100% DRAFT SCHEMATIC DESIGN SUBMITTAL

National Mall and Memorial Parks (NAMA)

Rehabilitate Seawalls and Shoreline – Design Services for Predesign (PD), Schematic Design (SD), and Design-Build (DB) Request for Proposal (RFP)

Coastal Modeling Report

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Rehabilitate Seawalls and Shoreline

National Mall and Memorial Parks

Washington, DC

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Coastal Modeling Report

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COASTAL MODELING

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Glossary

CEM	Coastal Engineering Manual
E	East
ft	Feet
ft/sec	Feet per second
in.	Inch
kip/ft	Kip per foot
MIKE21 FM	MIKE Flexible Mesh (Model)
MIKE21 FM HD	2D Depth-Averaged Flexible Mesh Hydrodynamic (Model)
MIKE21 FM SW	2D Flexible Mesh Spectral Wave (Model)
MLLW	Mean lower low water
NAMA	National Mall
NOAA	National Oceanic and Atmospheric Administration
NPS	National Park Service
NW	Northwest
Parks	Memorial Park
Q	Discharge
SLR	Sea level rise
SSW	South-south-west
SWL	Still water level
TOW	Top of wall
UACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
WL	Water level

1. Executive Summary

The National Park Service (NPS) is preparing for the rehabilitation of portions of the National Mall (NAMA) Tidal Basin seawall and the entire West Potomac Park seawall south of Arlington Memorial Bridge. This project evaluates 6,800 linear feet of seawall that is administered by the NPS through the National Mall and Memorial Parks (Park) and located in Washington, DC.

The purpose of the proposed project is to restore the functionality of the seawalls, restore the cultural landscape, improve visitor experience along the shorelines, minimize soil erosion and safety hazards, and provide some flood protection. The NPS proposes to rebuild and elevate the seawalls to re-establish the historic functional height of the walls in such a way as to provide for a sustainable solution that expands the lifecycle of the seawalls and future extensions of the wall to respond to changing climate patterns, including storms of greater intensity and frequency.

This report presents the results of the coastal modeling analyses that provide design insight in terms of coastal hazards including flood levels, scour velocities, wave overtopping, and wave loading. Hydrodynamic modeling was performed to develop design requirements for the proposed seawall that reduce risk and maintain functionality over the life of the design.

Coastal Model Description

- M&N existing MIKE21 hydrodynamic numerical model was used to compute the coupled influence of tides, wind, waves, and riverine flows at the project site. The existing model was extended and updated to include the Tidal Basin, the upstream Potomac River, and inlet and outlet gates. The model bathymetry along the seawall was refined using bathymetric and topographic survey data collected during spring 2022.
- Under a separate task and over a two-year effort that began in April 2022, water levels are being collected at the Inlet Bridge. The measured water levels are generally in agreement with water levels recorded at the National Oceanic and Atmospheric Administration (NOAA) Washington Channel tide gauge, with a difference in high water level of approximately -0.3 feet and a 20-minute time lag. Both the reduction in tidal range and the tidal lag from the channel to the basin are consistent with the inlet gates restricting tidal flow.
- The hydrodynamic model was calibrated using the data from the Inlet Bridge water level gauges, from April 29, 2022 to May 16, 2022, which captured a high-water level event. After conducting applying a Potomac River flow reduction, good agreements were reached between modeled and measured data, deeming the calibration appropriate and adequate.

Inlet and Outlet Gates

- The inlet gates located on the Inlet Bridge at the Potomac River side open during high tide and close during low tide; hence restricting the direction of the tidal flow into the Tidal Basin. Conversely, the outlet gates open at low tide and close during high tide.
- The existing gates do not function as flood control structures; they only provide a flushing mechanism within the Tidal Basin.
- The performance of the gates will impact flushing and circulation mechanisms in the Tidal Basin.
  - With partially opened gates, the current velocities are expected to increase at the gates due to flow constriction. Such impacts are localized at the immediate vicinity of the gates which could require especial design attention to the abutments and seawall near the gates. On the other hand, gate operation restrictions are not anticipated to cause changes to currents along the seawall in the away from the gates.
  - With closed gates, there will be no flow passing through the Tidal Basin and water quality issues could emerge in the Tidal Basin. Washington Channel could also require

maintenance dredging if excessive sedimentation occurs due to lack of flushing from the Tidal Basin.

Water Levels and Storm Surge

- Water levels are currently able to reach higher elevations at the Project site than when the NAMA seawall was first constructed. Water level analysis shows over time an increase in the number of events that can cause flooding at the Project site with water elevations higher than the current top of wall (TOW) elevation at 2.5 ft NAVD88 and an increase in the duration of flooding.
- The gradual increase in the frequency of flood events has lead to recommending raising the top of wall elevation to 5.5 and 4.75 feet NAVD88 at West Potomac Park and the Tidal Basin, respectively. The given recommendation is also linked to geometrical limitations associated with West Potomac Park upland elevations.
- SLR is expected to exacerbate the flood hazards at the Project site.
  - Under current conditions, the average interval between floods for the proposed Tidal Basin (West Potomac Park) TOW was estimated at 1.3 years (2.2 years); that number is expected to decrease to 3.3 months (1 year) under SLR.
  - These projections of frequent on-site flooding indicate that SLR will cause the Project site to be more frequently flooded by “clear sky” nuisance flooding and cause flood depths to increase over time.
- The effects of SLR on storm surge were evaluated using the hydrodynamic model.
  - The effects of SLR on storm surge were evaluated using three historical hurricanes at the project site: Isabel, Fran, and Sandy, in combination with increasing sea levels ranging from 0 feet (present) to 5 feet.
  - All modeling results indicated that SLE would lead to a reduction in storm surge at the Project Site. All three hurricanes show reductions in storm surge at the project site with SLR. However, for the 2050 SLR projection only a 3% reduction in surge was observed, which is considered negligible, and therefore, a one-to-one increment of SLR on storm surge is recommended for design purposes.

Current Velocities and Scour Potential

- Along the seawall in West Potomac Park, the peak currents are controlled by Potomac River high discharge. The maximum current modeled at West Potomac Park given a 50-year Potomac River discharge, conservatively assuming no overbank flooding, was 7.5 feet per second (ft/sec).
- In the Tidal Basin, the peak currents along the seawall are much lower than those along West Potomac Park during high river discharge events due to the one-way inlet gates; however, large currents were found at the inlet and outlet gates when the gates are open, but are only limited to the area at and near the gates. Tidal Basin maximum current at the seawall close to the gate was 1.4 ft/sec while the currents away from the gates did not exceed 0.7 ft/sec.
- The scour risks and stability of the bottom rocks were checked upon the peak currents along the seawall.
  - Given that the existing riprap protection at the project site consists of 18-inch rock (per field observations), the currents need to exceed 9.4 ft/sec to cause any stability issues. The simulated peak currents both at the West Potomac Park and Tidal Basin are all less than 9.4 ft/s, thus; no scour risk is associated with the existing 18-inch rock.
  - Given the maximum 7.5 ft/sec velocity along West Potomac Park, bedding/scour stone should have a stone size (D_{50}) of 7-inch.

Wave Loading and Wave Overtopping

- Wave modeling was performed for several design return periods using the same model in the hydrodynamic analysis.
  - The maximum significant wave heights for present day conditions vary from 2.2 feet for a 10-year return period to 4.1 feet at for a 100-year return period along West Potomac Park, and from 1.4 feet for a 10-year return period to 2.9 feet for a 100-year return period in the Tidal Basin.
  - The results under future SLR conditions showed little variation compared to present SLR.
- Wave loading and overtopping were calculated along West Potomac Park and within the Tidal Basin with proposed top of wall elevations 5.5 and 4.75 feet NAVD88, respectively.
- Overall, the largest wave heights, wave loads, and overtopping rates were found along West Potomac Park at southern locations near the Inlet Gate. This is consistent with the larger deterioration seen in this area.
- Maximum wave loads calculated for several design return periods were found to occur below the proposed top of wall elevation.
  - The maximum wave loads for present day conditions vary from 1.0 kip per foot (kip/ft) for a 10-year return period to 2.5 kip/ft for a 100-year return period on West Potomac Park, and from 0.9 kip/ft for a 10-year return period to 2.3 kip/ft for a 100-year return period in the Tidal Basin.
  - SLR does not impact the wave loads because the maximum wave loads were found to occur below the proposed top of wall elevation.
- Wave overtopping was calculated for several design return periods.
  - Along West Potomac Park, it was found that erosion of soil with patchy grass cover would occur for a wave overtopping event with a return period greater than 10-year[1]. In the Tidal Basin, erosion of soil with patchy grass cover would occur for a wave overtopping event with a return period greater than 50-year[2].
  - SLR will cause water levels to increase relative to the proposed top of wall elevations. Because the site will be more prone to flooding, erosion of soil with patchy grass cover due to wave overtopping is expected for a 5-year event within the Tidal Basin and along West Potomac Park[3]. With a 5-year event having a 20% probability of being exceeded per year, more maintenance will be needed along the backside of the walls to maintain consistent vegetation.
  - Preventative measures to reduce wave overtopping damage other than raising the seawall elevation may include: hardening a larger surface of West Potomac Park uplands, adding a splash pad on the top of wall, adding an impermeable parapet/guardrail on the top of wall.

[1] An overtopping return period greater than 10 years is expected along West Potomac Park for a 10-year return period wave event coinciding with water levels above the proposed +5.5 ft NAVD88 wall crest (roughly 7-yr return period at present-day).

[2] An overtopping return period greater than 50 years is expected along the Tidal Basin for a 50-year return period wave event coinciding with water levels above the proposed +4.75 ft NAVD88 wall crest (roughly 4-year return period at present day).

[3] An overtopping event with a return period greater than 5 years is expected in 2050 along (1) West Potomac Park for a 5-year wave event coinciding with water levels above the proposed +5.5 ft NAVD88 wall crest (roughly 4-yr return period in 2050), and (2) along the Tidal Basin for a 5-year wave event coinciding with water levels above the proposed +4.75 ft wall crest (roughly 2.5-year return period in 2050).

2. Introduction

The National Park Service (NPS) is preparing for the rehabilitation of portions of the National Mall (NAMA) Tidal Basin seawall and the entire West Potomac Park seawall. This project evaluates 6,800 linear feet of seawall that is administered by the NPS through the NAMA and Memorial Parks (Park) and located in the District of Columbia (see Figure 1).

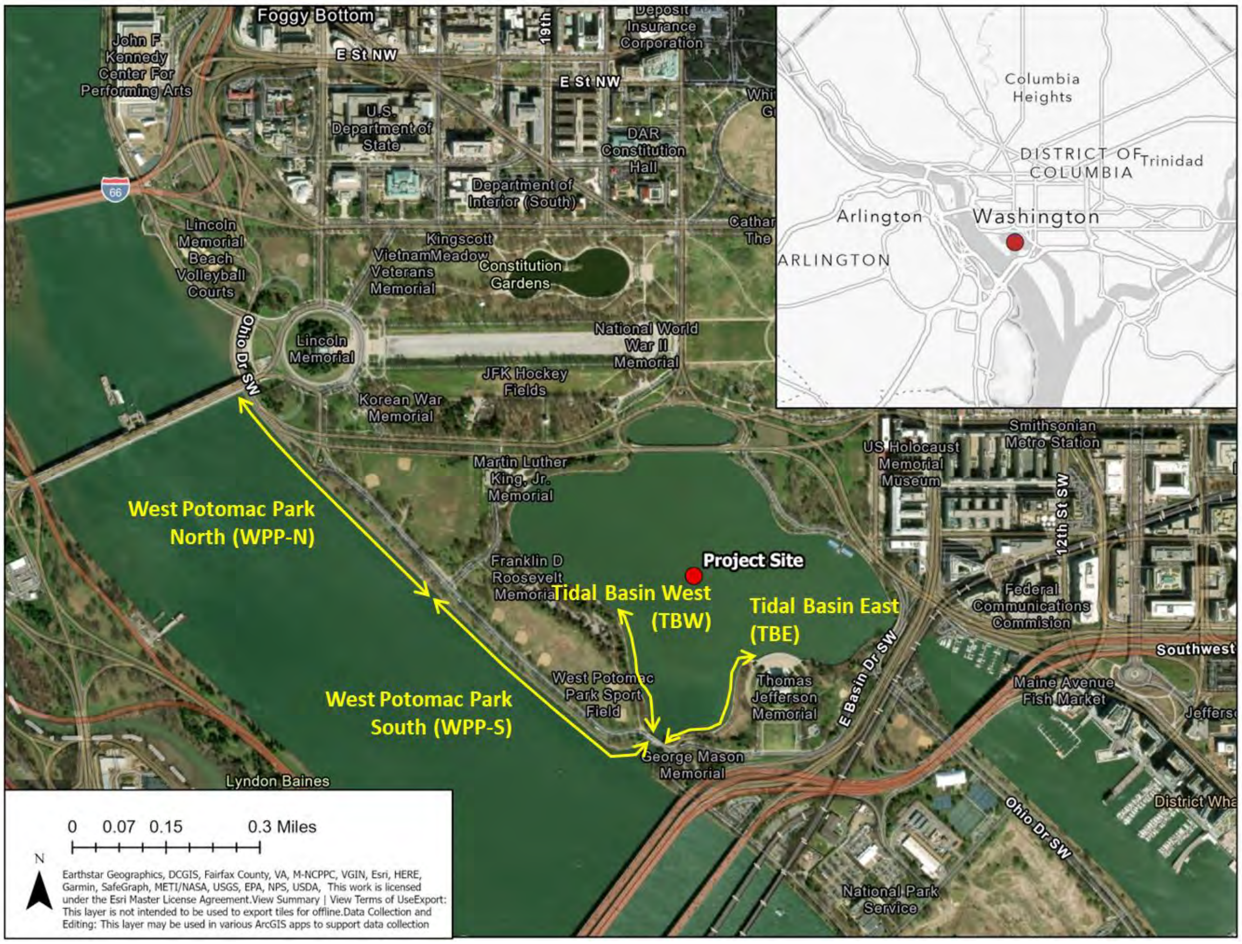

FIGURE 1. PROJECT SITE LOCATION MAP.

The purpose of the project is to restore the historic functional height of the seawalls, restore the cultural landscape, improve visitor experience along the shorelines, minimize soil erosion and safety hazards, and provide some flood protection. The NPS proposes to rebuild and elevate the seawalls to re-establish the historic functional height of the walls in such a way as to provide for a sustainable solution that expands the lifecycle of the seawalls and future extensions of the wall to respond to changing climate patterns, including storms of greater intensity and frequency.

This report presents the results associated with the coastal modeling analyses that provide design insight in terms of the understanding coastal hazards, including flood levels, scour velocities, and waves loading. Hydrodynamic modeling was performed to develop design requirements for the proposed seawall that reduce risk and maintain functionality over the proposed design life. The coastal modeling scope includes the following:

- Update existing hydraulic model with new bathymetric data collected in 2022.
- Update model to include Tidal Basin and Washington Channel areas.

- Create simulation for tidal gates.
- Calibrate circulation model using the water level data collected at the Tidal Basin inlet bridge.
- Calculate tide and storm current fields and scour potential.
- Evaluate non-linear interaction between sea level rise (SLR) and storm surge.
- Evaluate wave heights with and without SLR conditions for overtopping, wave loading, and scour on the seawall.
- Evaluate high river events for extreme conditions modeling.

The importance of this analysis is to inform the seawall design in order to provide adequate protection for the service life of the design.

3. Water Levels and Currents

3.1. Purpose and Goals

The goal of analyzing water levels and currents at the project site is twofold:

1. Inform the design about extreme water levels and currents and associated scour;
2. Evaluate the effects that SLR can have on water levels at the project site.

To attain the project goals, a hydrodynamic model has been employed in combination with water level data that have been collected at the project site. Model description and set up is provided in Appendix B.

3.2. Tidal Basin General Circulation

The Tidal Basin is a man-made tidal reservoir located between the Potomac River and the Washington Channel in Washington, DC. It serves as a mean of flushing out the Washington Channel by releasing captured water at the Tidal Basin twice a day at high tide. The general circulation at the Tidal Basin is shown in Figure 2. The inlet gates, located at the Potomac River, open during rising tides and close during falling tides while the outlet gates, located at the Washington Channel, open during falling tides and close during rising tides.

The opening/closing mechanisms of the inlet and outlet gates is fully driven by the tides; in other words, there is no other powered mechanism that controls the flushing system of the Tidal Basin (gate photos are shown Figure 3 and Figure 4; gate geometry and information is provided in Appendix A). The existing gates do not function as flood control structures; they only provide a flushing mechanism within the Tidal Basin.

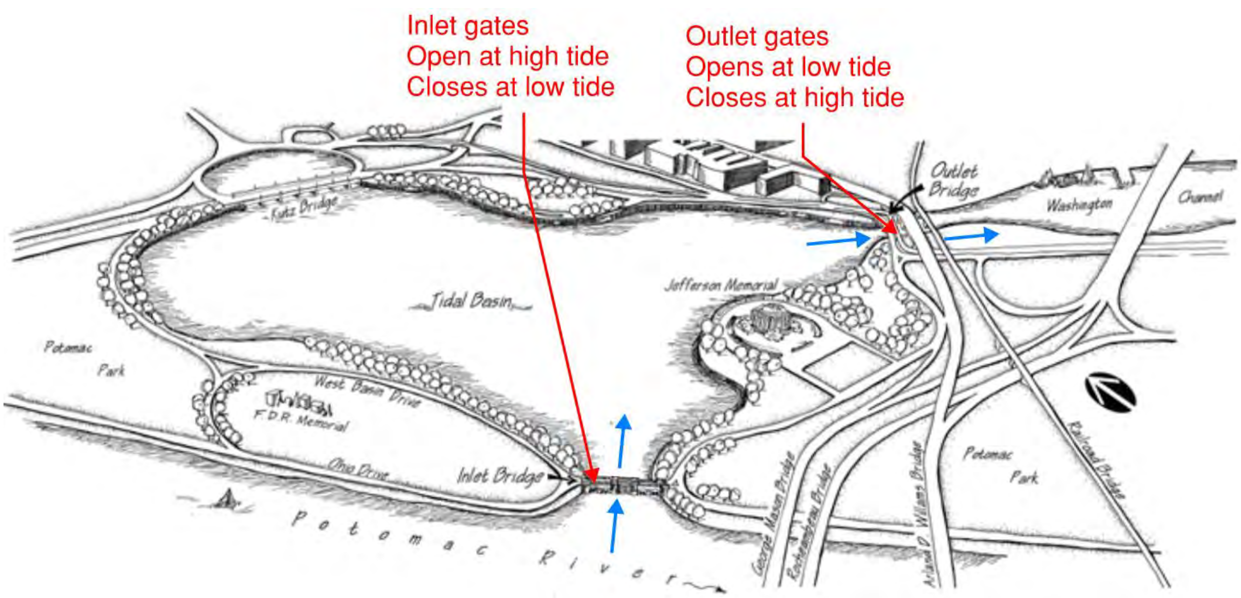

FIGURE 2. TIDAL BASIN GENERAL CIRCULATION SHOWN IN BLUE ARROWS (HEAR NPS, 2000).

FIGURE 3. TIDAL BASIN INLET BRIDGE: LOOKING AT IT FROM TIDAL BASIN (TOP), CENTRAL SPAN GATE (MIDDLE), AND INLET GATE (BOTTOM).

FIGURE 4. TIDAL BASIN OUTLET BRIDGE LOOKING AT IT FROM WASHINGTON CHANNEL.

3.3. Water Levels

3.3.1. Water Level and Seawall Design

Changes in flood frequency and duration over time were evaluated using the Washington Channel NOAA tide gage station data under the Climate Change and Natural Hazards report (Moffatt & Nichol, 2022). Water levels are currently able to reach higher elevations at the Project site than when the NAMA seawall was first constructed; therefore, water level frequently exceed the existing seawall as illustrated in Figure 5.

FIGURE 5. EXAMPLE OF FLOODING ON TIDAL BASIN WEST ON JUNE 7TH, 2022 WITH RECORDED WATER LEVEL AT 2.64 FT NAVD88.

As shown in Figure 6, flooding events with water levels exceeding the current top of wall elevation at 2.5 ft NAVD88 have increased both in frequency (number of events per year) and average duration since 1925. Water levels have exceeded the top of wall roughly 100 times per year during the past 20 years, whereas water levels exceeded the same elevation roughly 40 events per year during the past 90 years. These differences in flood frequency and duration are in part due to the slow increase in sea level, causing “clear sky” flooding above the NAMA seawall crest.

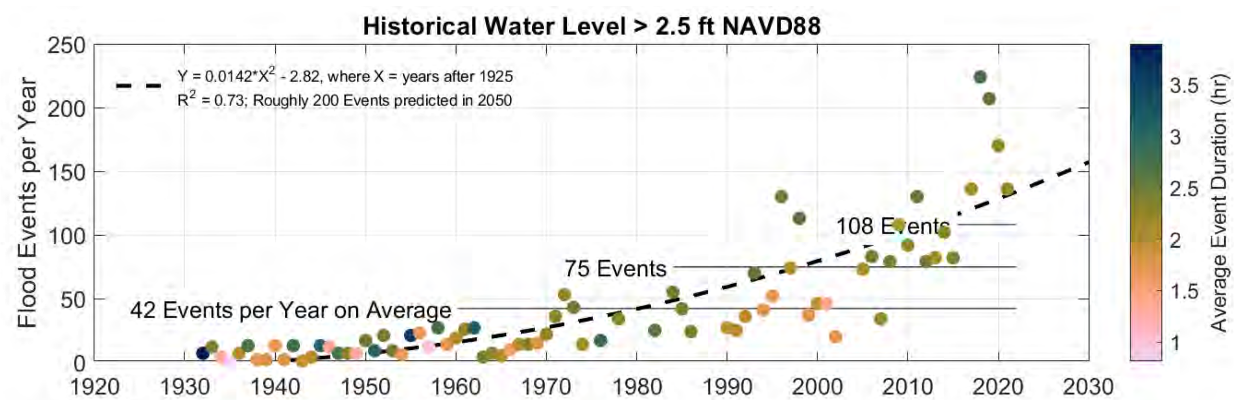

FIGURE 6. NUMBER OF EVENTS PER YEAR AND AVERAGE DURATION FOR FLOOD EVENTS EXCEEDING EXISITNG TOP OF WALL ELEVATION AT 2.5 FT NAVD88.

This gradual increase in the frequency of flood events has led to recommending raising the top of wall elevation to 5.5 and 4.75 feet NAVD88 at West Potomac Park and the Tidal Basin, respectively. The given recommendations are also linked to geometrical limitations associated with West Potomac Park upland elevations.

SLR is expected to exacerbate the flood hazards at the Project site. Water levels are expected to exceed the current TOW elevation roughly 330 days per year by 2050; raising the TOW elevation to 4.75 ft is anticipated to delay this frequency of flooding by roughly 70 years. Table 1 lists how the average interval between flood events is projected to change over time with SLR (see Appendix C for further results). Under current conditions, the average interval between floods for the proposed Tidal Basin (West Potomac Park) TOW was estimated at 1.3 years (2.2 years); that number is expected to decrease to 3.3 months (1 year) under SLR. These projections of frequent on-site flooding indicate that SLR will cause the Project site to be more frequently flooded by “clear sky” nuisance flooding, and cause flood depths to increase over time.

TABLE 1. AVERAGE INTERVAL BETWEEN FLOOD EVENTS WITH SLR FOR GIVEN TOP OF WALL ELEVATION.

SLR	+2.5 ft NAVD88 Current TOW	+4.75 ft NAVD88 Proposed TOW Tidal Basin	+5.5 ft NAVD88 Proposed TOW West Potomac Park
Present	2.7 days	1.3 years	2.2 years
2050	0.7 days	3.3 months	1.0 years
2070	0.6 days	12 days	3.2 months

3.3.2. Water Level Measuring Campaign

M&N began a two-year water level measuring campaign in April 2022 to evaluate the water levels at the Project site. The water level gages are mounted on the eastern side of the inlet gatesFigure 7; the recorded data were compared with water levels recorded by the NOAA tide gauge within the Washington Channel (NOAA CO-OPS Station 8594900).

Small differences were observed between tidal characteristics within the Tidal Basin and those at the NOAA gauge. The tidal range measured by M&N is roughly 0.3 ft smaller than the tidal range in Washington Channel (see Figure 8). Low tides within the Tidal Basin also lag those observed in the Washington Channel by roughly 20 minutes. Both the reduction in tidal range and the tidal lag are consistent with the inlet gates restricting tidal flow.

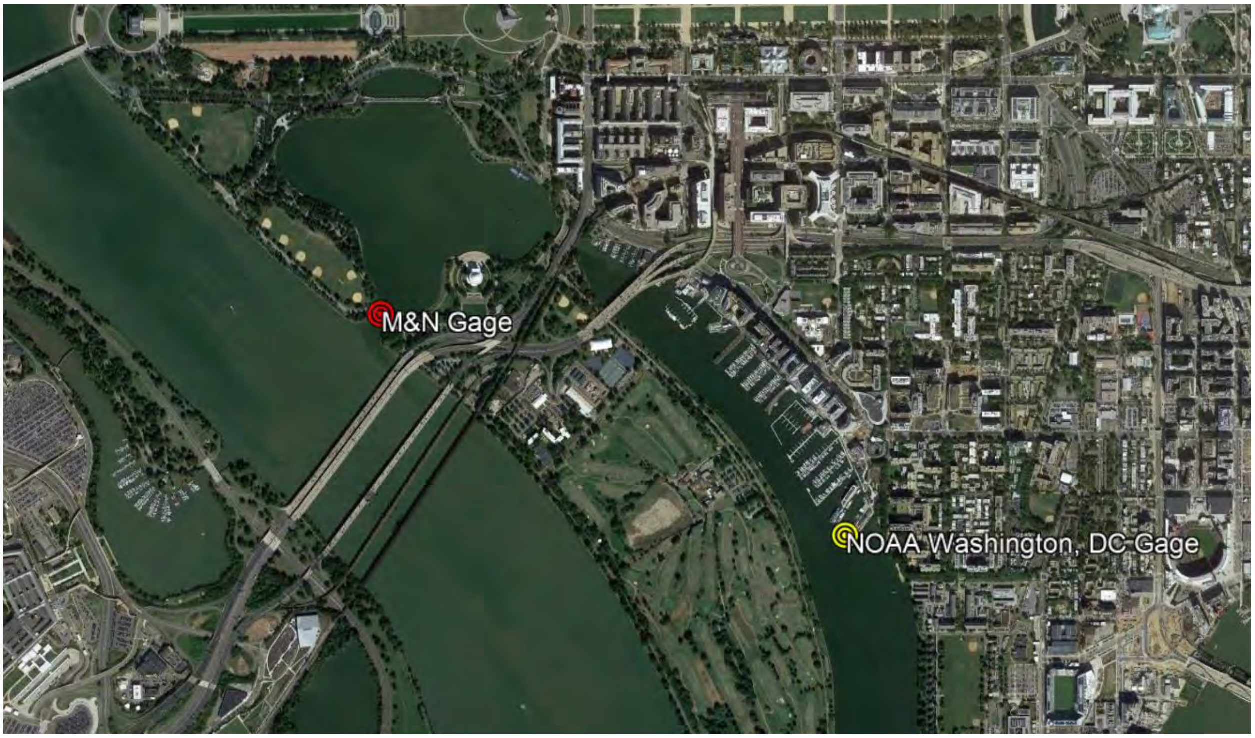

FIGURE 7. LOCATION OF M&N PRESSURE GAUGES RELATIVE TO THE TIDAL BASIN (TO THE NORTHEAST). THE POTOMAC RIVER IS LOCATED TO THE SOUTHWEST OF THE BRIDGE.

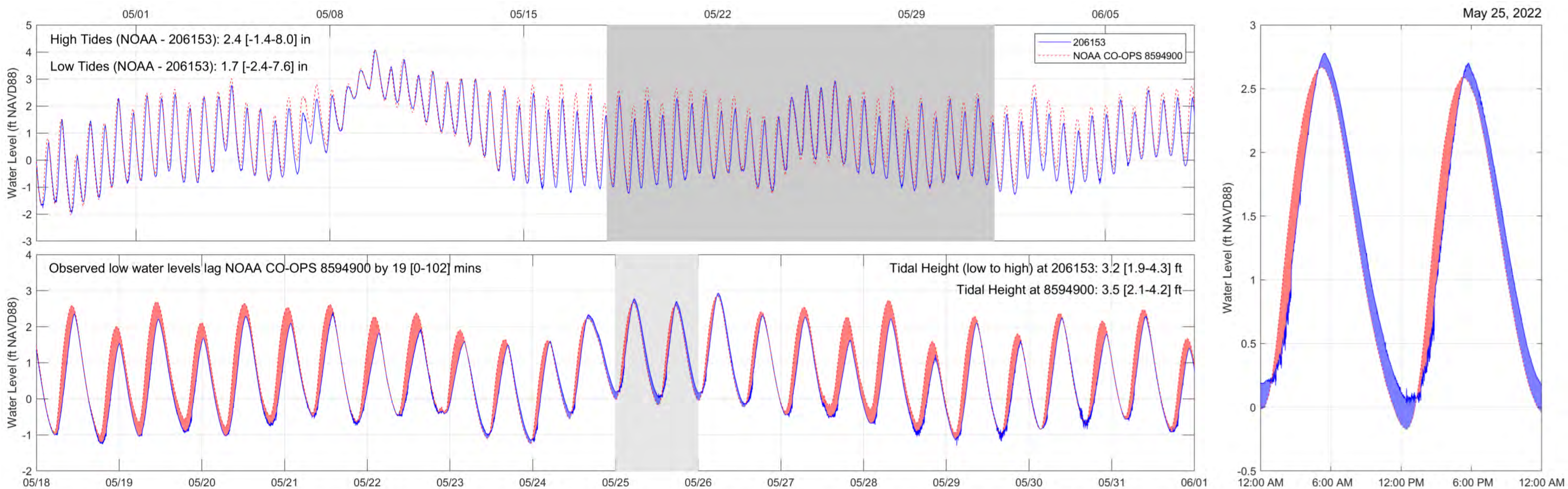

FIGURE 8. MEASURED WATER SURFACE ELEVATIONS AT M&N RBR GAUGE 206153 RELATIVE TO WATER LEVELS AT NOAA CO-OPS STATION 8594900 (TOP LEFT PANEL). THE DIFFERENCE BETWEEN MEASURED WATER LEVELS AND NOAA-RECORDED WATER LEVELS IS EMPHASIZED IN THE BOTTOM LEFT PANEL, WHICH SHOWS A TWO-WEEK PORTION OF THE FULL RECORD FOR ENHANCED DATA VISIBILITY; THE EXTENTS ARE SHOWN AS THE SHADED GREY REGION OF THE TOP LEFT PANEL. AN EXAMPLE COMPARISON OF MEASURED AND NOAA-RECORDED WATER LEVELS IS SHOWN AT RIGHT FOR THE DAY SHADED IN GREY IN THE BOTTOM LEFT PANEL. ALL COMPARISON METRICS ARE MEDIAN VALUES FOR THE FULL PERIOD OF DATA COMPARISON; THE FULL RANGE OF VALUES ARE PROVIDED IN BRACKETS.

3.3.3. Sea Level Rise Effects on Storm Surge

To evaluate the nonlinear effects SLR may have on storm surge, a range of possible future sea levels was considered in combination with historical hurricanes that had affected the project site. For each SLR scenario, the water level was raised across the entire model domain given the increase in sea level. A total of 18 simulations, which combine six SLR increments and three hurricanes, employed the following environmental and boundary conditions in the model, which coupled wind, waves, and water levels:

- SLR: 0, 1, 2, 3, 4, 5 ft
- Historical hurricanes
  - Hurricane Isabel 2003
  - Hurricane Fran 1996
  - Hurricane Sandy 2012
- Downstream boundary condition – historical water level hydrographs for historical hurricanes
- Upstream boundary conditions – historical discharge hydrographs for historical hurricanes

Following the model results for the 18 storm simulations, the non-linear effects of SLR on storm surge were calculated at West Potomac Park and Tidal Basin using the empirical non-linearity factor "NF":

$$NF = \frac{Water\ level_{with\ SLR} - SLR}{Water\ level_{witout\ SLR}}$$

If results indicate NF factor larger than 1, SLR would cause an increase in storm surge; conversely, if results indicate NF factor smaller than 1, SLR would cause a decrease in storm surge.

