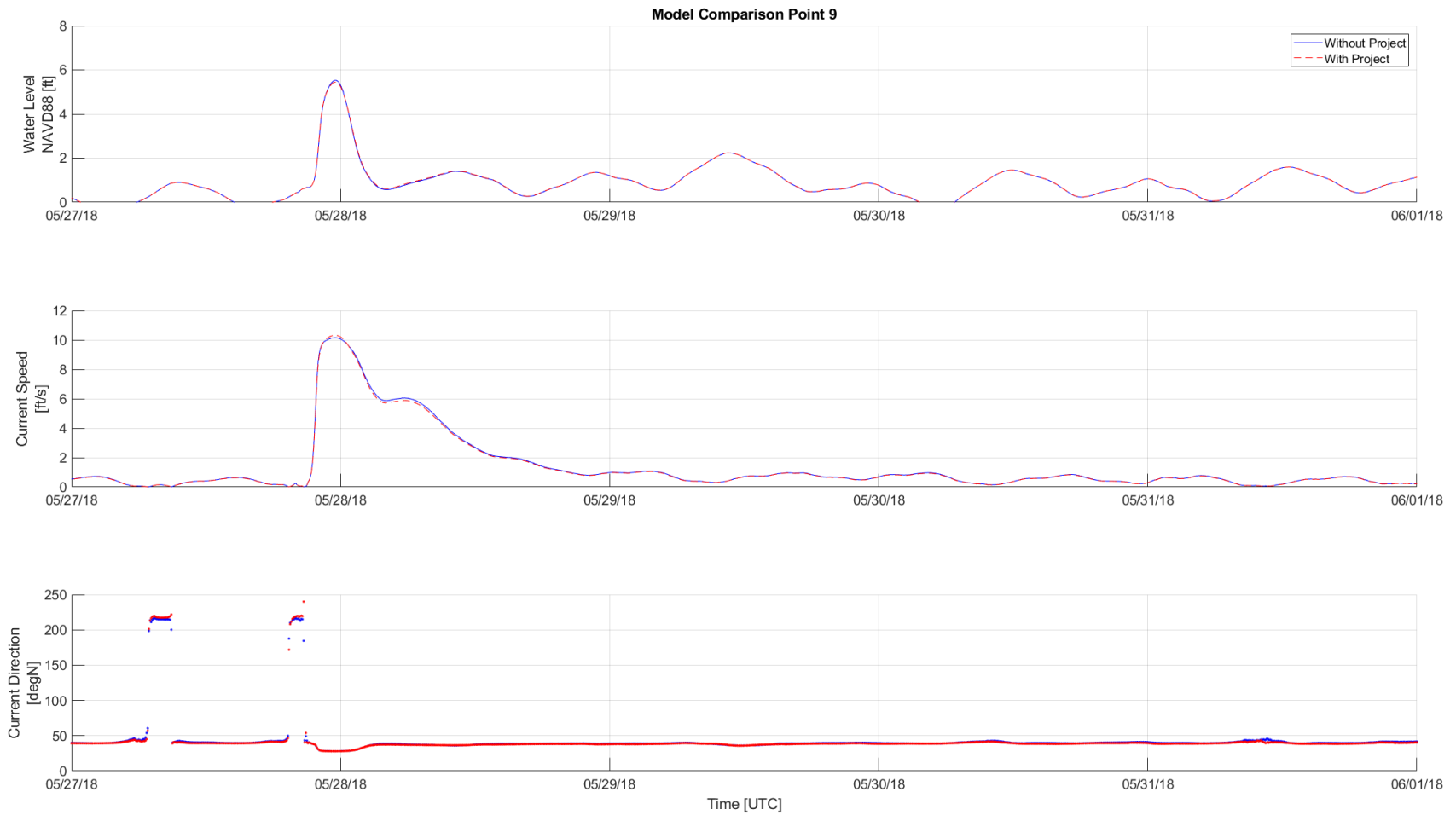


Figure B-27: Modeled with project (red) and without project (blue) project hydrodynamics during 100-year RP at model comparison point 9