Appendix B
Coastal Modeling Report

North/West Battery Park City Resiliency Project

Coastal Modeling Report in Support of 60% Design for NWBPCR

05/10/2024

Version Control

Document Dated	Version No.	Revision No.	Date Issued	Description	Reviewed By
3/31/2023	1	0	3/31/23	Submit as part of Draft 30% Internal Review Package	John Atkinson; Joe Marrone; Dan Rozell
4/14/2023	2	1	4/14/23	Submit as part of Draft 30% Package to BPCA	John Atkinson; Joe Marrone; Dan Rozell
4/28/2023	2	1	4/28/23	Report date was updated from 4/14 to 4/28 as part of the 30% Design Submission Package to BPCA. No changes from the version of the report submitted on 4/14 as part of the Draft 30% Design Development Package to BPCA	John Atkinson; Joe Marrone; Dan Rozell
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1/05/2024	6	0	1/05/24	Updated no-rise analysis with results from ADCIRC+SWAN modeling; updated wave modeling for modified alignment, platform elevations, and topo; updated overtopping and FBS crest elevations; updated loads on walls and under-	John Atkinson; Joe Marrone;

Document Dated	Version No.	Revision No.	Date Issued	Description	Reviewed By
				platform structures, and updated platform uplift forces.	
1/23/2024	6	1	1/23/24	Updated per TE comments. Submit as part of Draft 40% Review Package to BPCA.	John Atkinson
4/05/2024	7	1	4/05/2024	Updated Reach 1 alignment and top of wall elevation, no-rise analysis with results from 2D SWAN modeling; updated wave loads analysis results for Reach 5 and 7.	John Atkinson, Joe Marrone;
5/10/2024	7	2	5/10/2024	Updated per TE comments. Submit as part of 60% Review Package to BPCA.	John Atkinson, Joe Marrone;

Note:

1) All = Turner EE Cruz, Arcadis and its subconsultants (SCAPE/BIG/WSP, etc.)
2) Arcadis team = Arcadis and its subconsultants (SCAPE/BIG/WSP, etc.)

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Note to Reviewer: The appendices can found in the Procore submission of the 60% Design Submission

Acronyms & Abbreviations

ADCIRC	Advanced Circulation Model
BMCC	Borough of Manhattan Community College
BOD	Basis of Design
BPCA	Battery Park City Authority
CFR	Code of Federal Regulations
CLOMR	Conditional Letter of Map Revision
DFE	Design Flood Elevation
ERP	Emergency Response Plan
FBS	Flood Barrier System
FEMA	Federal Emergency Management Agency
FIRMs	Flood Insurance Rate Maps
FOS	Factors of Safety
HAT	Harbor and Tributaries
HRPK	Hudson River Park
HRPT	Hudson River Park Trust
LOMR	Letter of Map Revision
MOCEJ	Mayor's Office of Climate and Environmental Justice
MPT	Maintenance and Protection of Traffic
NAVD88	North American Vertical Datum of 1988 (2004.65)
NOAA	National Oceanic and Atmospheric Administration
NPCC	New York City Panel on Climate Change
NWBPCR	North/West Battery Park City Resiliency
NYCDEP	New York City Department of Environmental Protection
NYCDOT	New York City Department of Transportation
NYCEM	New York City Emergency Management
NYSDOT	New York State Department of Transportation
O&M	Operation and Maintenance
PANYNJ	Port Authority of NY and NJ
PATH	Port Authority Trans-Hudson
PDB	Progressive Design Build
PPE	Personal Protective Equipment
SBPCR	South Battery Park City Resiliency
SLR	Sea Level Rise
SWEL	Still Water Elevation
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey

1 Executive Summary

This technical report was prepared for the North/West Battery Park City Resiliency (NWBPCR) 60% design. The NWBPCR Project's primary goal is coastal storm flood risk reduction within inland of Battery Park City. It is proposed to achieve the goal of coastal storm risk reduction by implementing a flood barrier system (FBS) and integrating the system into the city landscape. The FBS will meet the design criteria for a 2050 100-year storm including rainfall, coastal storm surge with associated wave action, and predicted 2050s' 90th percentile SLR. It is the objective of the NWBPCR Project, once installed, to be accredited by FEMA. Accreditation requires a FEMA review of as-built plans and verification that the FBS meets all pertinent requirements and achieves acceptable risk reduction in practice. FEMA accreditation will acknowledge the reduction of risk resulting from the installation of the FBS and the change from the current flood zone to a lower risk category. The project should also seek to limit future tidal flooding due to SLR, and the interior drainage and pump station improvements will mitigate the potential impacts of coastal flooding.

Table 1: Coastal Modeling and Analysis Progress (30% to 60%)

Project Phase	Wave Modeling	Stillwater Level Determination	Engineering Analyses	Coastal Flooding	Impact of Butterfly Valves
Draft 30% Design Development	1D SWAN	Design still water level (SWEL) based on PFIRM and SLR projection	Overtopping rate analysis; wave loading analysis	PFIRM used	Depth-averaged Delft3D
30% Design	1D SWAN; FUNWAVE	Design still water level (SWEL) based on PFIRM and SLR projection	Carried over from previous phase	PFIRM used	Depth-averaged Delft3D
30% Consensus	1D SWAN	Design still water level (SWEL) based on PFIRM and SLR projection	Carried over from previous phase	PFIRM used	Depth-averaged Delft3D
Draft 40% Design Development	2D SWAN	Design still water level (SWEL) based on PFIRM and SLR projection	Alignment updated; Wall Height updated; Wave loading updated; Overtopping rate and wave runup updated	ADCIRC no-rise analysis	Depth-averaged Delft3D carried over
Draft 60% Design Development	2D SWAN	Design still water level (SWEL) based on PFIRM and SLR projection	Alignment updated; Wall Height updated; Overtopping rate updated; Wave loading updated; no-rise analysis updated	ADCIRC no-rise analysis carried over	Depth-averaged Delft3D carried over

Coastal Modeling and Analysis Progress

Coastal numerical modeling and analysis were progressed to support the 60% design development of the FBS. **Table 1** summarizes the progress since 40%.

Wave conditions near the FBS were evaluated using a two-dimensional wave model during this 60% design. A no-rise analysis was conducted for the entire project and a more focused analysis was done on how the Hudson River Park (HRPK) would be affected, specifically. Engineering analyses of wave overtopping rate and wave loading were updated with the two-dimensional wave modeling results in this 60% design with the updated Reach 1 alignment and top of wall elevation. The hydraulic impacts of butterfly valves (previously called sluice gates) were evaluated in 30% design using the depth averaged Delft3D model. No additional Delft3D modeling was conducted in 60% design pending further coordination with stakeholders and agencies.

The no rise analysis was updated in this 60% design as well.

Coastal Modeling and Analysis Updates

As part of the 60% design for the NWBPCR Project, a two-dimensional wave model was developed to better understand wave characteristics in the projects area and to update design wave conditions for engineering analyses. The FEMA 100-year SWEL of 11.3 feet NAVD88 with a sea level rise scenario of 2.5 feet (NPCC) was selected as the design flood elevation (elevation 13.8 feet NAVD88) for the 60% design.

Part of the 60% design tasks included a no-rise analysis for storm surge water level and water waves near the HRPK in Reach 1. The analysis for water waves is based on a northwestern wind occurring at the same time as the maximum water level which is a very conservative scenario. In the 60% design, a 2-dimensional (2D) wave analysis was developed, covering the entire project area, to evaluate wave attenuation concepts and provide updated detailed design conditions and the top of wall elevation for the flood defense along Reach 1. Elements, including floodwall, platforms, and wave attenuation features, etc., are shown in column “Assumptions” in **Table 2** and **Figure 1**. The 2D wave model analysis was completed using the Simulating Waves Nearshore (SWAN) numerical model package. The model illustrated shows negligible impacts to the NWBPCR project and inundations levels in HRPK are not impacted. No-rise analysis for water waves shows minor impacts on wave heights as well. The details of the wave model development are presented in Section 4.

A wave overtopping analysis was completed in compliance with the USACE Hurricane and Storm Damage Risk Reduction System Design (HSDRRS, USACE 2012) guidelines. The HSDRRS analysis considers the uncertainty of wave conditions and still water levels for overtopping analysis using the Monte Carlo simulation method and utilizes the Van der Meer’s overtopping formulas (TAW 2002), which have been adopted in the EurOtop manual (2018) with improvements. Utilizing the overtopping design criteria recommended in HSDRRS, the overtopping analysis was performed to determine the top-of-wall elevation of the proposed floodwall with the updated Reach 1 alignment and wall heights. In this overtopping analysis, the wave conditions are extracted from the 2D SWAN model results at approximately one wavelength from the floodwall; the ground elevation in front of the floodwall was used to adjust the wave height using a conservative breaker index of 0.8. Instead of using TAW (2002), the overtopping formula in the most recent EurOtop manual (2018) was used. All the wave model results, wave overtopping, and top of wall elevations were based on the latest 60% updated topography with final landscape. The 60% design of the recommended top of wall elevations along the alignment are shown in **Figure 1** and **Table 2**.

Figure 1: 60% alignment and recommended wall elevation (ft, NAVD88).

Table 2: 60% Recommended Top of Wall Elevation by Reach / Sub-Reach

Reach	Sub-Reach	Recommended Top of Wall Elevation per 30% Design (ft – NAVD88)	Recommended Top of Wall Elevation for 60% Design (ft-NAVD88)	Notes / Assumptions
1	North Moore	16.0	16.0	FEMA minimum – minimal wave action expected
1	BMCC north	16.5	18.0	Minimize the overtopping rate due to the new alignment close to BMCC buildings
1	BMCC south	17.5	18.0	Minimize the overtopping rate due to the new alignment close to BMCC buildings
2	All	16.5	16.5	Assumes platform replacement with new deck elevation or wave attenuation features above El. 12 (see Reach 4 results)
3	All	No Wall	No Wall	Similar to RFP results
4	All	16.5	16.5	Assumes platform replacement with new deck elevation, recurved flood wall in front of 300 Vesey St.
5	North of marina	16.5	16.5	Similar to east of marina
5	East of marina	16.5	16.5	Section of FBS furthest from Hudson River within marina
5	South of marina	16.5	16.5	Similar to east of marina; higher ground elevation at flood wall
6	North of Gateway	16.5	19.5	Recurved flood wall and connect with next section with 18' flood wall
6	Gateway	16.5	16.5	Assumes platform replacement with new deck elevation, recurved flood wall in front of Gateway Apartments
6	South of Gateway	19.5	19.5	Short platform distance from open water, recurved flood wall
7	North of South Cove	19.5	19.5	Short platform distance from open water—has large fetch from southwest
7	North part East of South Cove	16.5	16.5	Setback and higher ground between waterfront and FBS create less extreme wave conditions, recurved flood wall
7	South part of East of South Cove	16.5	18.0	This part wall is closer to shoreline with higher wave action, recurved flood wall

Wave runup along the floodwall was computed for 1% annual chance SWEL under the current condition. The runup calculation formulas in both the Shore Protection Manual (SPM 1984) and the EurOtop Manual were used. The runup calculation is used for the assessment of FEMA freeboard requirements and provides the reference for wall height changes in process of design optimization. It is illustrated in Section 5.2 that the 60% design of top-of-wall elevation meets the freeboard requirement for the current condition. However, freeboard is affected by sea level rise over the project life. With a SLR of 2.5 feet, both SWEL and wave heights will be increased, which will expose some segments of the floodwall to the risk of insufficient freeboard in the 2050's. The structure adaptability and sustainable design will prevent this risk based on the evaluation in future conditions.

Wave loads on vertical walls were calculated using the Goda formula (Table VI-5-53 in CEM 2011), which is consistent with HSDRRS and meets FEMA requirements (FEMA P-55, 2011). Wave dynamic loads for the 1% annual chance SWEL was estimated using the 1% annual chance SWEL associated wave height derived from FEMA PFIRM storm suite model results. The 500-year wave load was estimated by applying the 0.2% annual chance SWEL and associated wave height. The wave loads were applied to the wall as directed in HSDRRS Design Guidelines. Wave loading analysis in this document was updated based on 2D model results.

The floodgate hydraulics for the two canals underneath the platform, located above the PATH tunnels in Reach 5 will be evaluated with Delft3D or Flow3D in 90% design phase.

2 Introduction

The purpose of this technical memorandum is to document the coastal modeling analyses that were completed as part of the 60% design submittal for the North/West Battery Park Resiliency Project. The work described herein corresponds to the executed scope of work.

The modeling performed as part of 60% design includes:

- Two-Dimensional SWAN modeling to estimate wave conditions for overtopping and load calculations for the updated Reach 1 alignment and top of wall elevation.
- No-rise analysis for the entire project and specifically zooming into the HRPK area.
- Update the wave loading for platforms (decks and piles) and also bridge structures in Reach 5 and gravity walls in Reach 7.
- Additional modeling is expected to be completed during the 90% design for Delft3D modeling of the submerged gates.

2.1 Project Area

The Project Area is defined as the region loosely bounded by North Moore Street adjacent to the Borough of Manhattan Community College (BMCC) to the North and the South Cove on 1st Place to the South, as shown in **Figure 2**.

Figure 2: Project Area

2.2 Project Description

The project is located in lower Manhattan and includes the installation of a continuous FBS within Battery Park City (BPC) along the Hudson River waterfront between 1st Place in the south (tying into the SBPCR project) and the BPC North Esplanade in the north (tying into high ground). The flood barrier alignment continues across Route 9A/West Street and continues to the east along North Moore Street. In addition to the FBS, interior drainage improvements will be added including a near surface isolation (NSI) and pump station system along Route 9A/West Street and four tide gates along the flood barrier alignment on the northwest end of Rockefeller Park, West of Albany Street, under the ferry terminal, and West of Rector Park (**Figure 3).** In the 40% design, the preferred solutions for the alignment were calculated and are shown in **Figure 4**.