The results shown on Figure 9 and Figure 10 for West Potomac Park and the Tidal Basin, respectively, show reductions in storm surge with SLR as opposed to a surge increase. The decrease in storm surge can be attributed to the propagation of the storm surge and channel storage. When the surge travels upstream into Chesapeake Bay and in the Potomac River from the ocean, flooding takes place along the riverbanks, which contribute to the total capacity of storing and dissipating the surge; as the surge continues to move upstream, less volume of water reaches the project site compared to the mouth of the Potomac. Hence, a reduction in storm surge at the project site associated with SLR.

For all scenarios, the largest reduction in storm surge was less than 9% given 5 feet of SLR. However, for the 2050 SLR projection, only a 3% reduction in surge was observed, which is considered negligible. Thus, to be conservative for future design conditions, it is recommended to use a one-to-one increment of SLR on storm surge.

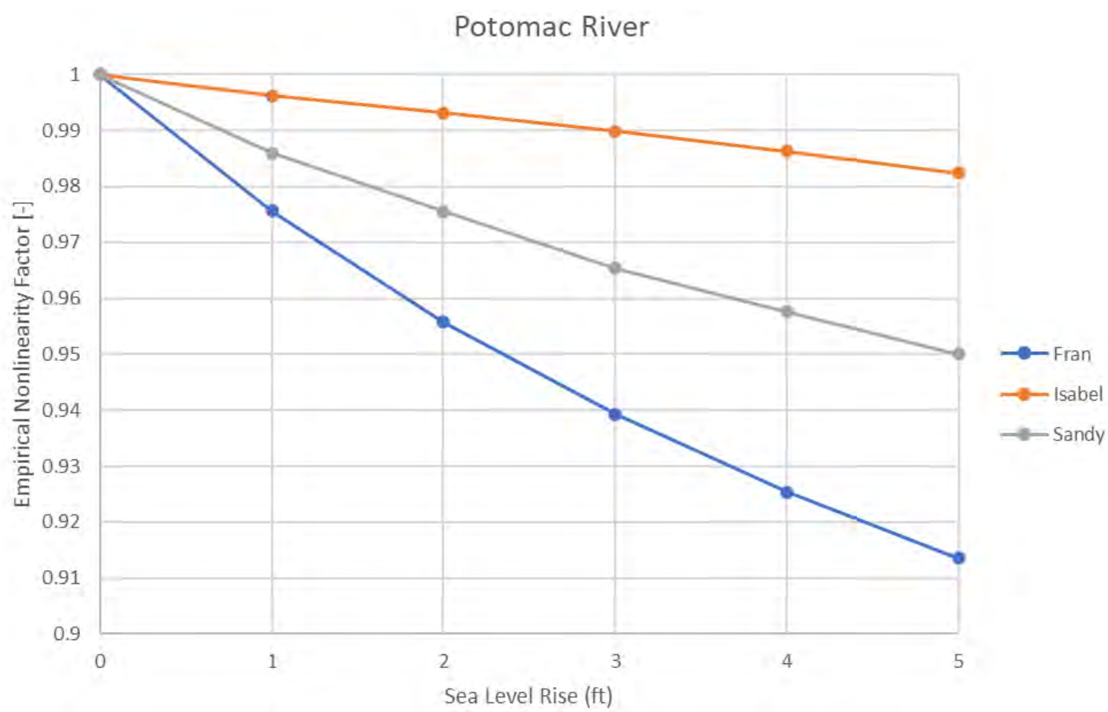

FIGURE 9. NON-LINEARITY FACTOR RESULTS AT WEST POTOMAC PARK FOR ALL MODEL SCENARIOS.

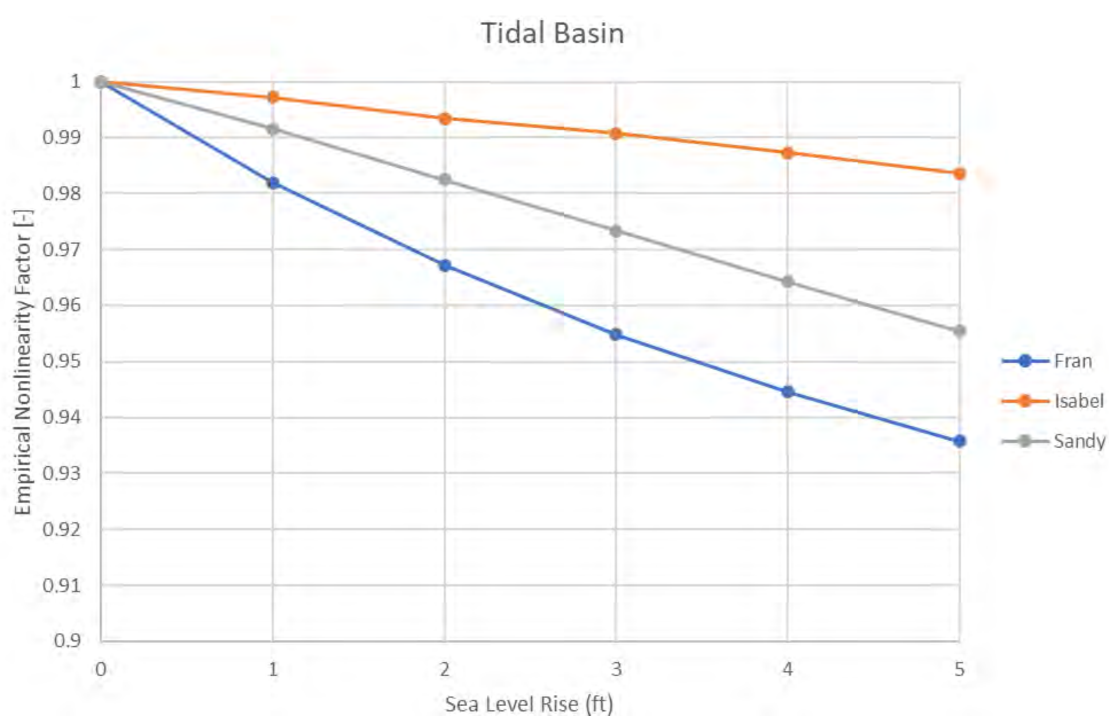

FIGURE 10. NON-LINEARITY FACTOR RESULTS AT TIDAL BASIN FOR ALL MODEL SCENARIOS.

3.4. Currents and Scour

The hydrodynamic model was employed to evaluate the currents at the project site with and without SLR conditions. The design scenarios shown in Table 2 summarize the boundary conditions and results for the West Potomac Park and Tidal Basin (see Appendix D for further results). Using the methods described in the Coastal Engineering Manual (USACE, 2002), the design current velocities were used to calculate the rock sizing associated with current-driven scour with a conservative slope of 1V:2H. Overall, our analysis found that:

- West Potomac Park: peak currents at the seawall are controlled by high river discharge from upstream Potomac River. Downstream water levels show little impacts on the current velocity; thus, the peak currents are not expected to increase by SLR.
- Tidal Basin:
  - Currents of 1.4 ft/sec were found adjacent to inlet gates when the gates are open, but this is limited to the area at and near the gates.
  - Peak currents at the seawall are much lower during high river discharge events due to the flow control of the one-way inlet gates. The peak currents at the Tidal Basin are associated with large increase gradients in water level. Overall peak currents away from the inlet gates were 0.7 ft/sec.
  - No significant change in peak current was observed under the SLR scenario.
- The scour risks and stability of the bottom rocks were checked upon the peak currents at the seawall site. Given that the existing riprap protection at the project site consists of 18-inch rock (per field observations), the currents need to exceed 9.4 ft/s to cause any stability issues. The simulated peak currents both at the Westpark Potomac Park and Tidal Basin are all less than 9.4 ft/s, so no scour risk is associated with the existing 18-inch rock. Given the maximum velocities within the Tidal Basin and along the wall in the Potomac River, bedding/scour stone should have a minimum rock size (D_{50}) of 7-inch (see Table 2).

TABLE 2. DESIGN CURRENTS BOUNDARY CONDITION AND RESULTS.

Site	Downstream Boundary	Upstream Boundary	Peak Currents	Rock Size
West Potomac Park	Extreme Low Tide (1967) Lowest WL = -6.4 ft NAVD88	50-yr Discharge per FEMA FIS at Little Falls $Q_{50\text{-}yr}$ = 395,000 cfs	7.5 ft/sec	7 in
Tidal Basin at inlet gates	Large WL variation (2018) WL Rising = 3 ft	No Flow	1.4 ft/sec	1 in
Tidal Basin away from inlet gates	Large WL variation (2018) WL Rising = 3 ft	No Flow	<0.7 ft/sec 1.4	1 in

4. Waves

4.1. Purpose and Goals

Wave conditions in the Tidal Basin and Potomac River are dominated by short-period locally generated wind waves. In this study, the extreme waves were fetch-limited and simulated using a spectral wave model. The model description and set up is provided in Appendix B and E. The purpose of wave modelling at the project site is to provide wave design conditions for:

- Wave overtopping
- Wave loading

4.2. Waves

The modeled wave results were extracted at multiple locations along the seawall at the Tidal Basin and West Potomac Park. The largest wave heights at West Potomac Park and Tidal are presented in the section; see Appendix E for more results.

4.2.1. Waves Under Present-day Sea Levels

The maximum significant wave height and highest peak wave period with 25-year return period are sown in Figure 11 and Figure 12, respectively. These wave conditions are the maximum values among all simulated scenarios associated with all wind directions. As shown, the waves at West Potomac Park are significantly higher than those in the Tidal Basin due to a longer fetch. The peak wave periods are below 4 sec, consistent with typical wave periods in wind-wave dominant areas. Appendix F provides the maximum significant wave heights and peak wave periods under 10-year, 50-year, and 100-year return periods.

Table 3 presents the maximum significant wave height, peak wave period, and mean wave direction at the West Potomac Park and Tidal Basin seawalls for 5-, 10-, 25-, 50-, and 100-year return periods. The 100-year wave height along the seawall does not exceed 3 ft inside the Tidal Basin, and just a little over 4 ft at West Potomac Park. The largest wave along the seawall at West Potomac Park comes from south-southeast (SSE) while the dominant wave direction along seawall in the Tidal Basin varies from northwest (NW) to east (E) depending on the longest fetch direction.

Overall, the largest wave heights, wave loads, and overtopping rates were found along West Potomac Park at southern locations near the Inlet Gate. This is consistent with the larger deterioration of the seawall seen in this area.

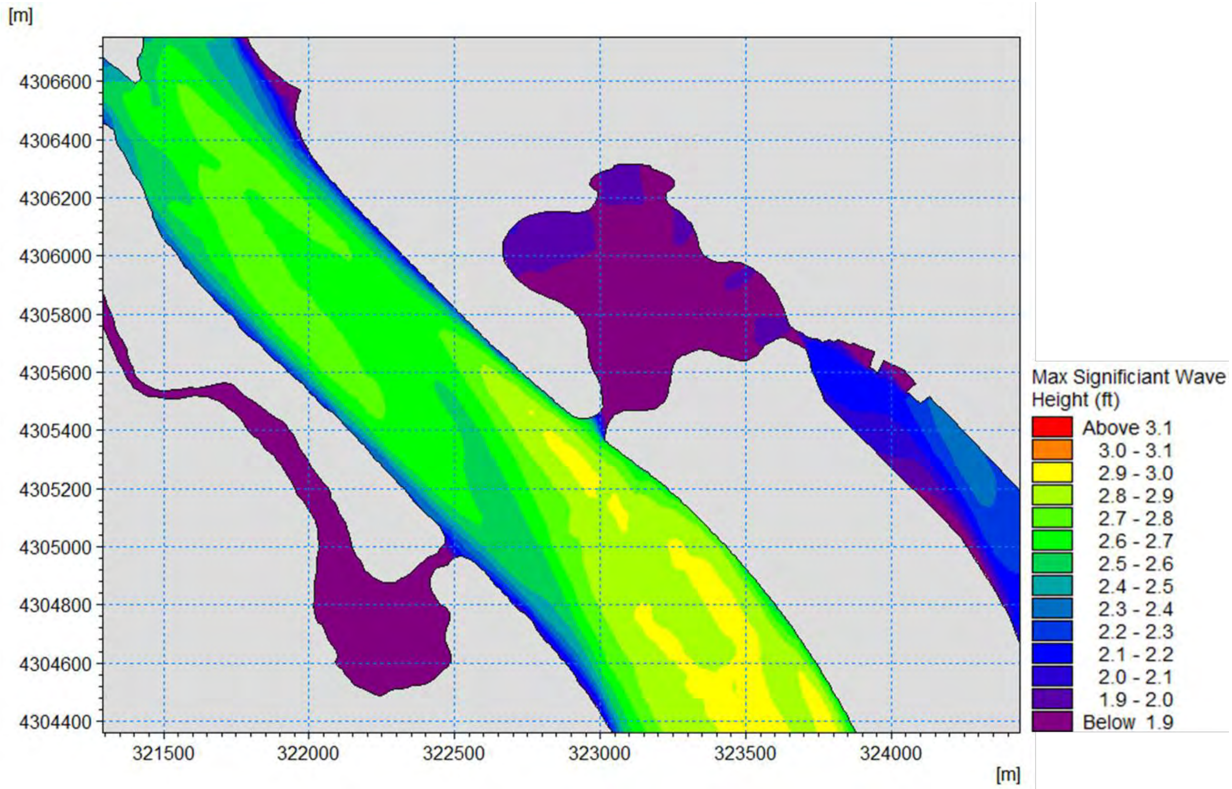

FIGURE 11. EXTREME WAVE MODELING RESULTS (25-YR, MAX SIGNIFICANT WAVE HEIGHT) – NO SLR.

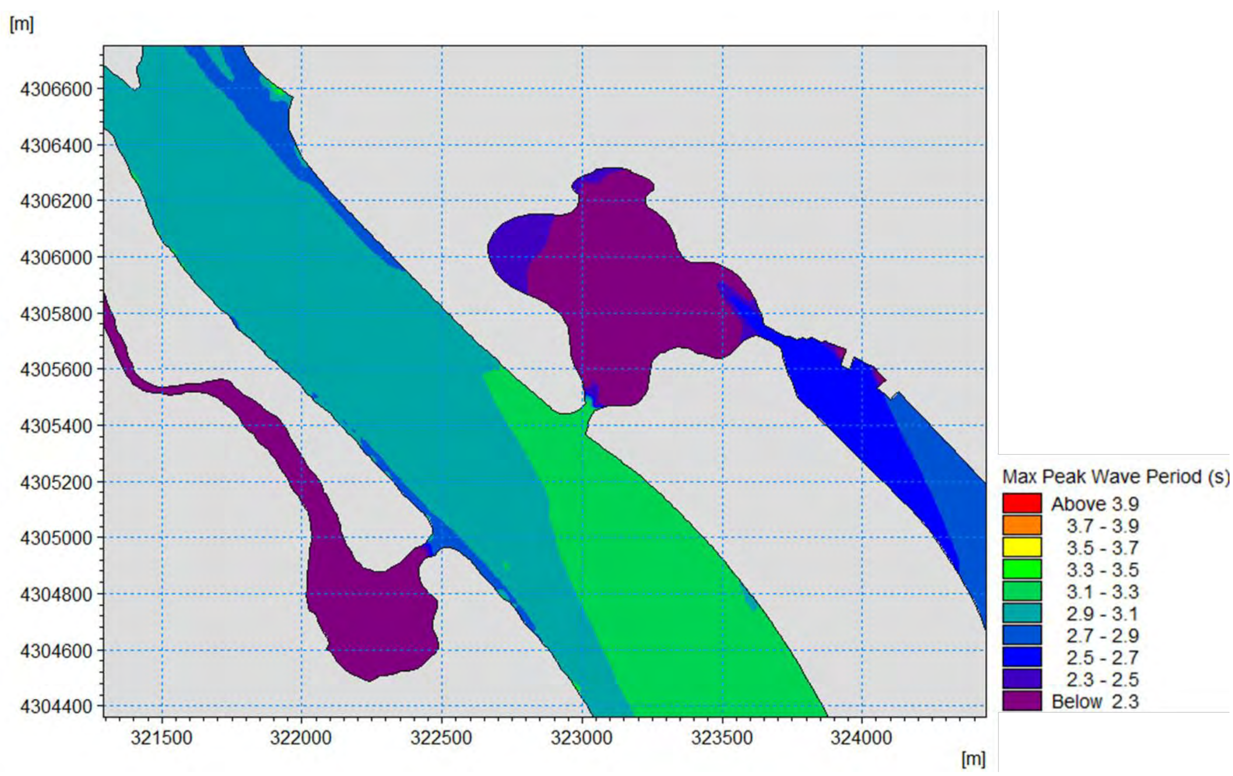

FIGURE 12. EXTREME WAVE MODELING RESULTS (25-YR, MAX PEAK WAVE PERIOD) – NO SLR.

TABLE 3. EXTREME WAVE CONDITIONS AT THE PROJECT SITE – PRESENT-DAY SEA LEVELS.

Return Period	West Potomac Park Seawall*			Tidal Basin Seawall*		
	Sig. Wave Height (ft)	Peak Wave Period (s)	Wave Direction (from deg N)	Sig. Wave Height (ft)	Peak Wave Period (s)	Wave Direction (from deg N)
100 years	4.1	3.7	SSE	2.9	2.6	N
50 years	3.4	3.4	SSE	2.4	2.5	N
25 years	2.8	3.1	SSE	2	2.3	N
10 years	2.2	2.8	SSE	1.6	2.1	N
5 years	1.8	2.6	SSE	1.4	2.1	N

*Waves at RP1 are presented here for West Potomac Park. Waves at BP5 are presented here for Tidal Basin.

4.2.2. Wave Results with Sea Level Rise by 2050

Model results with SLR by 2050 (1.3 ft increase since 2005) are presented in this section. The maximum significant wave height and highest peak wave period under 25-year return period with SLR are illustrated in Figure 13 and Figure 14, respectively. Appendix F provides the plots of maximum wave heights and peak wave periods under 10-year, 50-year, and 100-year return periods; Table 4 includes the modeled wave conditions at the seawalls of West Potomac Park and Tidal Basin with +1.3 ft SLR. The projected SLR slightly increase wave heights along the seawall of West Potomac Park (less than 0.1 foot) and little to no effect on wave period or direction; at the seawalls along the Tidal Basin, no changes are observed to the wave results due to SLR.

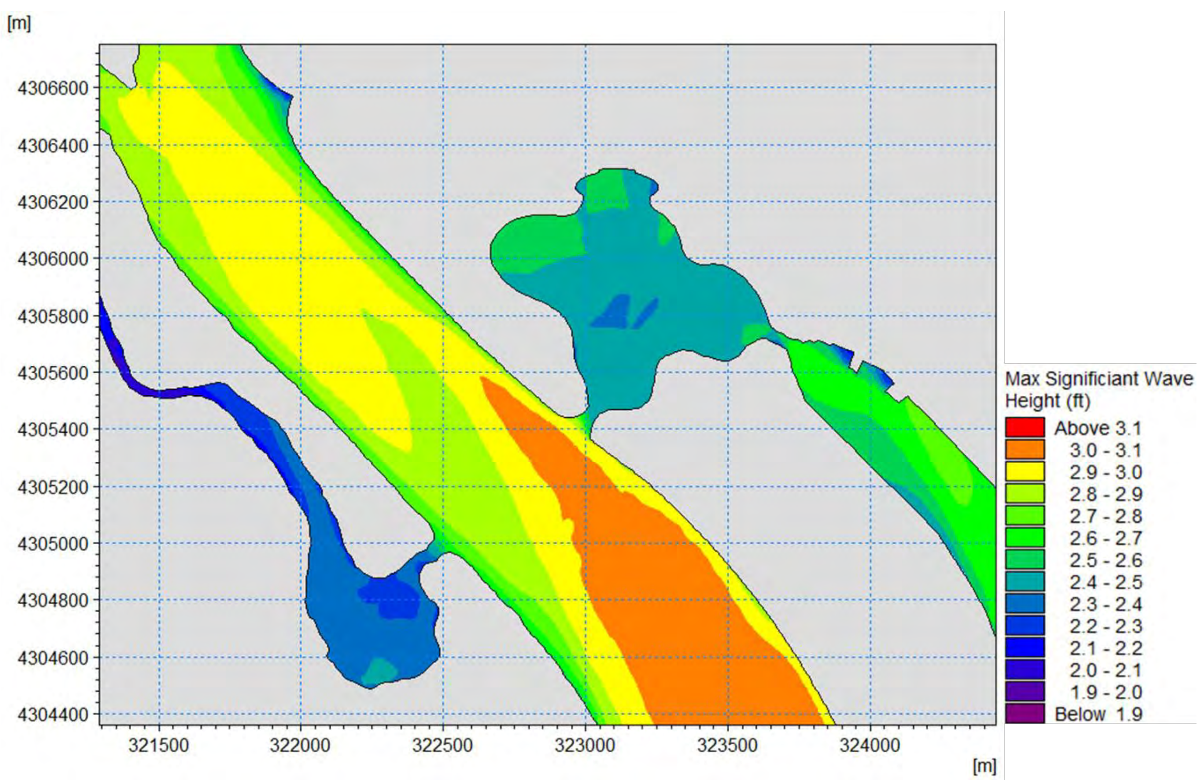

FIGURE 13. EXTREME WAVE MODELING RESULTS (25-YR, MAX SIGNIFICANT WAVE HEIGHT) , +1.3 FT SLR.

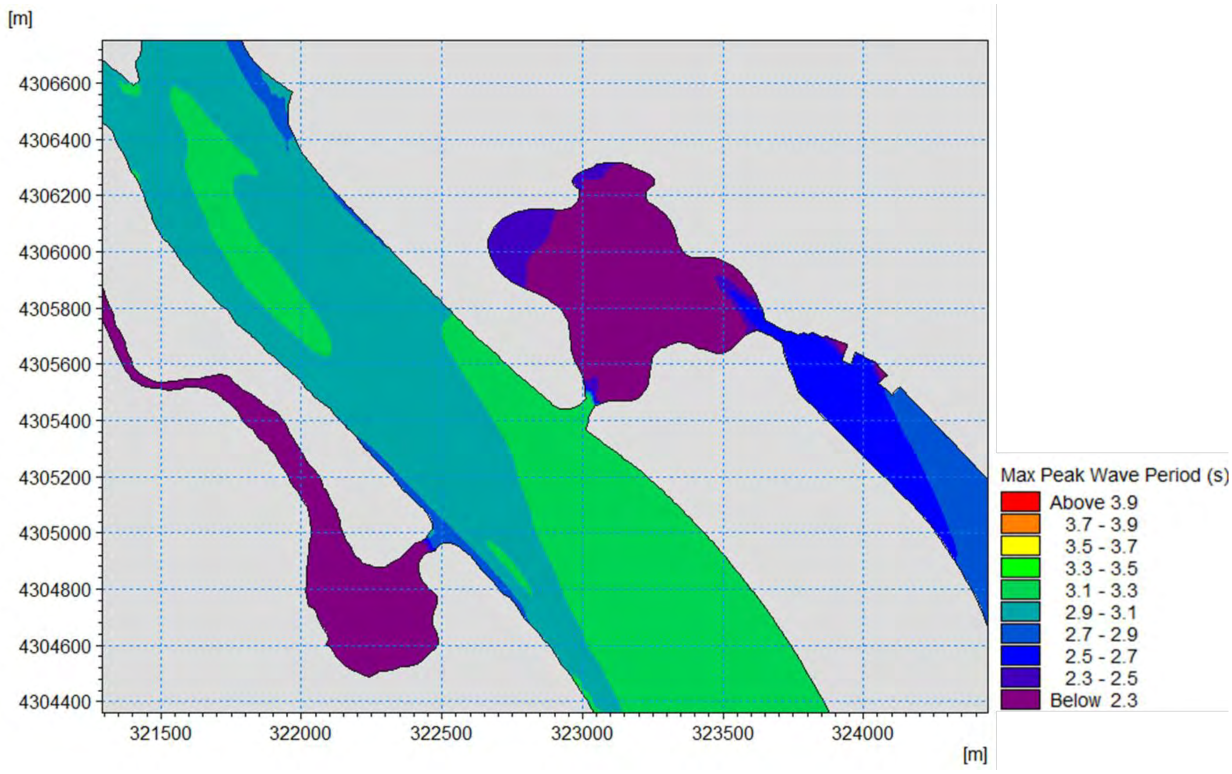

FIGURE 14. EXTREME WAVE MODELING RESULTS (25-YR, MAX PEAK WAVE PERIOD) , +1.3 FT SLR.

TABLE 4. EXTREME WAVE CONDITIONS AT THE PROJECT SITE, +1.3 FT SLR.

Return Period	**West Potomac Park Seawall***			**Tidal Basin Seawall***		
	Sig. Wave Height (ft)	Peak Wave Period (s)	Wave Direction (from deg N)	Sig. Wave Height (ft)	Peak Wave Period (s)	Wave Direction (from deg N)
100 years	4.2	3.7	SSE	2.9	2.6	N
50 years	3.4	3.4	SSE	2.4	2.5	N
25 years	2.9	3.2	SSE	2	2.3	N
10 years	2.3	2.8	SSE	1.6	2.1	N
5 years	2	2.8	SSE	1.4	2.1	N

*Waves at RP1 are presented here for West Potomac Park. Waves at BP5 are presented here for Tidal Basin.

4.3. Wave Overtopping

Wave overtopping was estimated following methods described in the EurOtop (2018) manual for vertical walls along West Potomac Park and Tidal Basin with TOW elevations at +5.5 ft and +4.75 ft NAVD88, respectively. To determine the range of potential overtopping rates, water surface elevations were varied from mean lower low water (-1.40 ft NAVD88) to the maximum water surface elevation associated with 5-, 10-, 25-, 50-, and 100-year return period wave conditions at present-day sea levels and with SLR in 2050 (SSP4-8.5 at +1.3 ft of SLR since 2005). Results were calculated at extraction points shown in Figure 32; maximum results are shown in this section and complete results in Appendix G.

4.3.1. Present-day Sea Level

Consistent with previous preliminary overtopping estimates, wave overtopping rates are higher along West Potomac Park at southern than at northern locations, and highest along the southern portion of the Tidal Basin (i.e., BP5) due to the combination of larger available fetch and relatively direct wave incidence. West Potomac Park overtopping rates are shown in Figure 15 for the controlling location (RP1) for a range of water levels and wave return periods with the EurOtop (2018) thresholds for erosion; water levels associated with the start of some erosion at the wall crest are provided in Table 6. Tidal Basin overtopping results are shown in Table 5 and Figure 16.

Along West Potomac Park, it was found that erosion of soil with patchy grass cover would occur for a wave overtopping event with a return period greater than 10-year; an overtopping return period greater than 10 years is expected along West Potomac Park for a 10-year return period wave event coinciding with water levels above the proposed +5.5 ft NAVD88 wall crest (roughly 7-yr return period at present-day).

In the Tidal Basin, no erosive overtopping is expected for 5-year wave conditions at present-day sea levels, and erosion of soil with patchy grass cover would occur for a wave overtopping event with a return period greater than 50-year. An overtopping return period greater than 50-year is expected along the Tidal Basin for a 50-year return period wave event coinciding with water levels above the proposed +4.75 ft NAVD88 wall crest (roughly 4-year return period at present day). Waves with water levels above the wall crest, however, are generally expected to produce some erosion of patchy grass cover, with erosion increasing with water level and wave return period.

TABLE 5. TIDAL BASIN PRESENT-DAY SEA LEVEL - WATER LEVELS ASSOCIATED WITH WAVE OVERTOPPING EROSION THRESHOLD FOR DISCRETE WAVE RETURN PERIODS.

Wave Return Period	**Wave Height** (ft)	**Limiting WL Elevation** (ft NAVD88)	**Approx. WL Return Period (year)**
5-yr	1.4 ft	-	-
10-yr	1.6 ft	5.1 ft	5 yr
25-yr	2.0 ft	5.0 ft	4 yr
50-yr	2.4 ft	4.8 ft	3 yr
100-yr	2.9 ft	4.5 ft	1 yr

TABLE 6. WEST POTOMAC PARK PRESENT-DAY SEA LEVEL – WATER LEVELS ASSOCIATED WITH WAVE OVERTOPPING EROSION THRESHOLD FOR DISCRETE WAVE RETURN PERIODS.

Wave Return Period	**Wave Height** (ft)	**Limiting WL Elevation** (ft NAVD88)	**Approx. WL Return Period** (year)
5-yr	1.8 ft	-	-
10-yr	2.2 ft	5.6 ft	7 yr
25-yr	2.8 ft	5.3 ft	6 yr
50-yr	3.4 ft	5.1 ft	5 yr
100-yr	4.1 ft	4.7 ft	2 yr

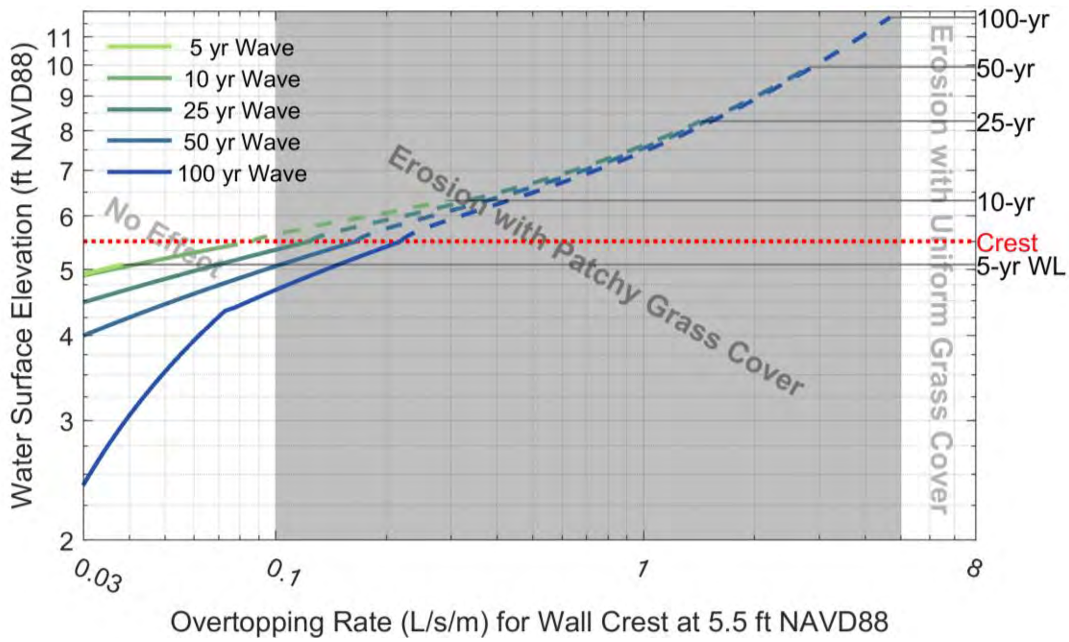

FIGURE 15. WEST POTOMAC PARK PRESENT-DAY SEA LEVEL - OVERTOPPING RATES FOR THE SOUTHERNMOST LOCATION FOR A RANGE OF WATER LEVELS AND WAVE RETURN PERIODS RELATIVE TO EUROTOP (2018) THRESHOLDS FOR EROSION AT THE WALL CREST. EXTREME WATER LEVELS ARE SHOWN AS GREY LINES AND ARE LABELLED ON THE RIGHT SIDE OF THE FIGURE. GREY SHADING INDICATES OVERTOPPING IS EXPECTED TO ERODE PATCHY GRASS COVER ON SEAWALL CRESTS BUT NOT CAUSE DAMAGE TO UNIFORM GRASS COVER.