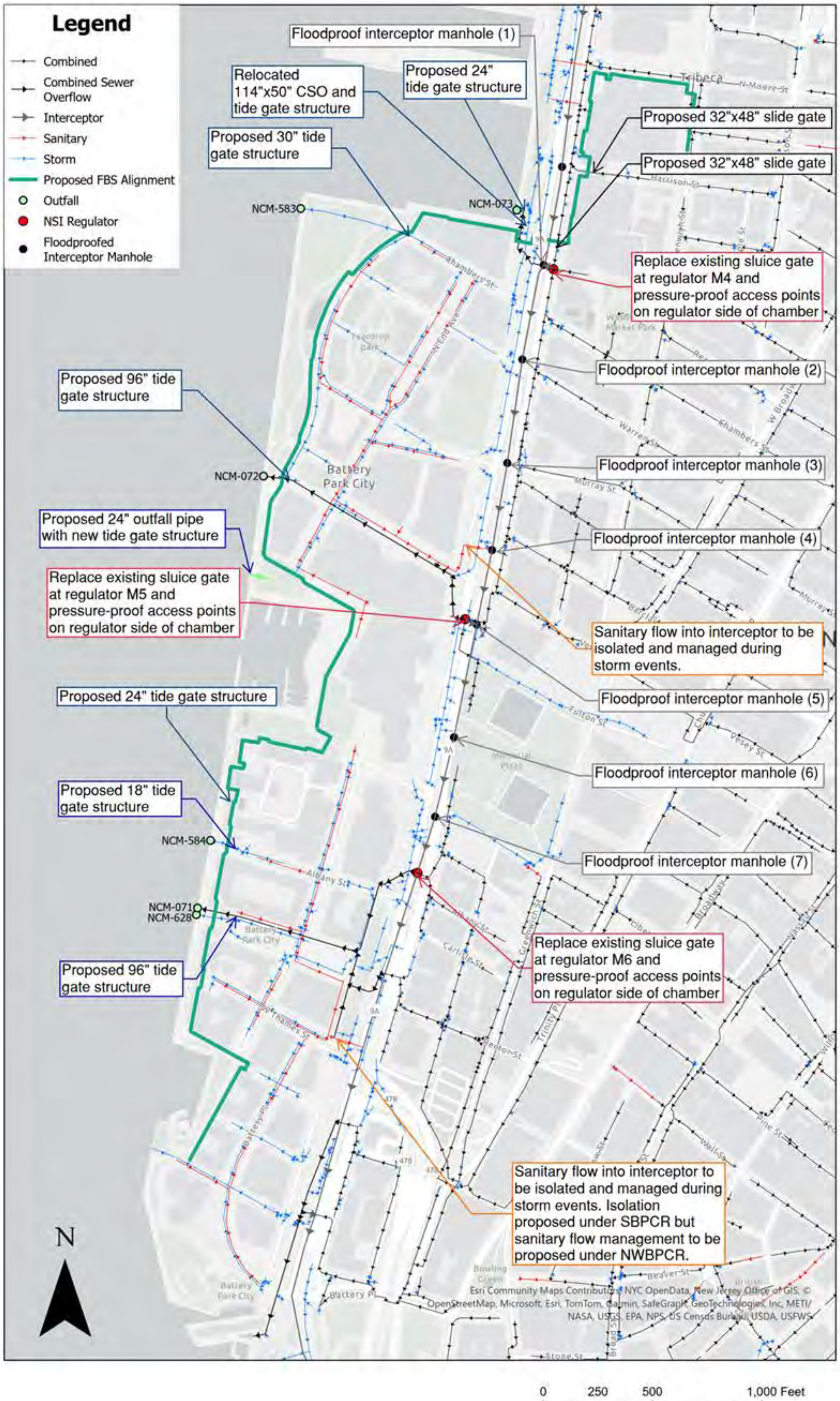


Figure 3: Proposed Interior Drainage Isolation Approach

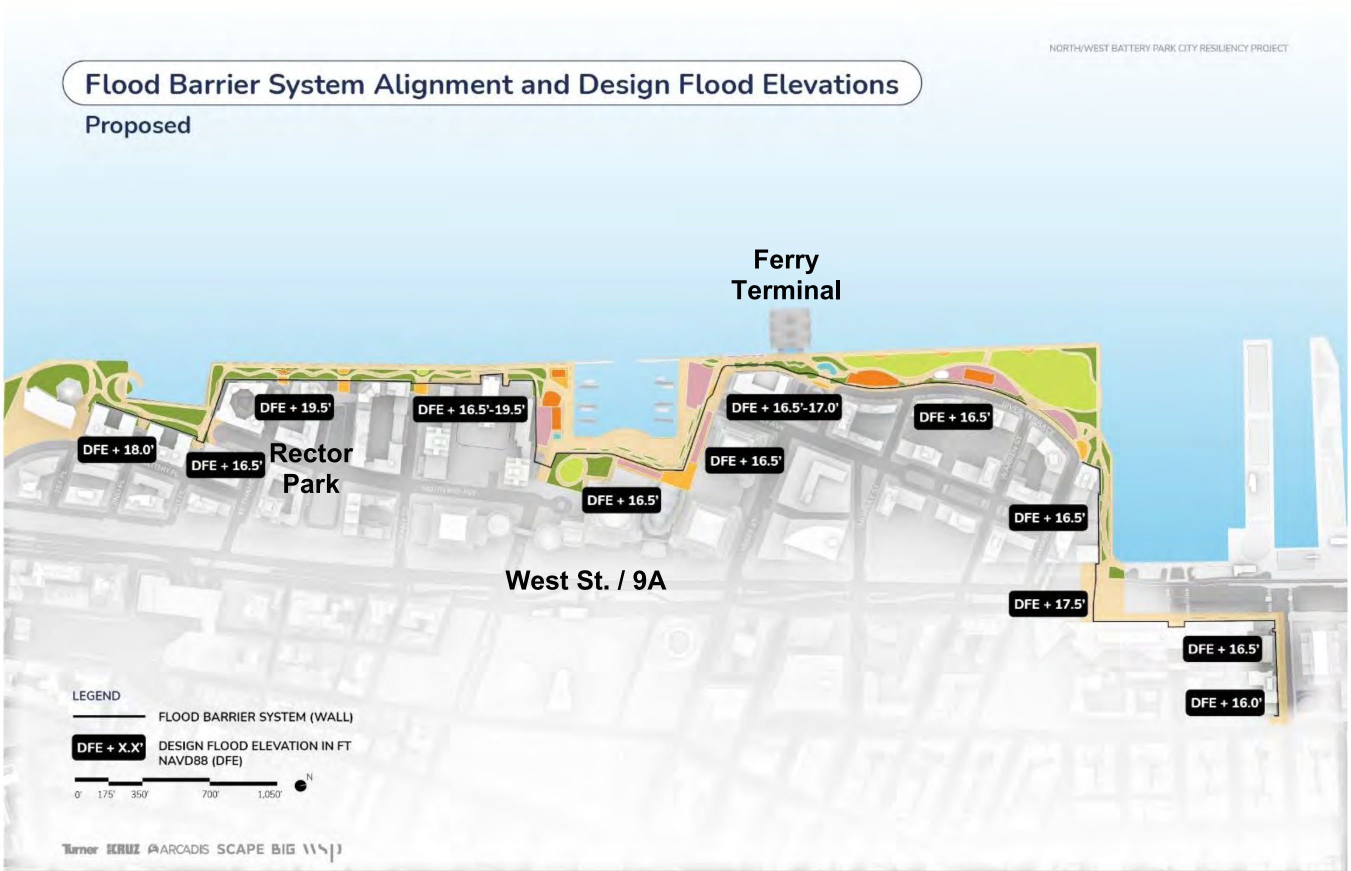


Figure 4: Project site and alignments and recommended top of wall elevation by reach.

3 Design Water Levels

3.1 Stillwater Elevation (SWEL)

The NWBPCR FBS is designed to mitigate the flooding risk associated with FEMA's 100-year storm flood event and address potential impacts of climate change over the design life of the project and beyond. The most recent 1% annual chance still water elevation (SWEL) released from the Preliminary Flood Insurance Rate Map (PFIRM) is the best available data and is consistent with other lower Manhattan coastal resiliency projects and the NYC Building Code. For the 60% design, the SWEL of 11.3 feet (NAVD88) between FEMA preliminary Flood Insurance Study (PFIS) Transects NY-19 and NY-20 was selected as representative of the project area, as shown in **Figure 5.** It should be noted that the PFIRM was successfully appealed by NYC and new maps are under development.

The PFIRM SWEL was also compared to the USACE North Atlantic Comprehensive Coastal Study (NACCS), which was completed in response to Hurricane Sandy. Figure 6 shows the FEMA save points and adjacent NACCS save points. The SWELs at various return intervals from three NACCS save points near the project area are listed in **Table 2** for comparison. This comparison confirmed that a 1% annual chance SWEL of 11.3 ft (NAVD88) is appropriate for use in the analysis.

Flood Source	Transect	Starting Wave Conditions for the 1% Annual Chance			Starting Stillwater Elevations[1] (ft NAVD88) Range of Stillwater Elevations[2] (ft NAVD88)			
		Coordinates	Significant Wave Height (ft)	Peak Wave Period (sec)	10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
East River	NY-14	N 40.709185 W 73.987873	2.58	2.65	6.8 6.6 - 6.8	9.6	11 10.9 - 11	14.3 14.3 - 14.5
East River	NY-15	N 40.708799 W 73.997024	2.93	3.09	6.8 6.7 - 6.8	9.7	11.1 10.9 - 11.1	14.6
East River	NY-16	N 40.705313 W 74.003819	3.12	3.51	6.8 6.6 - 6.8	9.8 9.7 - 9.8	11.2 10.9 - 11.3	14.7 14.7 - 14.8
East River	NY-17	N 40.701590 W 74.010682	3.37	3.21	6.9 6.9 - 7.4	9.9	11.3 10.7 - 11.3	14.8 14.8 - 14.9
Upper New York Bay	NY-18	N 40.702983 W 74.017383	5.01	5.17	6.9	9.9	11.3 11 - 11.3	14.9 14.7 - 14.9
Hudson River	NY-19	N 40.709054 W 74.018613	5.02	5.81	6.9 6.9 - 7.3	9.8 9.1 - 9.8	11.3 10.5 - 11.3	14.8 14.3 - 15.1
Hudson River	NY-20	N 40.715891 W 74.017227	4.94	5.82	6.6	9.7	11.2 10.7 - 11.2	14.7 14.3 - 14.7

Figure 5: PFIS Transect NY-19.

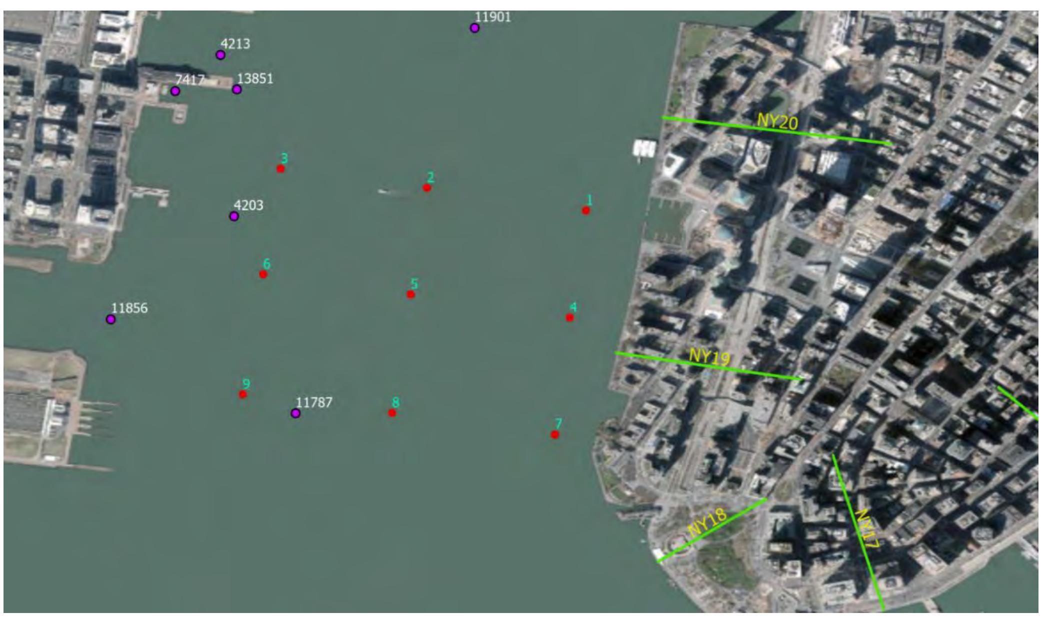


Figure 6: NAACS data at purple points. FEMA comparison points are red.

Table 3: NACCS Return Interval SWELs (feet, NAVD88)

NACCS Save_Point	10Yr	20Yr	50Yr	100Yr	200Yr	500Yr
11787	7.7	8.6	10.0	11.3	12.8	14.9
4203	7.7	8.6	9.9	11.2	12.7	14.8
11901	7.6	8.5	9.9	11.2	12.7	14.8

3.2 Sea Level Rise

In keeping with surrounding projects, the NWBPCR project design includes measures to address the potential effects of future SLR based on predicted values established by the New York City Panel on Climate Change (NPCC,2019). The project Technical Criteria document specifies that the 2050s 90th percentile SLR projection was selected by BPCA as most appropriate for this project. This is consistent with other Lower Manhattan climate adaptation projects.

The NPCC released updated SLR projections specific to the NYC region in January 2015 (Horton et al. 2015) and re-iterated the same numerical values in 2019. The projected SLR for the 2050s, 2080s, 2100s, and extrapolations to the 2120s (extrapolations based on 2080s to 2100s rates) are shown in **Table 4**.

Table 4: SLR Projections for NYC (Feet)

Year	10th Percentile	50th Percentile	90th Percentile
2050s	0.7	1.3	**2.5**
2080s	1.1	2.4	4.8
2100s	1.3	3.0	6.3
2120s	1.5	3.6	7.8

The rate of historical sea level change at the Battery in Lower Manhattan from 1856 to 2015 is less than the 10th percentile sea level projections. The 90th percentile SLR has a theoretical 90% chance of non-exceedance and is therefore conservative in nature.

The 90th percentile SLR in the 2050s of 2.5 feet has been established by the City and adopted by the NYSDEC (6 CRR-NY 490) as a criterion for design. Including the 2050s 90th percentile SLR projection, the 100-year SWEL for a future scenario is estimated as 11.3 feet (NAVD88) + 2.5 feet, resulting in a final design SWEL of 13.8 feet (NAVD88)**.** Linearly superimposing SLR directly to the current days' SWEL was confirmed to be a valid approach by NPCC (Orton, 2015). The 2050s 90th percentile sea level projection (2.5 feet) also meets the 2080s 50th percentile sea level projection (2.4 feet); in other words, the 2050s 90th percentile SLR scenario is also nearly equivalent to the 2080s 50th percentile SLR scenario. The 2080s SLR projections are approximately 60 years beyond expected construction, encompassing the 50-year design life. Therefore, it can be concluded that the design for the 2050s 90th percentile SLR projection (2.5 feet) also satisfies the 2080s 50th percentile SLR projection (2.4 feet).

3.3 Design Water Levels

Based on the review of FEMA's 1% and 0.2% annual chance SWELs and NYSDEC SLR recommendation, the design water levels used in 60% design are 100-year current days SWEL (11.3 ft), 100-year SWEL with 2050s 90^{th} percentile SLR (13.8 ft), 500-year current days SWEL (14.8 ft), and 500-year SWEL with 2050s 90^{th} percentile SLR (17.3 ft), respectively. All elevations are referenced to NAVD88.

4 Wave Modeling

For 60% design, two-dimensional (2D) SWAN modeling was conducted to better understand the wave characteristics in the project area and to improve the estimation of wave conditions in front of the FBS, subsequently to update the engineering analysis of wave overtopping, wave loading and wave runup and to optimize the 60% design FBS. The 2D model domain covers from south of Governor's Island to north of Hudson River Park. The project area is well resolved with the cell size ranging from 8 ft to 700 ft. The mesh was refined along the shoreline and the proposed protection alignment. **Figure 7** presents the model domain and closeup of the project area. The stationary mode was used for wave modeling. A spatially uniform wind forcing is assumed. Detailed model configuration, simulation scenarios and model results are presented in following sections.

A separate 2D SWAN model of no-rise analysis was conducted for HRPT. The goal was to replicate similar results to a previous impact analysis completed for the North Project in 2020 using the MIKE 3FM model (AECOM 2022).The boundary conditions used for the no-rise analysis are different than those described above. Instead, the same conditions were used with the 100-year water level with SLR and the worst scenario of northwestern winds.

Figure 7: Model domain and SWAN unstructured mesh

4.1 Model Configuration

4.1.1 Elevation Data

The 2D SWAN wave model incorporates topo-bathymetric data from different resources: USGS Coastal National Elevation Database (CoNED), and the bathymetric and topographic survey data provided by BPCA for existing conditions, from low to high priority, respectively. CoNED data is used for the offshore area, while the survey data collected during the scoping phase of the NWBPCR project, shown in **Figure 8**, provides nearshore bathymetry and topography within the project area. All elevations are referenced to the NAVD88 datum. In terms of the proposed project conditions, the topography data is revised to reflect the implementation of the proposed 60% design of the final grading, landscape, flood barrier and raised platforms. ArcGIS was utilized to merge and process all data sources to develop the digital elevation model, which was converted to the 2D SWAN model mesh.

Any future survey data will be under review by the coastal engineering team and to evaluate the impacts on the wave modeling, but most likely will not change after the 60% design unless significant changes are made to the proposed grading and landscape.

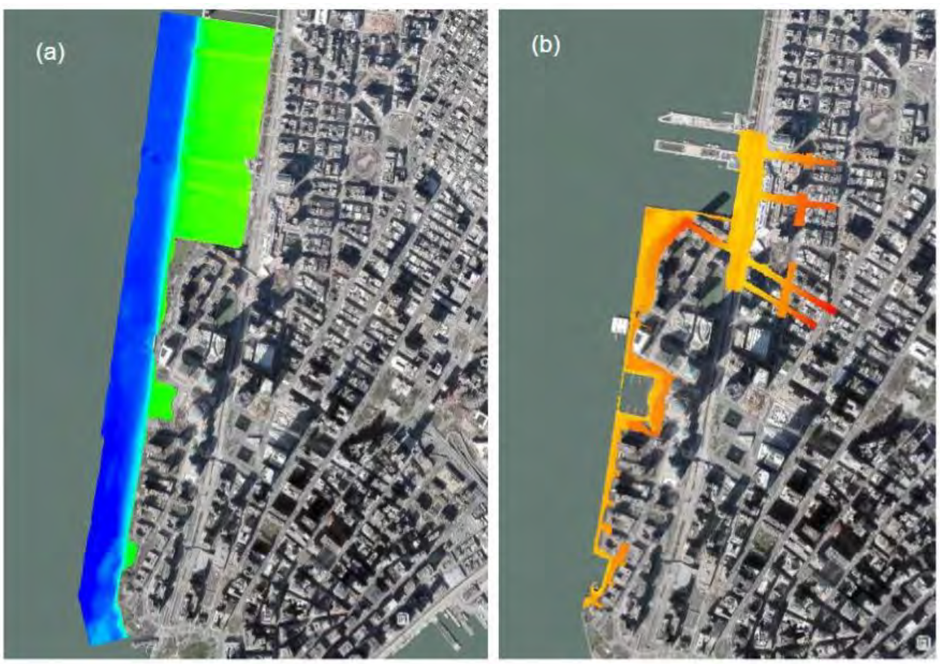


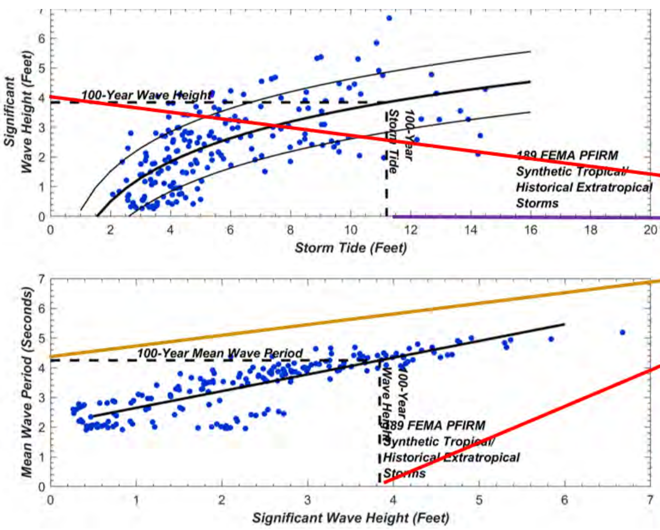
Figure 8: Extent of the (a) bathymetric survey and (b) topographic survey

4.1.2 Model Boundary Conditions and Initial Conditions

Wave conditions are required at both the north and south open boundaries. Wave heights and peak wave periods were derived from FEMA PFIRM storm suite model results for design water levels under current conditions using the same method used to derive incident wave conditions for 1D SWAN transect modeling. Different from the 1D SWAN boundary condition development, instead of the locations of save points at the transect ends near the project site, wave heights and peak periods were derived at locations at the north and south boundaries, respectively.

In order to obtain the incident wave conditions at the model boundary and wind speed over the model domain associated with storm surge, a statistical evaluation of wave height versus still water level (regression method) was performed to understand the offshore wave heights and periods for water levels near to the project's design water level. The FEMA storm set developed for the Preliminary Flood Insurance Study was used, which includes both historic and synthetic storm events. The FEMA storm data was checked against the NACCS data set. The wind speed and storm tide level relationships were also calculated in a similar manner. Comparative FEMA data were extracted from the points in **Figure 6** as shown in **Figure 9**. The comparison of the data in **Table 5** indicates general agreement of data sets.

The 3 black curves in the top graph of **Figure 9** are simple polylogarithmic functions in the logarithmic of *n* (which in this case is storm tide), that best fit (in a least-squares sense) the data. The top and bottom curves are the function plus and minus one standard deviation, which is also estimated from the data.