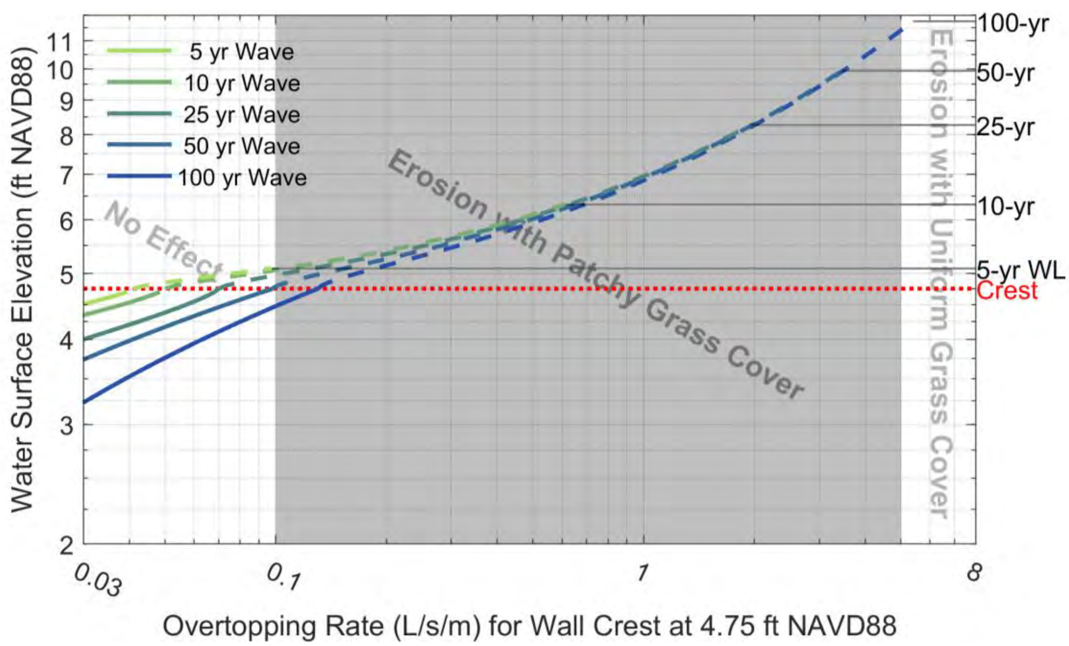

FIGURE 16. TIDAL BASIN PRESENT-DAY SEA LEVEL - OVERTOPPING RATES FOR THE SOUTHERNMOST LOCATION OVER A RANGE OF WATER LEVELS AND WAVE RETURN PERIODS RELATIVE TO EUROTOP (2018) THRESHOLDS FOR EROSION AT THE WALL CREST. EXTREME WATER LEVELS ARE SHOWN AS GREY LINES AND ARE LABELLED ON THE RIGHT SIDE OF THE FIGURE. GREY SHADING INDICATES OVERTOPPING IS EXPECTED TO ERODE PATCHY GRASS COVER ON SEAWALL CRESTS BUT NOT CAUSE DAMAGE TO UNIFORM GRASS COVER.

4.3.2. Future (2050) Conditions with Sea Level Rise

Site-specific wave heights along West Potomac Park and Tidal Basin are generally projected to increase by less than 5% with SLR by 2050; changes in wave period and wave direction with SLR are similarly small (typically less than 2%). Because the extreme wave conditions remain similar, the water levels associated with the initiation of wall-top soil erosion with patchy grass cover with SLR are nearly identical to the results for present-day sea level. Instead, the greatest effect of SLR on overtopping is through increased extreme water levels. As shown in Figure 17 and Figure 18, future water levels associated with return periods of 5-year or greater are projected to be higher than the wall crest.

Overtopping rates for all tested combinations of extreme wave conditions and extreme water levels are projected to cause at least some erosion at the wall crest, in areas of patchy grass cover. However, these estimates of erosion of wall-top soil will largely depend on the presence (during rapid inundation) or absence (during slow inundation) of rapid water currents within the flooded area above the wall crest.

TABLE 7. WEST POTOMAC FUTURE (2050) SEA LEVEL - WATER LEVELS ASSOCIATED WITH WAVE OVERTOPPING EROSION THRESHOLD FOR DISCRETE WAVE RETURN PERIODS.

Wave Return Period	**Wave Height** (ft)	**Limiting WL Elevation** (ft NAVD88)	**Approx. WL Return Period**
5-yr	2.0 ft	5.7 ft	1 yr
10-yr	2.3 ft	5.6 ft	<1 yr
25-yr	2.9 ft	5.3 ft	<1 yr
50-yr	3.4 ft	5.1 ft	<1 yr
100-yr	4.2 ft	4.6 ft	<1 yr

TABLE 8. TIDAL BASIN FUTURE (2050) SEA LEVEL - WATER LEVELS ASSOCIATED WITH WAVE OVERTOPPING EROSION FOR DISCRETE WAVE RETURN PERIODS.

Wave Return Period	**Wave Height** (ft)	**Limiting WL Elevation** (ft NAVD88)	**Approx. WL Return Period**
5-yr	1.4 ft	5.1 ft	<1 yr
10-yr	1.6 ft	5.1 ft	<1 yr
25-yr	2.0 ft	5.0 ft	<1 yr
50-yr	2.4 ft	4.8 ft	<1 yr
100-yr	2.9 ft	4.5 ft	<1 yr

Preventative measures to reduce wave overtopping damage other than raising the seawall elevation may include: hardening a larger surface of West Potomac Park uplands, adding a splash pad on the top of wall, adding an impermeable parapet/guardrail on the top of wall.

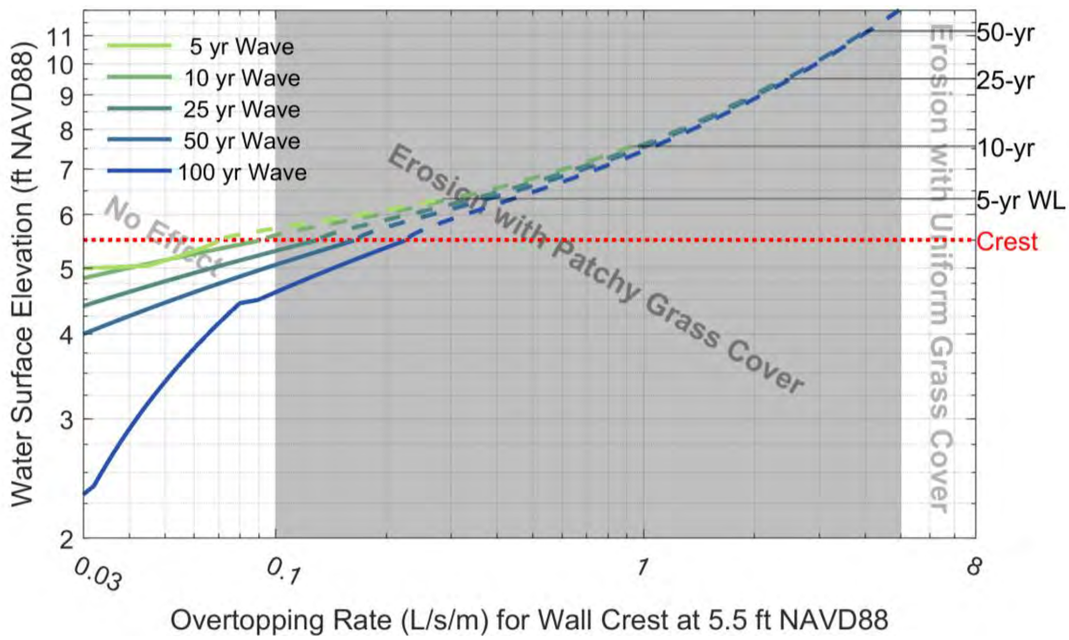

FIGURE 17. WEST POTOMAC PARK FUTURE (2050) SEA LEVEL - OVERTOPPING RATES FOR THE SOUTHERNMOST LOCATION OVER A RANGE OF WATER LEVELS AND WAVE RETURN PERIODS RELATIVE TO EUROTOP (2018) THRESHOLDS FOR EROSION AT THE WALL CREST. EXTREME WATER LEVELS ARE SHOWN AS GREY LINES AND ARE LABELLED ON THE RIGHT SIDE OF THE FIGURE. GREY SHADING INDICATES OVERTOPPING IS EXPECTED TO ERODE PATCHY GRASS COVER ON SEAWALL CRESTS BUT NOT CAUSE DAMAGE TO UNIFORM GRASS COVER.

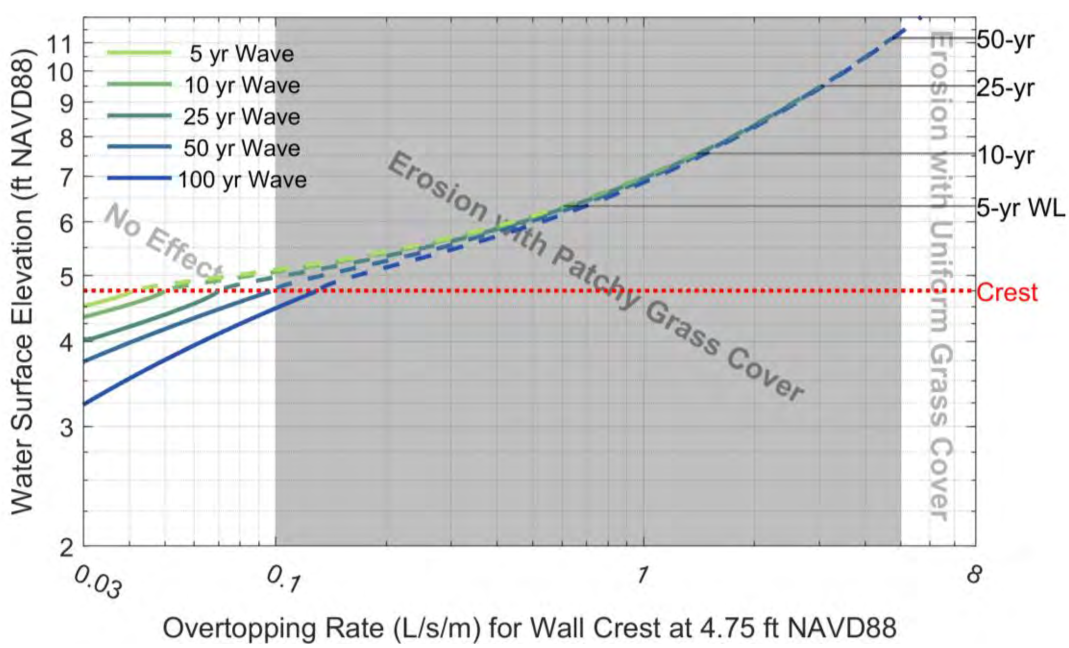

FIGURE 18. TIDAL BASIN FUTURE (2050) SEA LEVEL - OVERTOPPING RATES FOR THE SOUTHERNMOST LOCATION OVER A RANGE OF WATER LEVELS AND WAVE RETURN PERIODS RELATIVE TO EUROTOP (2018) THRESHOLDS FOR EROSION AT THE WALL CREST. EXTREME WATER LEVELS ARE SHOWN AS GREY LINES AND ARE LABELLED ON THE RIGHT SIDE OF THE FIGURE. GREY SHADING INDICATES OVERTOPPING IS EXPECTED TO ERODE PATCHY GRASS COVER ON SEAWALL CRESTS BUT NOT CAUSE DAMAGE TO UNIFORM GRASS COVER.

4.4. Wave Loading

Wave loads were calculated following methodology by Goda (1974) as outlined in the CEM (USACE, 2002) and as described in Appendix H. Four combined wave and water level retorn periods (10-, 25-, 50-, and 100-year events) were used to compute the horizontal wave-induced horizontal forces for each of the locations depicted in Figure 32. The maximum wave loads for West Potomac Park and Tidal Basin are presented din this section.

Results for the wave load analysis are summarized in Table 9 and Table 10 which provides the maximum horizontal wave load, water level, and associated wave crest elevation approximation at which the maximum load occurs; bay bottom (Z_{toe}), and top of wall (Z_{crest}) elevations are also provided as reference. Magnitude of the estimated wave loads vary depending on the assumed design event but generally range between 2.5 kip/ft to 1.0 kip/ft for the West Potomac Park, and between 2.3 kip/ft to 0.9 kip/ft for the Tidal Basin.

Wave loads in the Potomac River wall are largest at the south and decrease towards the. Meanwhile, although wave heights fronting the Tidal Basin are 20% -30% smaller, wave loads are of comparable magnitude to those in the Potomac River wall. This is primarily explained by the fact that the design waves approach the wall in a more perpendicular direction consistently receiving the largest loads, where wave incidence is 1° and 35° (compared to an average of 66° at the Potomac River Wall).

TABLE 9. WAVE LOADS AND ASSOCIATED PARAMETERS AT WEST POTOMAC PARK.

Point ID	Bay Bottom (Z_{toe}, ft, NAVD88)	Top of Wall (Z_{crest}, ft. NAVD88)	Return Period (yr.)	Max. Horizontal Force (kip/ft)	Still Water Level (SWL, ft, NAVD88)	Design Wave Height (ft)	Approx. Wave Crest (ft, NAVD88)
RP1	-3	5.5	100	2.5	2.5	7.4	7.7
RP1	-3	5.5	50	1.9	1.5	6.1	5.8
RP1	-3	5.5	25	1.4	1.5	5.0	5.0
RP1	-3	5.5	10	1.0	0.0	4.0	2.8

TABLE 10. WAVE LOADS AND ASSOCIATED PARAMETRS AT THE TIDAL BASIN.

Point ID	Bay Bottom (Z_{toe}, ft, NAVD88)	Top of Wall (Z_{crest}, ft. NAVD88)	Return Period (yr.)	Max. Horizontal Force (kip/ft)	Still Water Level (SWL, ft, NAVD88)	Design Wave Height (ft)	Approx. Wave Crest (ft, NAVD88)
BP5	-4	4.5	100	2.3	0.0	5.2	3.7
BP5	-4	4.5	50	1.7	-1.0	4.2	1.9
BP5	-4	4.5	25	1.3	-1.4	3.6	1.1
BP5	-4	4.5	10	0.9	-1.4	2.9	0.6

Although results presented in this section use wave heights as modeled under present-day sea level conditions (Section 4.2.1), it is assumed that the provided wave loads are also applicable with SLR because:

1. Wave height increase with the projected SLR is negligible. On average, there is only a 1% increase on the modeled wave heights under future SLR conditions with respect to present-day sea level conditions.
2. Highest wave loads are not associated with high water levels. The maximum wave-induced loads on the walls occur with water levels ranging from -1.4 feet (NAVD88) to +2.0 feet (NAVD88), which are below the top of wall elevations. Therefore, any future increment on water levels will not have an effect on the wave-induced loads.

5. References

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Appendix A: Tidal Basin Inlet and Outlet Gates

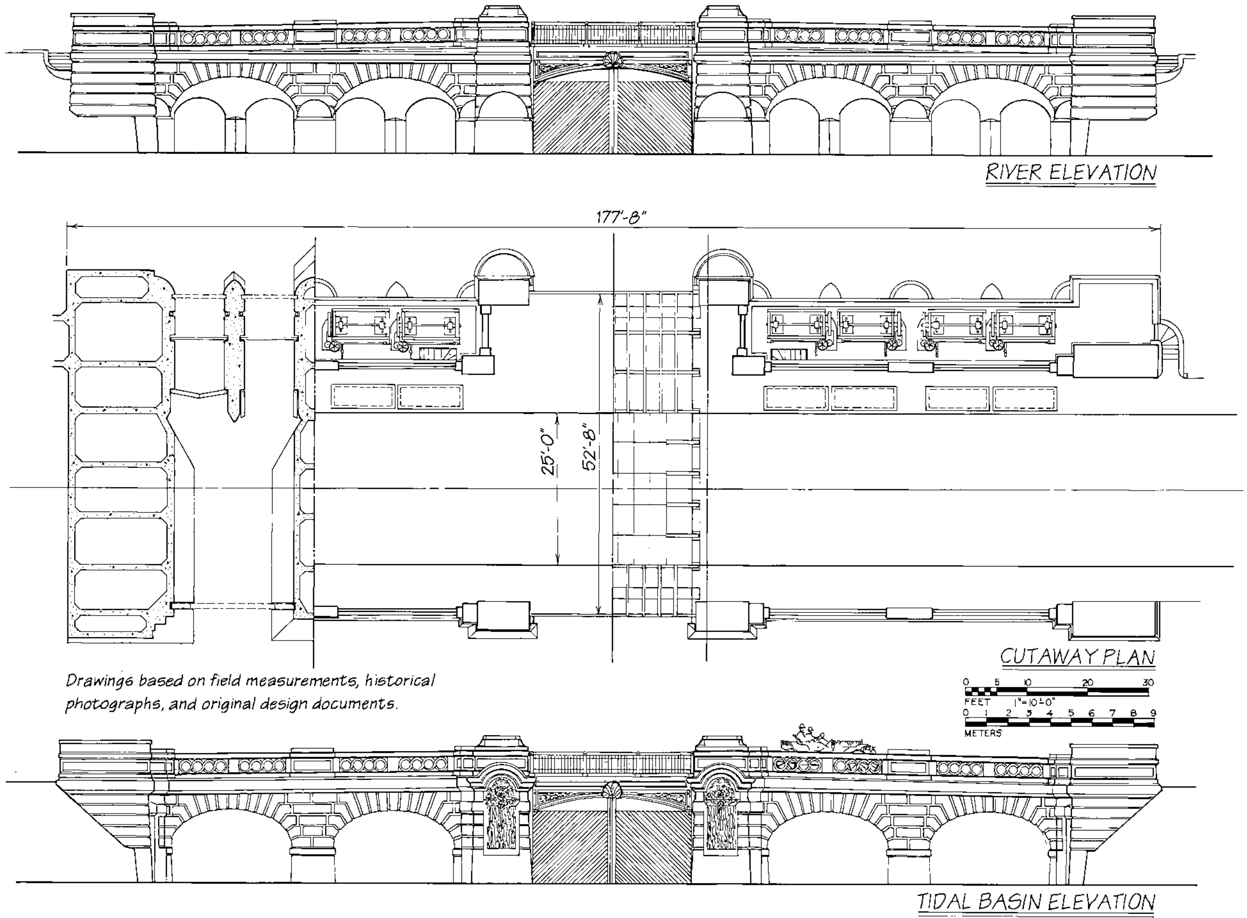

FIGURE 19. TIDAL BASIN INLET BRIDGE GEOMETRY (HEAR NPS, 2000).

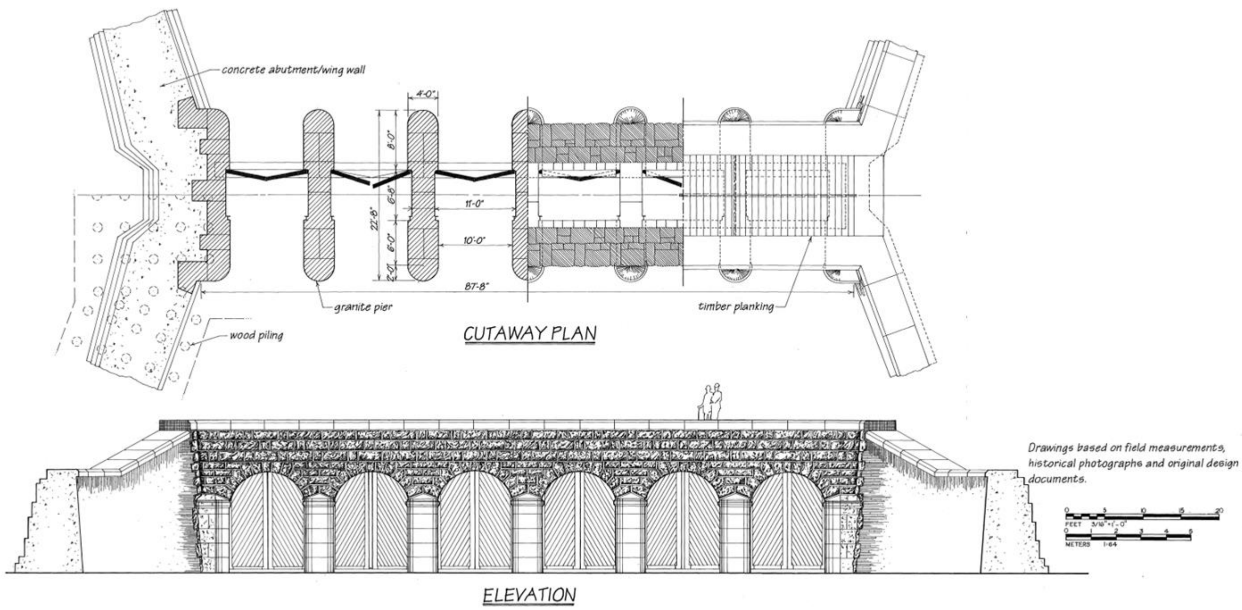

FIGURE 20. TIDAL BASIN OUTLET BRIDGE GEOMETRY (HEAR NPS, 2000).

TABLE 11. SUMMARY OF GATE GEOMETRIES AND OPERATIONS.

Descriptor	Inlet Gate	Outlet Gate
Type of gate	a. Automatic steel-plated swing gates b. Iron curtain gates c. Vessel gate	a. Double-leaf gates that close during rising tides
Opening size	a. h = NA, w = NA b. h = NA, w = NA c. h = 12 ft, w = 26 ft	a. h = 12 ft, w = 10 ft
Number of openings	a. 8 b. 8 c. 1	a. 6
Original operation intent	a. Open during flood tide and close during ebb tide b. Manually operated through wheels and cranes. Intended to close off the Tidal Basin during flooding c. Allows passage of vessels through central span	a. Automatically closed by the rising of the high tide
Current operation status	a. NA b. NA c. Permanently closed	a. 3 out of 6 gates appear to remain open at all times. Existing wooden structure (believed to block the passing of kayakers) constricts the flow

(NA = not available)

Appendix B: Hydrodynamic Model Development

Model Development

The 2D MIKE21 Flexible Mesh (FM) model suite developed by DHI was selected to study hydrodynamics for the NAMA Seawall Project. MIKE21 FM is a modeling suite capable of simulating water level and velocity variations in response to a variety of environmental forcing in riverine, coastal, and estuarine environments. The Mike21 FM model is also a FEMA-accredited model that has been approved for National Flood Insurance Program usage.

In this study, the 2D depth-averaged hydrodynamic (MIKE21 FM HD) module was used to model freshwater inflow, currents, and tides. The modeling focuses on providing water levels and velocities within the project area under a wide range of upstream freshwater inflows, downstream water levels, and local winds. The 2D spectral wave (MIKE21 FM SW) module was also coupled with the HD module to account for the interactions between the hydrodynamics and wind-generated waves.

Model Domain and Bathymetry

The model domain for the NAMA Seawall Project is illustrated in Figure 21. The inlet gate connects the Tidal Basin to the Potomac River, and the outlet gate links the Tidal Basin to the Washington Channel. The entire flexible mesh has 29,133 nodes and 55,173 elements. The mesh element size varies from 18 feet at the inlet gates to 300 feet at the remote areas far away from the site. The entire model domain expands approximately 16 river miles on the Potomac River. The section of the Potomac River included in the mesh has sufficiently covered the longest fetch that generates the largest wind waves at the site. Other major tributaries to the Potomac River, such as the Anacostia River, are also included in the model domain.

Figure 22 depicts the model bathymetric surface, which was generated with multiple data sources including:

- Local survey collected early in 2022 at the project site
- U.S. Army Corps of Engineers (USACE) Hydro surveys within the federal navigation channels
- NOAA nautical chart 12289

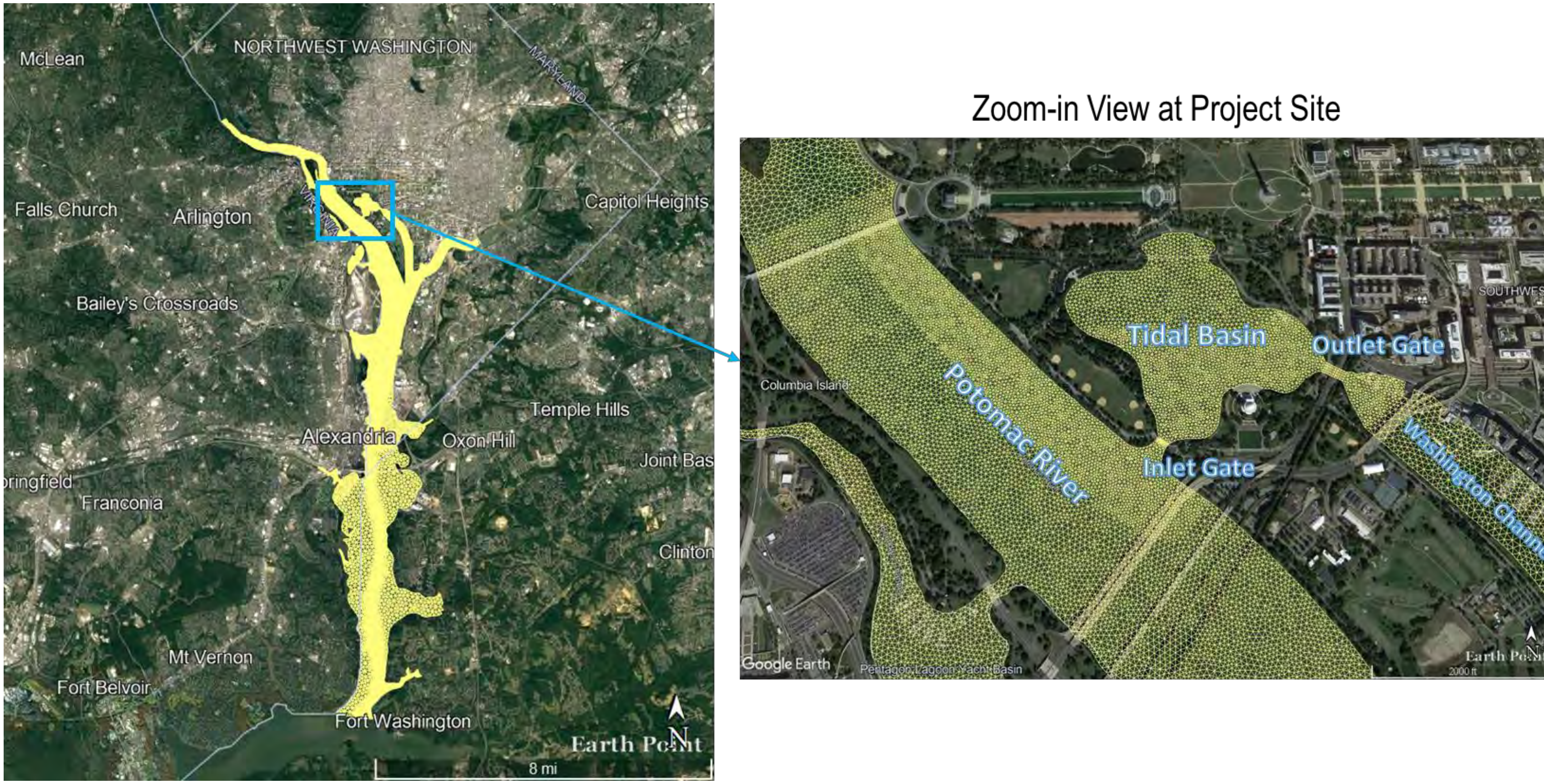

FIGURE 21. MODEL DOMAIN AND FLEXIBLE MESH.

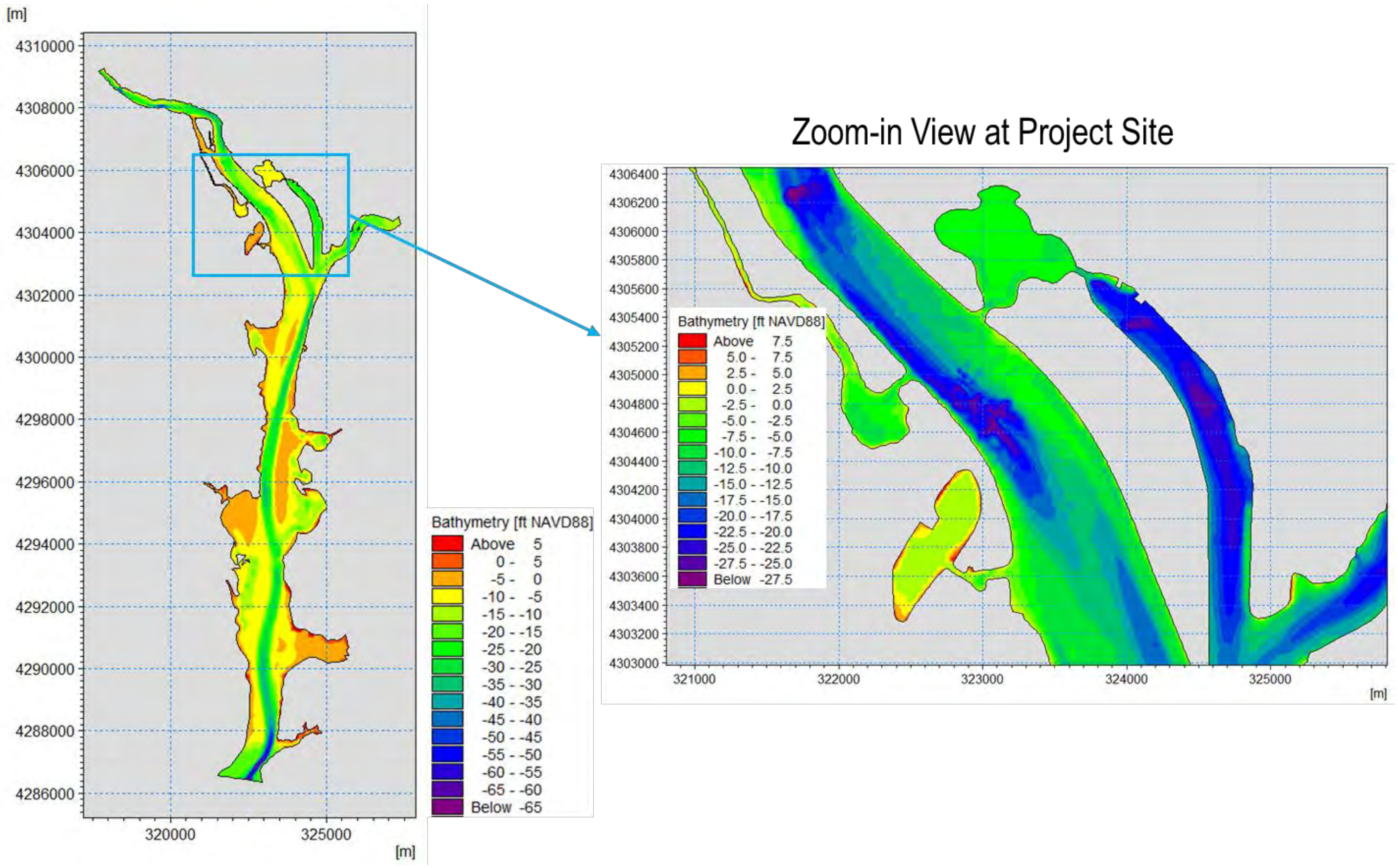

FIGURE 22. MODEL BATHYMETRIC SURFACE (UTM-18, m).

Tidal Basin Gate Setup

In this modeling study, both the inlet and outlet gates were assumed to be fully functional as designed. Due to the incomplete information regarding gate geometry, the numbers and dimensions of the inlet and outlet gates were estimated with our best knowledge and are listed below:

- Inlet Gates:
  - Central gate closed
  - Eight, 8.2-foot-wide by 4.6-foot-high openings
- Outlet Gates:
  - Six, 9.8-foot-wide by 11.8-foot-high openings

In the numerical model, the inlet and outlet gates were set as one-way flap gates; they only open under one direction of the flow and close when the flow direction reverses. The inlet gates open when the flow goes from the Potomac River to the Tidal Basin; the outlet gates open when the flow goes from the Tidal Basin to the Washington Channel.

It is found in the analysis that the performance of the gates will impact the current conditions at the gates, as well as the water flushes out to Washington Channel. With partially opened gates, the currents are expected to increase at the gate due to smaller openings; however, such impacts to the currents are more localized, no significant changes to the design currents along the seawall in Tidal Basin. With closed gates, there will be no flow passing through the tidal basin. Washington Channel could require maintenance dredging if excessive sedimentation occurs due to lack of flushing from the Tidal Basin.

Model Calibration

Besides the water level measurements at the inlet bridge, other gauged data from U.S. Geological Survey (USGS) and NOAA were collected and processed for model calibration. Table 12 provides the list of these data including both water levels and flow rates.

The calibration period is from April 29, 2022 to May 16, 2022. This is the first available data period from the RBR gauge that was deployed at the inlet bridge on the Tidal Basin side. There was a high water level event captured on May 10, 2022, as shown in Figure 23.

TABLE 12. GAUGED DATA USED IN THE NUMERICAL MODEL.

Data Type	Gauge Name	Gauge ID	Data Period	Use
Water Levels	Potomac River at Wisconsin Ave.	USGS 01647600	10/2009 - current	Calibration
	Inlet Gate, Tidal Basin Side	RBR 206153	04/27/2022 - current	Calibration
	Washington, DC	NOAA 8594900	04/1931 - current	Calibration
	Anacostia River NR Buzzard Pt	USGS 01651827	07/2019 - current	Boundary Condition
	Fourmile Run above HWY1 at Alexandria, VA	USGS 01652545	10/2007 - current	Boundary Condition
	Potomac River at Indian Head	USGS 01655480	03/2022 - current	Boundary Condition
Discharge	Little Falls	USGS 01646500	1930 – current (daily); 10/1990 – current (15-min)	Boundary Condition
	Cameron Run at Alexandria, VA	USGS 01653000	10/1990 – current (15-min)	Boundary Condition

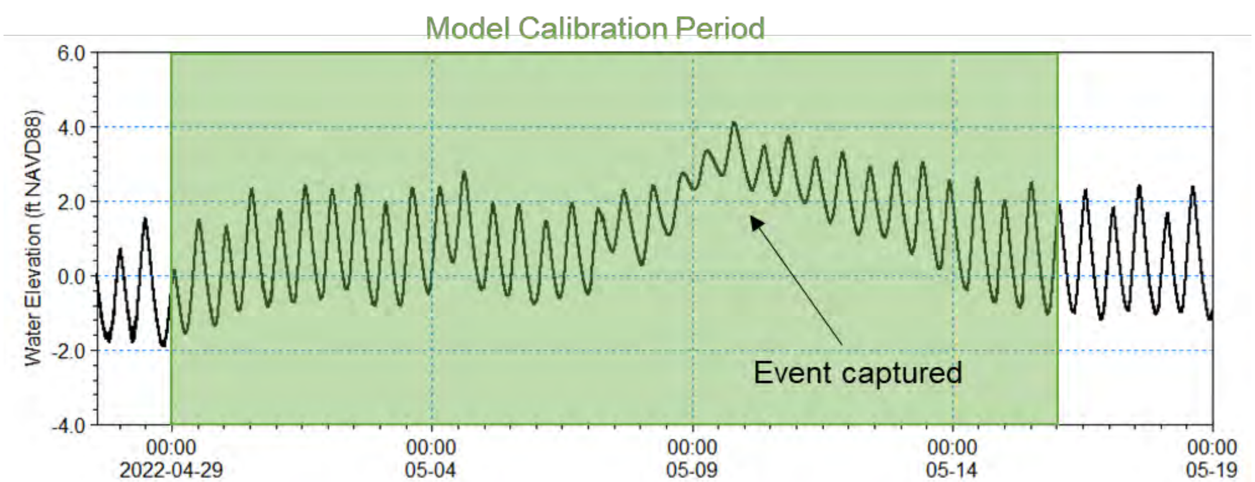

FIGURE 23. RBR WATER LEVEL MEASUREMENTS AT INLET BRIDGE (RBR GAUGE 206153) DURING CALIBRATION PERIOD.

The boundary conditions (model environmental forcing) used in the calibration included:

- Upstream: Potomac River discharge (Q) at the USGS gauge 01646500
- Downstream: Average water level (WL) between USGS gauge 01652545 and USGS gauge 01655480

The model was calibrated based on water levels at the USGS gauge 01647600 location (see Figure 24). To achieve good water level correlation between the model and gauged data, a 20% flow reduction at the upstream Potomac River discharge boundary condition was applied.

The modeled water levels were compared with field measurements at the RBR inlet bridge gauge and NOAA tide gauge in Washington Channel (NOAA 8594900). The comparison results are illustrated in Figure 25 to Figure 27. Good agreements were reached at both gauges, especially the one in Washington Channel; thus, the calibration was deemed appropriate and adequate.

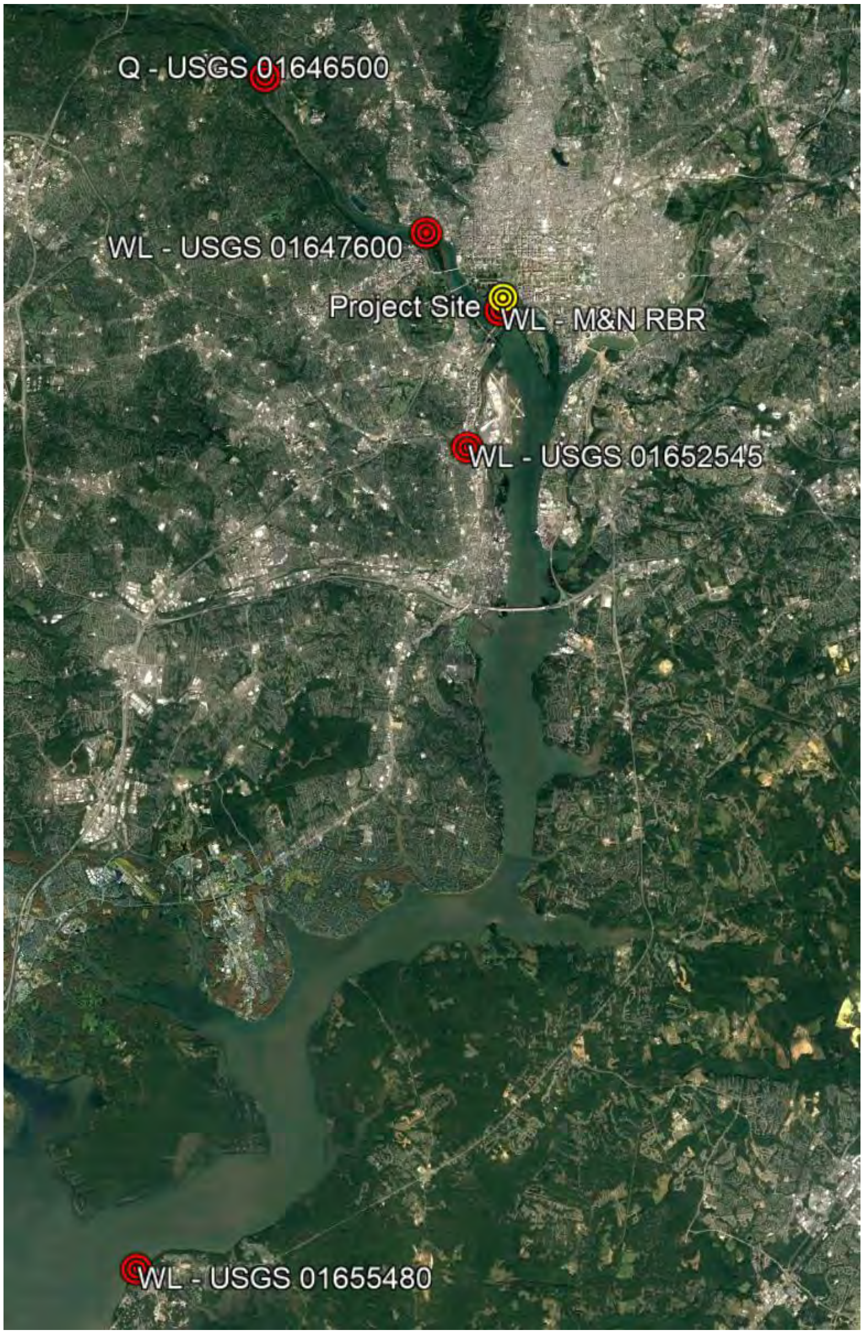

FIGURE 24. GAUGES USED IN MODEL CALIBRATION.

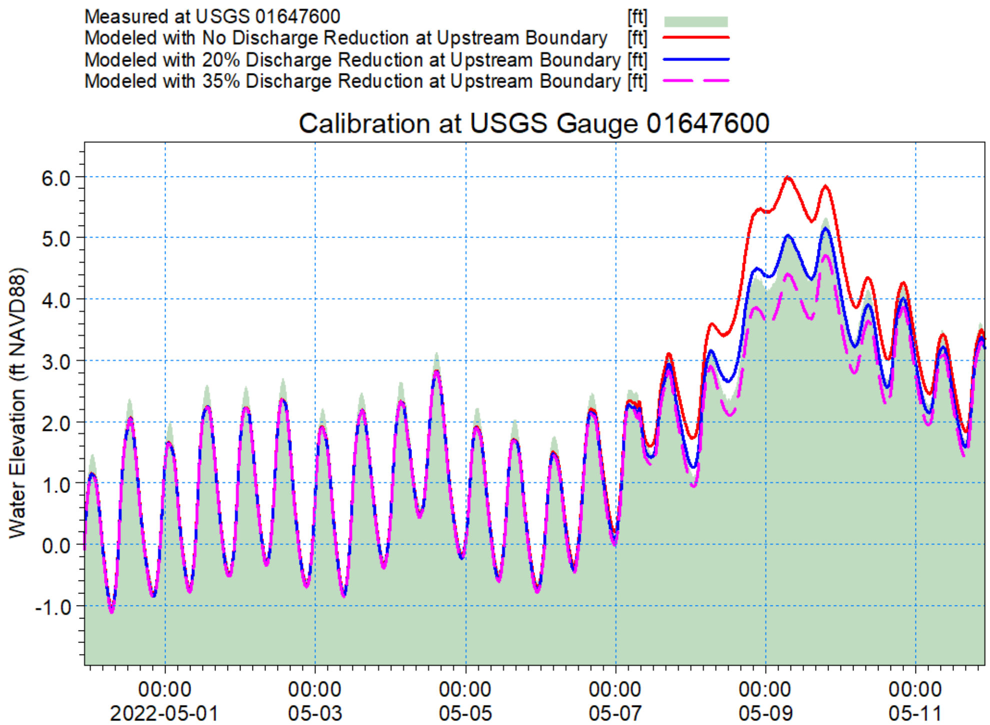

FIGURE 25. SENSITIVITY TESTS WITH UPSTREAM FLOW REDUCTION – WATER LEVEL COMPARISON AT USGS GAUGE 01647600.

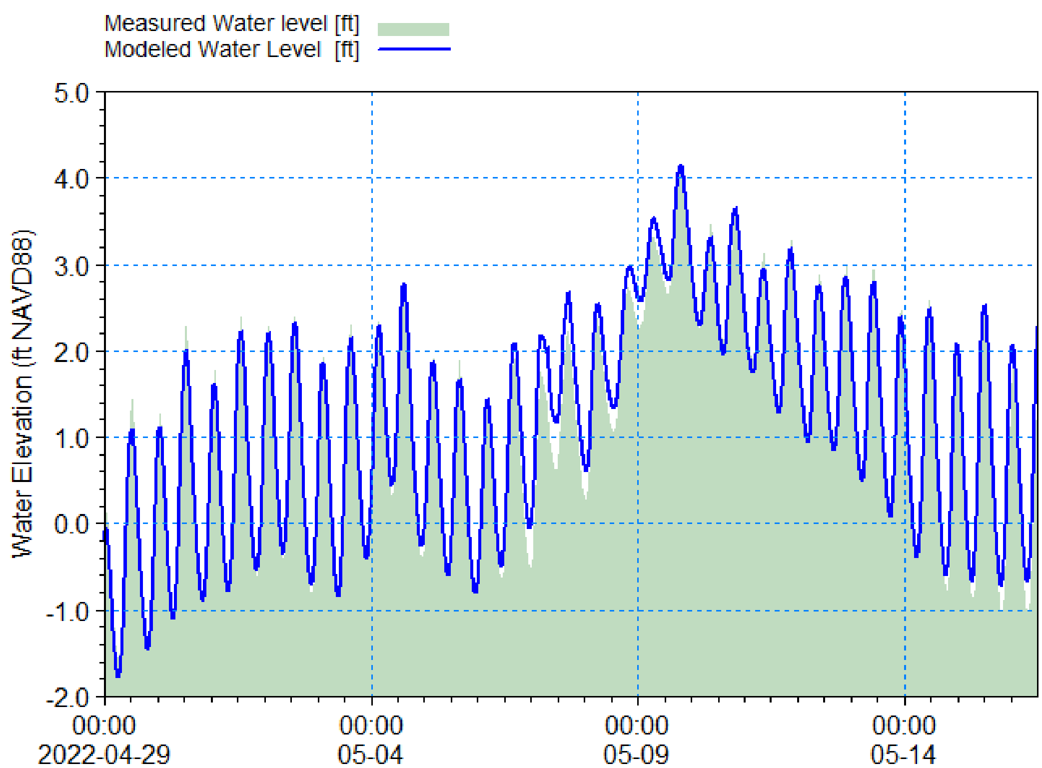

FIGURE 26. WATER LEVEL COMPARISON AT RBR INLET BRIDGE GAUGE DURING CALIBRATION PERIOD (GREEN SHADED: MEASURED, BLUE SOLD LINE: MODELED).

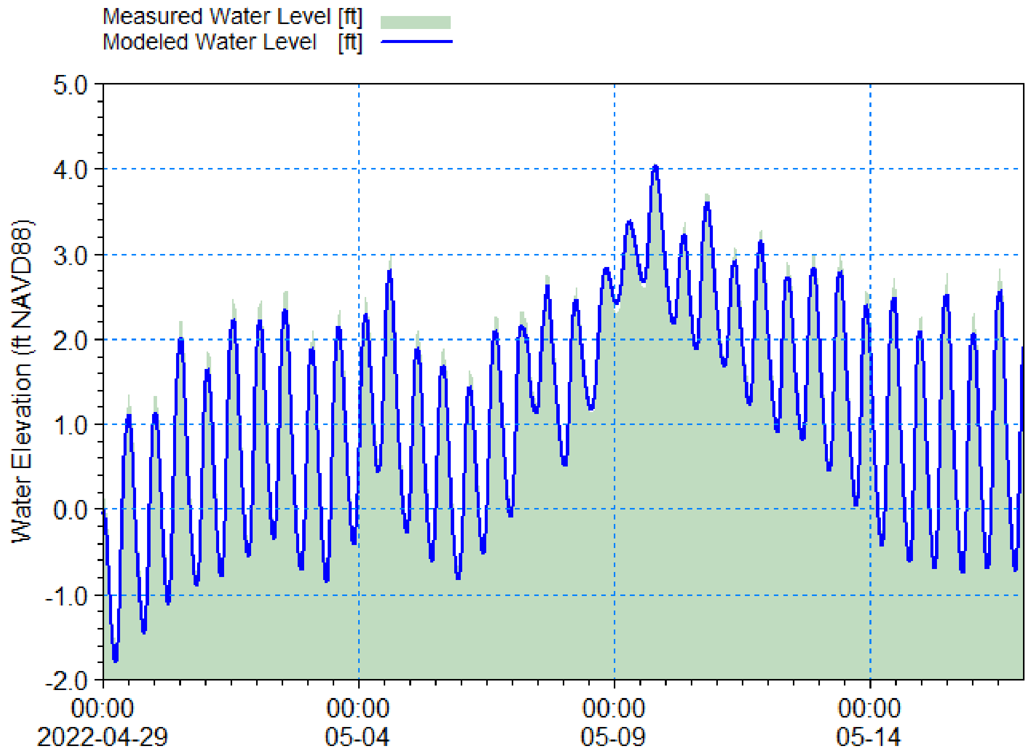

FIGURE 27. WATER LEVEL COMPARISON AT NOAA TIDE GAUGE 8594900 (WASHINGTON CHANNEL) DURING CALIBRATION PERIOD (GREEN SHADED: MEASURED, BLUE SOLD LINE: MODELED).

Appendix C: Water Level Analysis

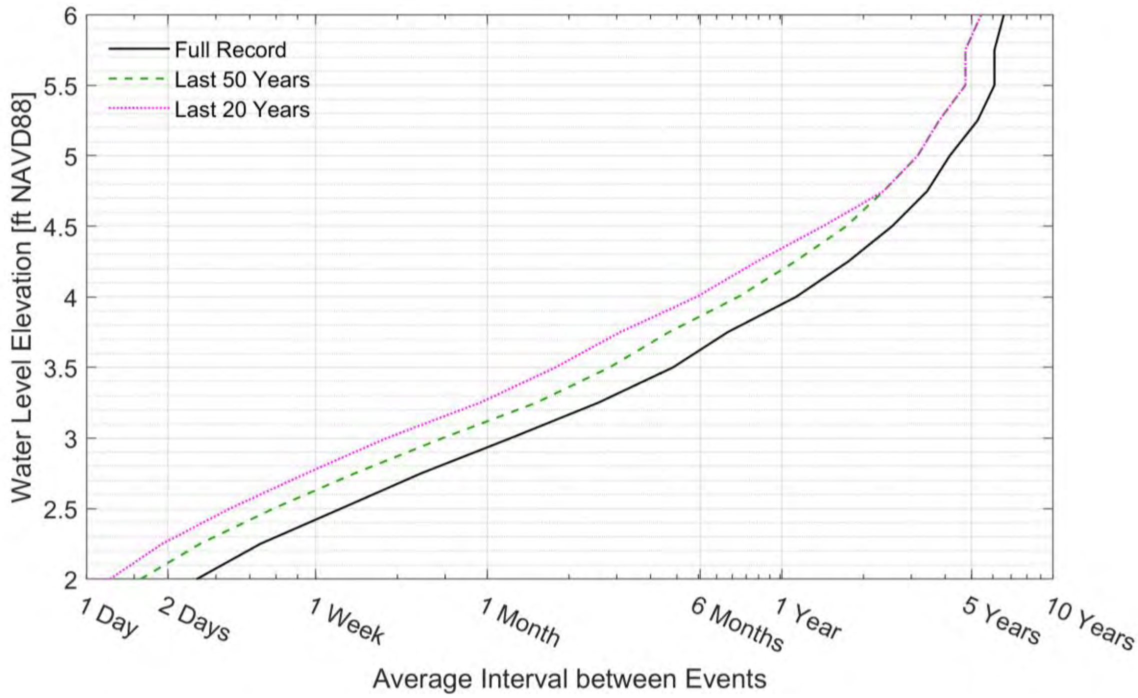

FIGURE 28. AVERAGE INTERVAL BETWEEN FLOODING EVENTS AS A FUNCTION OF WATER LEVEL RECORD LENGTH AND FLOOD ELEVATION THRESHOLD.

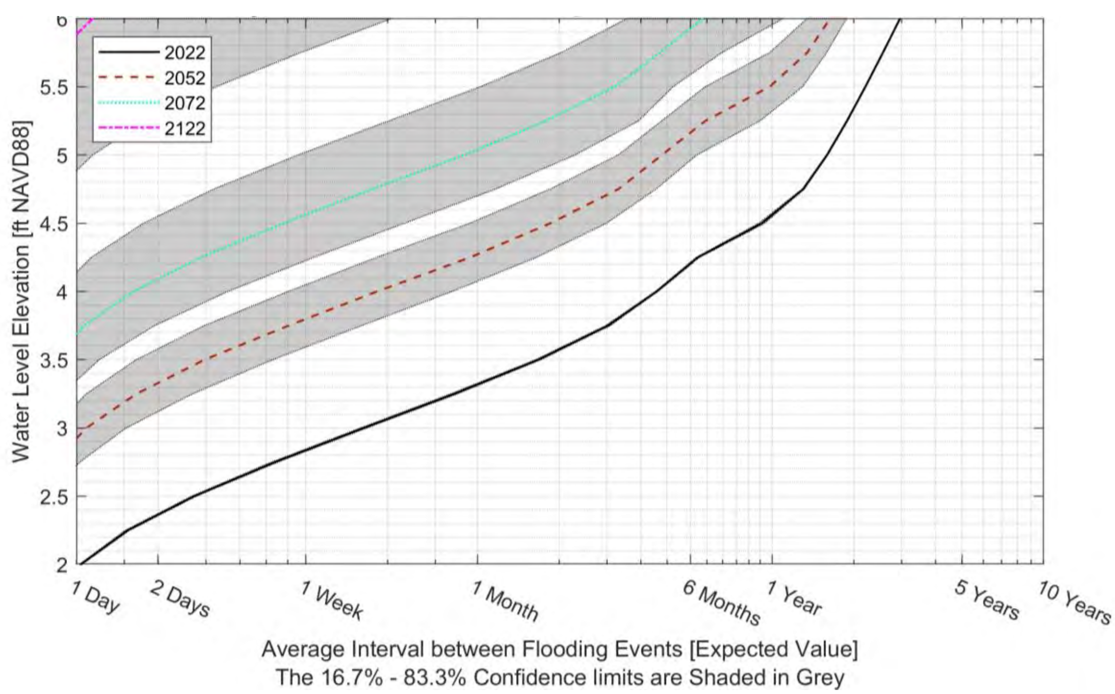

FIGURE 29. AVERAGE INTERVAL BETWEEN FLOODING EVENTS AS A FUNCTION OF WATER LEVEL RECORD LENGTH AND FLOOD ELEVATION THRESHOLD UNDER SLR PROJECTION (IPCC SSP4-8.5).

Appendix D: Maximum Currents

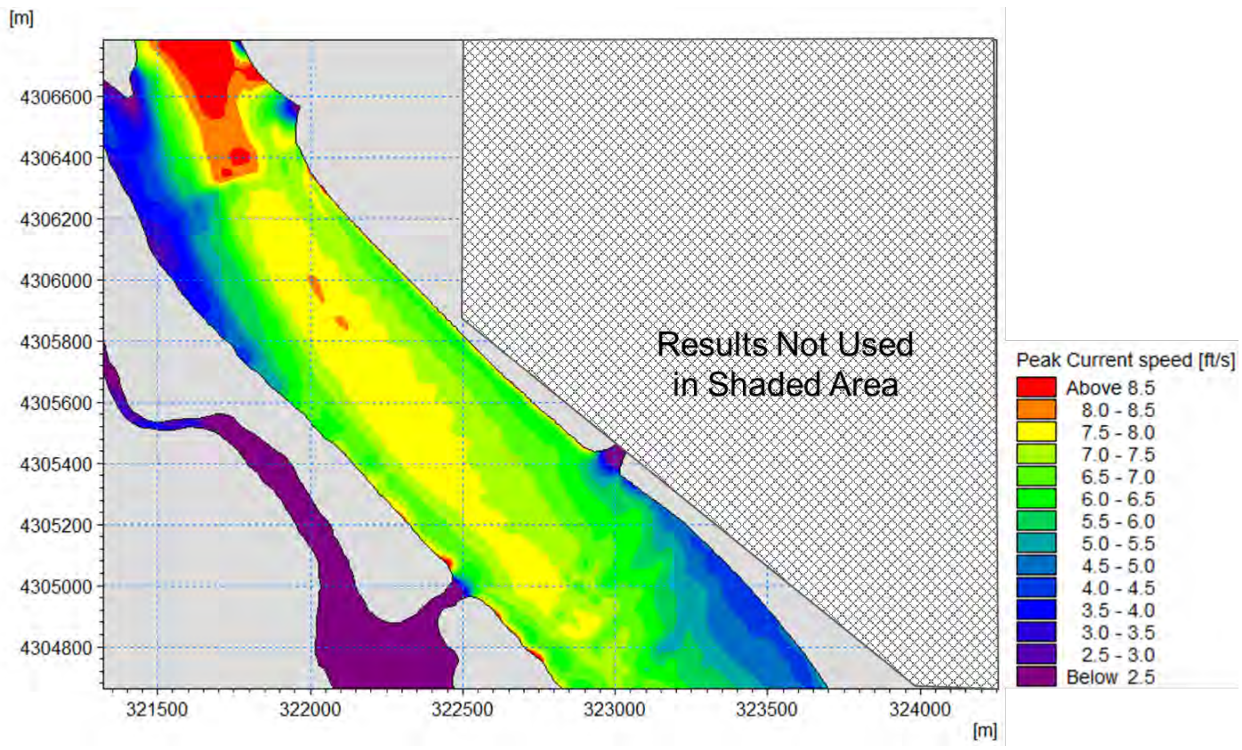

FIGURE 30. MAXIMUM CURRENTS AT WEST POTOMAC PARK.

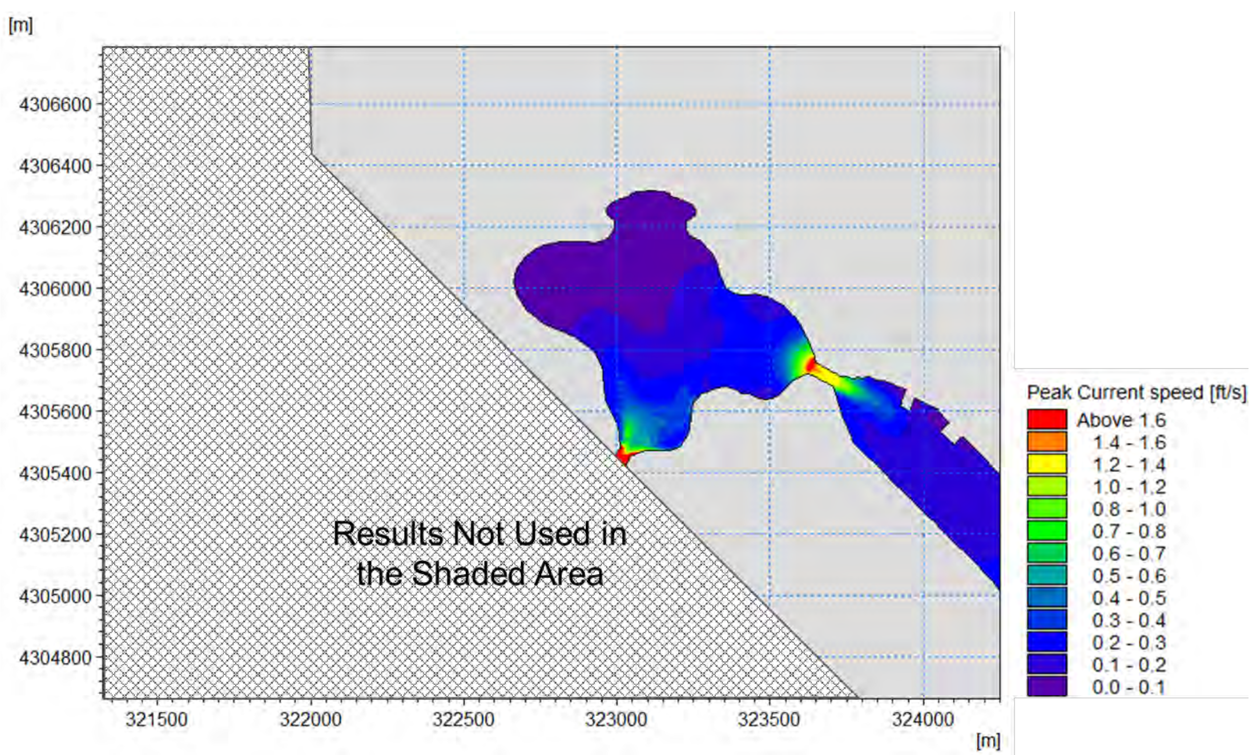

FIGURE 31. MAXIMUM CURRENTS AT TODAL BASIN.

Appendix E: Wave Model Development

The DHI MIKE21 FM SW module was used for determining the design waves. This third-generation model simulates the growth, decay, and transformation of wind-generated waves and swells in offshore and coastal areas. The 2D FM used in the wave modelling was the same as the one used for water level and current modelling; the details of model domain and bathymetry are discussed in Appendix B.

The model was forced by a set of boundary conditions with varied wind speeds and directions and water levels. The local wind conditions were developed based on measurements at Washington National Airport. Per CEM (USACE, 2002), the wind speed needs to be adjusted from the averaging time of observation to an averaging time appropriate for wave prediction so that maximum wave growth can be ensured. As shown in Table 14, for Tidal Basin, 15-min duration is recommended, and for West Potomac Park, 45-min wind duration were applied.

TABLE 13. EXTREME WIND SPEEDS AT PROJECT SITE.

Return Period	2-minute Wind Speed (kts)*	15-minute Wind Speed (kts)	45-minute Wind Speed (kts)
5 years	39.9	35.3	34.3
10 years	43.2	38.2	37.1
25 years	48.7	43.0	41.8
50 years	54.3	48.0	46.7
100 years	61.6	54.4	52.9

*Observed wind averaging time at airport gauge.

The water levels applied in the model correspond to the same return period of the wind that was used to be conservative (see Table 14). However, it is found that the wind waves are not sensitive to the water elevation applied. The extreme water level information is available in the Climate Change and Natural Hazards Report (Moffatt & Nichol, 2022). The locally generated wind waves were predicted for both present-day 2050 sea levels and +1.3 feet of SLR by 2050 (per SSP4-8.5 projections). Table 15 summarizes the model scenarios for estimating the design waves with various return periods.

TABLE 14. EXTREME WATER LEVELS AT THE PROJECT SITE (MOFFATT & NICHOL, 2022).

Return Period	Water Surface Elevations (ft NAVD88 in 2022)
5 years	5.09
10 years	6.32
25 years	8.27
50 years	9.94
100 years	11.75

TABLE 15. MODELING SCENARIOS FOR DESIGN WAVES.

Return Period	Design Wind Speed	Wind Direction	Stillwater Level	SLR
5 years	Washington National Airport Wind; 15-min Averaged winds (Tidal Basin); 45-min Averaged wind (West Potomac Park)	All compass with 22.5-degree bin	5-year WL	Present-day sea levels; SLR by 2050 (+1.3 feet since 2005)
10 years			10-year WL	
25 years			25-year WL	
50 years			50-year WL	
100 years			100-year WL	

Appendix F: Wave Results

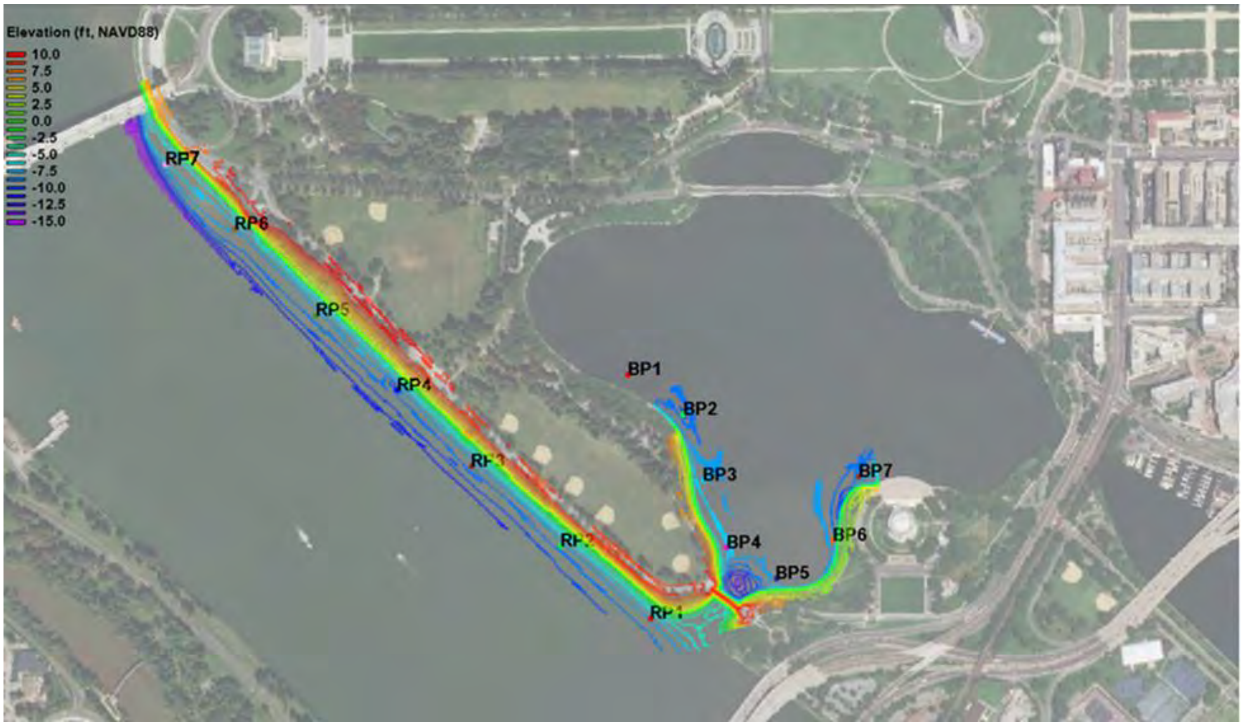

FIGURE 32. EXTRACTION LOCATIONS OF WAVE RESULTS.