Point #	Storm Tide (100YR)	Hs (ft)	Tm (s)
1	11.2	3.84	4.25
2	11.2	4.79	4.31
3	11.2	4.40	4.26
4	11.2	3.82	4.28
5	11.2	4.77	4.34
6	11.2	4.49	4.28
7	11.2	3.77	4.25
8	11.2	4.78	4.35
9	11.2	4.58	4.30

Figure 9: 100-year storm significant wave height and period extracted from FEMA data set.

Table 5: Comparison of NACCS and FEMA Wave Heights and Periods 100-year and 500-year data

NACCS 100yr

Point	Storm Tide (100YR)	Hs (ft)	Tp (s)	Peak Wind Speed (mph)
11787	11.3	4.14	3.82	73.82
4203	11.3	3.72	3.58	72.47
11901	11.3	3.76	3.70	74.42

FEMA 100yr

Point	Storm Tide (100YR)	Hs (ft)	Tp (s)	Peak Wind Speed (mph)
11787	11.3	4.78	4.35	82.46
4203	11.3	4.18	4.20	82.91
11901	11.3	4.80	4.25	83.14

NACCS 500yr

Point	Storm Tide (100YR)	Hs (ft)	Tp (s)	Peak Wind Speed (mph)
11787	14.8	4.78	4.04	84.79
4203	14.8	4.31	3.76	83.46
11901	14.8	4.32	3.91	85.44

FEMA 500yr

Point	Storm Tide (100YR)	Hs (ft)	Tp (s)	Peak Wind Speed (mph)
11787	14.8	5.38	4.67	96.62
4203	14.8	4.70	4.51	96.86
11901	14.8	5.42	4.54	97.12

Note:

Hs=significant wave height
Tp = Peak Wave Period
Storm Tide is the still water elevation in feet, NAVD88

It should be noted that the significant wave height reported on the FEMA transect for the 1% annual exceedance probability (AEP) SWEL is somewhat larger than the wave boundary condition used in the current analysis because the methods used to derive the wave conditions are different and the locations are different (**Figure 5** vs. **Figure 10**).

In the FEMA PFIRM study, the average wave condition is derived from several (approximately 7) of the synthetic storms that generated storm tides approximately equal to the 1% AEP SWEL at a specific location. In contrast, the analysis presented here uses the entire storm suite to derive a regression correlation between storm tide and wave height as well as between the wave height and mean wave periods. For a given SWEL, e.g. 1% AEP SWEL of 11.3-ft NAVD88, the corresponding wave height and wave period are derived from the regression relationship and reported in **Table 5** shown in **Figure 9**. The peak period is then obtained from the mean period using the commonly used factor of 1.1.

The method of the regression relationship is used because it provides a more representative wave period based on the complete storm suite dataset. The method of regression relationship for determining the wave boundary condition for the 2D SWAN analysis provides insight to wave dynamics of the storm suite and confidence in the data to be used.

The significant wave heights of the starting wave in FEMA's FIS report were not significantly different from the wave height extracted in the current analysis at three offshore points shown in **Figure 6** (in the middle of the river, points 2, 5 and 8). The wave heights extracted at points 1, 4, and 7 near the shoreline are notably smaller than the wave height in the FIS since the latter were obtained at locations 2,500 feet offshore (further offshore than points 1, 4, and 7 in **Figure 6**).

A sensitivity test confirmed that the model results and overtopping rates are not very sensitive to the wave boundary conditions because any incident deepwater wave is dissipated over the raised platform where reduced water depth limits wave height.

This regression method was applied to obtain the incident wave conditions and wind speeds associated with 100 year and 500-year storm surge nearby the south and north model boundaries. The extracted wave conditions are almost the same, less than 0.1 feet. So the same waves are applied to both boundaries to reduce the number of simulations which is shown in Table 6.

4.1.3 Wind Forcing and Simulation Scenarios

During the 40% design, the regression method was applied to derive wind speeds associated with 100-year and 500-year return interval SWELs near the south and north model boundaries to cover the model domain. The derived wind speeds are assumed spatially uniform. All possible wind directions combined with incident waves from the south and north boundaries are setup for wave simulations. Assuming that the incident waves at the south boundary only enter the domain from the south, the possible associated wind directions are from the southern quadrants, ranging from 203 to 293 degrees in 15-degree intervals, clockwise from the north. Similarly, associated with incident waves entering the domain from the north boundary, the possible wind directions are from the north quadrants, ranging from 278 to 353 degrees in 15-degree intervals. **Figure 10** presents all possible wind directions and incident waves directions. **Table 6** summarizes all simulation cases. In total, four scenarios (100-year, 100-year + SLR, 500-year, and 500-year + SLR SWELs) and 52 cases were simulated.

For the no-rise wave model, a conservative scenario of northwestern wind, at angle of 293 degrees, occurring at the same time as the maximum water level was the only scenario modeled in 2D SWAN.

Table 6: 2D SWAN wave model boundary conditions

Model Configuration	Input Parameter	100-year	500-year
Static water level	SWEL (ft) w/o or w/ SLR	11.3/13.8	14.8/17.3
Wind forcing	Wind speed (mph)	78	87
	Wind directions (CW from true north)	Associated with waves from south: 203, 218, 233, 248, 263, 278, 293 Associated with waves from north: 278,293, 308, 323, 338, 353	
Wave boundary condition	Hs (ft)	4.48	5.04
	Tp (sec)	4.18	4.48

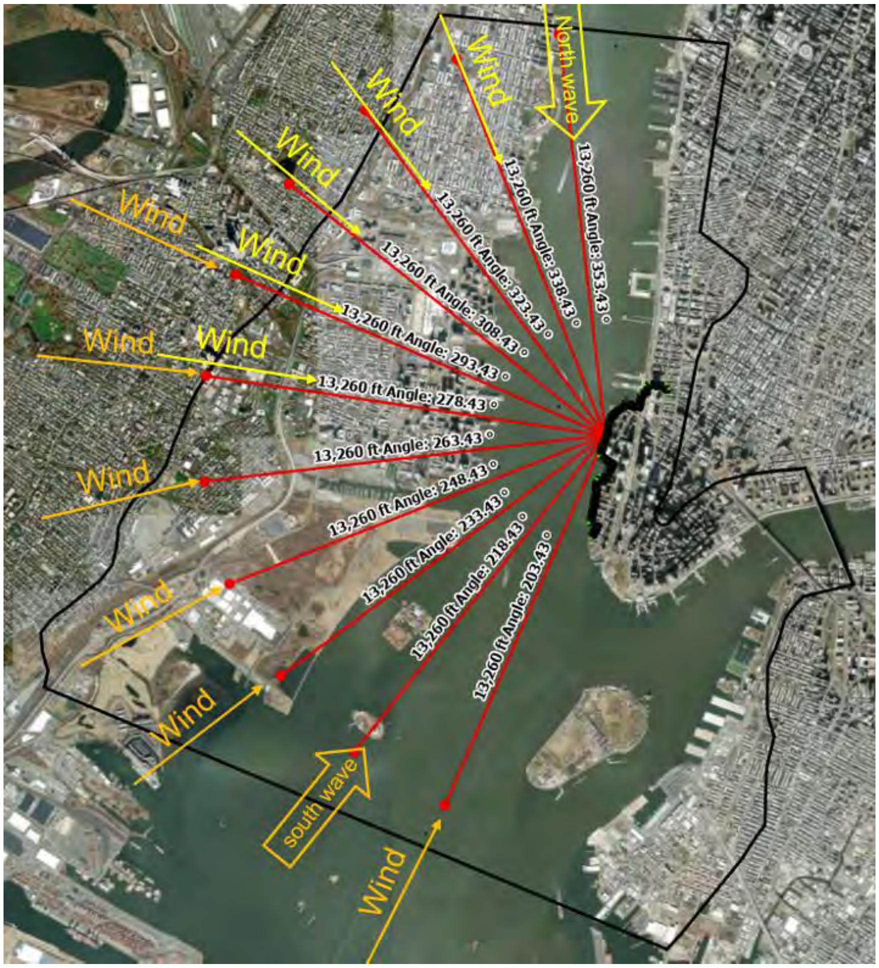


Figure 10: Combination of incident waves and wind directions

4.2 Model Results and Design Wave Conditions

A 2D SWAN model (version 41.45) was developed to predict how wave conditions in the Hudson River transform to Battery Park City and the FBS under both existing topography (without project) and proposed topography (with project) conditions. During the 40% design, 13 simulations (wind directions listed in Table 6) for each topography condition and design water level were modeled. A total of 104 simulations were performed (13 wind directions for 4 design water levels and 2 topography conditions). The waves are simulated and driven by incident waves and winds, and propagated to the project site. The design wave conditions for each topography and design water level were determined based on the 13 wind direction simulations for overtopping and wave loading calculations.

SWAN is a third-generation phase-decoupled model (Booij et al., 1999). SWAN is capable of simulating the important wave dynamic processes from offshore to the shallow areas near a project site. The following relevant wave processes were simulated in SWAN:

- Wind-wave generation (wave growth due to winds)
- Shoaling (increase in wave height as waves enter shallower water)
- Refraction (change in direction of waves caused by changing water depth)
- Energy dissipation due to bottom friction, depth-limited breaking, white capping, and wave-wave nonlinear interactions

SWAN has been applied for the determination of wave conditions in coastal areas by a global community of researchers and engineering consultants. The model is frequently updated with the most current developments from the research community.

The wave model results are extracted at 152 locations, every 50 ft along the alignment and about one wave length away from the alignment. The design wave conditions for each location are the maximum wave selected from 13 wave simulations with possible wind directions and incident waves. The waves parallel to the alignment were not be selected for wave overtopping analysis due to the small overtopping rates. The 152 locations are shown in **Figure 11**.

Figure 11: 2-D SWAN 152 extraction locations along the alignment.

Figure 12 through **Figure 15** show the design wave heights of 2-D SWAN modeling at extraction locations along the alignment for different topography conditions with current 100-year and future 100-year design water levels.

Results show the natural wave attenuation as waves propagate from the deeper water of the Hudson River onshore and in some locations indicate further attenuation over the platform where water depths are shallower. These are the recommended locations for estimating a representative wave condition for subsequent analyses. A sensitivity analysis was completed to evaluate how the noted variation in offshore wave heights would impact onshore results and an appropriate approach was selected.

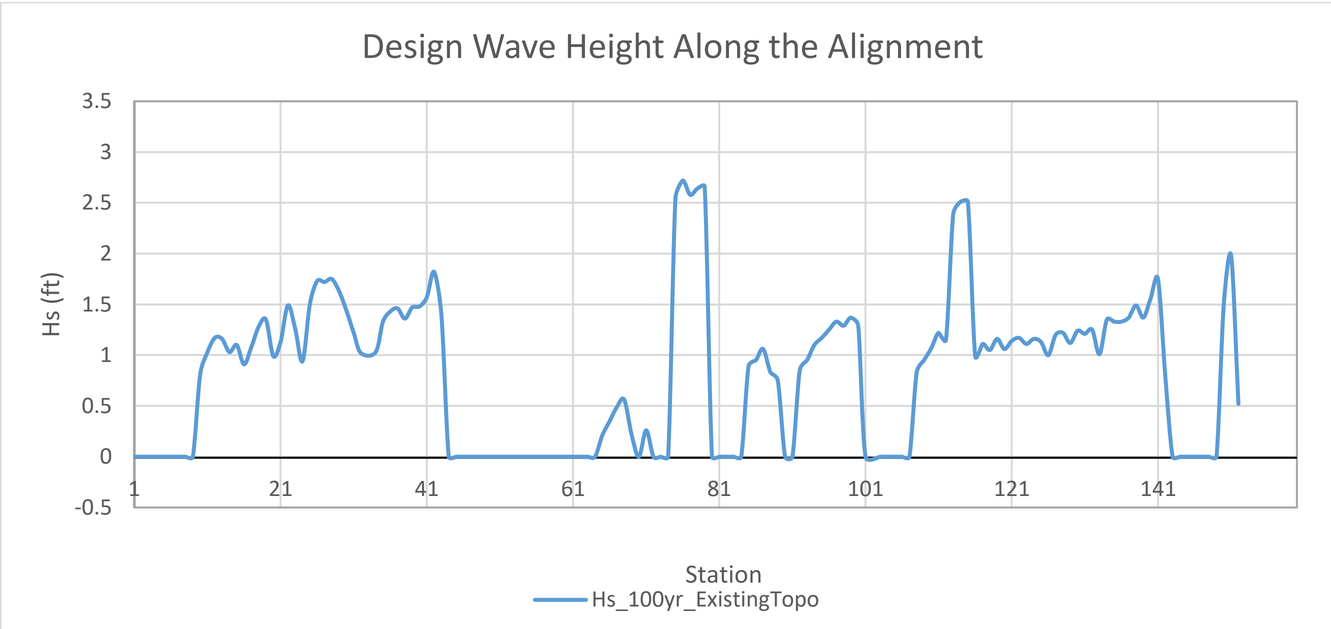


Figure 12: Design wave heights along the alignment for 100-year water level and existing topography.

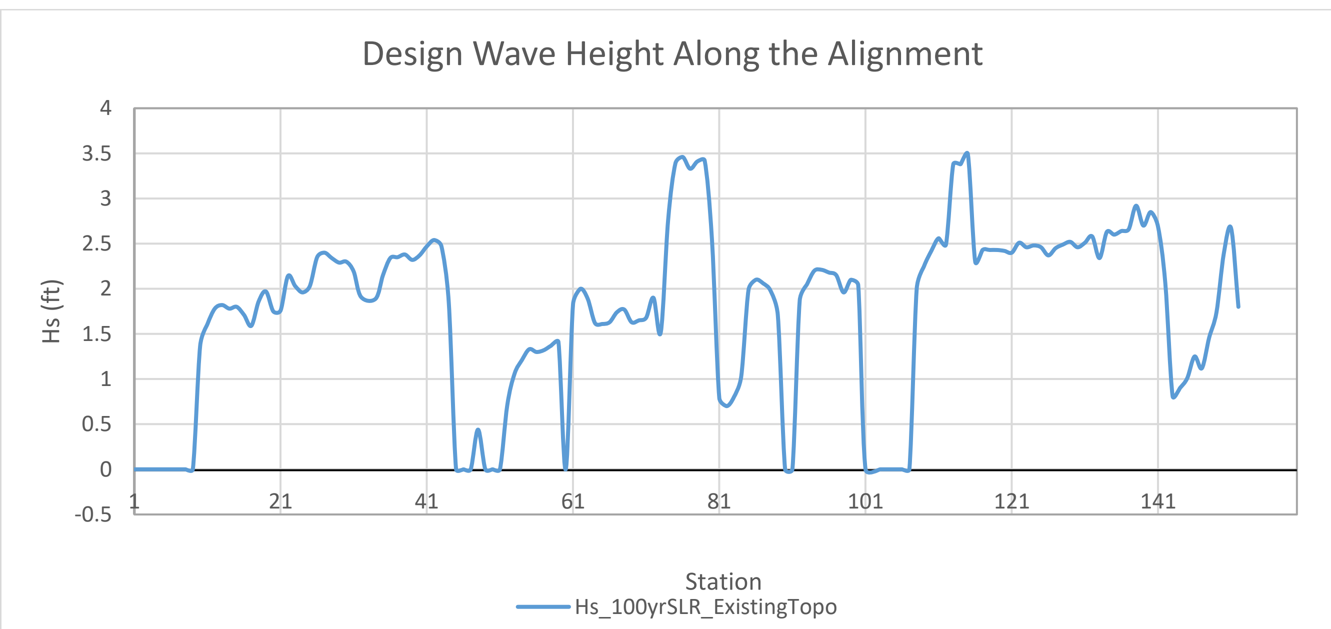


Figure 13: Design wave heights along the alignment for 100-year + SLR water level and existing topography.

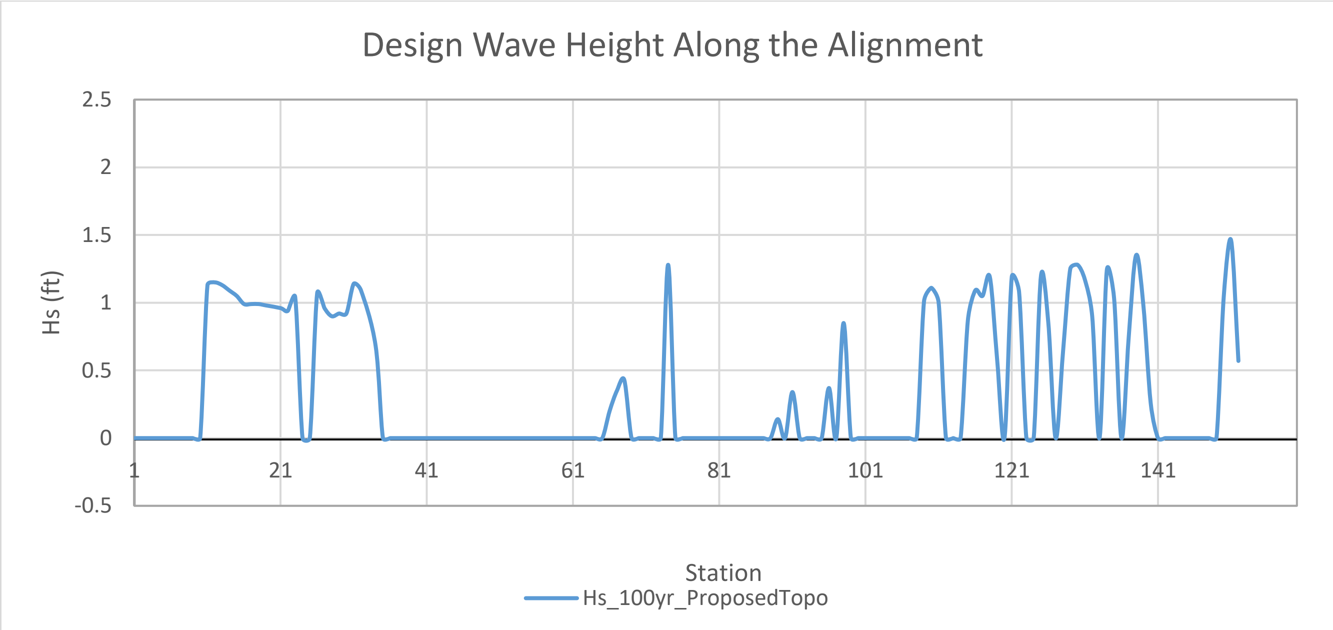


Figure 14: Design wave heights along the alignment for 100-year water level and proposed topography.