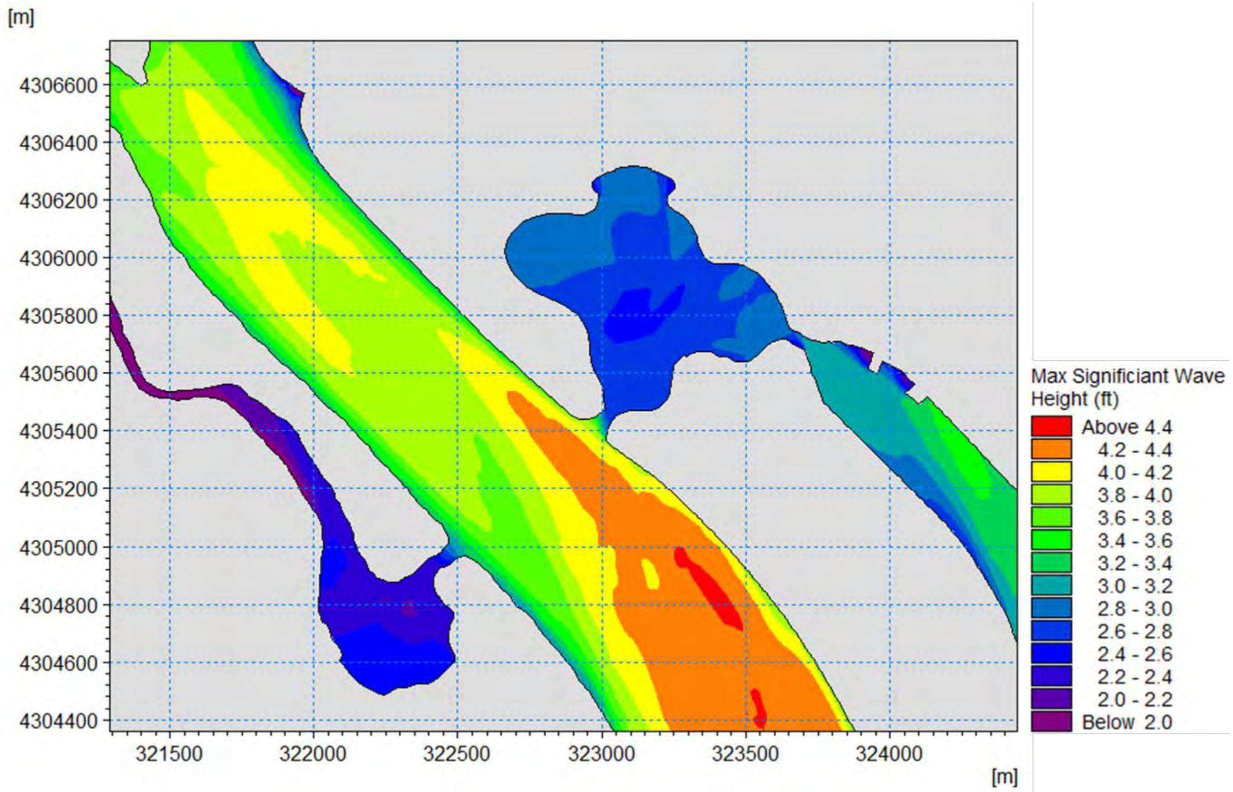

FIGURE 33. EXTREME WAVE MODELING RESULTS (100-YR, MAX SIGNIFICANT WAVE HEIGHT) PRESENT-DAY.

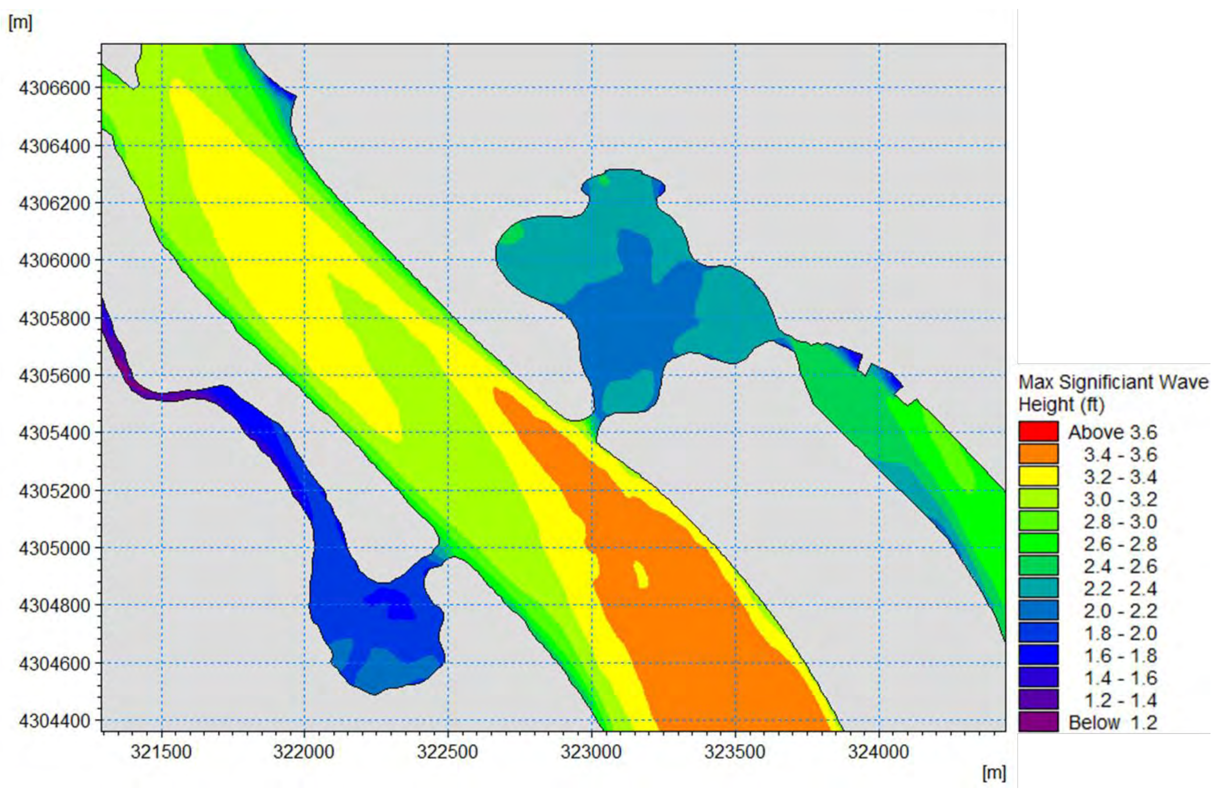

FIGURE 34. EXTREME WAVE MODELING RESULTS (50-YR, MAX SIGNIFICANT WAVE HEIGHT) PRESENT-DAY.

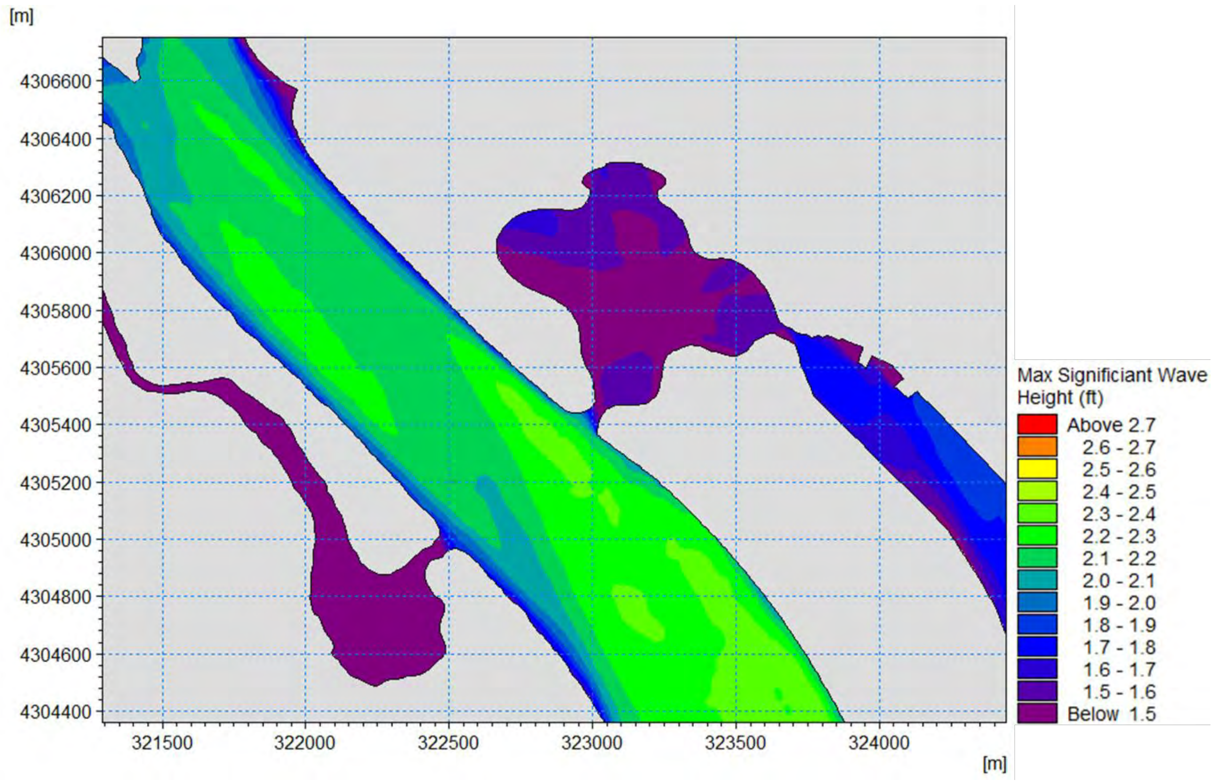

FIGURE 35. EXTREME WAVE MODELING RESULTS (10-YR, MAX SIGNIFICANT WAVE HEIGHT) PRESENT-DAY.

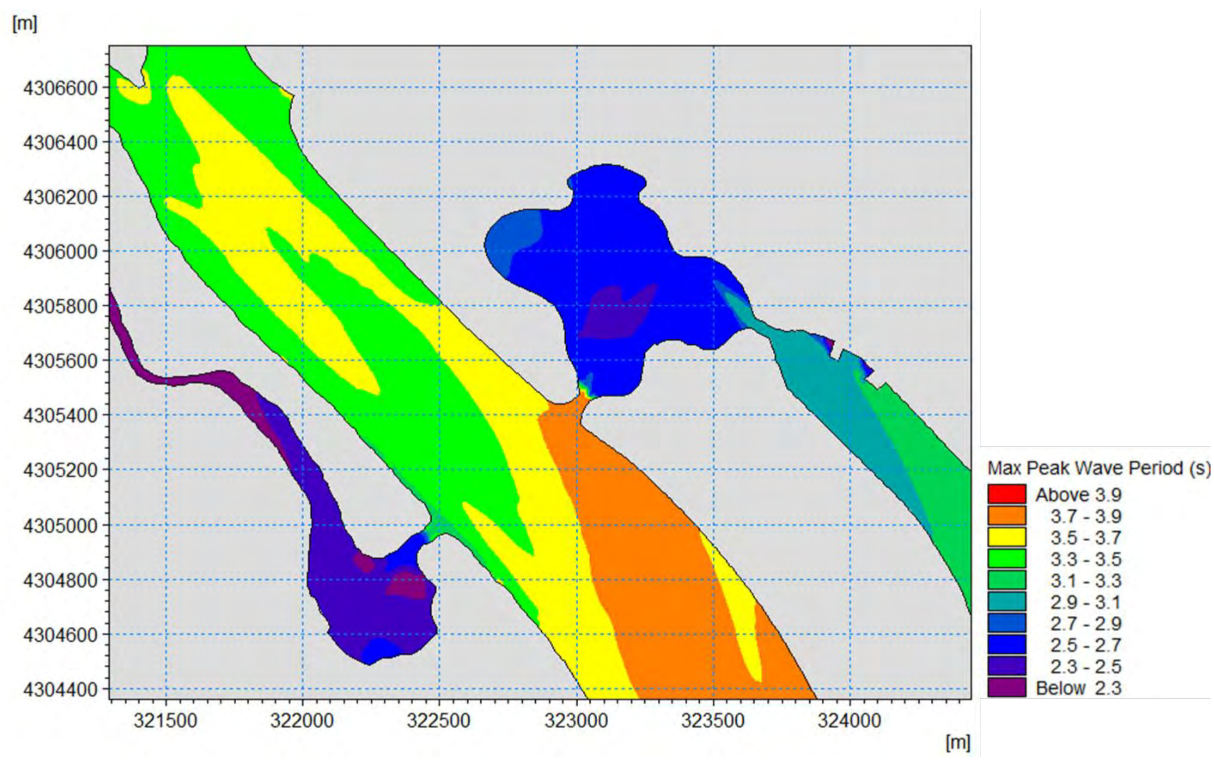

FIGURE 36. EXTREME WAVE MODELING RESULTS (100-YR, MAX PEAK WAVE PERIOD) PRESENT-DAY.

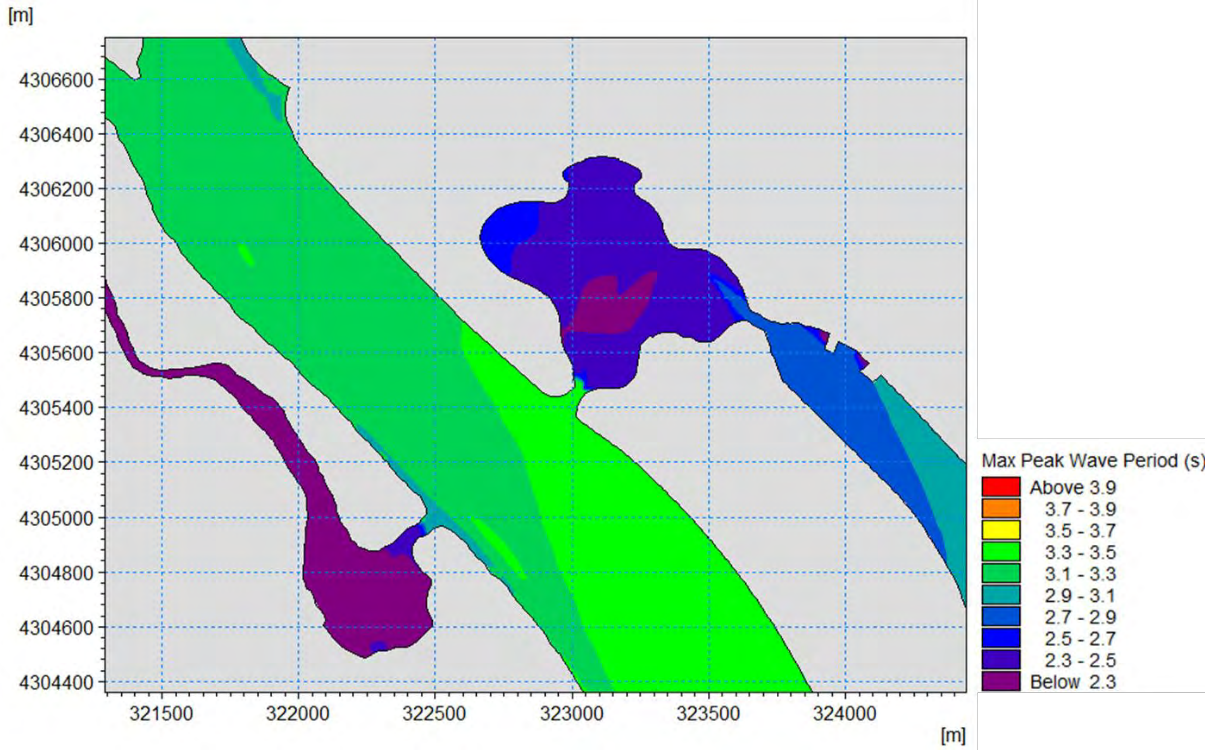

FIGURE 37. EXTREME WAVE MODELING RESULTS (50-YR, MAX PEAK WAVE PERIOD) PRESENT-DAY.

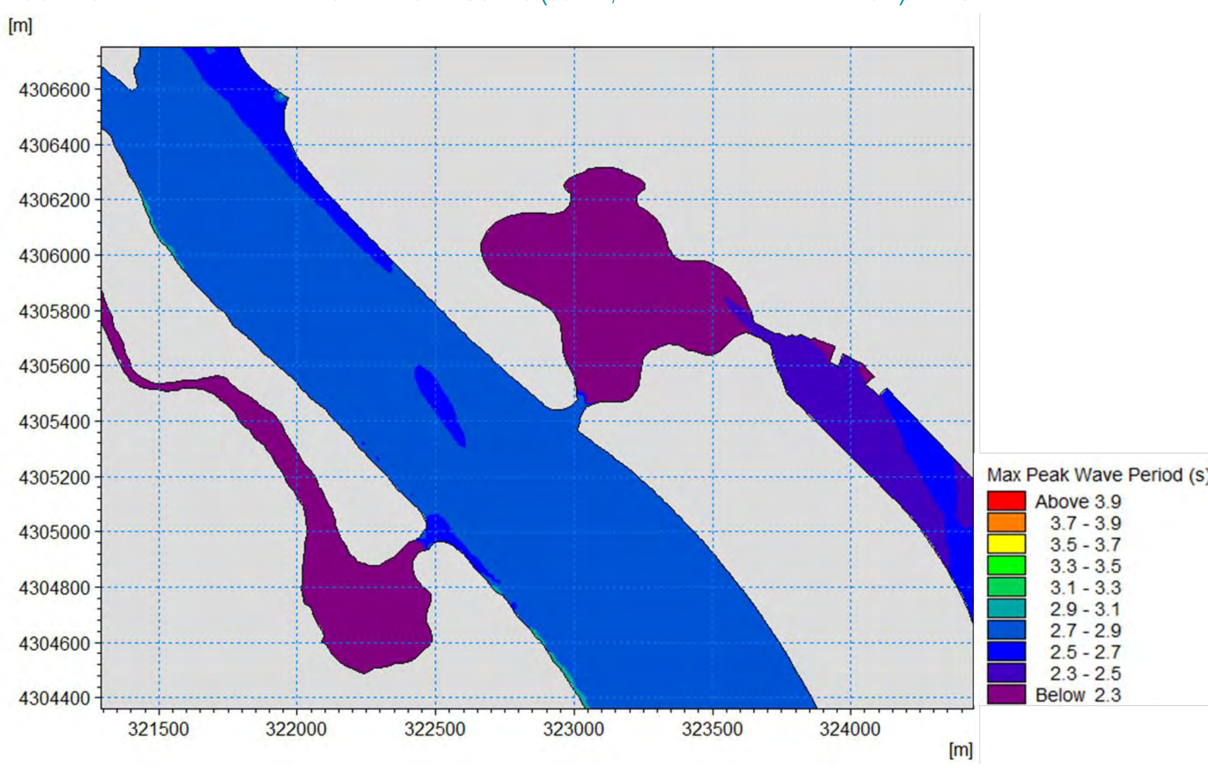

FIGURE 38. EXTREME WAVE MODELING RESULTS (10-YR, MAX PEAK WAVE PERIOD) PRESENT-DAY.

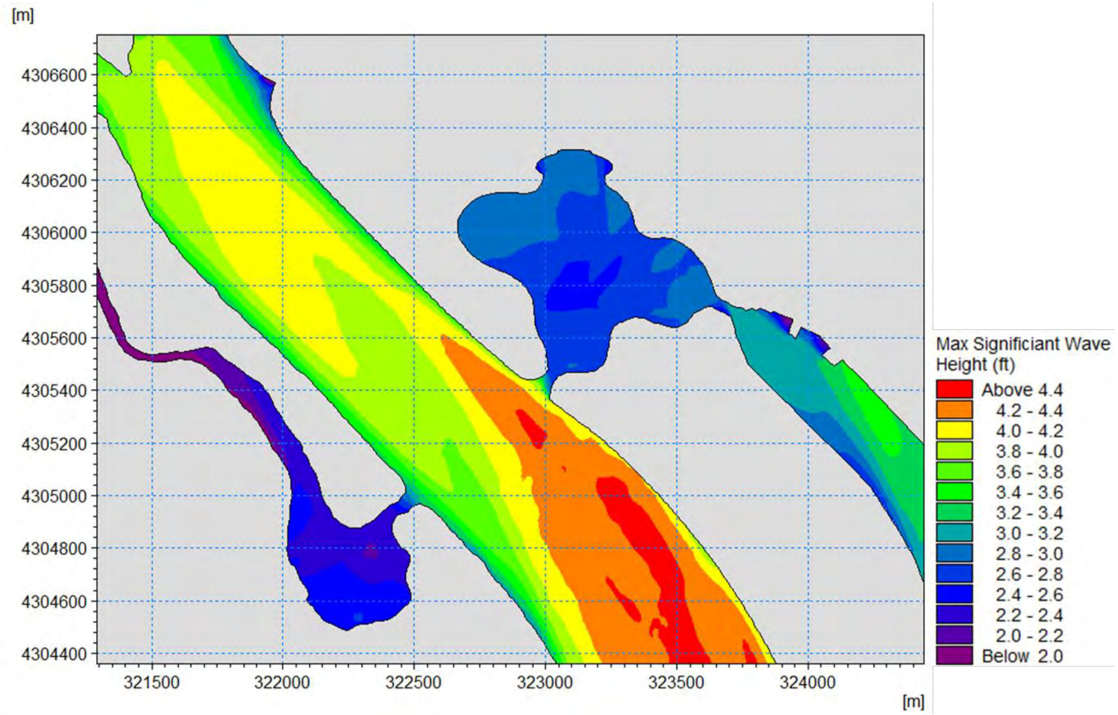

FIGURE 39. EXTREME WAVE MODELING RESULTS (100-YR, MAX SIGNIFICANT WAVE HEIGHT) UNDER SLR BY 2050.

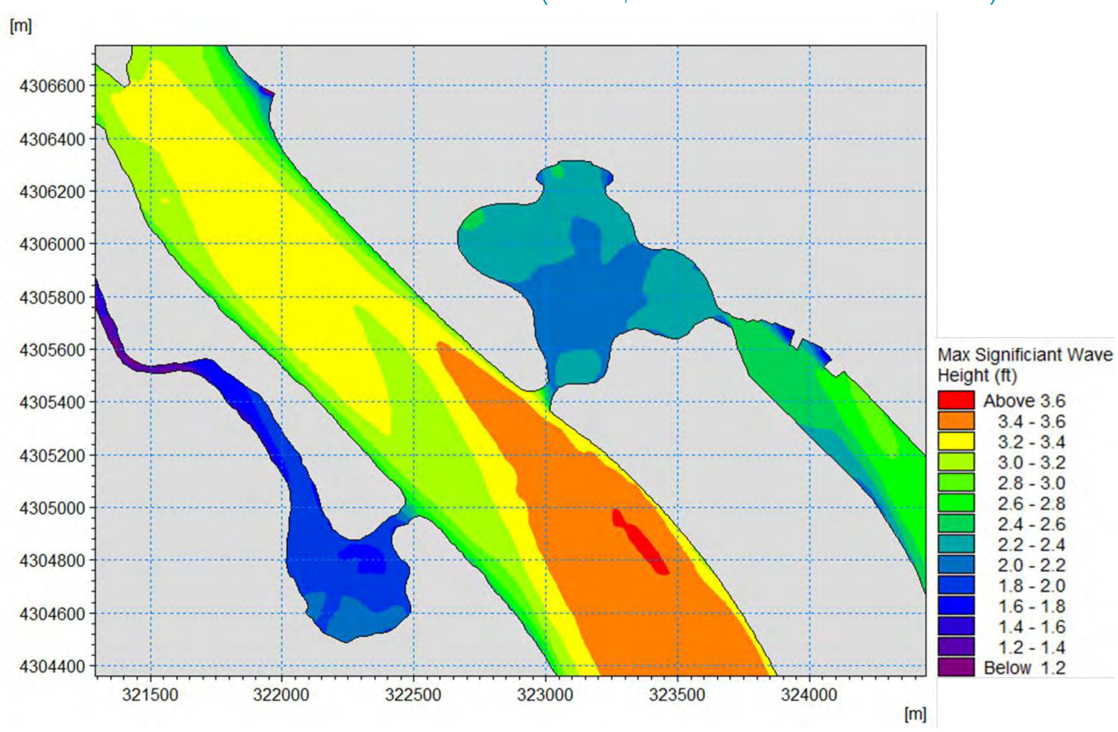

FIGURE 40. EXTREME WAVE MODELING RESULTS (50-YR, MAX SIGNIFICANT WAVE HEIGHT) UNDER SLR BY 2050.

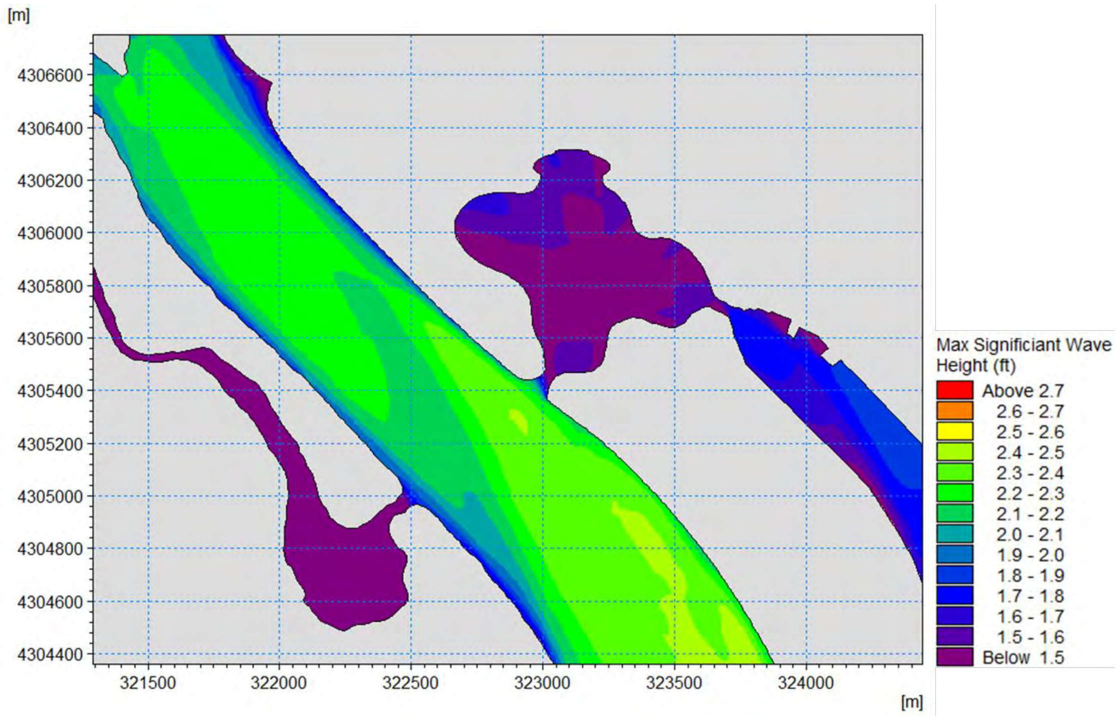

FIGURE 41. EXTREME WAVE MODELING RESULTS (10-YR, MAX SIGNIFICANT WAVE HEIGHT) UNDER SLR BY 2050.

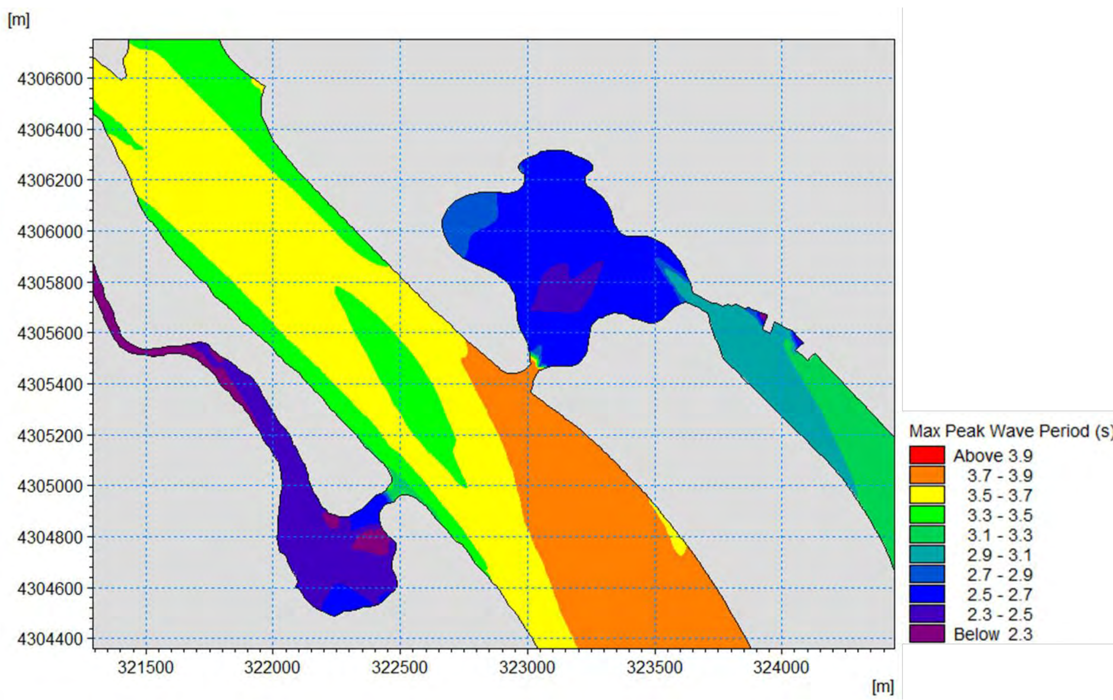

FIGURE 42. EXTREME WAVE MODELING RESULTS (100-YR, MAX PEAK WAVE PERIOD) UNDER SLR BY 2050.

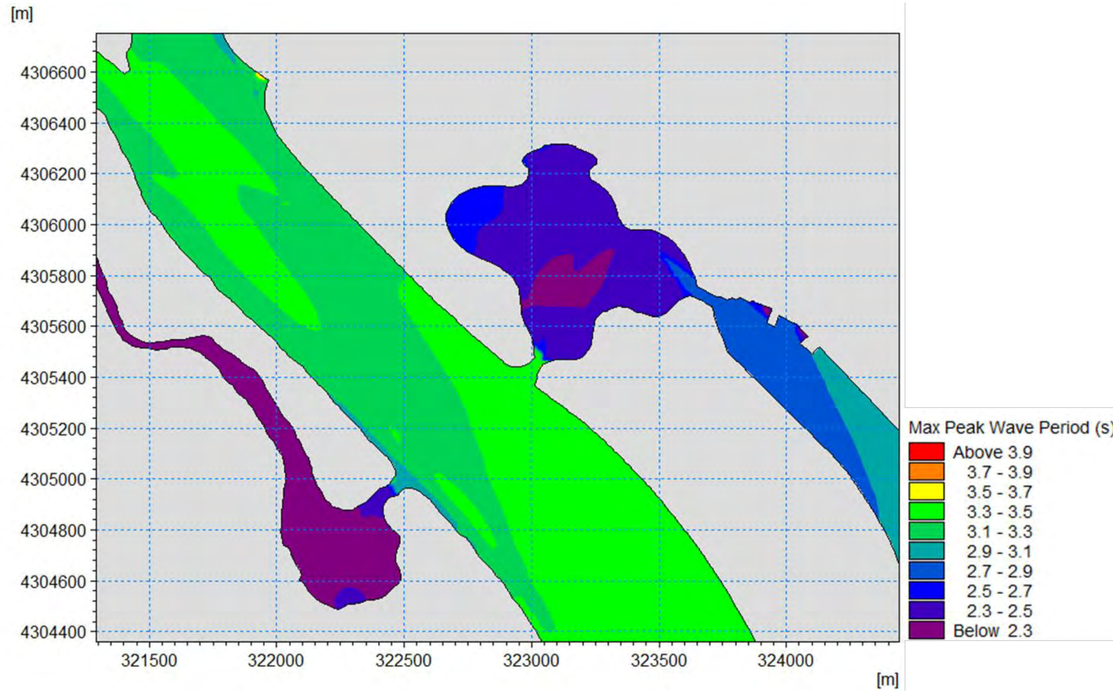

FIGURE 43. EXTREME WAVE MODELING RESULTS (50-YR, MAX PEAK WAVE PERIOD) UNDER SLR BY 2050.