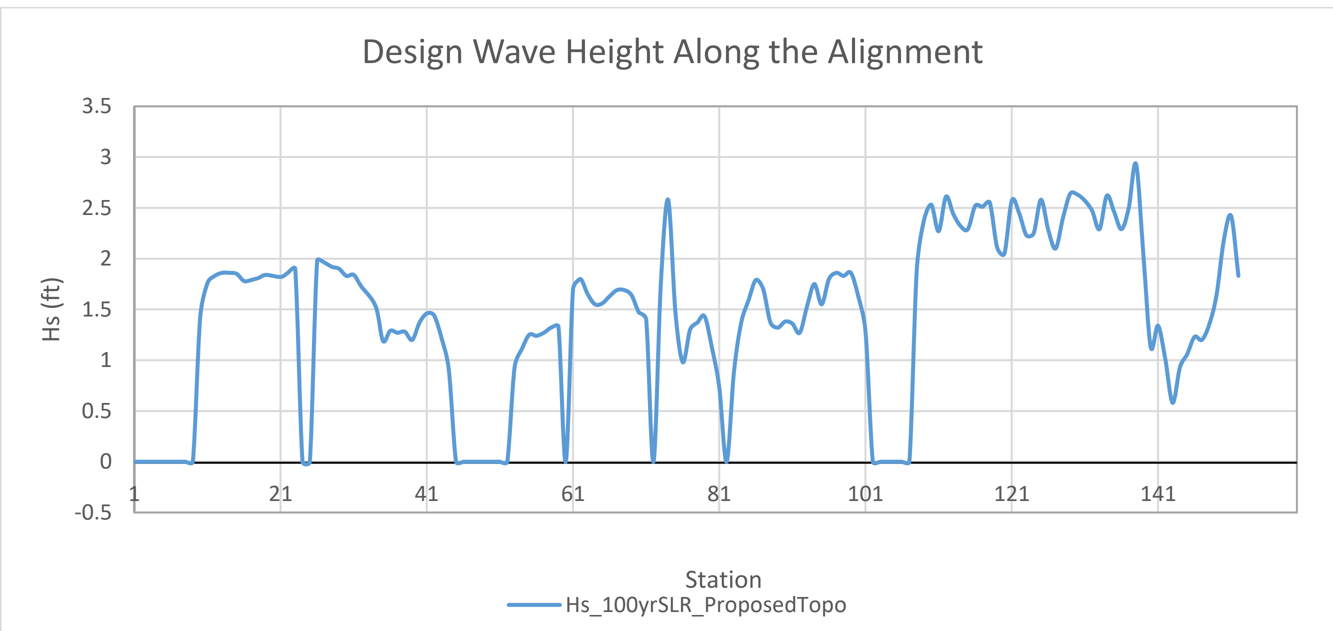


Figure 15: Design wave heights along the alignment for 100-year + SLR water level and proposed topography.

5 Wave Overtopping, Wave Loading and Runup

5.1 Wave Overtopping Rate Estimation

The wave overtopping rates are used as criteria for selection of structure crest elevation. USACE guidelines (HSDRRS) recommend the design criteria for both 50 percentile and 90 percentile overtopping rates, calculated by Monte Carlo simulation (probabilistic approach), less than 0.03 cfs/lf and 0.1 cfs/lf, respectively. The EurOtop manual also presents the deterministic approach which is called the “design or safety assessment” approach since it’s with partial safety factor. The overtopping assessment for interior drainage design used the deterministic formula to be conservative. The technical details for each method are described in the following sections.

5.1.1 Probabilistic overtopping assessment

For 60% design with updated alignment and wall heights, a wave overtopping assessment was completed in accordance with the Hurricane and Storm Damage Risk Reduction System Design (HSDRRS) Guidelines (USACE, 2012). The overtopping formulas for vertical walls (Van der Meer and Bruce 2014) explained in Chapter 7 of the EurOtop manual (2018) were used in conjunction with the Monte Carlo simulation method recommended in HSDRRS. The analysis estimated the overtopping rate and its uncertainty due to the uncertainties in estimated water level, wave height, and wave period. For a given structure crest elevation, overtopping rates at given probabilities were obtained and the 50th and 90th percentile overtopping rates were used as criteria for selection of the structure crest elevation.

For the Monte Carlo simulations, the water level, wave height, and wave period were varied using the mean values and standard deviations determined as follows:

- 10% of the design water level or 0.6 ft (HSDRRS), whichever is smaller;
- 20% of given wave height; and
- 20% of given wave period.

The coefficients in overtopping formulas were assumed to be normally distributed with the standard deviations specified in the EurOtop manual (2018). For the Monte Carlo simulation analysis, 10,000 simulations were run based on variations in the parameters noted above.

The analysis was repeated for a series of structure crest elevations to examine overtopping rate response to changes in structure crest elevation. Per the HSDRRS, overtopping rates were output for both the 50% and 90% confidence levels. A sensitivity test of overtopping rates using various crest elevations demonstrates diminishing improvement in the overtopping rate as the wall height increases. This suggests that other methods of reducing the overtopping rate, such as wave attenuating features, may be more effective and yield additional benefits with respect to cost and aesthetics. Such features will be investigated as the design progresses.

The HSDRSS recommends overtopping limits based on the 50th percentile and 90th percentile overtopping rates calculated using the Monte Carlo simulation described above. For vertical structures, the 50th percentile overtopping rate should be less than 0.03 cfs/lf (cubic feet per second per linear foot) and less than 0.1 cfs/lf for the 90th percentile. Overtopping calculations use the wave height at a distance one wavelength from the FBS. **Figure 16** shows zero overtopping for the 100-year water level and **Figure 17** shows the probabilistic overtopping results for 152 locations along the alignment for the proposed topography condition for the 100-year with SLR, and all the overtopping rates meet both 50th and 90th design criteria. Wave overtopping rates were also computed using the two most common deterministic methods, EurOtop and Franco, and the latter was found to be less conservative than the probabilistic method. The deterministic method is a mean value approach with the inclusion of the uncertainty of prediction. The mean values for each variable are the mean variable plus its one standard deviation.

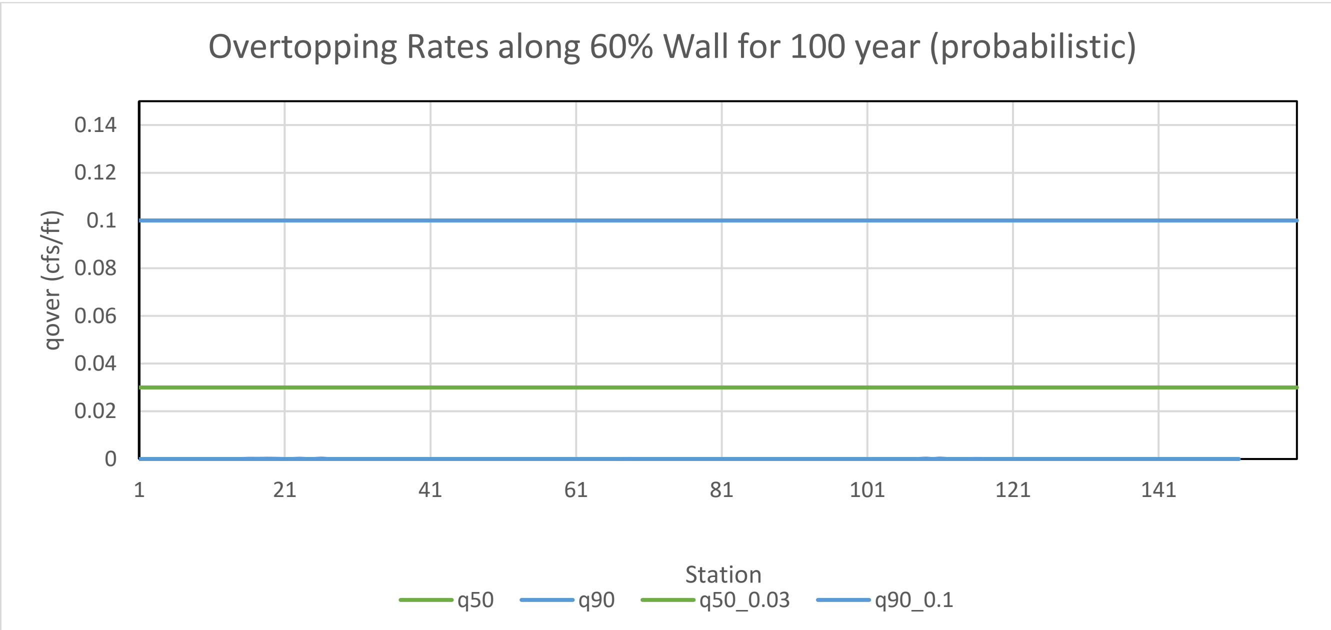


Figure 16: Overtopping rates for proposed topography and 100-year water level and design criteria

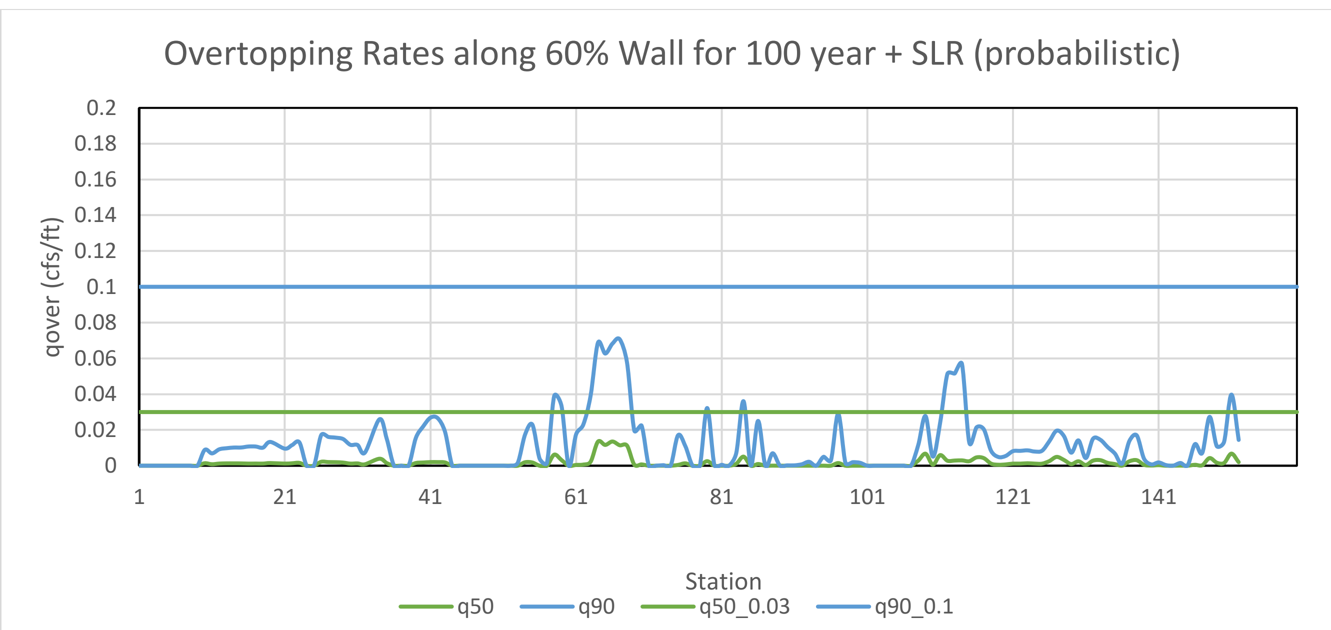


Figure 17: Overtopping rates for proposed topography and 100-year + SLR water level

It is important to note that the USACE maximum overtopping rate of 0.03 cfs/lf ensures that the structural integrity of the FBS is protected from wave over wash. However, over the course of a 3-hour high tide cycle, this maximum allowable overtopping rate could equate to over 2,000 gallons per linear foot. This is a substantial amount of water, but it is still relatively small in comparison to the expected interior stormwater flows from accompanying rainfall. For example, the maximum allowable overtopping would still be less than 5% of the stormwater flow created by a rainfall of 8.79 inches over 24 hours (a 100-year rain event).

The overtopping rates for the selected elevations are all below the 0.03 cfs/lf criteria. Additionally, the modeled overtopping rates all assume 2.5 feet of SLR. Overtopping rates for current conditions are expected to be even lower. If there is a need in particular areas to further reduce the overtopping rate, a recurved wall can be added to the FBS. A recurved wall is a type of seawall with a seaward overhang as part of the structure with the intent of reducing wave overtopping by deflecting up rushing water back toward open water (Pearson, 2005).

5.1.2 Deterministic overtopping assessment

For 60% design, the deterministic approach is also used for interior drainage system design. The deterministic approach is different from the probabilistic approach in hydraulic parameters of water level, wave height, and wave period. Whereas in the probabilistic approach, the coefficients are determined by the mean values plus or minus a random value within one standard deviation range. The deterministic approach uses the fixed coefficients by mean values plus one standard deviation, which is with inclusion of the uncertainty of the prediction. Based on the calculation of overtopping results using different methods, the deterministic approach results are about equivalent 75 percentile overtopping rates in probabilistic approach results.

5.1.3 Bullnose application for overtopping reduction

One of the methods applied to optimize the 40% design is a recurved wall top treatment (also called a bullnose) to reduce the wave overtopping. The reduction of wave overtopping is very effective when the recurve has adequate free board and is located at the upper portion of the wave run-up extent. An example of what the recurved wall looks like is shown in **Figure 18. Figure 19** shows the changes of overtopping due to the application of a recurved wall. The blue line is for 40% wall height without bullnose. The partial bullnose application is one of the design optimizations which apply at different segments shown **in Figure 1.**

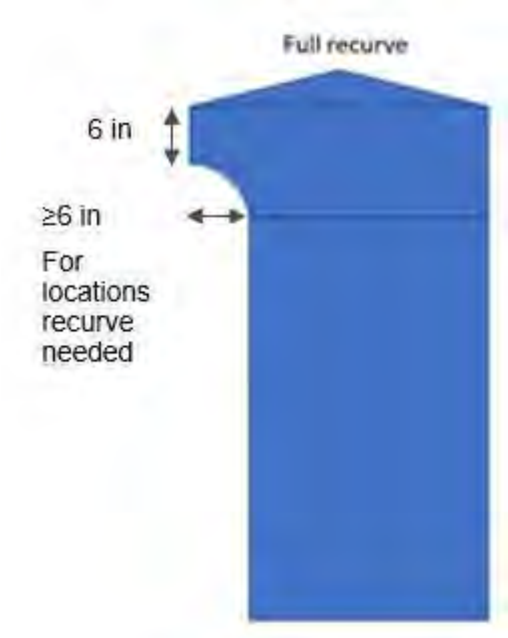


Figure 18 Example of Recurved Wall Application

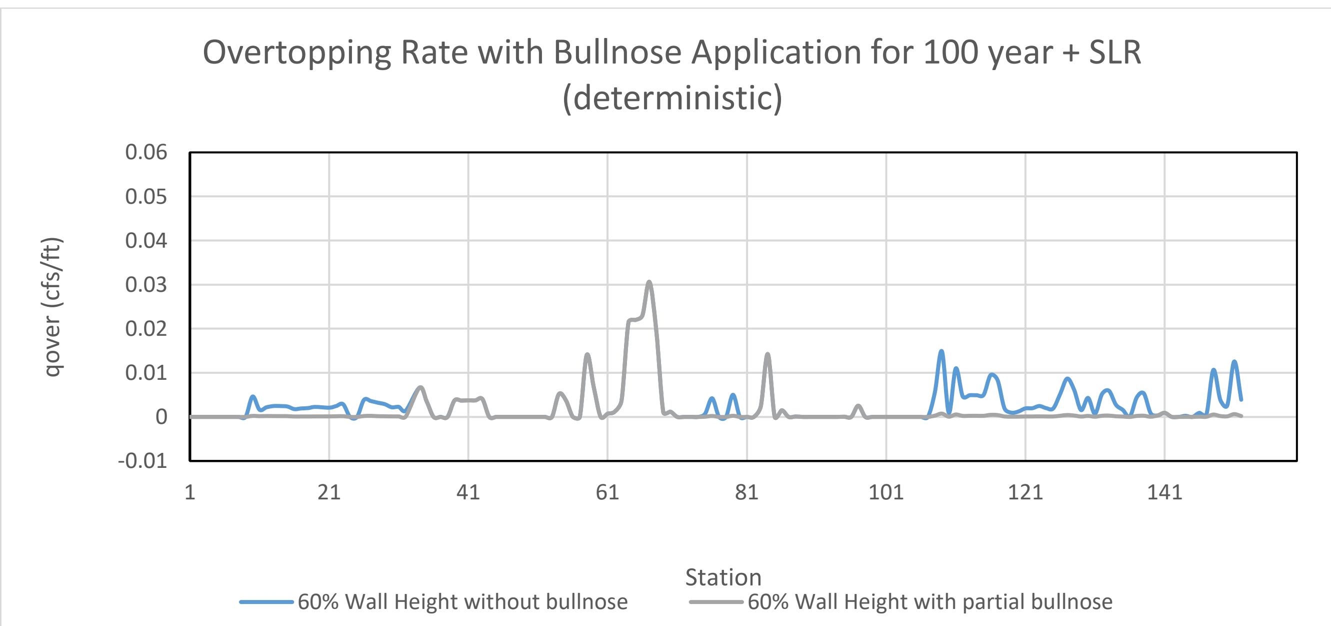


Figure 19: Overtopping reduction due to the bullnose application

5.1.4 Optimized Top of Wall Elevations

The 30% top of wall elevations are based on multiple criteria including FEMA requirements, BPCA design goals, and site-specific design needs. In order to meet the FEMA accreditation requirement outlined in 44 CFR 65.10, (b) (1) (iii) and (iv), the top of wall elevations must be based on the design still water elevations, location specific wave heights, wave runup, and overtopping rates. Wave overtopping occurs when a wave “runs up” the flood defense structure and exceeds the structure’s crest elevation, allowing water to enter the protected side of the flood defense system. Therefore, the structure crest or top of wall elevation is established to prevent or limit overtopping and water on the interior side of the structure. While a criterion of no overtopping is preferred by BPCA, practical considerations often necessitate some overtopping with SLR in extreme storm conditions. When overtopping is considered in the flood defense design, the volume of water from wave overtopping must still be limited based on interior drainage, safety, and structural considerations. In conformance with BPCA and the SBPCR Project, the design goal is to have no overtopping for existing storm conditions (SLR is not considered) and to meet FEMA freeboard requirements. Likewise, the design goal is to minimize wave overtopping to within acceptable levels (as defined by FEMA and USACE guidelines) under the design still water level and wave conditions for future SLR conditions.