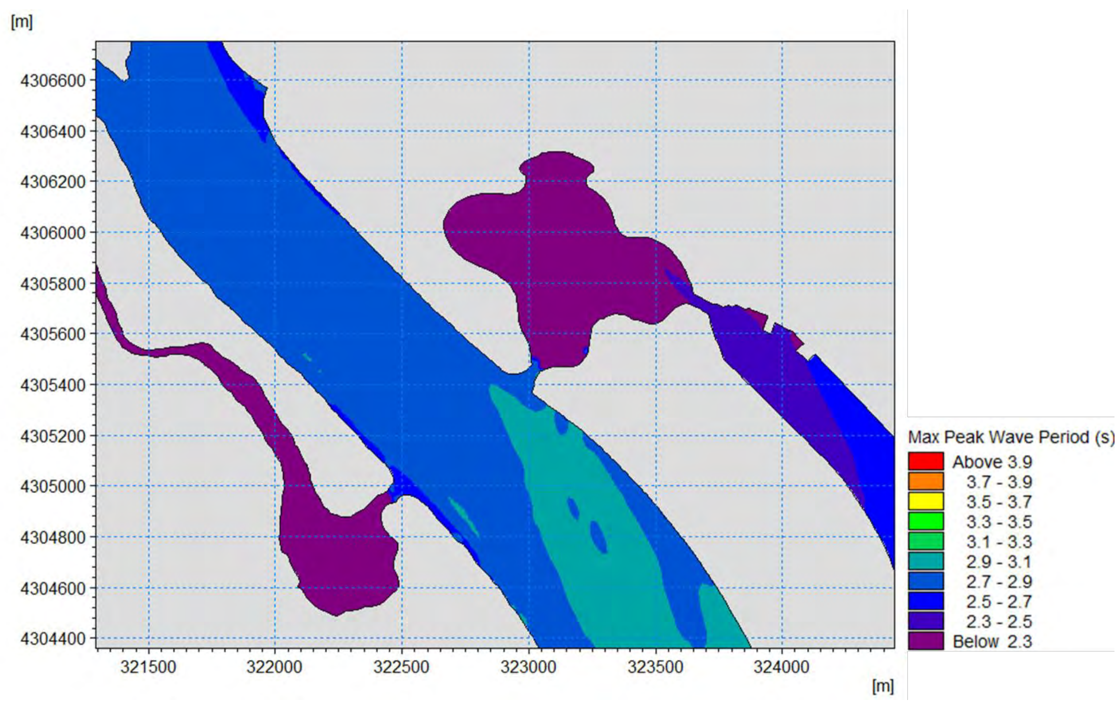

FIGURE 44. EXTREME WAVE MODELING RESULTS (10-YR, MAX PEAK WAVE PERIOD) UNDER SLR BY 2050.

Appendix G: Overtopping Results

Overtopping rates are provided for each analysis location, with and without sea level rise, below. Each figure presents overtopping for a range of water levels and wave return periods relative to EurOtop (2018) thresholds for erosion at the wall crest. Extreme water levels are shown as grey lines and are labelled on the right side of the figures. Grey shading indicates overtopping is expected to erode patchy grass cover on seawall crests but not cause damage to uniform grass cover.

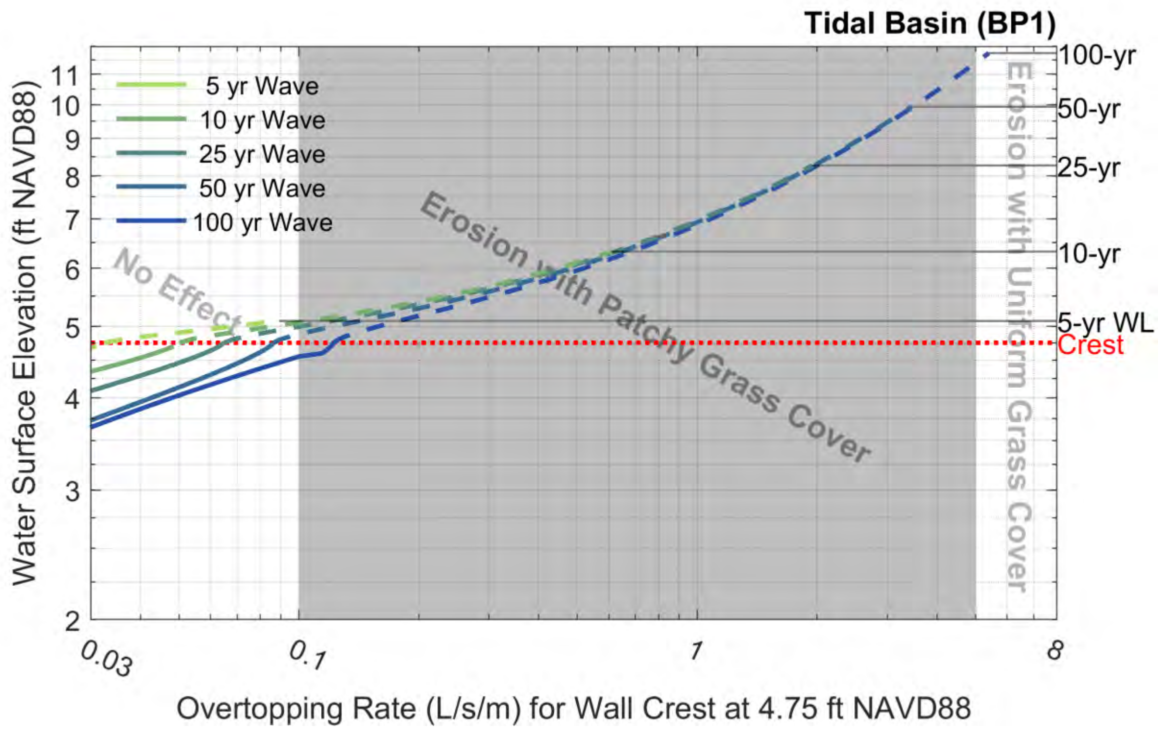

FIGURE 45. OVERTOPPING RATES FOR THE TIDAL BASIN (BP1) FOR PRESENT-DAY SEA LEVELS.

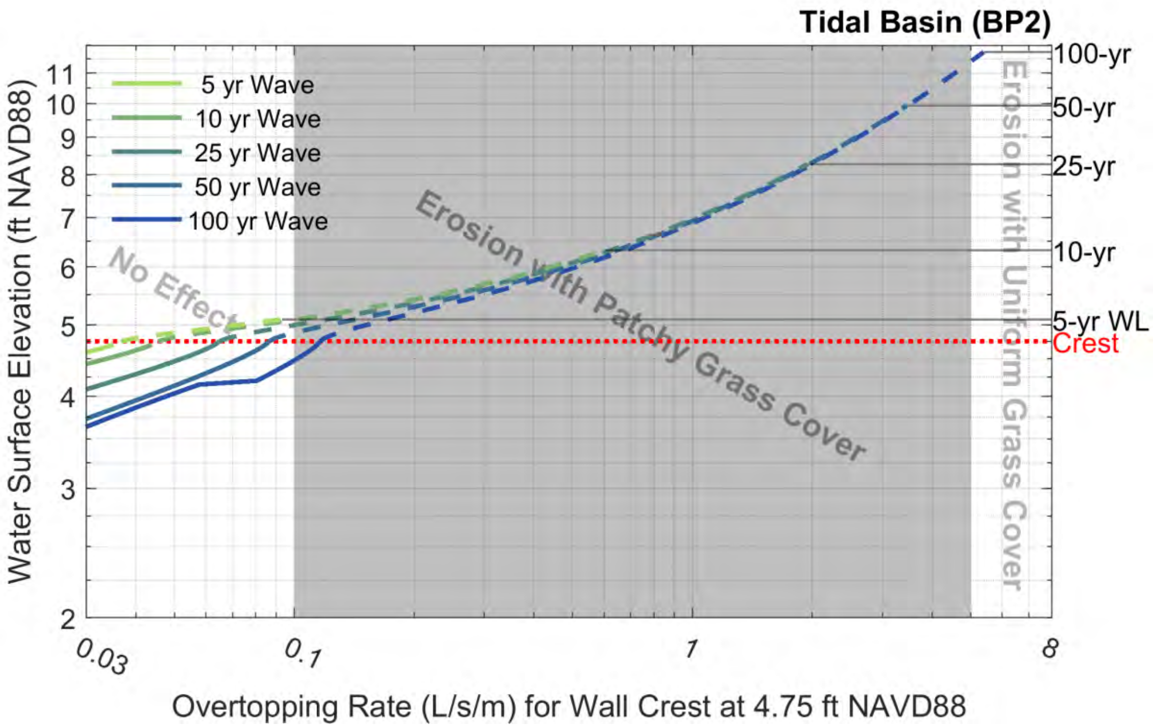

FIGURE 46. OVERTOPPING RATES FOR THE TIDAL BASIN (BP2) FOR PRESENT-DAY SEA LEVELS.

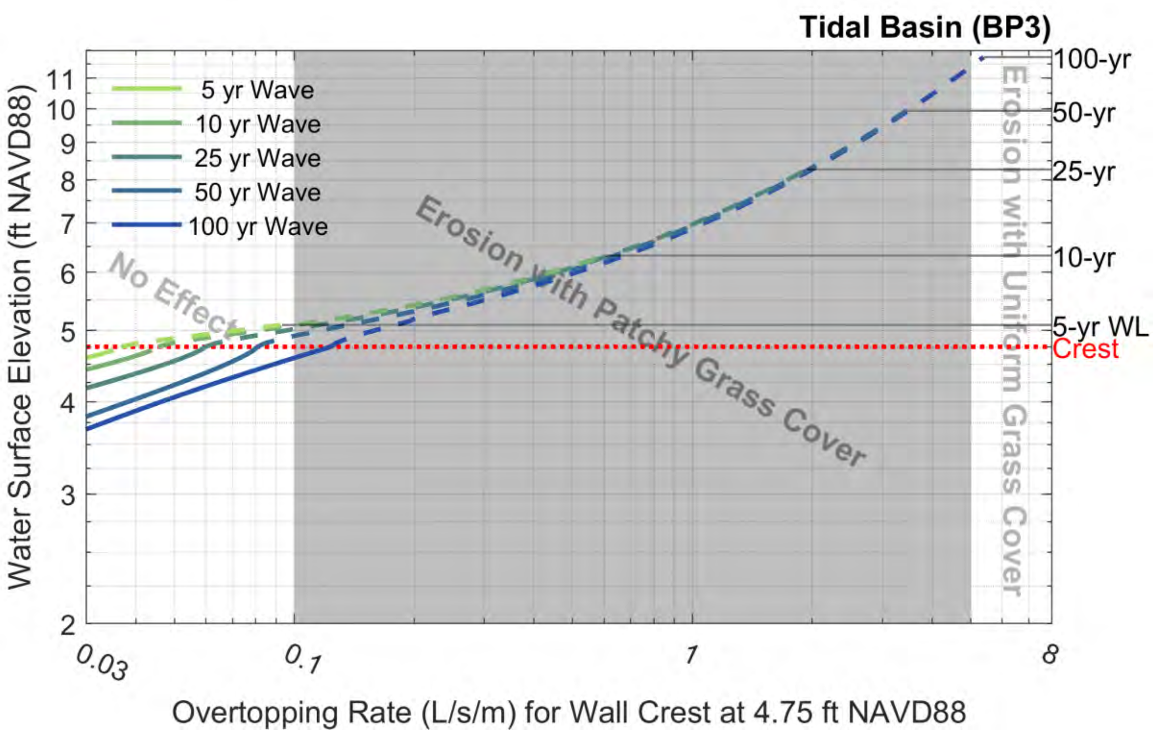

FIGURE 47. OVERTOPPING RATES FOR THE TIDAL BASIN (BP3) FOR PRESENT-DAY SEA LEVELS.

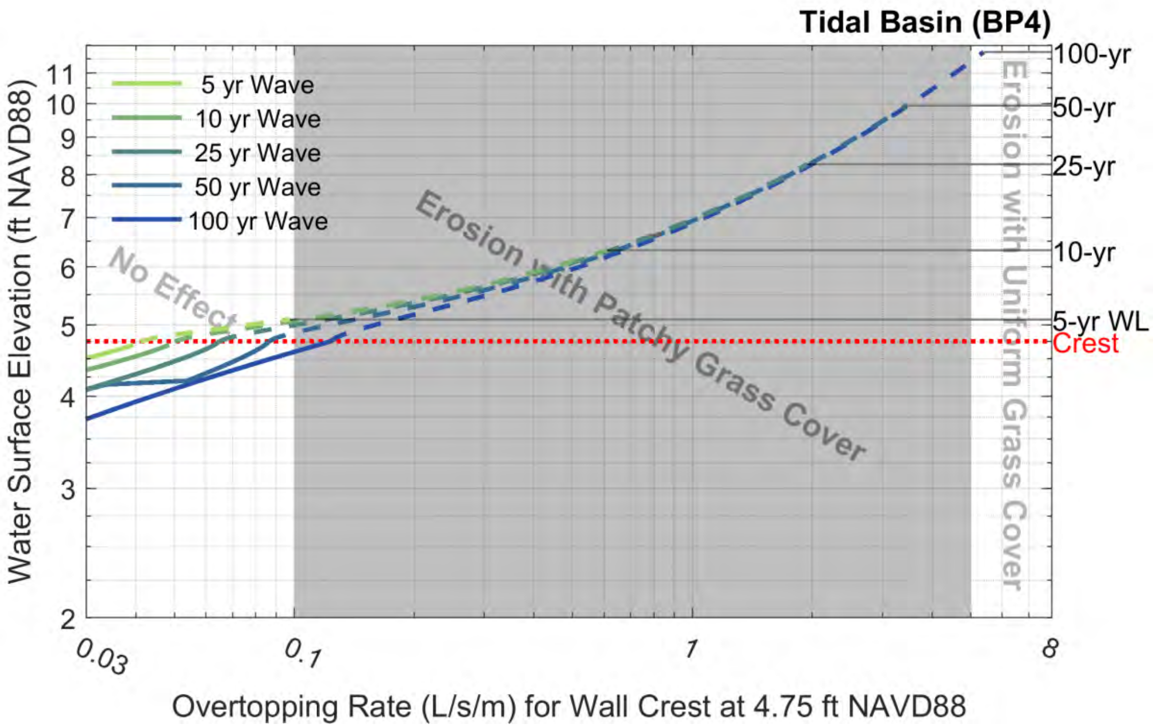

FIGURE 48. OVERTOPPING RATES FOR THE TIDAL BASIN (BP4) FOR PRESENT-DAY SEA LEVELS.

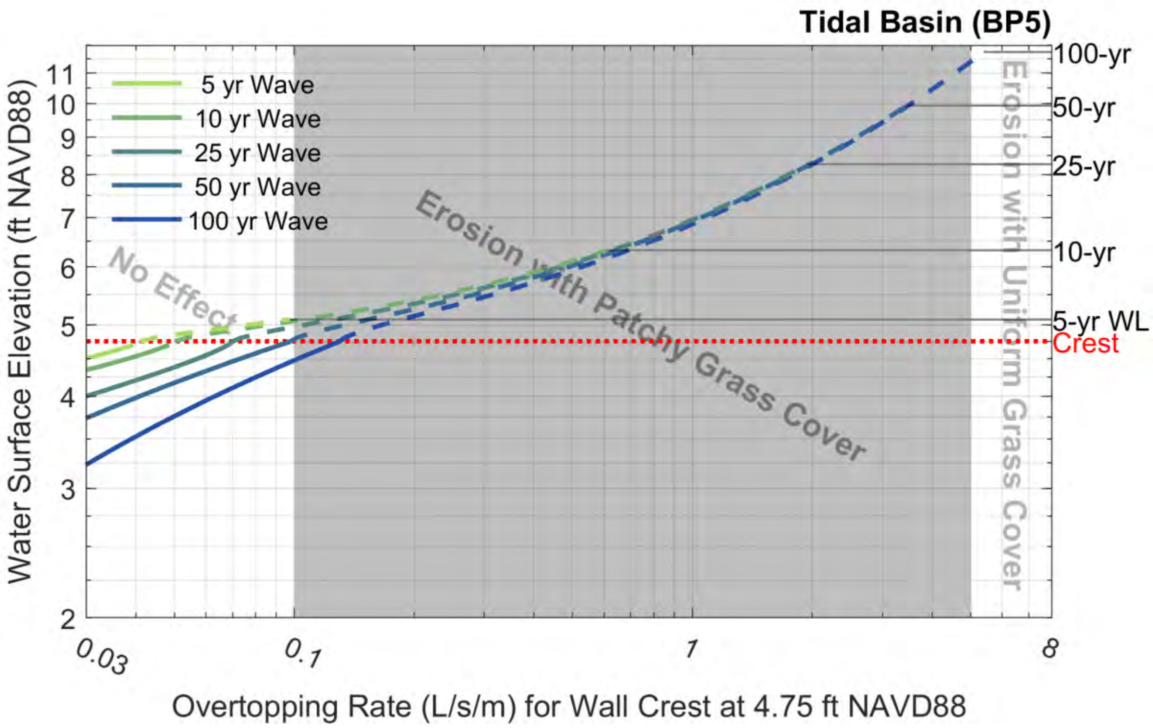

FIGURE 49. OVERTOPPING RATES FOR THE TIDAL BASIN (BP5) FOR PRESENT-DAY SEA LEVELS.

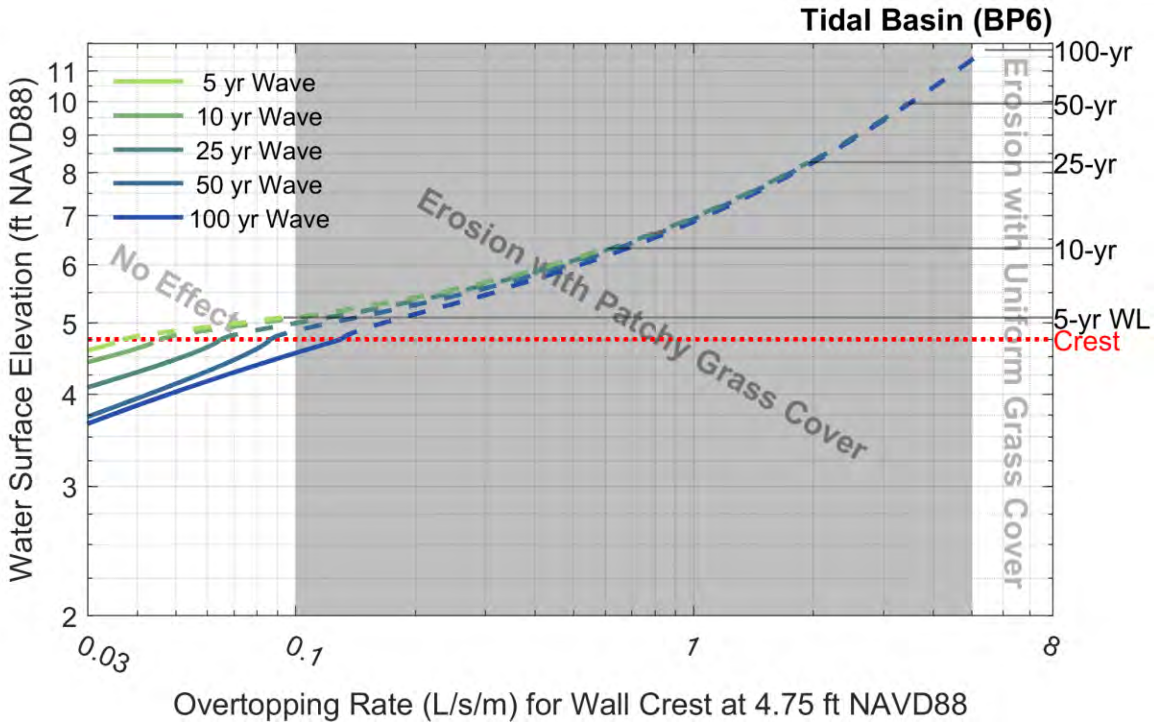

FIGURE 50. OVERTOPPING RATES FOR THE TIDAL BASIN (BP6) FOR PRESENT-DAY SEA LEVELS.

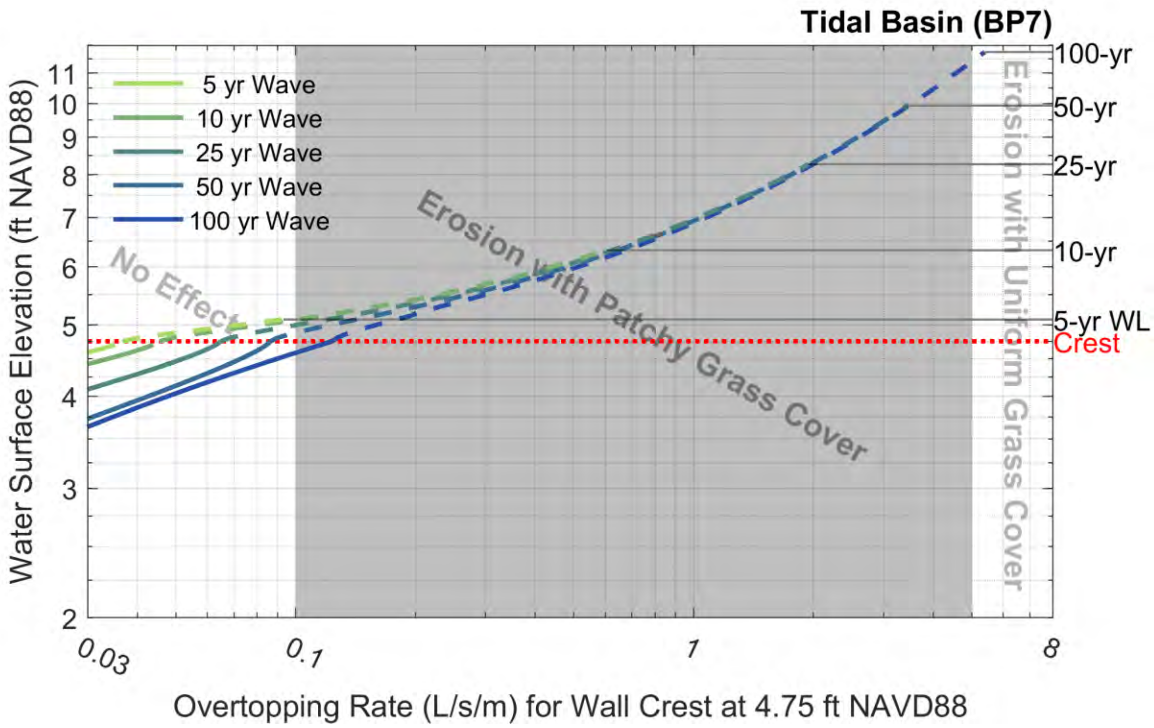

FIGURE 51. OVERTOPPING RATES FOR THE TIDAL BASIN (BP7) FOR PRESENT-DAY SEA LEVELS.

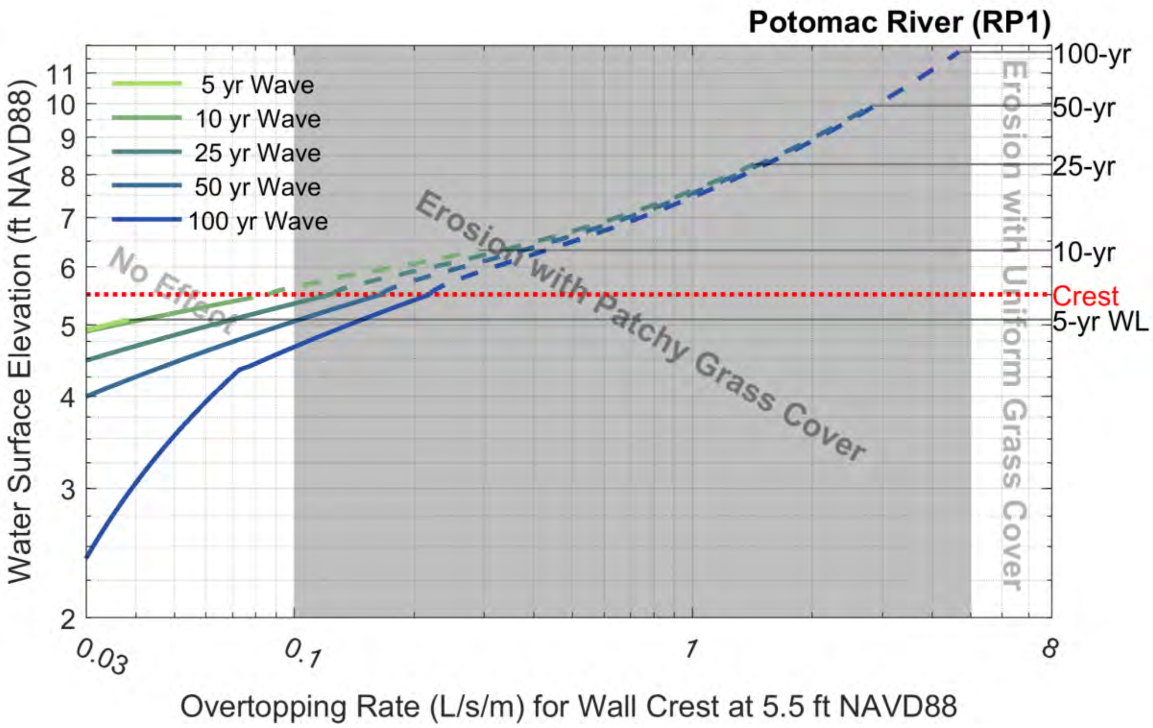

FIGURE 52. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP1) FOR PRESENT-DAY SEA LEVELS.

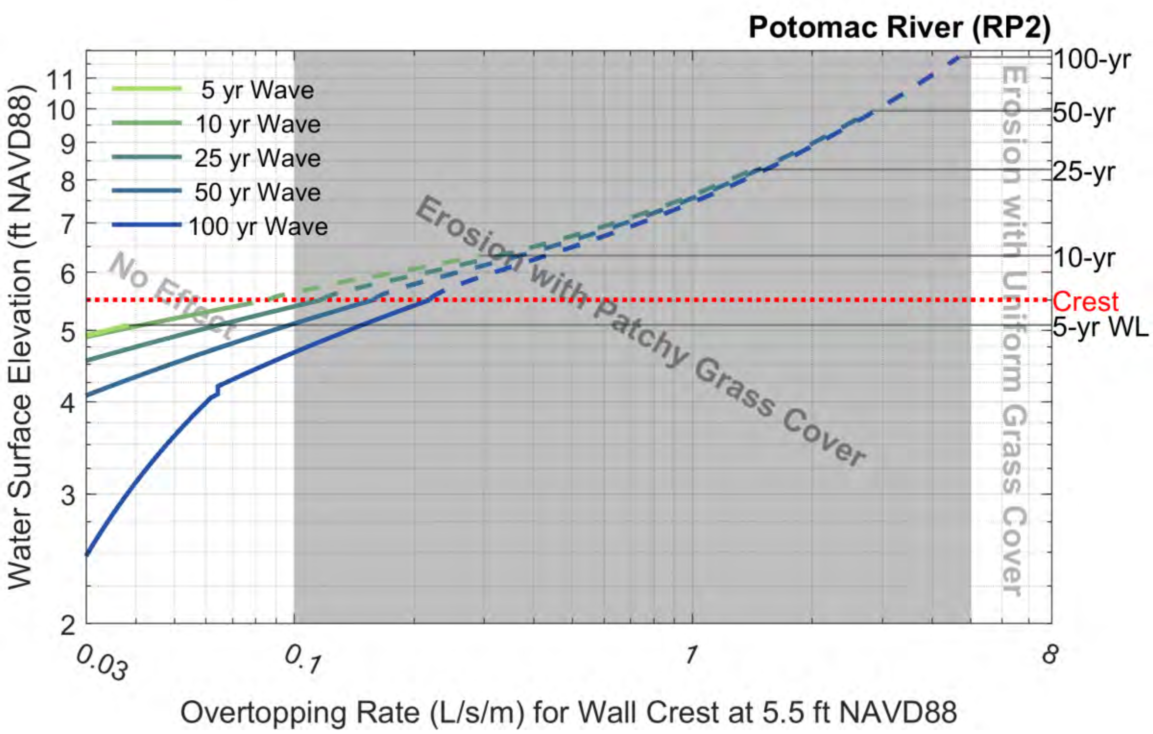

FIGURE 53. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP2) FOR PRESENT-DAY SEA LEVELS.

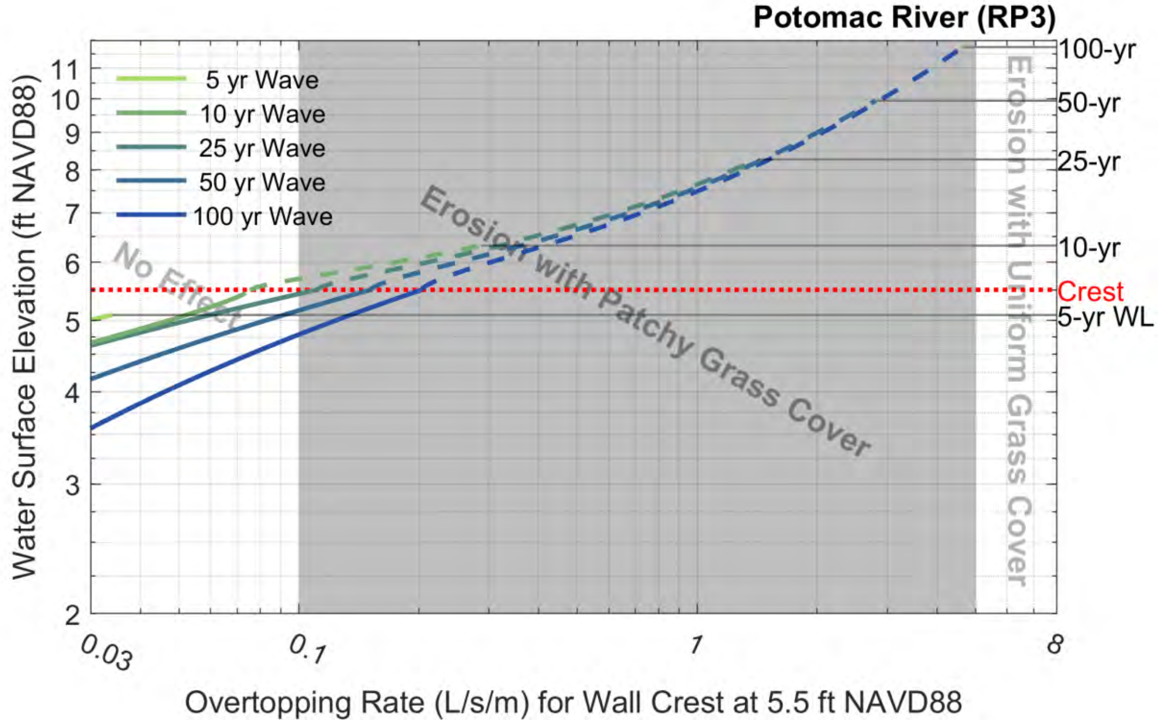

FIGURE 54. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP3) FOR PRESENT-DAY SEA LEVELS.