Wave attenuation features have the potential to reduce the top of wall elevation by controlling the amount of wave energy reaching the flood wall and limiting wave runup and overtopping. A key question that continues to be addressed during and post-30% design is the interplay of wave attenuation features and top of wall elevations. The size (height and width), configuration, and spacing of wave attenuation features will be further studied and how the various attenuation configurations impact wall height will be determined.

The 60% wall height recommendations are summarized in **Figure 1** and **Table 2** of the Executive Summary. These 60% design wall heights are within the range of prior estimates by the owner’s engineer (AECOM, 2022). It should also be noted that the minimum achievable top of wall elevation is 16 ft (NAVD88). This is required to meet the FEMA accreditation requirement of a minimum of 2 feet of freeboard above the design SWEL which by the end of the design life is expected to be 13.8 ft (NAVD88). FEMA has an additional requirement of 1 foot of freeboard above the 100-year SWEL + wave runup. In the 40% elevation study, the intent was to explore how wall height influenced overtopping with the intent of minimizing overtopping while looking at where there might be diminishing benefits from higher wall heights. For the 60% elevation study, the wall heights are obtained from the detailed study of overtopping, interior drainage, and proposed finished grading and topography. The effectiveness of different overtopping mitigation measures, including wall height adjustment, wave attenuation, recurved wall, and raised platform, are studied, and provide the guidance for optimizing the design. The updated wall heights meet the following design criteria: i) FEMA 44 CFR 65.10 freeboard requirements for present day 100-year water level without future SLR. ii) the overtopping rate criteria for structural stability established by USACE for 100-year water level with future SLR. iii) overtopping water volume will not significantly impact the interior drainage system or ponding (less than one foot maximum depth), and iv) select the overtopping mitigation measures to meet as close as possible a condition of zero wave overtopping by limiting overtopping to < 0.0004 cfs/ft (cited as negligible overtopping by EuroTop guidance). The final design criteria is desired by BPCA especially for some residential areas immediately adjacent to the FBS. The location map is shown in **Figure 1**.

5.2 Wave Runup Calculation

Wave runup calculations were performed for both current conditions (100-year design water level, which will be required for the CLOMR) and future conditions (100-year + SLR design water level). The results show that 40% design recommended wall heights meet CLOMR requirements, shown in **Figure 20**. However, it is important to note that there may be instances that require a variance from the runup requirement if the 2.5-ft of SLR occurs in the future. But the design optimization, such as recurved wall, may have different runup for the future conditions with SLR. In the future, it is anticipated that an exemption from this requirement would be sought from FEMA. The mitigation measures and adaptable designs allow for the exemption are provided in the design.

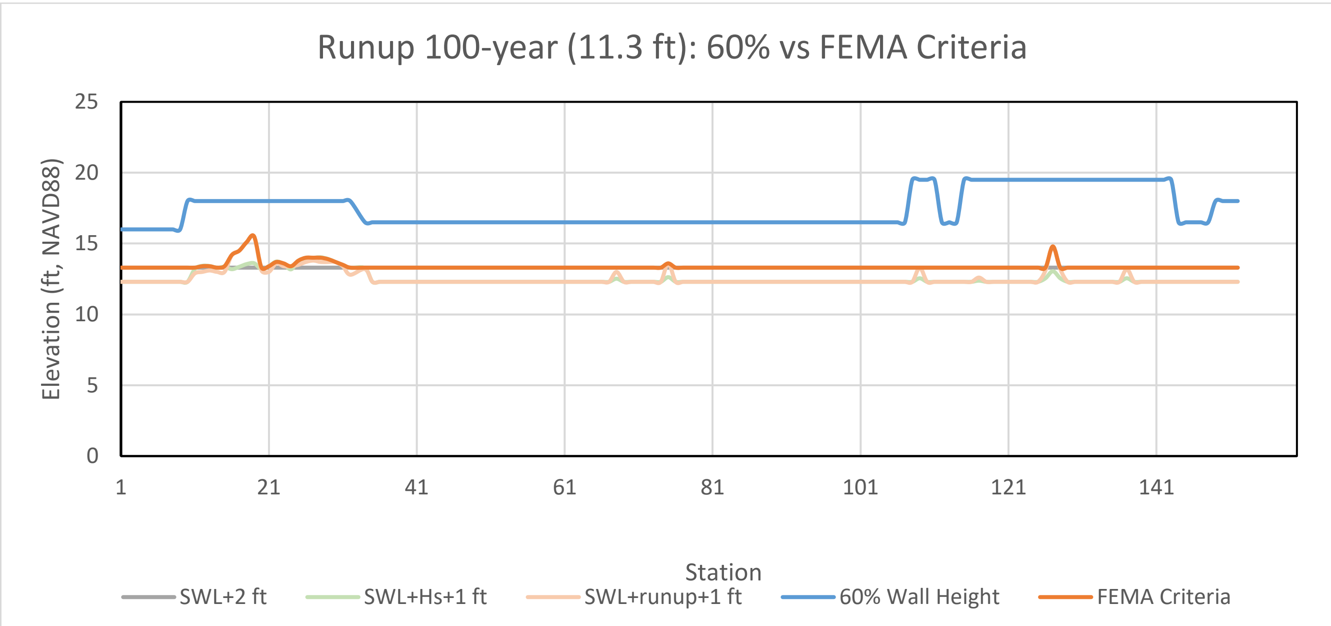


Figure 20. FEMA design criteria of free board for CLOMR

5.3 Wave Induced Load on Flood Walls

Wave loads were calculated using the SWAN 2D model results for design water levels and structure heights. The analysis was performed using HSDRRS design guidelines (**Figure 21**), the Goda method (**Figure 22**) and FEMA requirements at locations every 50 feet along the flood wall alignment, shown in Figure 11. The results of the wave loading analysis are summarized in the attached excel sheet of March2024 Wall Heights and Wave Loading.

The Excel sheet shows the Goda wave pressures per transect based off the 100-year with SLR storm conditions. No additional factor of safety has been applied to estimate p_1, p_2, p_3, p_u and F_h (horizontal design wave load).

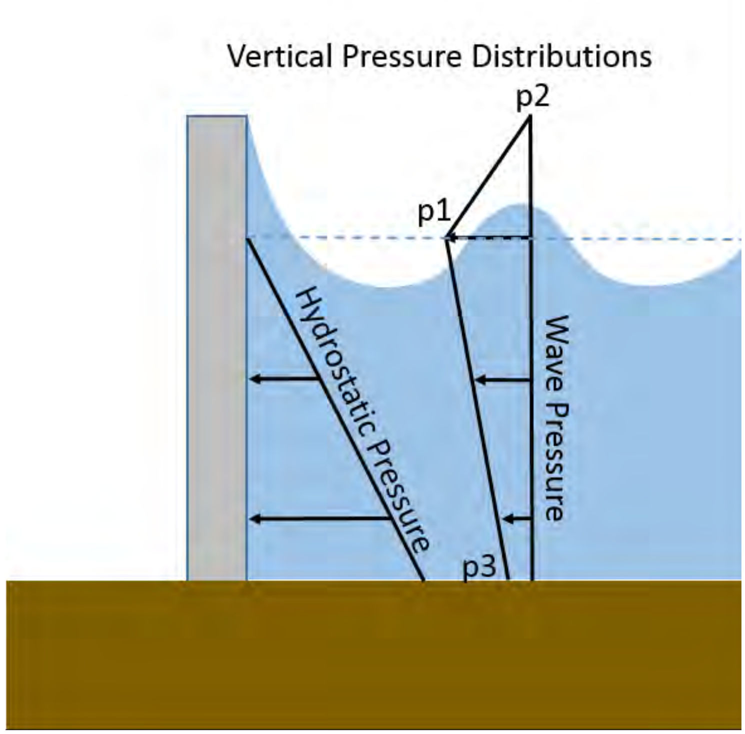


Figure 21: Definition Sketch of Wave Force Calculations

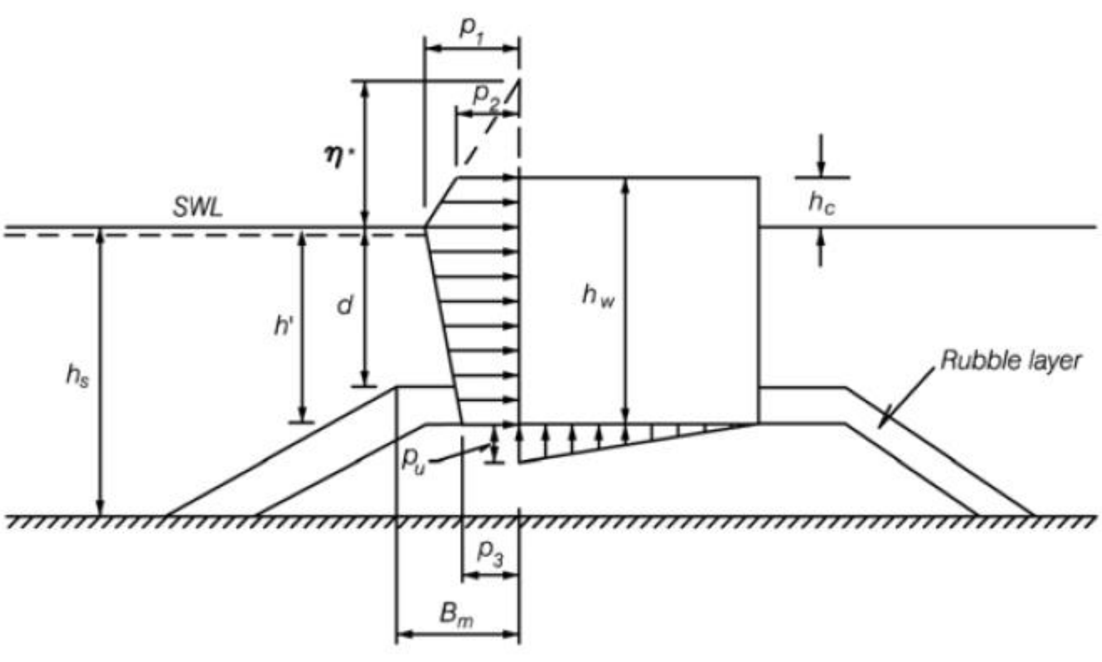


Figure 22. Definition Sketch of the Goda Formula for Irregular Waves

5.4 Wave Induced Load on Piles

The wave conditions, used for wave load calculations on piles of platforms, were extracted from locations in front of platforms. The greater of wave heights acting on the platforms were selected for wave load calculations.

Variables defined in wave forces calculations are shown in the **Figure 23** (CEM Figure VI-5-125):

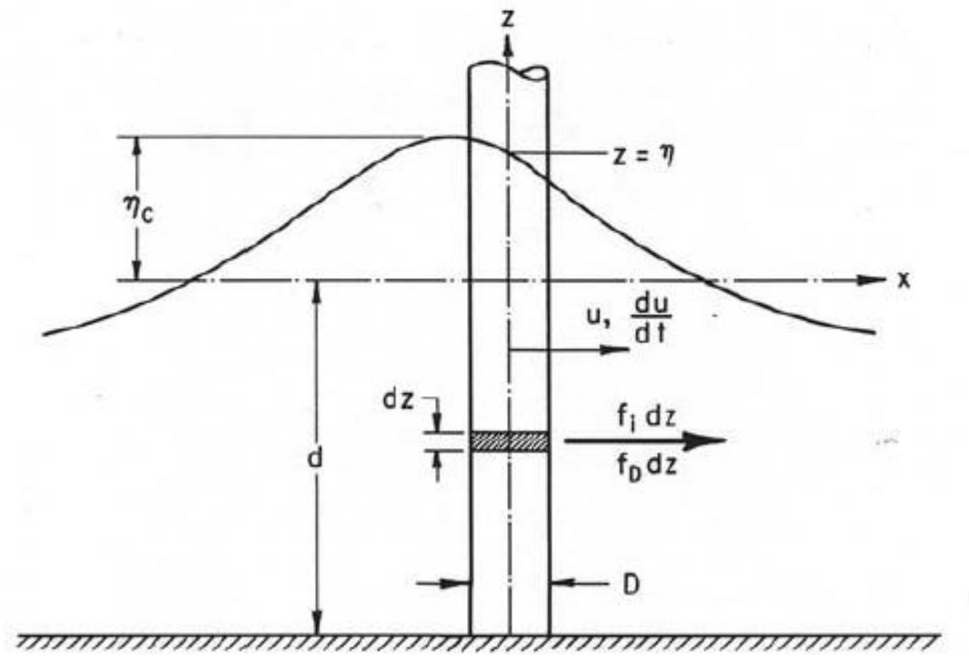


Figure 23: Variable definition for wave induced loads on piles

The Morrison formula was used for the horizontal force per unit length of a vertical cylindrical pile due to waves. The details refer to Section 6.1.3.2 of Forces on Exposed Piles and Columns in Guide Specification for Bridges Vulnerable to Coastal Storms (AASHTO, 2022, CEM, 2011).

$$F = C_d \frac{\rho_w}{2} AU|U| + C_m \rho_w V \frac{dU}{dt}$$

$$F_{total} = \int_{-h}^{z} C_d \frac{\rho_w}{2} AU|U| \, dz + \int_{-h}^{z} C_m \, \rho_w V \frac{dU}{dt} \, dz$$

Where:

$\rho\_w =$ mass density of water taken as 2.0 slugs/ft^3

U = horizontal component of water particle velocity (ft/s)

A = project area per unit length (A=D for circular pile, ft^2/ft)

V = displace volume per unit length ($V = (\pi D\text{^}2)/4$, ft^3/ft)

$C\_d =$ Morison drag coefficient (0.65 used in calculation)

$C\_m =$ Morison inertia coefficient (1.6 used in calculation)

dU/dt = horizontal acceleration of water particles (ft/s^2)

To determine the total force on a vertical pile, the above F, the force per unit elevation, must be integrated over the immersed length of the pile (Dean and Dalrymple 1992. See formula 8.34). For emerged piles, the integration can be carried from sea bottom to the mean free surface, but for submerged piles, the integration is from sea bottom to the elevation of pile below the sea wall.

The resultant total force location is varied between d and d/2 for deep water and shallow water respectively, based on the lever arm terms in formula 8.38 of Dean and Dalrymple 1992. The water surface elevation or the topo of pile was used for leading piles in this study to be conservative. The half water depth or mid location of exposed pile was used for trailing piles due to the shoaling effects due to the slope,

Each platform has four rows of drilled piles (or shafts). The piles next to the sea wall are the leading piles and any piles behind the leading piles are trailing piles. The linear wave theory was used in this study, assuming both of the leading and trailing piles are in same wave field. The total force on trailing piles should be a portion of that on the nearest leading piles due to the different heights of piles, The total force on trailing piles can be calculated as follows:

$$F_{trailing} = \frac{L_{trailing}}{L_{leading}} F_{total,leading}$$

Where:

$L_{trailing}$ = height of trialing pile

$L_{leading}$ = height of the nearest leading pile

The different water levels, including MLLW, MSL, 100-year, 100-year with SLR, 500-year, and 500-year with SLR, were simulated for the wave model and wave forces on piles were calculated because the wave forces could be smaller with higher water levels due to the smaller velocity when the piles were submerged.

The wave conditions used for wave forces on platforms and the greatest wave induced forces at the water levels below or equal to 500-year with SLR are shown in **Table 7**.

Table 7: Wave Force on Leading Piles of Platforms (500-year SWEL + SLR)

Platform	Wave Height (ft)	Wave Period (s)	Wave Length (ft)	Wave Force on 48” Pile (lbf)	Wave Force on 36” Pile (lbf)	Force Location (ft, NAVD88)
Reach 2	2.8	3.0	40.8	436.6	248.6	-5.3
Reach 4	5.2	4.1	84.4	1789.7	1161.6	-5.3
Reach 6	5.1	4.1	84.1	1679.6	1088.2	-5.3

5.5 Wave Induced Load on Decks and Seawall

The hydrostatic and hydrodynamic loads on platforms considered the following forces: buoyancy, drag and inertia forces, associated forces with added mass, and vertical slamming forces.

Wave forces acting on the decks include two forces: uplift force on the deck with the waves under the decks, and the horizontal force on the slabs along the deck. Both forces are presented in Section 6.1.2 of Guide Specifications for Bridges Vulnerable to Coastal Storms (AASHTO, 2008, 2022).

The horizontal force on the seawall or rail around the platform edge were calculated by p_1 as described in AASHTO 6.1.3.3 in **Figure** 24:

The variables used in forces calculations as shown in **Figure 25**.

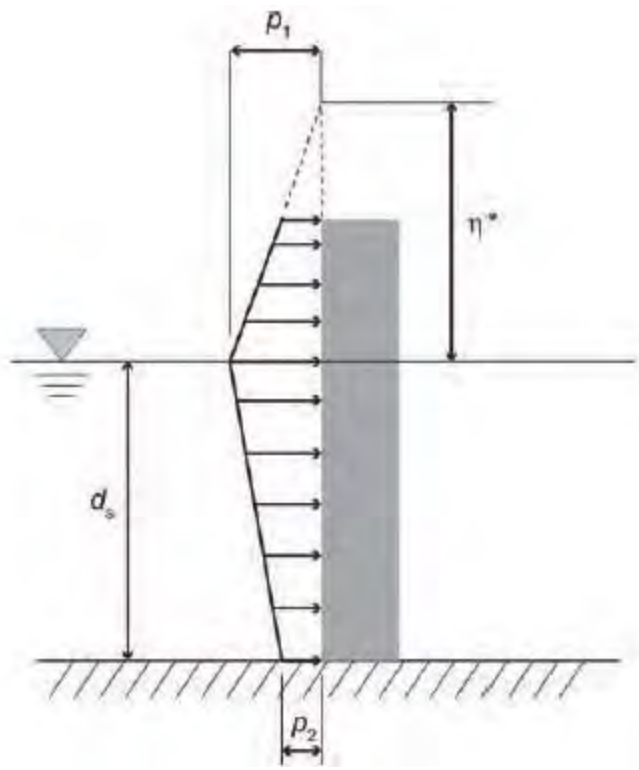


Figure 24 Definitions of Wave Pressures on the Flood Wall

$$p_1 = \frac{1}{2}(1 + cos\theta)\alpha_1 \gamma_w H_{max}$$

$$p_2 = \frac{p_1}{\cosh\left(\frac{2\pi d}{\lambda}\right)}$$

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{4\pi d/\lambda}{\sinh\left(4\pi d/\lambda\right)}\right]^2$$

Where:

p_1 = pressure at storm water level (kip/ft^2)

p_2 = pressure at mudline (kip/ft^2)

α_1 = coefficient

θ = angle between direction of wave approach and a line normal to the structure (deg)

The pressure distributions are interpolated based on the definition figure above. The pressure on seawalls of platform were presented in **Table 8**.

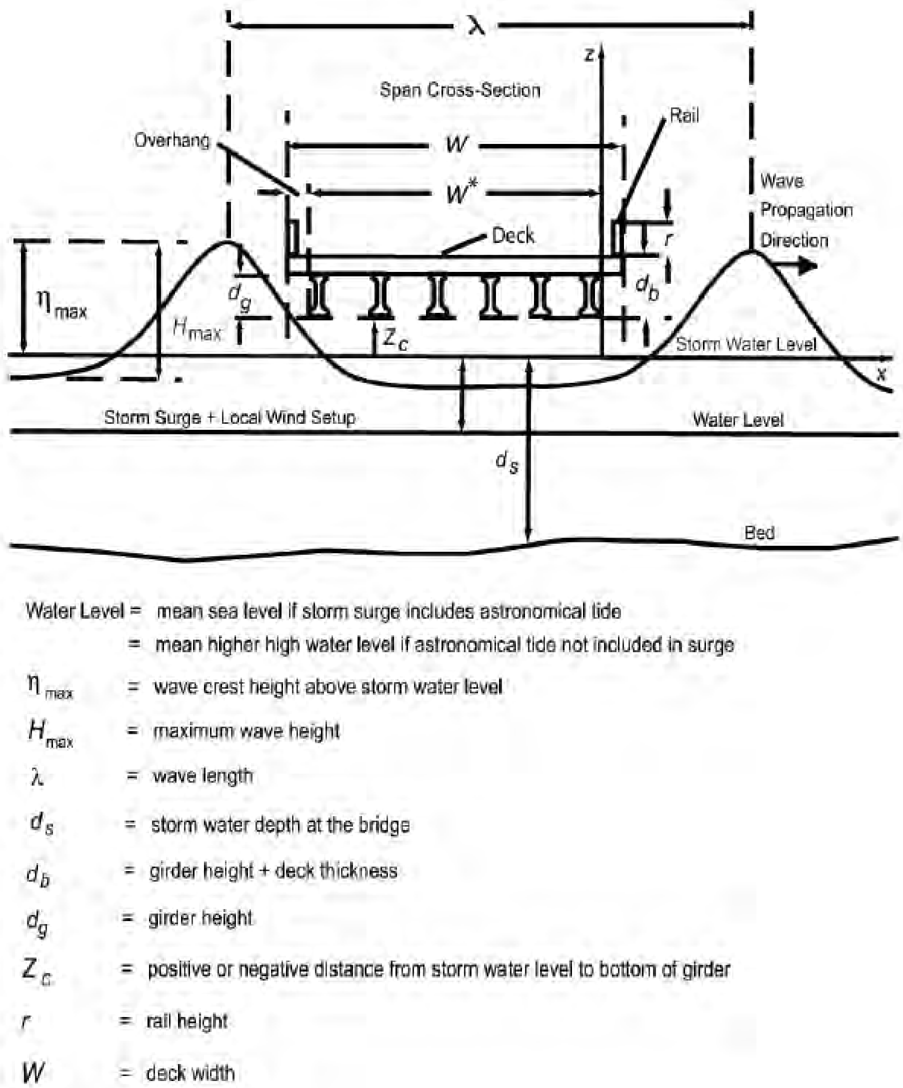


Figure 25. Variable definition used for wave induced loads on decks

The maximum quasi-static vertical force on the deck, F_{V-MAX} in lbf/ft of length of deck, were calculated using formula in Section 6.1.2.2 in AASHTO guide mentioned before. The equation is presented here:

$$F_{V-MAX} = \gamma_w \bar{W} \beta \left(-1.3\frac{H_{max}}{d_s} + 1.8\right)\left[1.35 + 0.35\tanh\left(1.2T_p - 8.5\right)\right]\left(b_0 + b_1 x + \frac{b_2}{y} + b_3 x^2 + \frac{b_4}{y^2} + \frac{b_5 x}{y} + b_6 x^3\right) TAF$$

Where:

$\bar{W} = \left[\lambda - \left(\frac{\lambda}{H_{max}}\right)\left(Z_c + \frac{H_{max}}{2}\right)\right]$

If $\frac{\bar{W}}{W} < 0.15\ then\ \bar{W} = 0.15W$

If $\eta_{max} - Z_c <= 0, then\ \beta = 0$; If $0 < \eta_{max} - Z_c < d_b, then\ \beta = \frac{\eta_{max} - Z_c}{d_b}$; If $\eta_{max} - Z_c > d_b, then\ \beta = 1$;

$x = \frac{H_{max}}{\lambda}$; $y = \frac{\bar{W}}{\lambda}$;

γ_w = unit wight of water (64.2 lbf/ft^3)

Z_c = vertical distance from bottom of deck cross section to the storm water level (ft)

TAF = trapped air factor (1.0 for conservative)

Limited wave energy passes through the seawall along the edge of platform to the underside of the deck. The seawall around the platform acts as a wave barrier. The wave heights under the deck are the transmitted waves. The transmission coefficient is given by Wiegel (1960) and Kreibel and Bollmann (1996) as

$$T_F = \frac{2k(d-w) + \sinh\left(2k(d-w)\right)}{2kd + \sinh\left(2kd\right)}$$

$$K_t = T_F^{1/2}$$

The variable definition is shown in **Figure 26**.

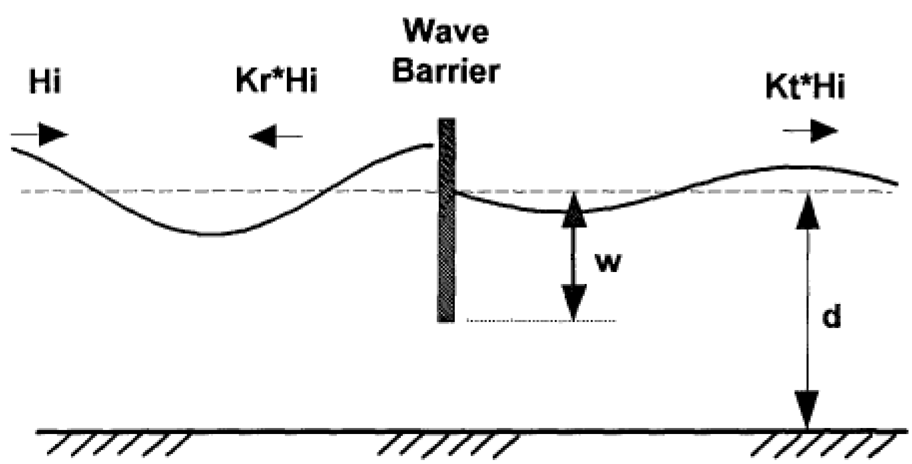


Figure 26. Variables in Wave Transmission Coefficient

The calculated vertical force on the deck using the AASHTO formula based on the physical experiment was compared with the theoretical formula derived by Dean and Dalrymple (1991) and they were similar. The AASHTO results were used due to its wide use for structure design. The uplift force results were presented in **Table 8.**

Table 8. Wave Induced Uplift Force on Decks and Pressure Distribution on Seawalls or Gravity Wall (500-year SWEL + SLR)

Platform	Uplift Force (kip/ft)	Pressure at Top of Sea Wall (psf)	Pressure Elevation (ft, NAVD88)	Pressure at Bottom of Sea Wall (psf)	Pressure Elevation (ft, NAVD88)
Reach 2	62.6	138.5	15.0	49.6	-5.3
Reach 4	65.8	215.2	12.0	148.9	-5.3
Reach 6	66.7	218.2	12.0	149.3	-5.3
Reach 5	NA	104.4	8.8	12.0	-17.0
Reach 7	NA	188.2	13.5	150.0	7.0

6 No Rise Analysis

A coupled hydrodynamic and wave model was used to conduct an analysis to determine the impact of the FBS on 100-year storm tide elevations at properties adjacent to and outside of the flood protection alignment. A representative 100-year storm event was simulated using two models: one configured to represent existing conditions and one configured to represent future conditions with the flood protection alignment in place (new alignment and landscape will be updated in next submission). Simulations were performed for the 100-year storm tide event occurring both with present day sea level and again with the 90th percentile SLR projection in the 2050s of 2.5ft. Comparison of the results from the modeled scenarios is used to quantify the change in maximum still water elevation potentially induced by the project immediately post-construction and in the future.

The coupled ADvanced CIRCulation (ADCIRC) and SWAN model (ADCIRC+SWAN) model was used to perform this impact analysis. ADCIRC is a hydrodynamic model that solves the vertically integrated continuity and conservation of momentum equations for water level and current velocity (Luettich et al., 1992). ADCIRC uses an unstructured triangular mesh, which is ideal for complex coastal geometries because it avoids many of the complications that are evident when trying to resolve small topographical features near the shoreline. When paired with the SWAN model, the ADCIRC+SWAN modeling system provides a tightly coupled model that solves for sea surface elevation, currents, and waves on an identical unstructured mesh and provides for real-time interaction between the models (Dietrich et al., 2011). ADCIRC+SWAN incorporates the physical processes of wind- and wave-induced momentum transfer, which are essential to properly simulate storm surge. The ADCIRC+SWAN system is a state-of-the-art system that has been successfully used for many years across the coastal regions of the United States to simulate astronomic- and wind-induced changes in water level, velocity, and waves. It is the modeling system used by FEMA to delineate coastal flood hazard areas for NYC and by the USACE-ERDC for the NACCS study.

The following section discusses the selection of the representative 100-year storm event, the setup of the ADCIRC+SWAN simulations, and the demonstration of no-impact in the simulation results. For the 60% design phase, the no-rise analysis was updated for the HRPK region in the study area. Only the 2D SWAN was run for this analysis. The study results matched well with the prior estimates by the owner's engineer with differences of 1-3 inches (AECOM, 2020). The SWAN modeling results are shown in Section 6.4.

6.1 Selection of a Representative 100-Year Storm Tide Event

To determine a representative 100-year storm tide event, all of the tropical and extratropical storms used by FEMA to develop the 100-year storm tide shown on the PFIRMs were reviewed. The FEMA storms are called synthetic storms and are developed by FEMA to represent hypothetical storm events, based on tropical storm parameters (storm track, storm size, storm speed, and atmospheric pressure distributions) that are statistically representative for the NYC region (FEMA, 2014a). Storm events that produced peak storm tide near the project area of similar magnitude to the 100-year storm tide were identified by comparing the maximum simulated water surface elevation for each individual storm to the 100-year still water elevation of 11.3 ft. The comparisons were made at eight sampling points near the FBS as shown in **Figure 27**. The closest matches identified are synthetic storms named NJA_0007_006 and NJB_0003_010 which were selected for the modeling. The tracks and a snapshot of the wind fields at landfall are shown for the selected storms in **Figure 28 and 29**.

Figure 27.Sampling points used to identify which of the FEMA synthetic storms generated maximum water surface elevation approximately equal to FEMA's 100-year statistical still water elevation.

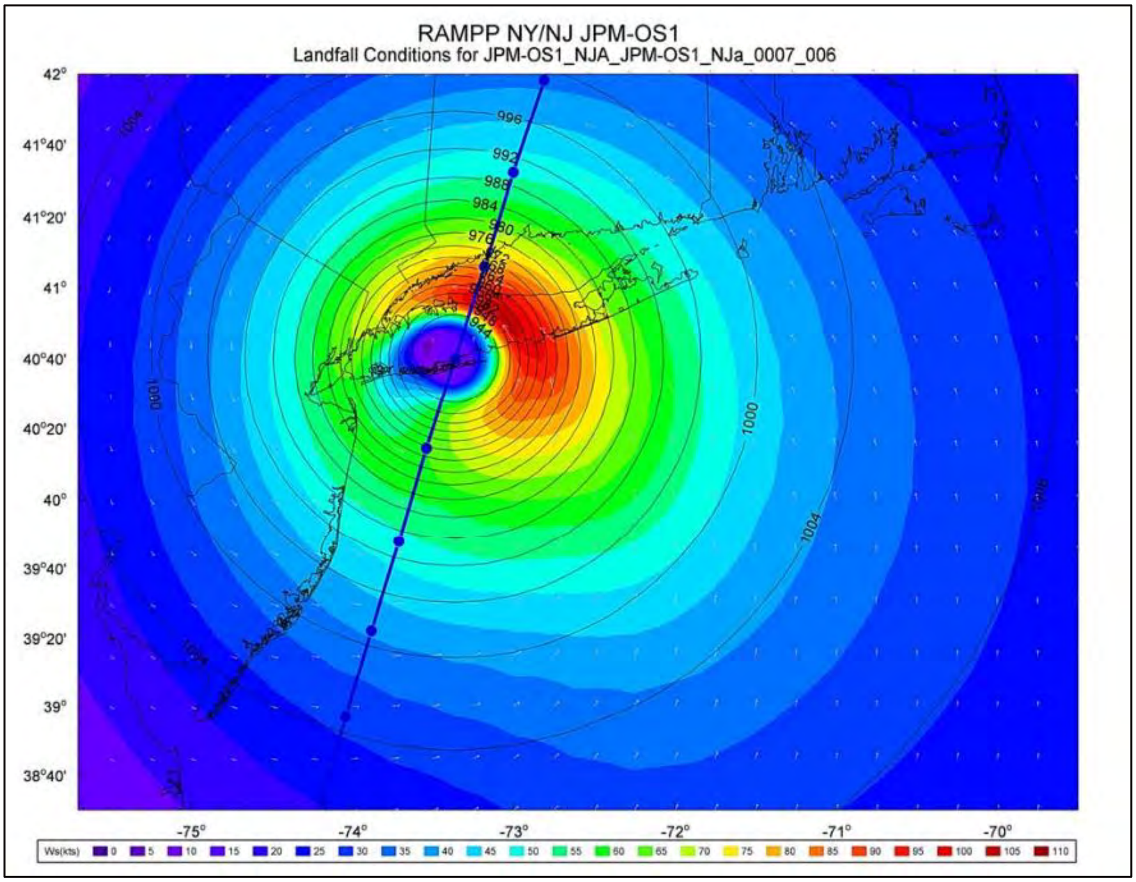


Figure 28. Storm track (blue), pressure contours, and wind vectors (white) for FEMA synthetic storm NJA-0007-006. Blue dots along the track represent eye location every six hours.