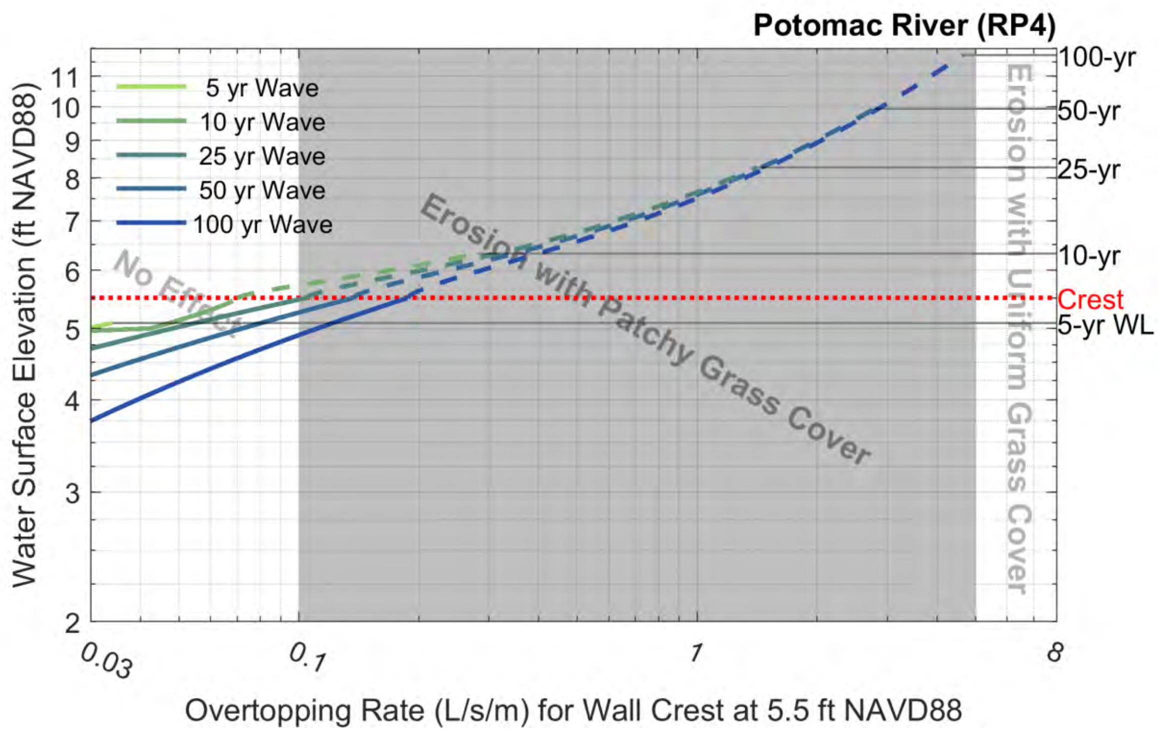

FIGURE 55. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP4) FOR PRESENT-DAY SEA LEVELS.

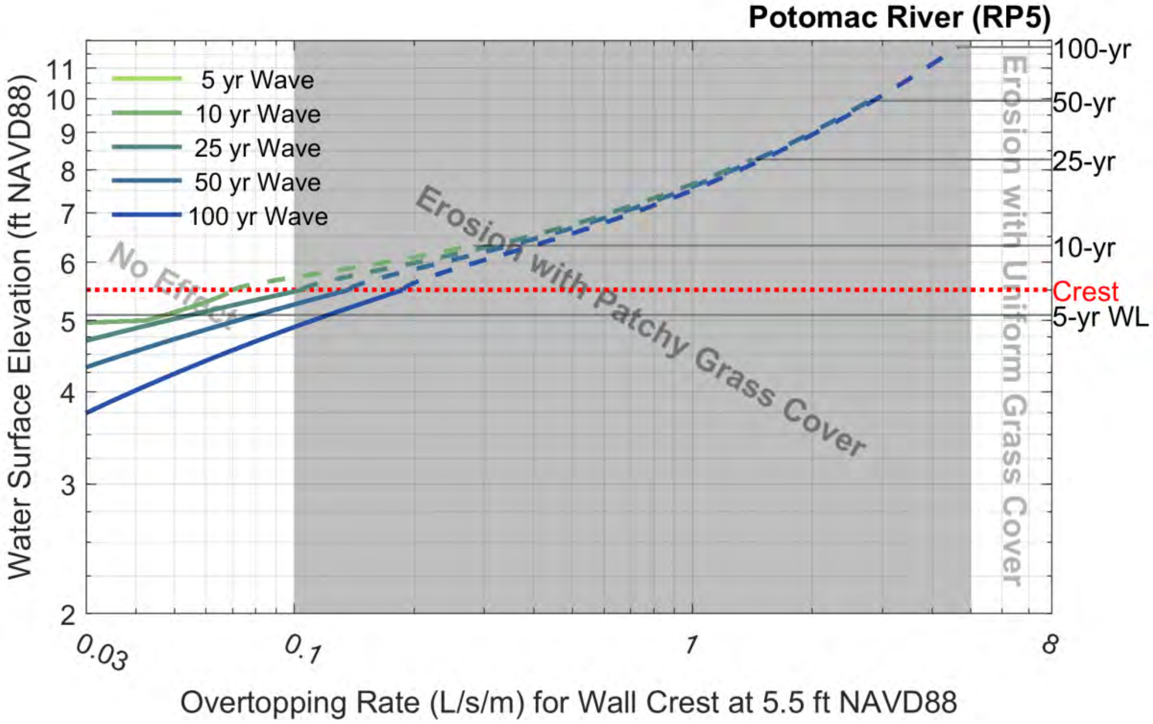

FIGURE 56. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP5) FOR PRESENT-DAY SEA LEVELS.

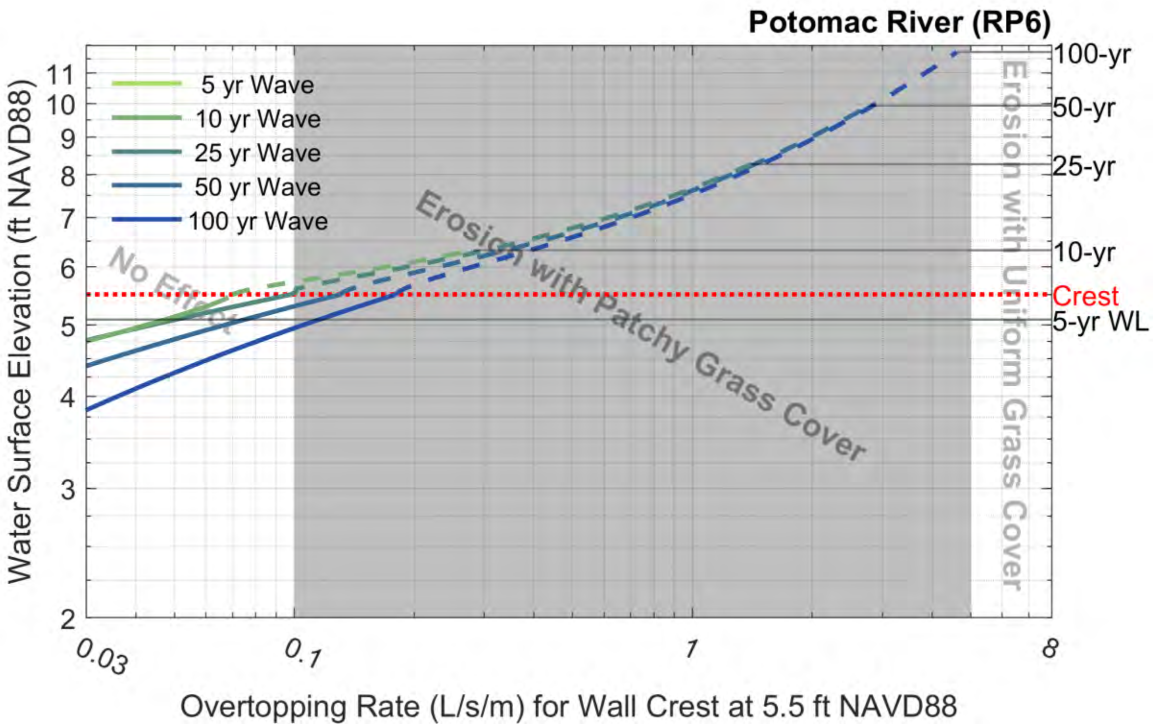

FIGURE 57. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP5) FOR PRESENT-DAY SEA LEVELS.

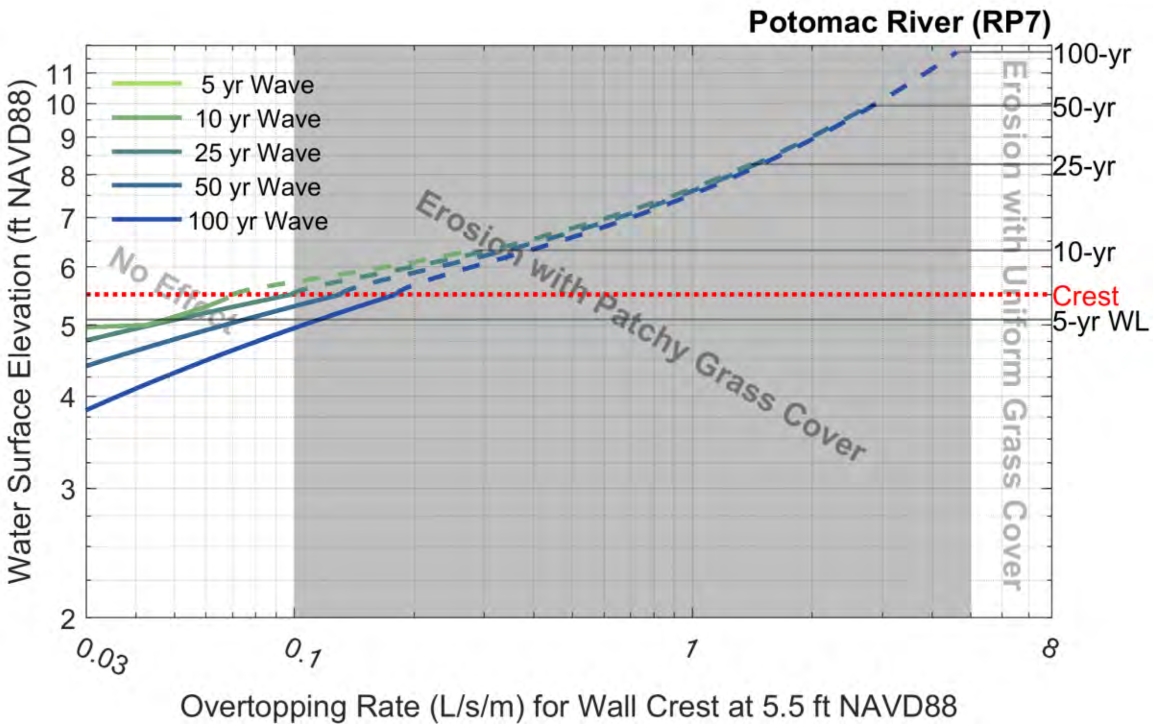

FIGURE 58. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP7) FOR PRESENT-DAY SEA LEVELS.

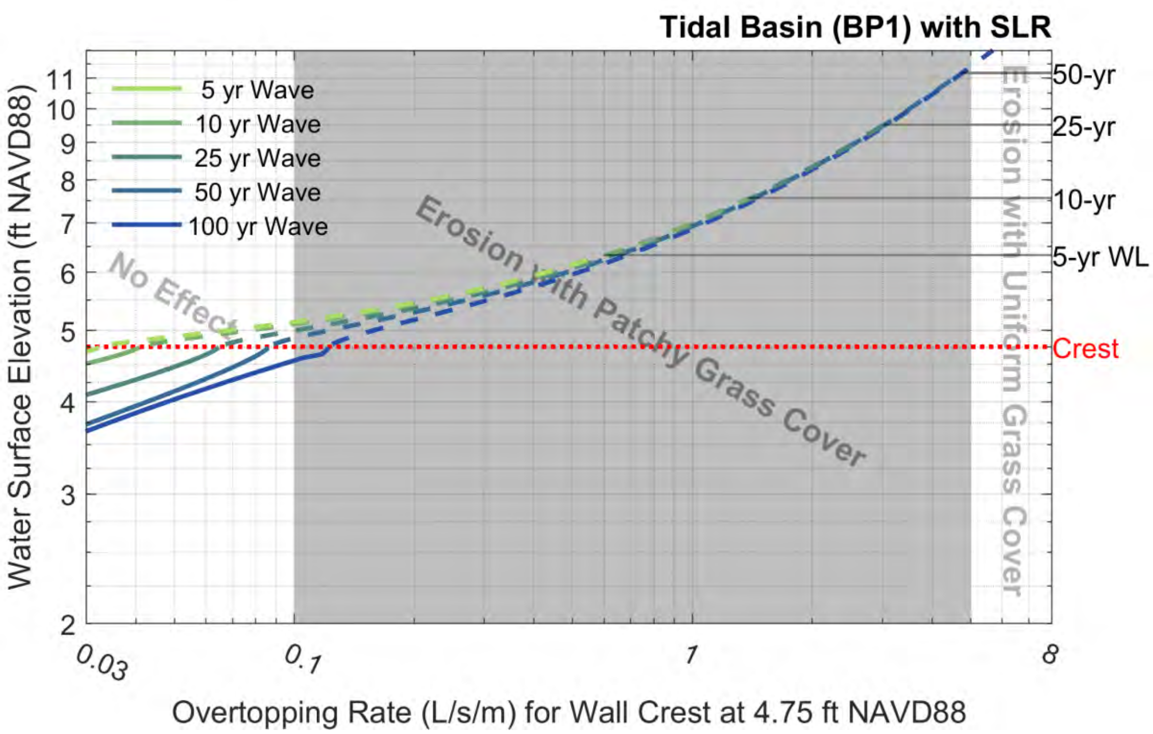

FIGURE 59. OVERTOPPING RATES FOR THE TIDAL BASIN (BP1) FOR FUTURE (2050) SEA LEVELS.

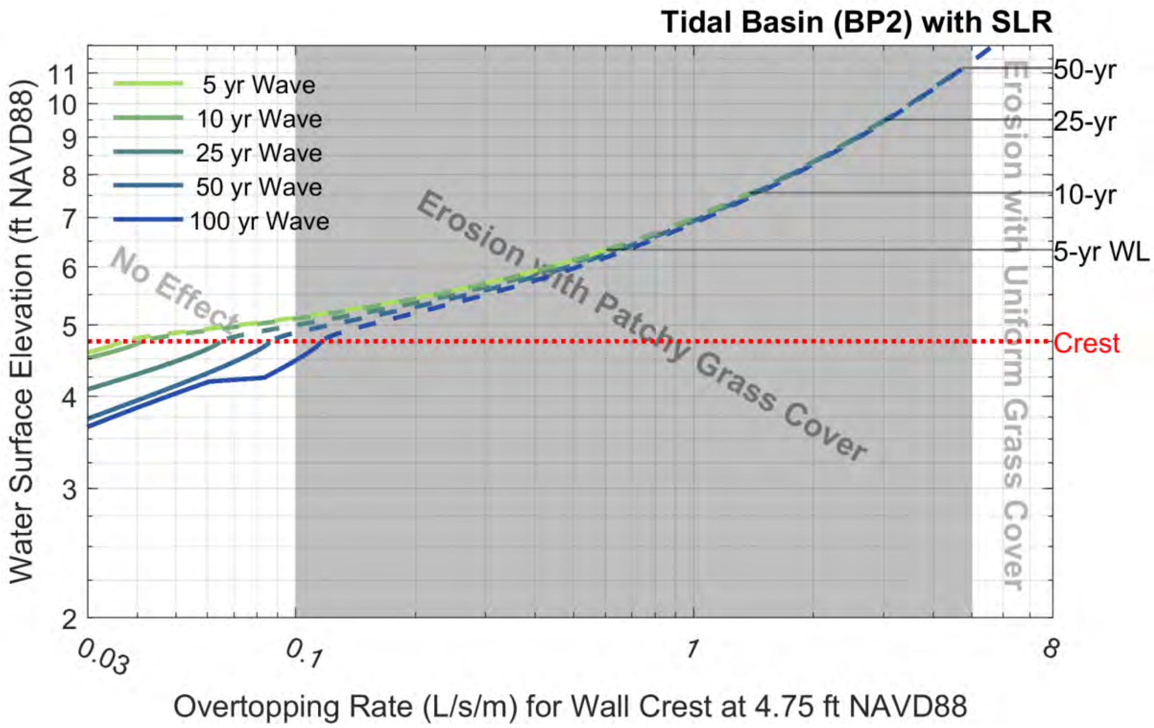

FIGURE 60. OVERTOPPING RATES FOR THE TIDAL BASIN (BP2) FOR FUTURE (2050) SEA LEVELS.

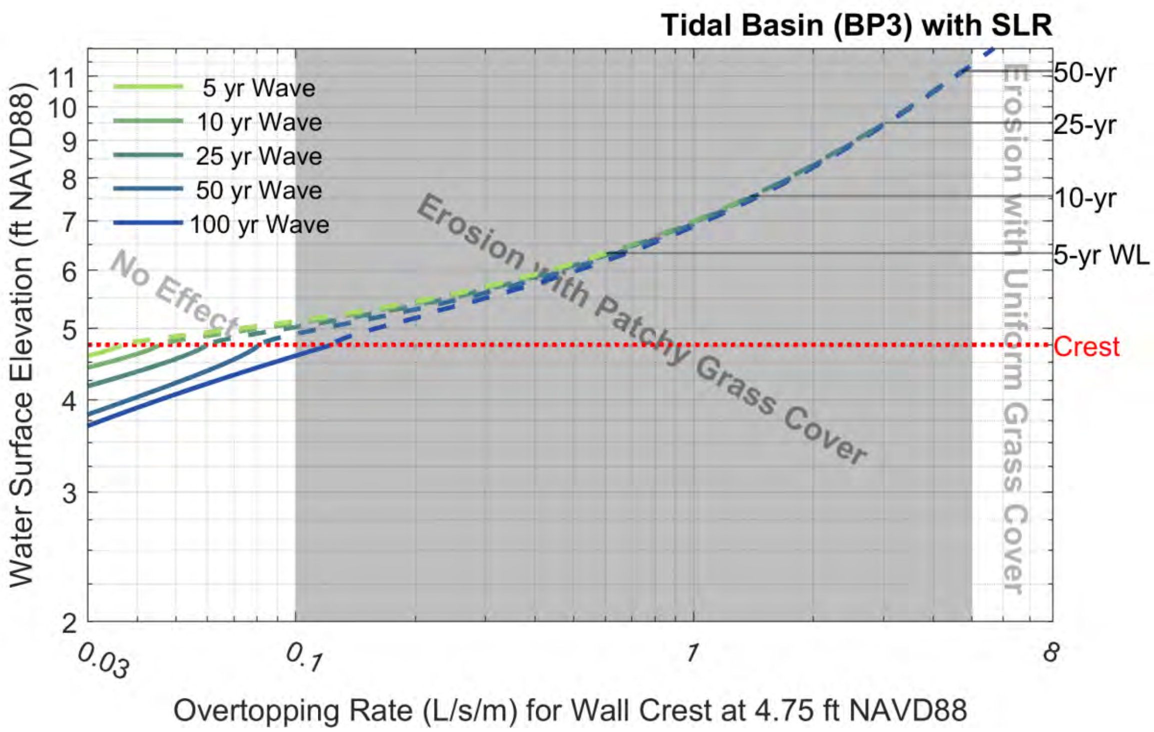

FIGURE 61. OVERTOPPING RATES FOR THE TIDAL BASIN (BP3) FOR FUTURE (2050) SEA LEVELS.

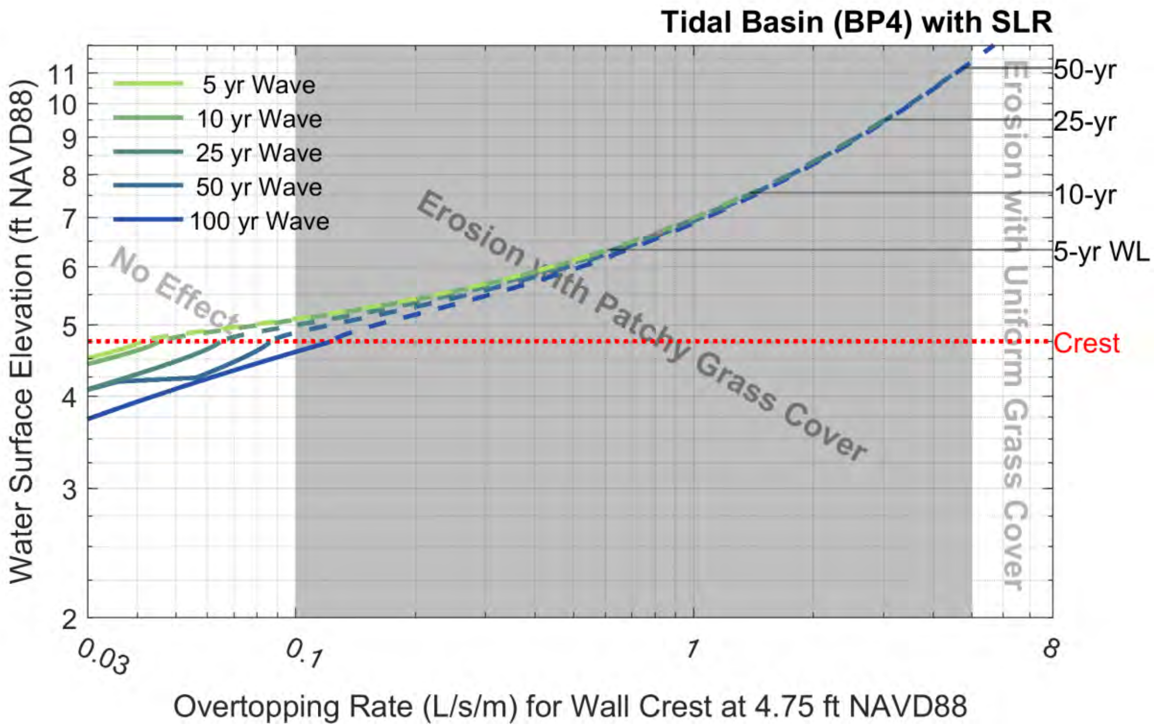

FIGURE 62. OVERTOPPING RATES FOR THE TIDAL BASIN (BP4) FOR FUTURE (2050) SEA LEVELS.

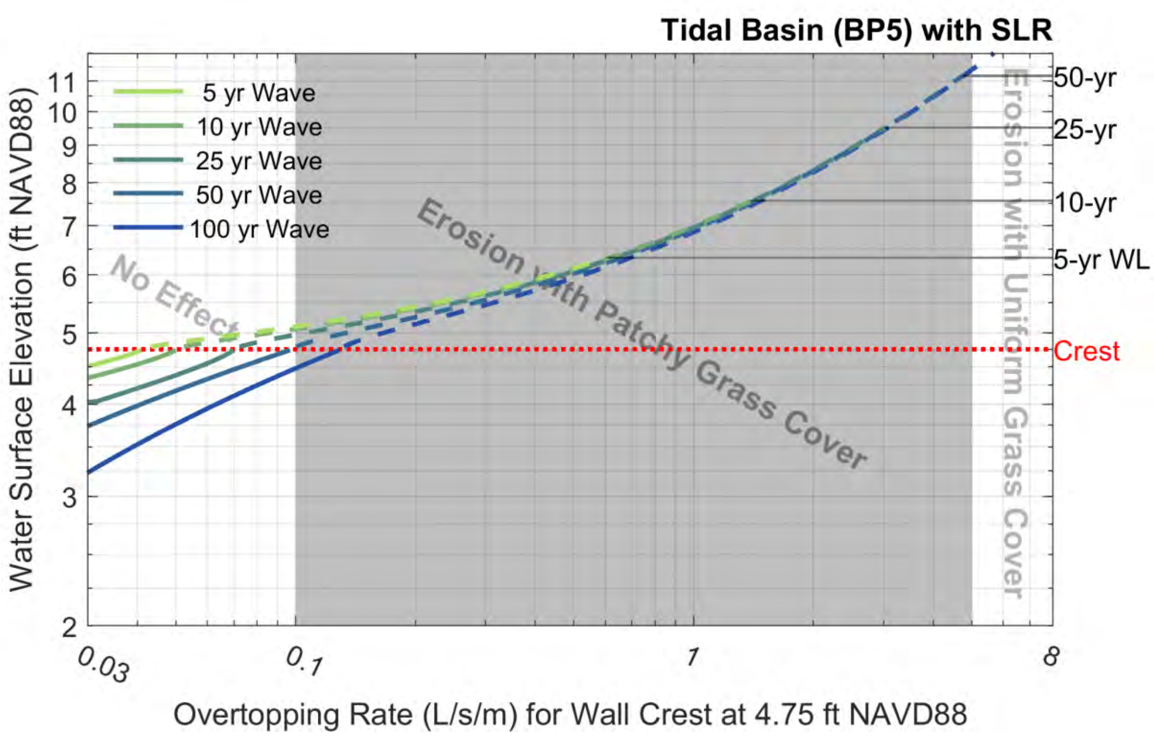

FIGURE 63. OVERTOPPING RATES FOR THE TIDAL BASIN (BP5) FOR FUTURE (2050) SEA LEVELS.

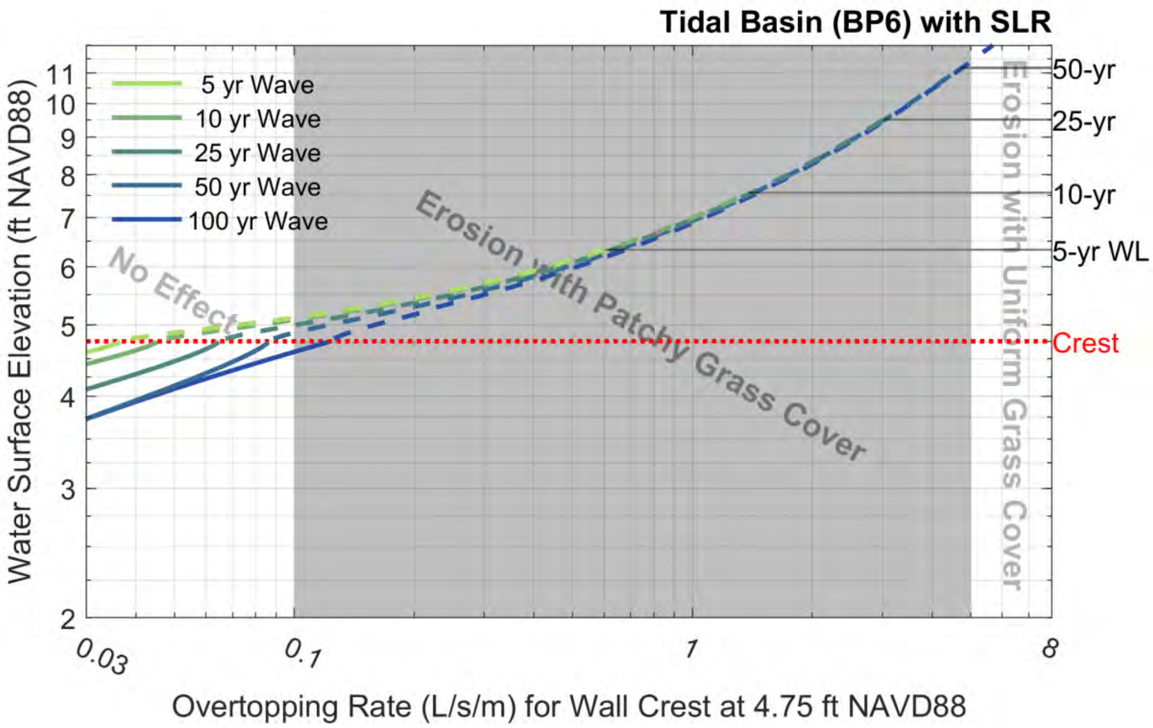

FIGURE 64. OVERTOPPING RATES FOR THE TIDAL BASIN (BP6) FOR FUTURE (2050) SEA LEVELS.

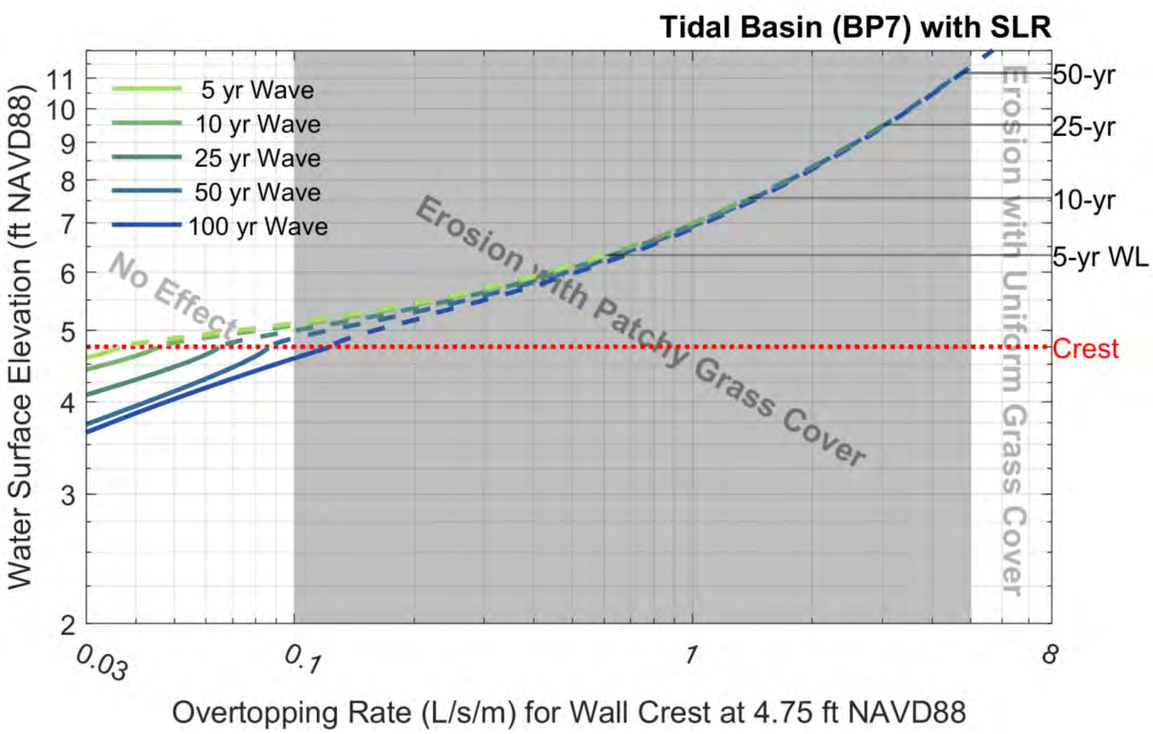

FIGURE 65. OVERTOPPING RATES FOR THE TIDAL BASIN (BP7) FOR FUTURE (2050) SEA LEVELS.