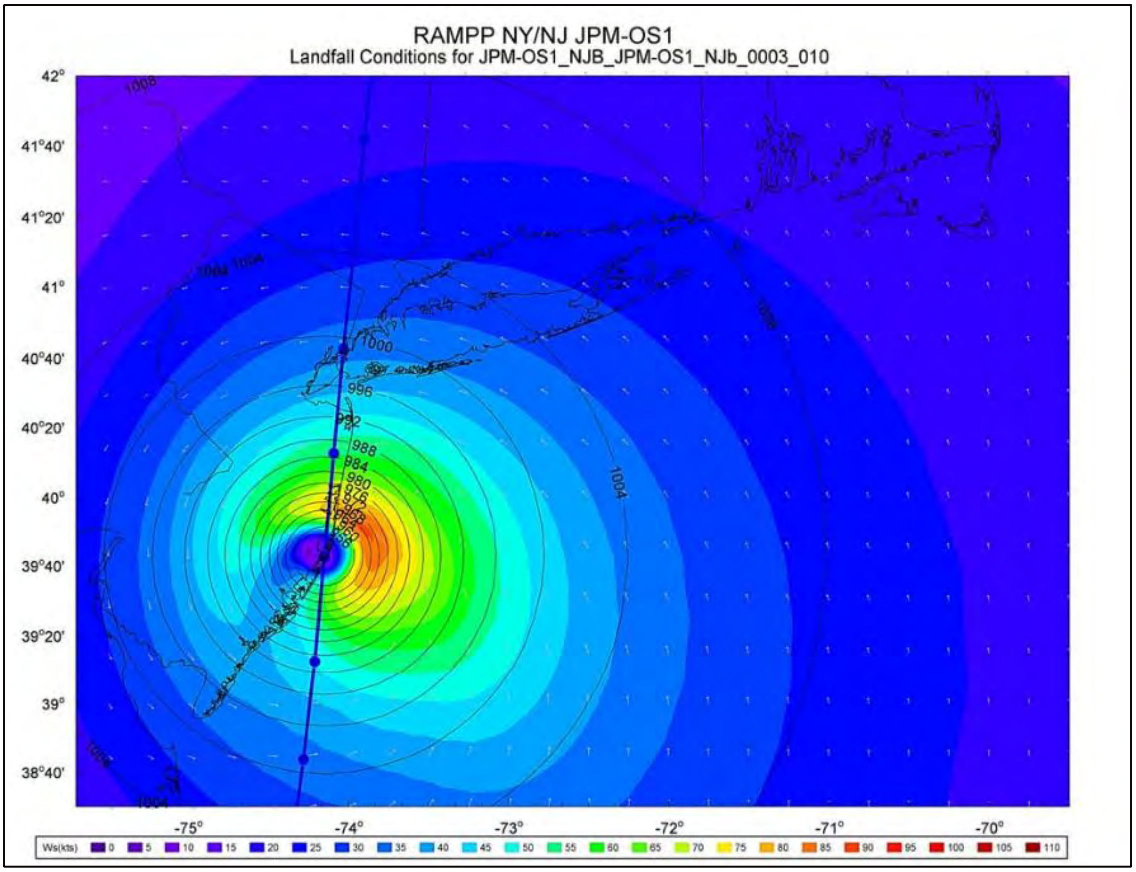


Figure 29. Storm track (blue), pressure contours, and wind vectors (white) for FEMA synthetic storm NJB-0003-010. Blue dots along the track represent eye location every six hours.

6.2 ADCIRC+SWAN Simulation Setup

The representative storm tide event was simulated using the same models used by FEMA during the analysis performed to generate the PFIRMs. The bathymetric finite element grid, or mesh, used for the ADCIRC + SWAN simulations was based on the mesh used in the FEMA PFIRM study, and the full model extent is shown in **Figure 30**. The mesh contains 1.8M elements (triangles) and 924k nodes (vertices). The mesh is used to define the landscape, elevations, and other parameters needed for the computations and is an important input for the simulations. The mesh was enhanced with more resolution (i.e. smaller triangles) in the areas adjacent to the FBS with resolution between 20 – 25 m (66 – 68 ft) at the FBS alignment. Two versions of the mesh input file were created, one with existing topography and one with the FBS elevations. The FBS was added to the mesh by raising the nodal elevations in the model along the FBS to a crest elevation of 7 m (23 ft), NAVD88. Other than the elevated nodes representing the FBS, all other mesh parameters are identical. An image of the mesh resolution in the project area with contours of the mesh elevations for both existing and proposed landscape are shown in **Figure 31**. A zoom-in of the north can be seen in **Figure 32** and of the south in **Figure 33** (new alignment and landscape will be updated in next submission).

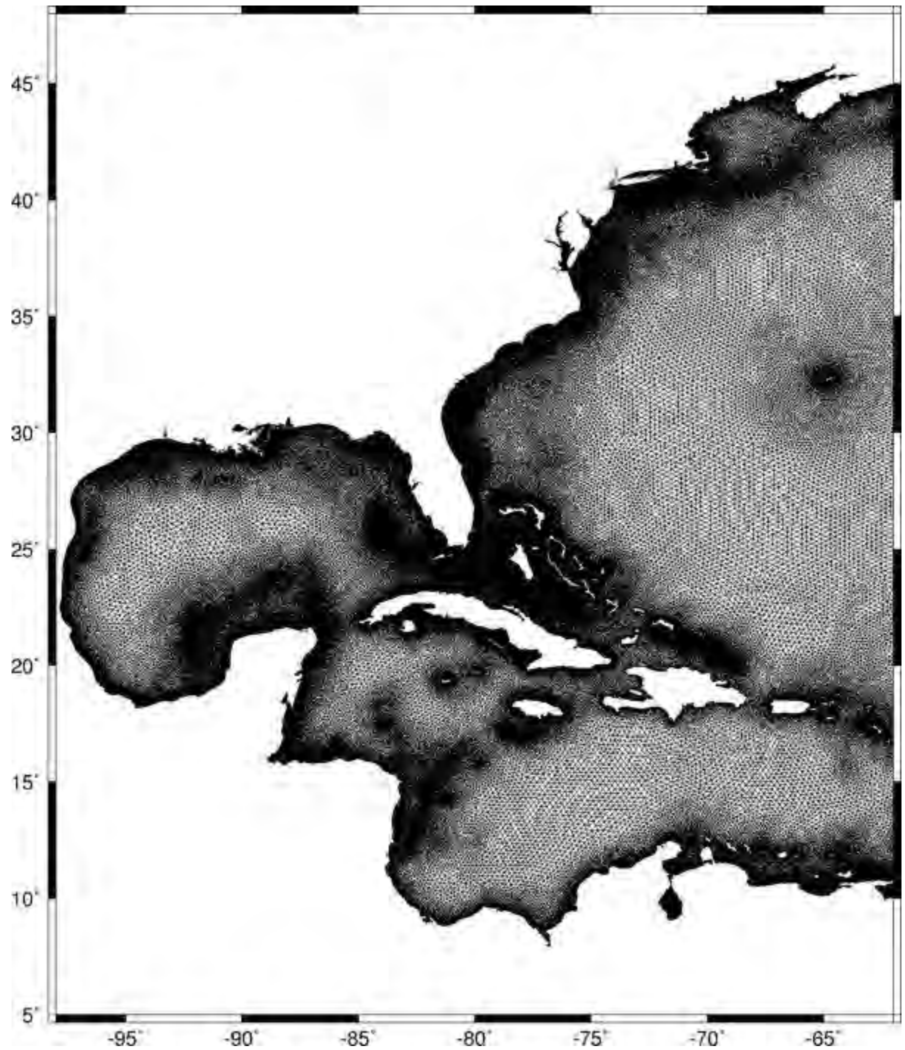


Figure 30. The total extent of the ADCIRC+SWAN model mesh used for the analysis.

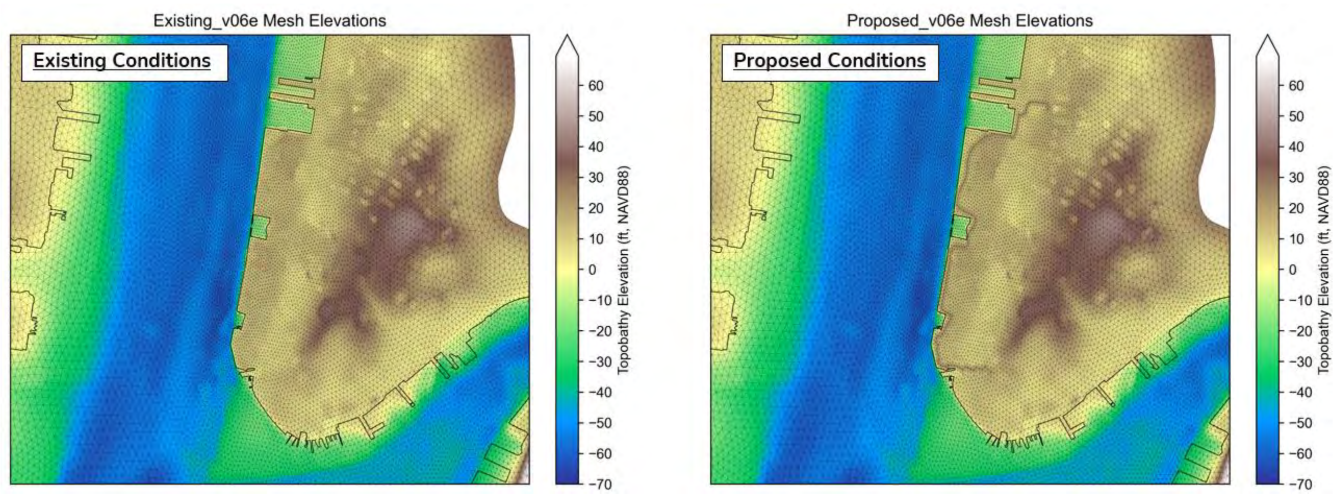


Figure 31. Comparison of the two meshes used in the analysis. The FBS alignment can be seen as the line of raised nodes in the mesh representing the proposed conditions (right).

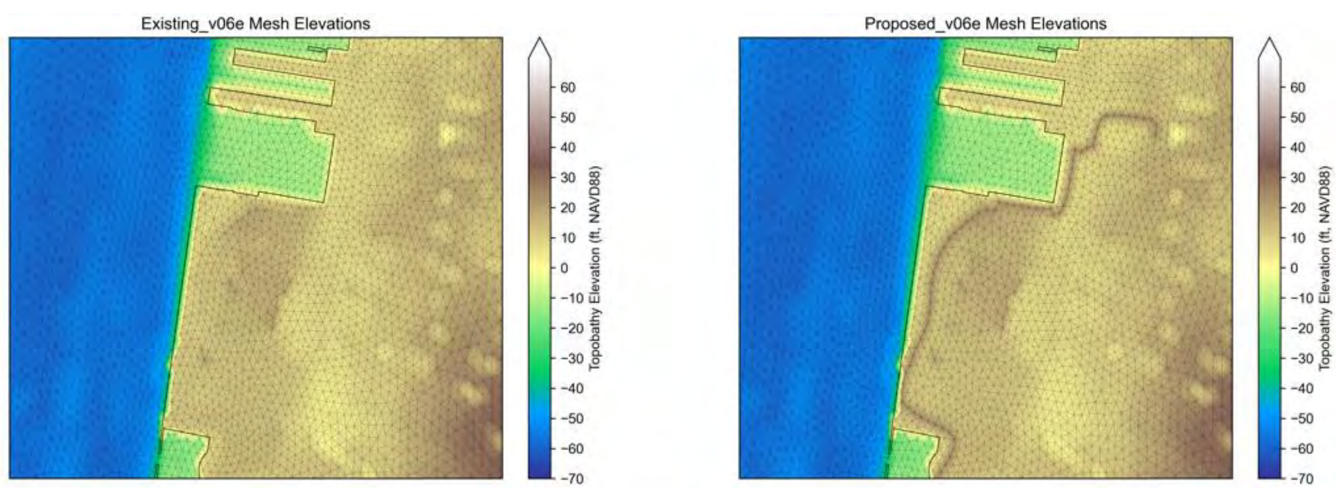


Figure 32. North

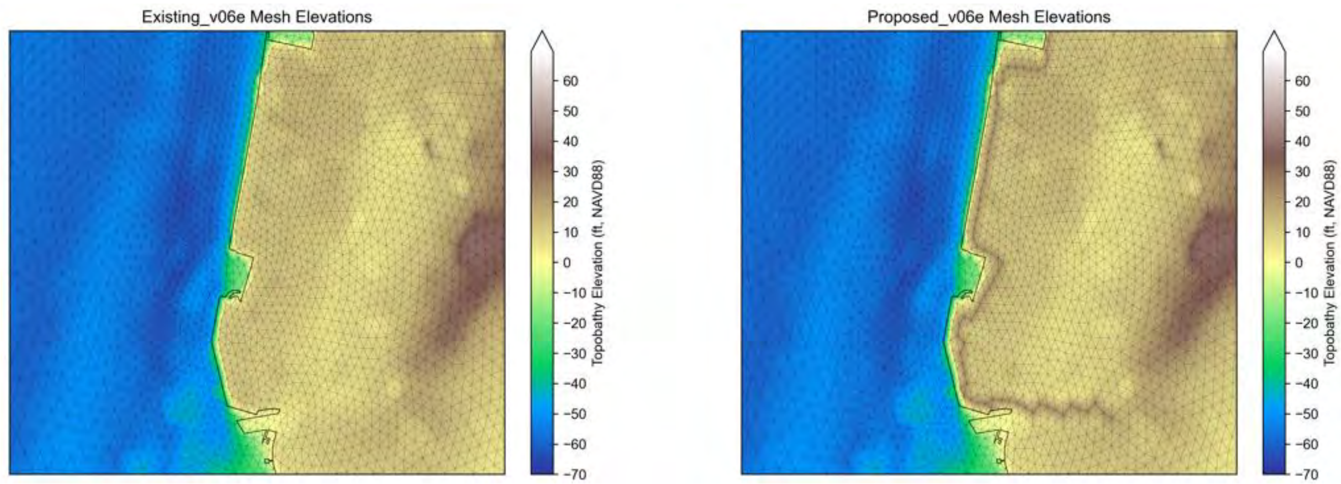


Figure 33. South

6.3 Water Level Comparisons

Simulations of the representative 100-year storm tide event were performed for the following four scenarios:

- With and Without the FBS Alignment - Current-Day Sea Levels
- With and Without the FBS Alignment - 90th Percentile SLR Projection in the 2050s

The potential impact of the proposed FBS alignment is estimated by comparing the computed maximum water surface elevation from simulations with and without the alignment in place. **Figure 34** and **Figure 35** are maximum water surface elevation difference plotswith the upper panel for present day sea level and the lower panel for future sea level. The areas shown in white are where differences are within +/- 0.25 ft and can be seen to include most of the modeled region. Some additional differences can be seen immediately next to the alignment where the model mesh is “wet” in the without simulation and “dry” in the with simulation, which appears as a large decrease in water surface elevation. The differences along the wall are a numerical result of the granularity of the mesh and do not represent a physical decrease in the water level. The conclusion of the modeling is that no negative consequences are predicted to the spatial distribution of flooding, the extent of inundation, or the maximum still water elevation (i.e. storm surge) due to the proposed FBS alignment.

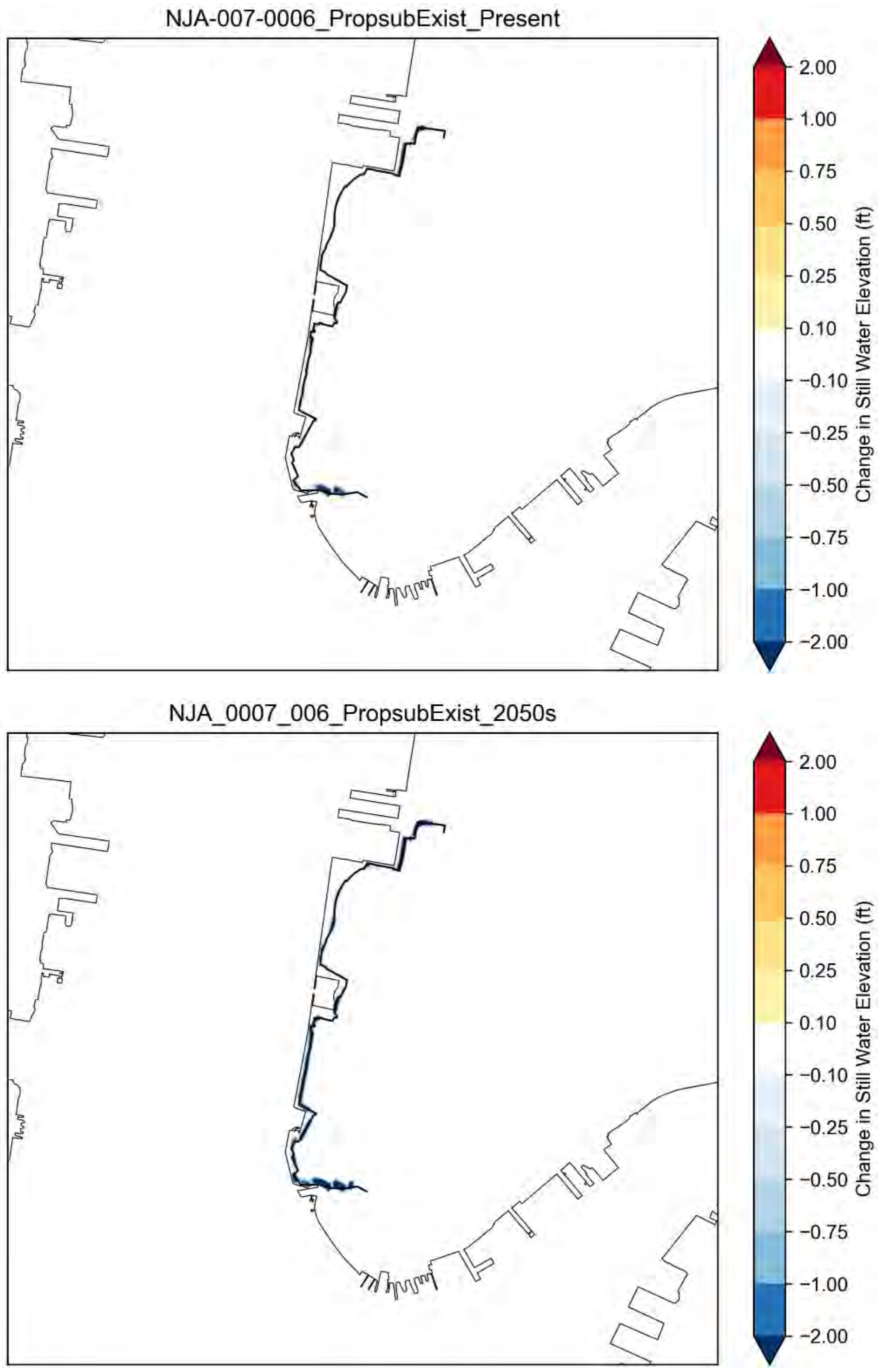


Figure 34. Difference in maximum Still Water Elevation for Storm NJA-0007-006 (Upper: 100-year, Lower: 100-year+SLR)