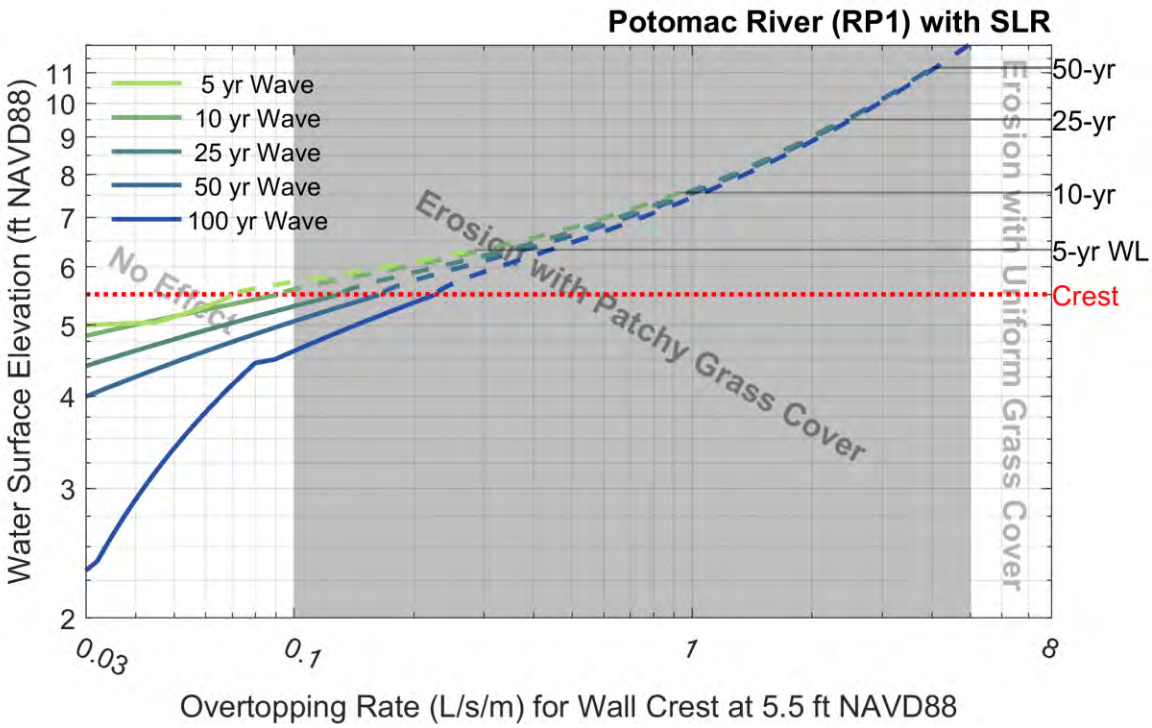

FIGURE 66. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP1) FOR FUTURE (2050) SEA LEVELS.

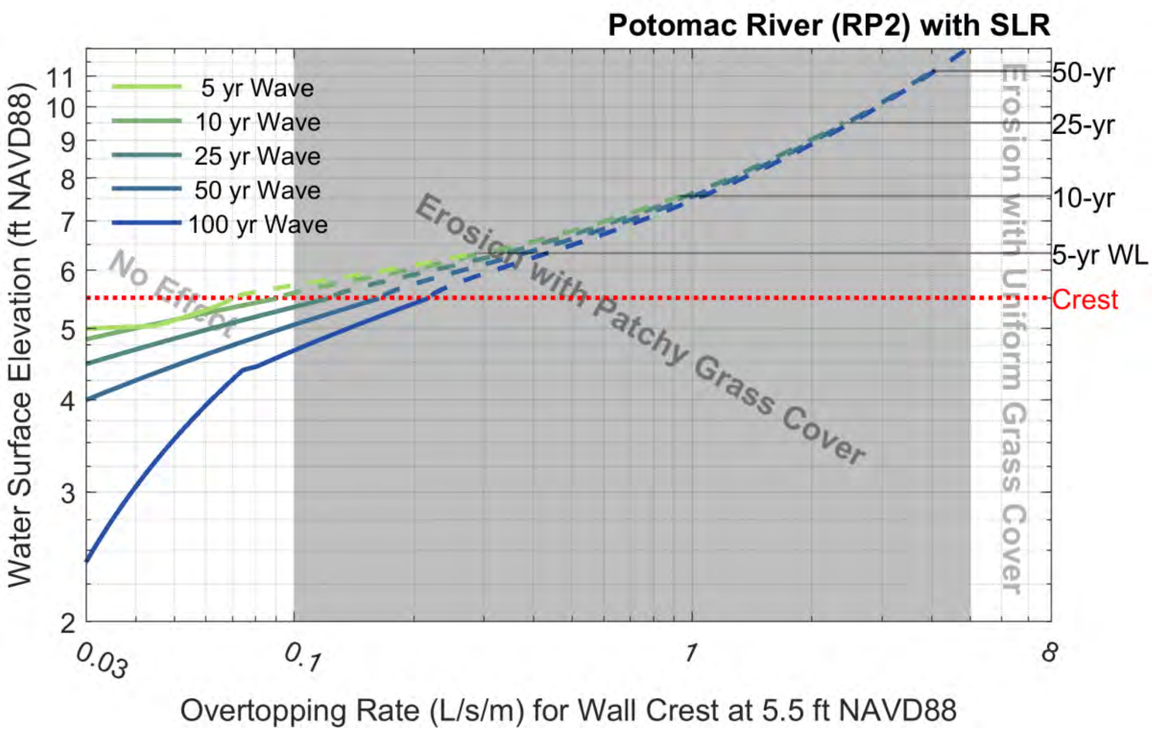

FIGURE 67. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP2) FOR FUTURE (2050) SEA LEVELS.

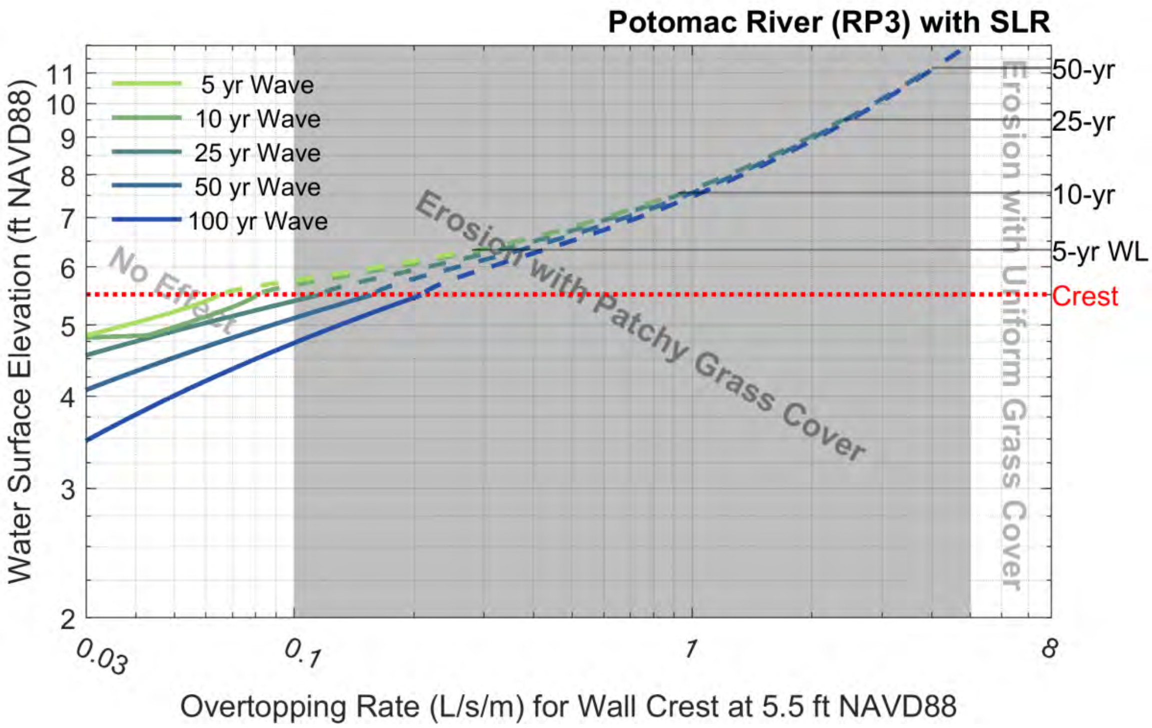

FIGURE 68. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP3) FOR FUTURE (2050) SEA LEVELS.

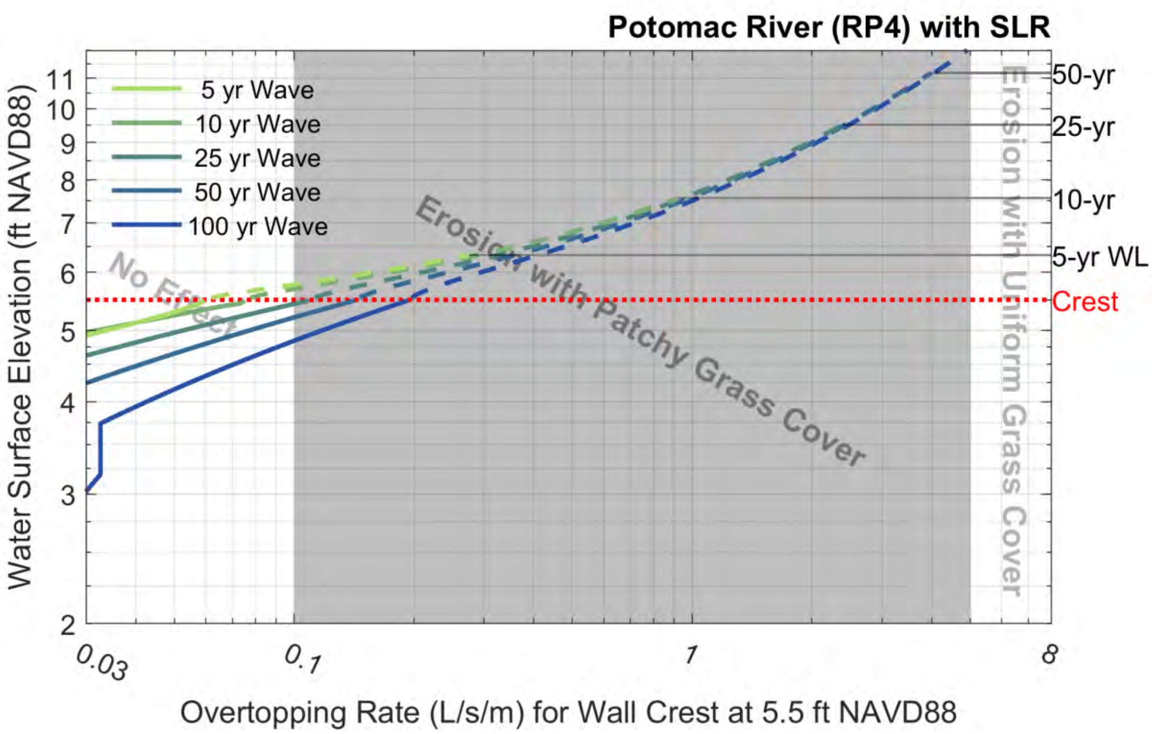

FIGURE 69. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP4) FOR FUTURE (2050) SEA LEVELS.

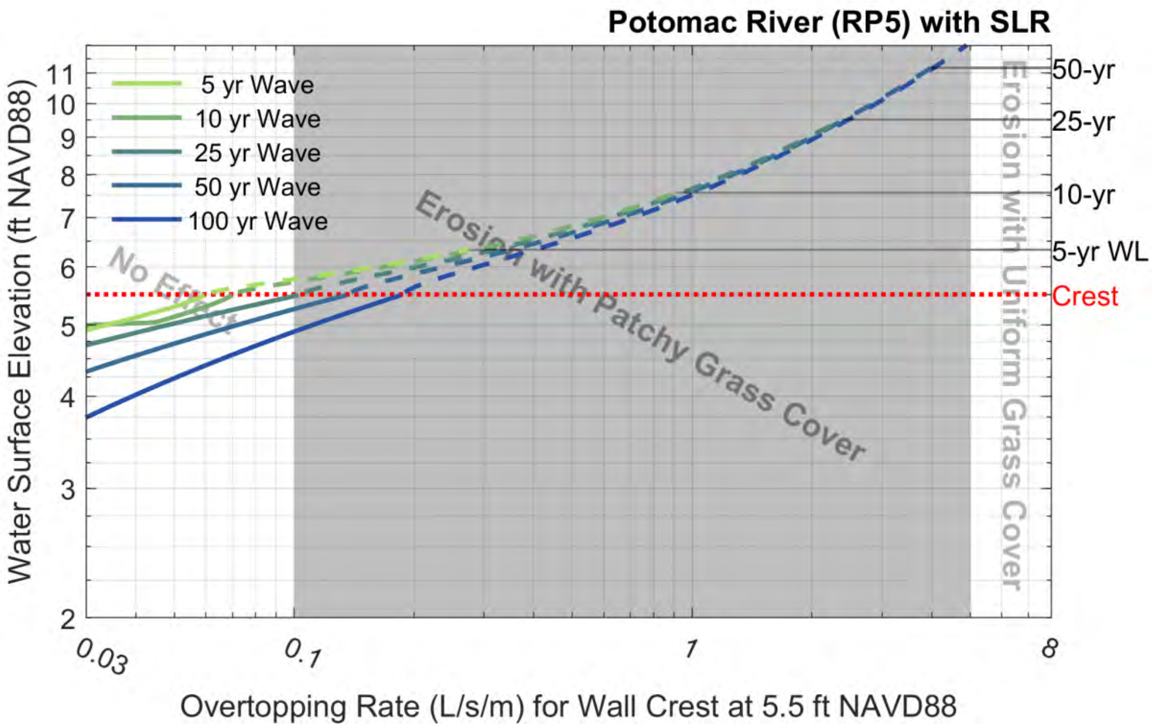

FIGURE 70. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP5) FOR FUTURE (2050) SEA LEVELS.

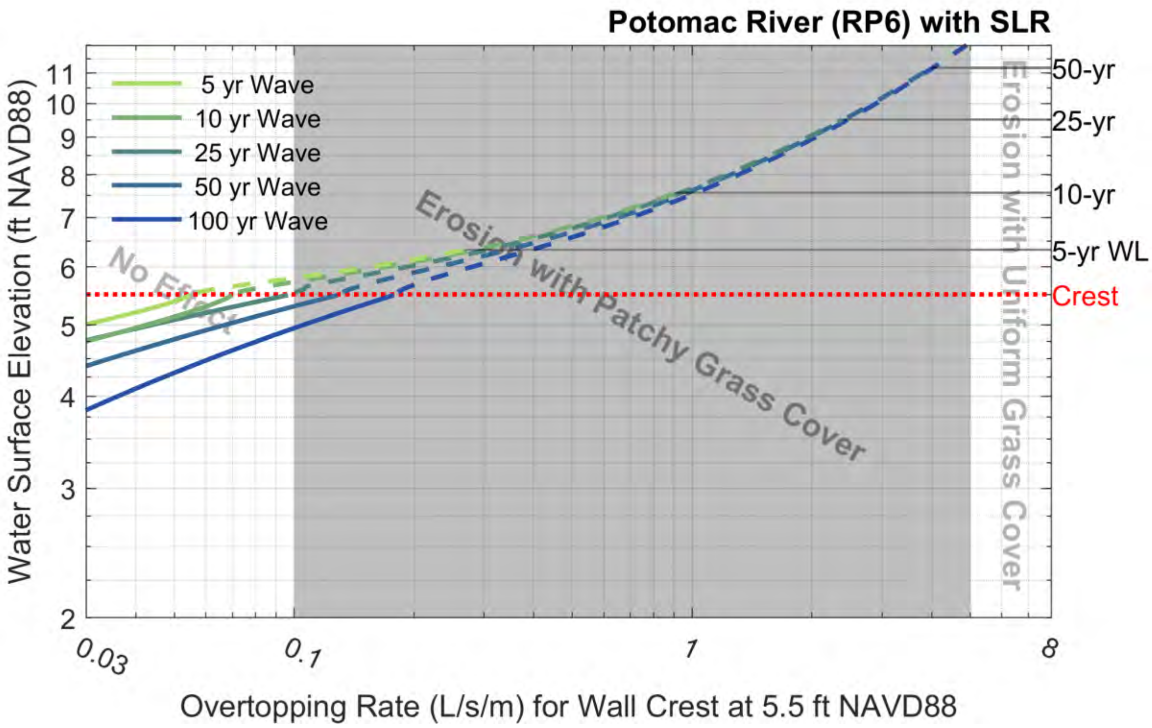

FIGURE 71. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP5) FOR FUTURE (2050) SEA LEVELS.

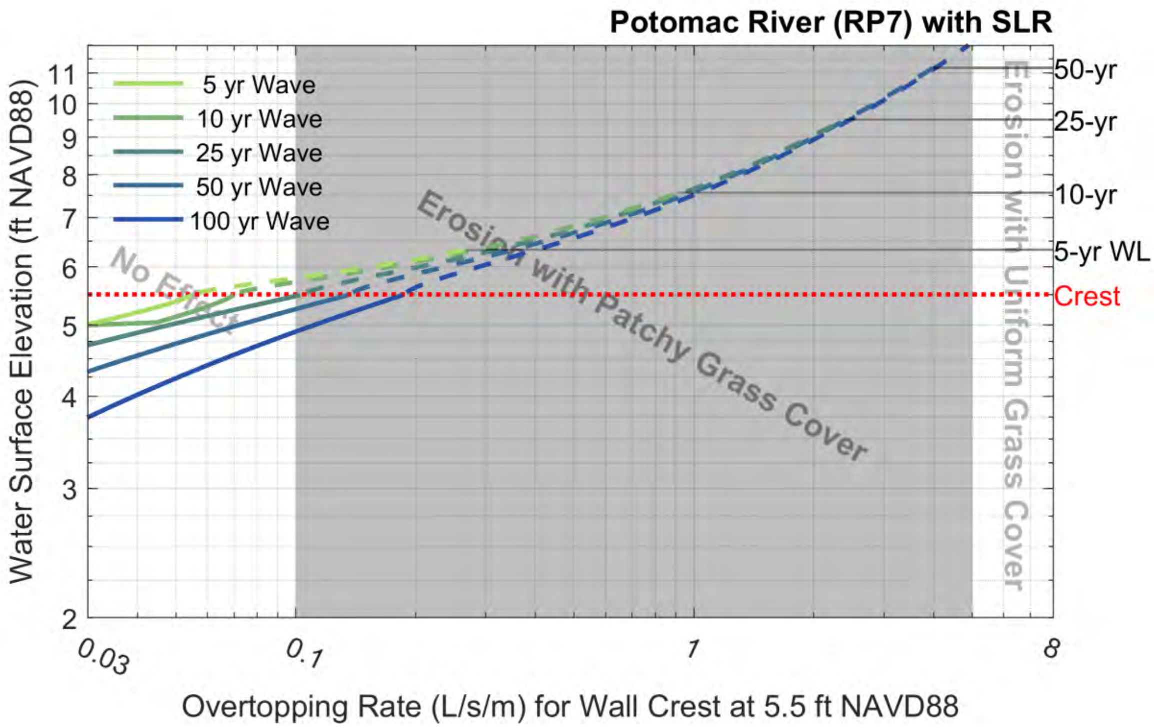

FIGURE 72. OVERTOPPING RATES FOR THE POTOMAC RIVER (RP7) FOR FUTURE (2050) SEA LEVELS.

Appendix H: Wave Loads

For each location and design condition, the horizontal wave-induced force was evaluated for a range of still water level (SWL) conditions: Starting from the MLLW elevation, and then every 0.5 foot increment up to the water level associated with the design condition.

A design wave height was computed for each wave load calculation. H_{des}, was defined as:

$$H_{des} = 1.8 \times H'_s$$

To account for depth-induced wave breaking, H'_s, is the minimum between the depth-limited wave height and the modeled significant wave height:

$$H'_s = \min(0.78 \times d, H_s)$$

Where d is the water depth at the toe of the structure and H_s is the modeled significant wave height for each design condition (Section 4.2.1).

Additionally, a reduction on the horizontal wave forces was considered in case of oblique wave incidence.

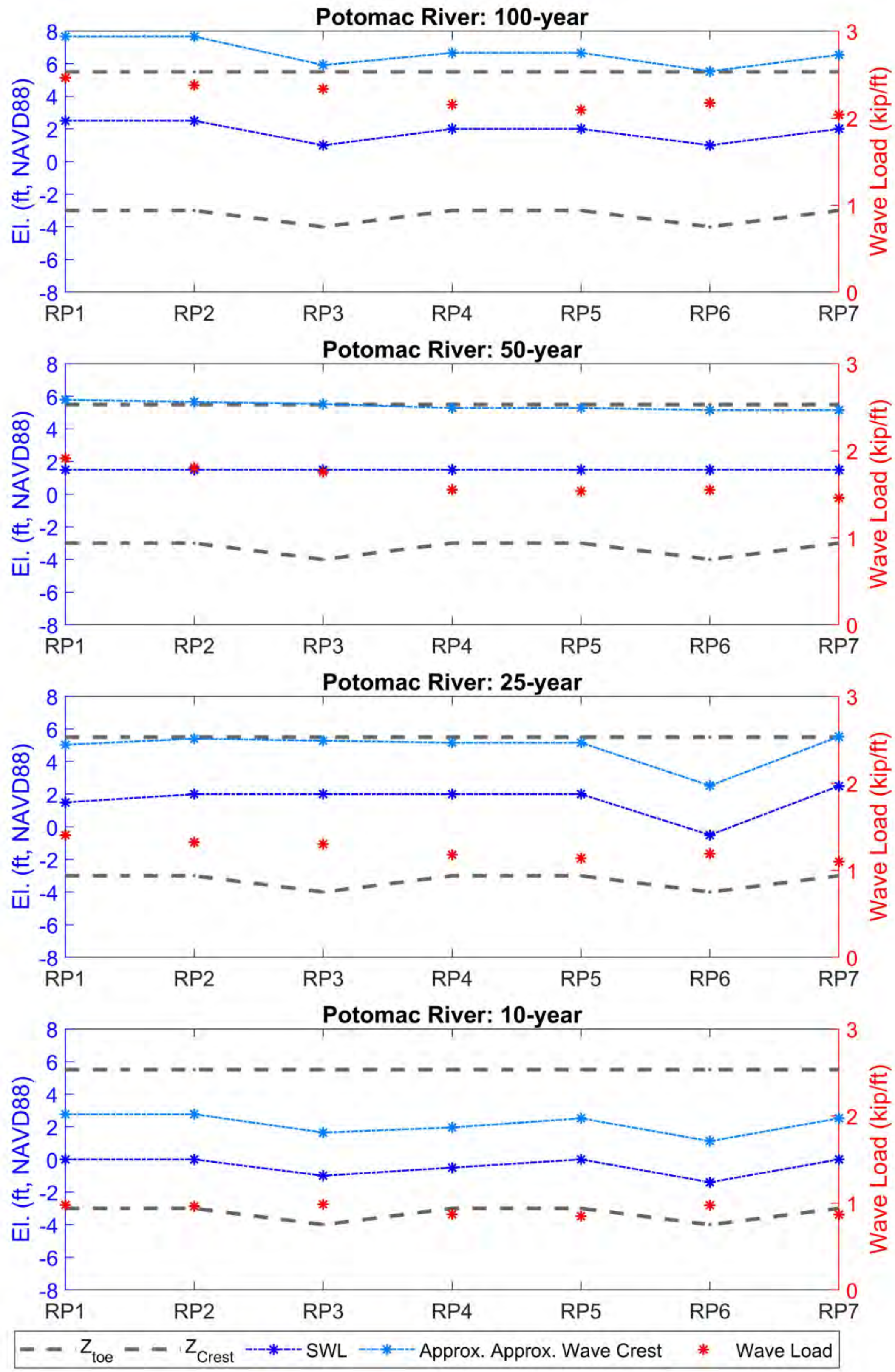

FIGURE 73. WAVE LOAD (RIGHT AXIS) AND ASSOACIATED ELEVATIONS (LEFT AXIS) AT THE POTOMAC RIVER.

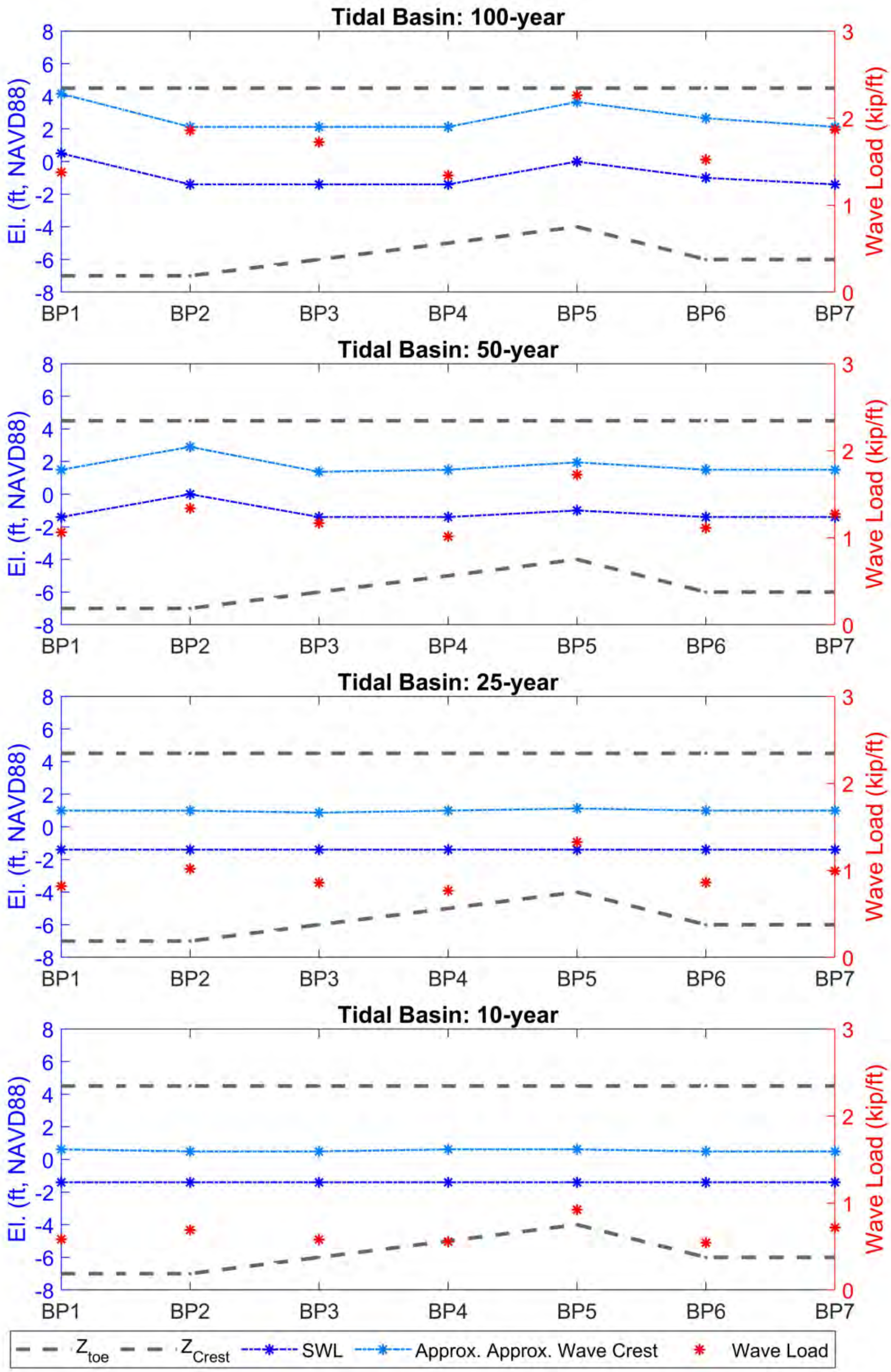

FIGURE 74. WAVE LOAD (RIGHT AXIS) AND ASSOACIATED ELEVATIONS (LEFT AXIS) AT THE POTOMAC RIVER.

TABLE 16. WAVE LOADS AND ASSOCIATED PARAMETERS AT WEST POTOMAC PARK.

Point ID	Bay Bottom (Z_{toe}, ft, NAVD88)	Top of Wall (Z_{crest}, ft. NAVD88)	Return Period (yr.)	Max. Horizontal Force (kip/ft)	Still Water Level (SWL, ft, NAVD88)	Design Wave Height (ft)	Approx. Wave Crest (ft, NAVD88)
RP1	-3	5.5	100	2.5	2.5	7.4	7.7
RP2	-3	5.5	100	2.4	2.5	7.4	7.7
RP3	-4	5.5	100	2.3	1.0	7.0	5.9
RP4	-3	5.5	100	2.2	2.0	6.7	6.7
RP5	-3	5.5	100	2.1	2.0	6.7	6.7
RP6	-4	5.5	100	2.2	1.0	6.5	5.5
RP7	-3	5.5	100	2.0	2.0	6.5	6.5
RP1	-3	5.5	50	1.9	1.5	6.1	5.8
RP2	-3	5.5	50	1.8	1.5	5.9	5.7
RP3	-4	5.5	50	1.8	1.5	5.8	5.5
RP4	-3	5.5	50	1.6	1.5	5.4	5.3
RP5	-3	5.5	50	1.5	1.5	5.4	5.3
RP6	-4	5.5	50	1.5	1.5	5.2	5.2
RP7	-3	5.5	50	1.5	1.5	5.2	5.2
RP1	-3	5.5	25	1.4	1.5	5.0	5.0
RP2	-3	5.5	25	1.3	2.0	4.9	5.4
RP3	-4	5.5	25	1.3	2.0	4.7	5.3
RP4	-3	5.5	25	1.2	2.0	4.5	5.2
RP5	-3	5.5	25	1.1	2.0	4.5	5.2
RP6	-4	5.5	25	1.2	-0.5	4.3	2.5
RP7	-3	5.5	25	1.1	2.5	4.3	5.5
RP1	-3	5.5	10	1.0	0.0	4.0	2.8
RP2	-3	5.5	10	1.0	0.0	4.0	2.8
RP3	-4	5.5	10	1.0	-1.0	3.8	1.6
RP4	-3	5.5	10	0.9	-0.5	3.5	2.0
RP5	-3	5.5	10	0.8	0.0	3.6	2.5
RP6	-4	5.5	10	1.0	-1.4	3.6	1.1
RP7	-3	5.5	10	0.9	0.0	3.6	2.5

TABLE 17. WAVE LOADS AND ASSOCIATED PARAMETRS AT THE TIDAL BASIN.

Point ID	Bay Bottom (Z_{toe}, ft, NAVD88)	Top of Wall (Z_{crest}, ft. NAVD88)	Return Period (yr.)	Max. Horizontal Force (kip/ft)	Still Water Level (SWL, ft, NAVD88)	Design Wave Height (ft)	Approx. Wave Crest (ft, NAVD88)
BP1	-7	4.5	100	1.4	0.5	5.2	4.2
BP2	-7	4.5	100	1.9	-1.4	5.0	2.1
BP3	-6	4.5	100	1.7	-1.4	5.0	2.1
BP4	-5	4.5	100	1.3	-1.4	5.0	2.1
BP5	-4	4.5	100	2.3	0.0	5.2	3.7
BP6	-6	4.5	100	1.5	-1.0	5.2	2.7
BP7	-6	4.5	100	1.9	-1.4	5.0	2.1
BP1	-7	4.5	50	1.1	-1.4	4.1	1.5
BP2	-7	4.5	50	1.3	0.0	4.1	2.9
BP3	-6	4.5	50	1.2	-1.4	4.0	1.4
BP4	-5	4.5	50	1.0	-1.4	4.1	1.5
BP5	-4	4.5	50	1.7	-1.0	4.2	1.9
BP6	-6	4.5	50	1.1	-1.4	4.1	1.5
BP7	-6	4.5	50	1.3	-1.4	4.1	1.5
BP1	-7	4.5	25	0.8	-1.4	3.4	1.0
BP2	-7	4.5	25	1.0	-1.4	3.4	1.0
BP3	-6	4.5	25	0.9	-1.4	3.2	0.9
BP4	-5	4.5	25	0.8	-1.4	3.4	1.0
BP5	-4	4.5	25	1.3	-1.4	3.6	1.1
BP6	-6	4.5	25	0.9	-1.4	3.4	1.0
BP7	-6	4.5	25	1.0	-1.4	3.4	1.0
BP1	-7	4.5	10	0.6	-1.4	2.9	0.6
BP2	-7	4.5	10	0.7	-1.4	2.7	0.5
BP3	-6	4.5	10	0.6	-1.4	2.7	0.5
BP4	-5	4.5	10	0.6	-1.4	2.9	0.6
BP5	-4	4.5	10	0.9	-1.4	2.9	0.6
BP6	-6	4.5	10	0.5	-1.4	2.7	0.5
BP7	-6	4.5	10	0.7	-1.4	2.7	0.5