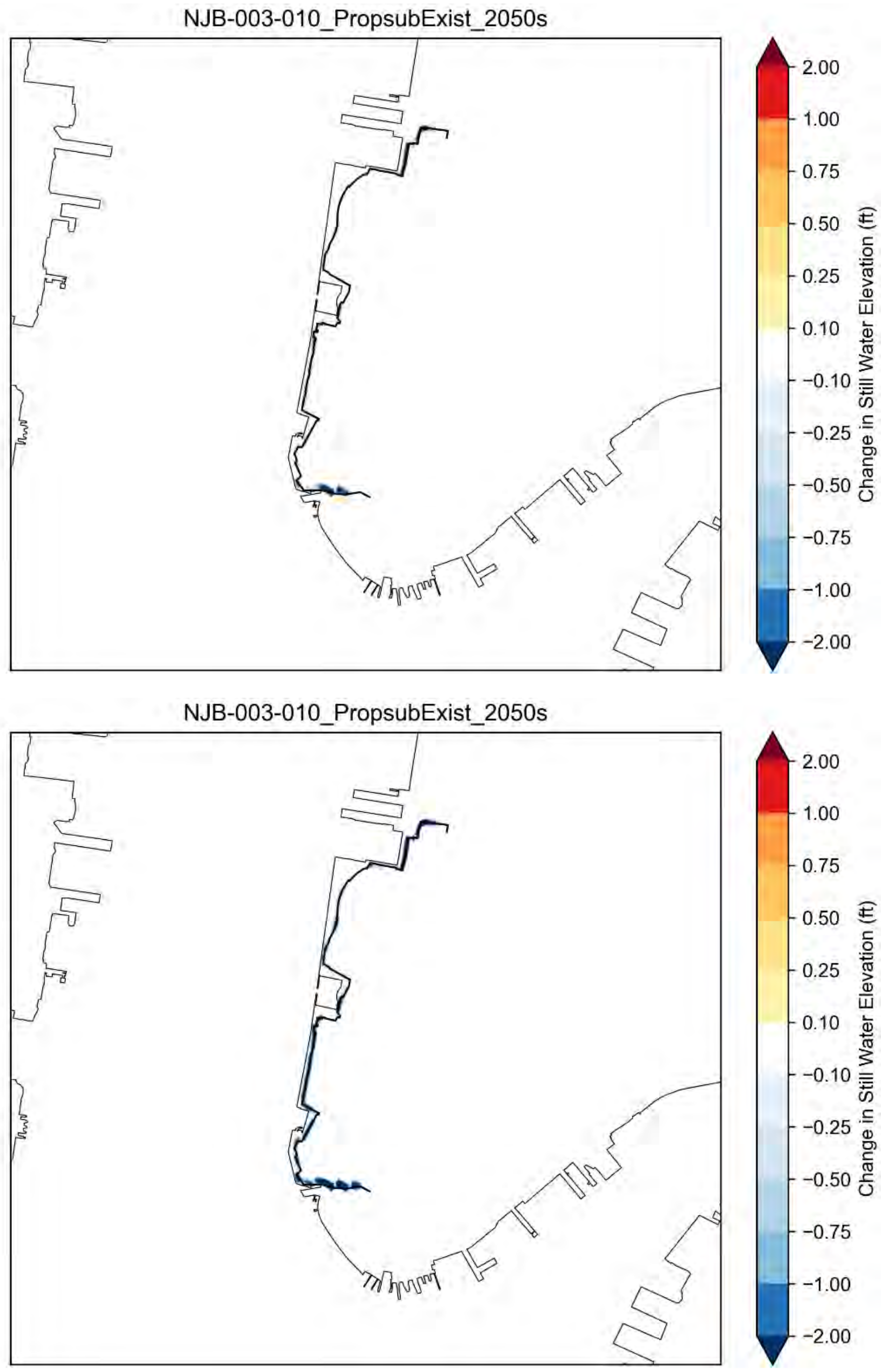


Figure 35. Difference in Maximum Water Surface Elevation for Storm NJB-0003-010 (Upper: 100-year, Lower: 100-year+SLR)

6.4 HRPT No-Rise Analysis

The no-rise analysis for storm surge water level shows negligible impacts due to the NWBPCR project. Inundation levels in HRPK are not impacted (flooding depth of approximately 3 to 5 feet under current storm conditions, and approximately 5.5 to 8 feet depth under future SLR storm conditions) as shown in **Figure 36**. The no-rise analysis for water waves is based on a northwestern wind occurring at the same time as the maximum water level which is conservative scenario. As shown in **Figure 37**, the water waves show minor impacts on wave heights, less than 6 inches near the wall and 1 to 3 inches away from the wall. Wave heights of 1.0 to 2.0 feet for current conditions and 2.0 to 3.0 feet for future SLR conditions remain with a small variation. The maximum wave impacts of 1 to 6 inches are estimated to be present for less than approximately 1 hour.

The model results show that the FBS has negligible impacts on flood levels or depths for the project area, including HRPT properties. FBS has minor impacts on wave heights as follows: For 100-year water level (about 3 ft depth in HRPT area), there are 2-3 inches increase of wave height. For 100-year with SLR (about 5 ft flood depth in HRPT area), there are about 2-6 inches increase of wave height. Higher increases are near the structure and only 1-3 inches increase away from the structures.

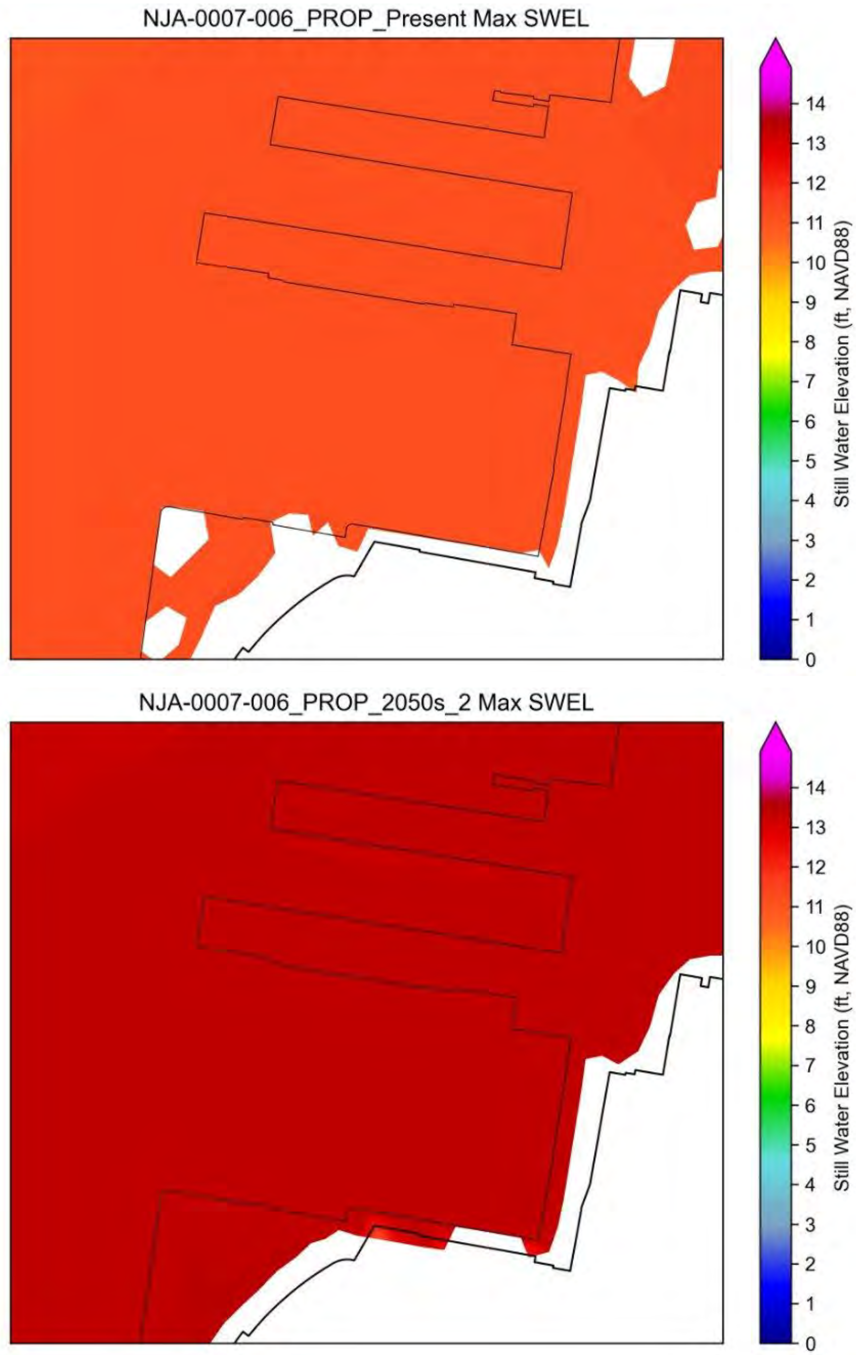


Figure 36 Inundation map and surge water level (Zoom-in HRPT area) for the 1% Annual Maximum Storm and 1% Annual +2050s SLR (Upper: 100-year, Lower: 100-year+SLR)

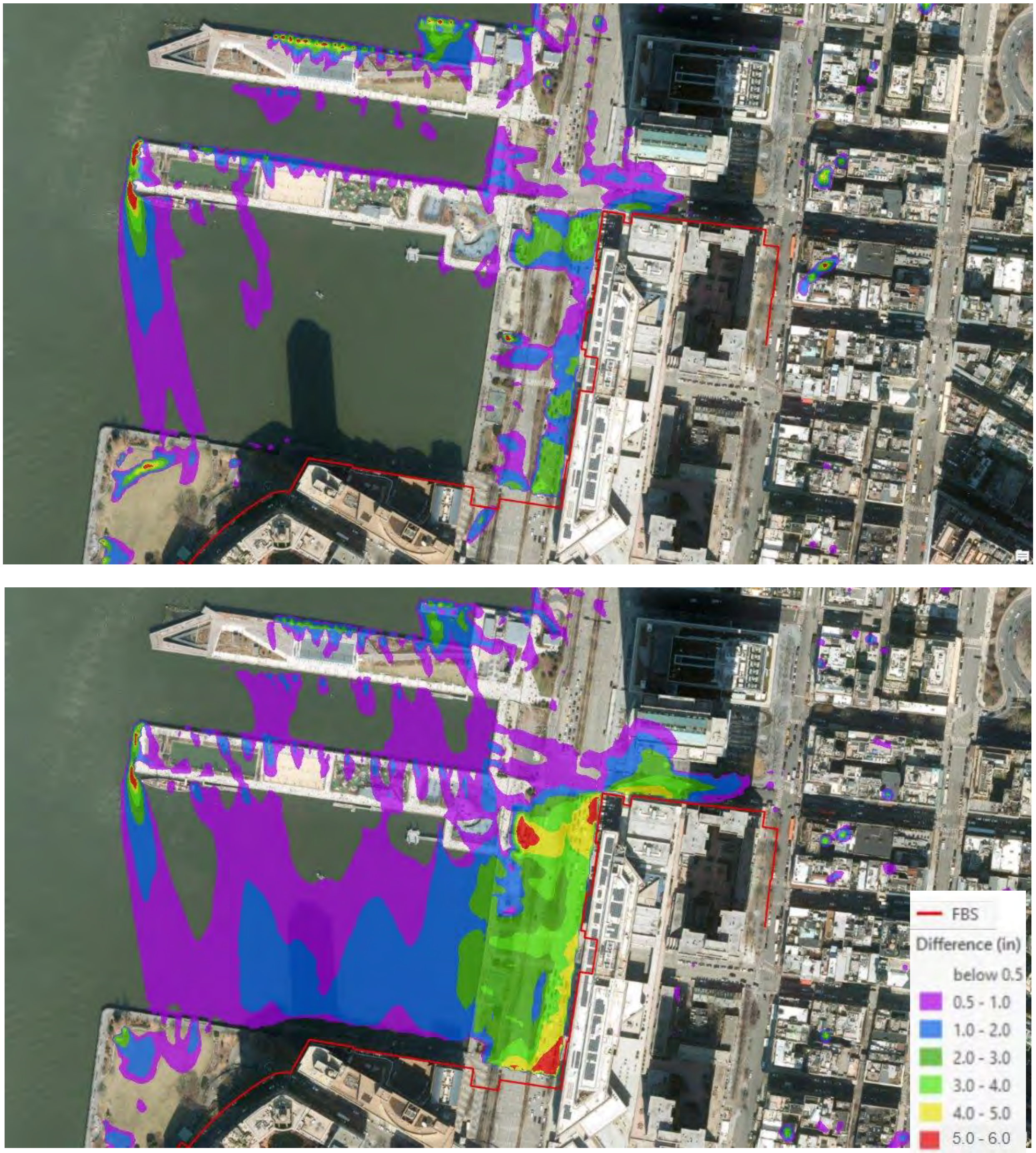


Figure 37 Difference Plots Wave Heights (HRPK Area) proposed minus existing (Upper: 100-year, Lower: 100-year+SLR)

7 Submerged Butterfly Valves

7.1 Description and Modeling Objectives

In Reach 5 on the inland side of the North Cove Marina, there are two channels underneath the platform, located above the PATH tunnels. The PATH tunnels were built in the 1910s and predate the fill and construction of Battery Park City. Battery Park City was built on land over the Hudson River in an area used as a landfill for the excavation of the World Trade Center. No fill was allowed over the existing PATH tunnel, so a platform was built to prevent additional loads on the PATH-tunnels and created interior channels. The channels are open to the tidal water level fluctuations from the Hudson River on the Marina side and closed on the inland side. During storm surge flooding events, surge may enter the open channels. To provide a complete line of defense, the channels need to be closed during storm surges when water levels are expected above EL +8.00 ft (NAVD88) which could result in uplift on the overdecking and storm surge flow through any stormwater drains into the channels. In all other conditions, the channels must be open to allow regular tidal flushing to maintain existing water quality of the channel waters. As part of the 30% design, hydrodynamic modeling is used to provide guidance on how the size of the gate structures at the tunnel openings may impact circulation and flushing of the water in and out of the tunnel spaces.

The approximate surface area of the open water in each of the existing tunnels is: 130 ft x 495 ft = 64,500 ft^2. From the Battery NOAA tide station, the largest diurnal tide difference is 5.1 ft (1.55 m), which combined with the open water area equates to a tidal prism of about 330,000 CF. With approximately 6.2 seconds between high and low tide, this gives an average flow rate of 15 cfs through the tunnel entrances. The hydrodynamic model described below is used to refine the flowrate estimate for existing conditions and to compute the flowrate for a range of potential flow area restrictions representing various gate designs and sizes.

7.2 Modeling Approach

A Delft3D model was constructed to simulate the impact of various gate configurations. Delft3D is an industry standard and well used modeling tool within the industry and commonly used for simulating currents and depths in lakes, bays, and oceans resulting from surface wind forces, astronomic tides, and Coriolis accelerations. The model can be run in depth averaged mode or in full 3D mode and can include weirs and flow blockages which are used here to represent the closing of the channel by the gate structures. A depth averaged mode was used. The model domain is shown in **Figure 38** and includes both channels and the North Cove Marina. All of the piles that support structures are included in the model as flow blockages. The computational domain mapped to model coordinates is shown in **Figure 39**. The cell size is 0.5 m and the timestep is 0.6 seconds. The boundary condition applied along the western boundary of the model domain (in the Hudson River) is a daily fluctuation of tidal elevation from the Battery NOAA gauge and all other boundaries are closed. No additional inflows into the channels are applied. The Delft3D modeling was not conducted during the 40% design stage as further coordination with stakeholders and agencies was required. However, additional modeling using Delft3F or Flow3D is scheduled for 90% design.

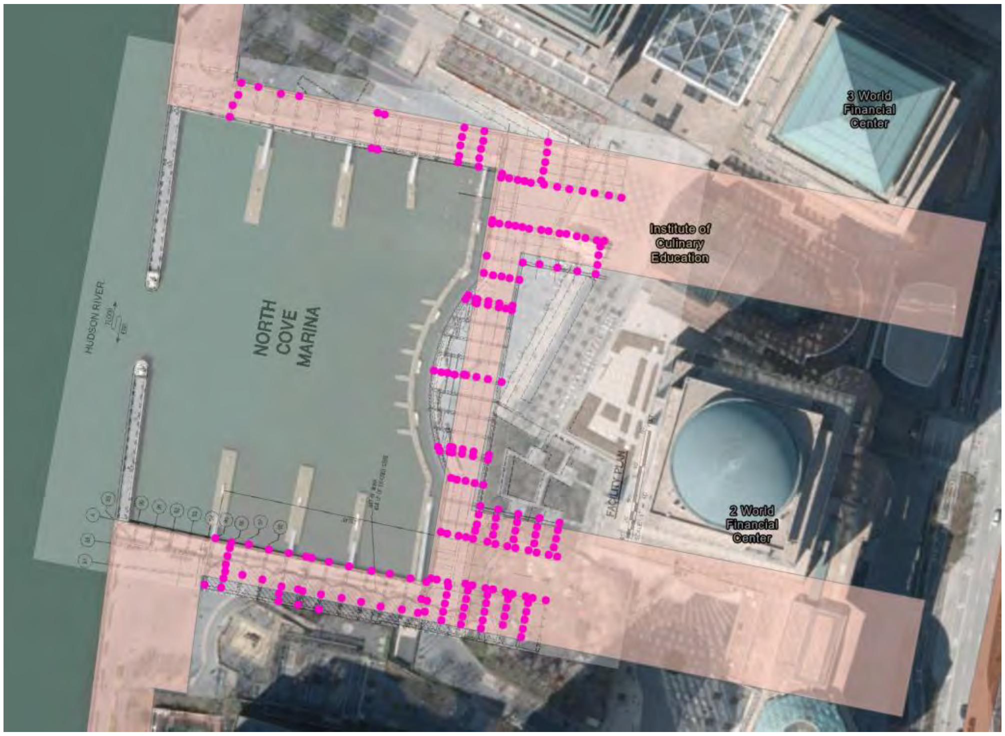


Figure 38: Plan view of the marina, the two channels, and the pile supports. Pink dots represent some digitized pile locations.

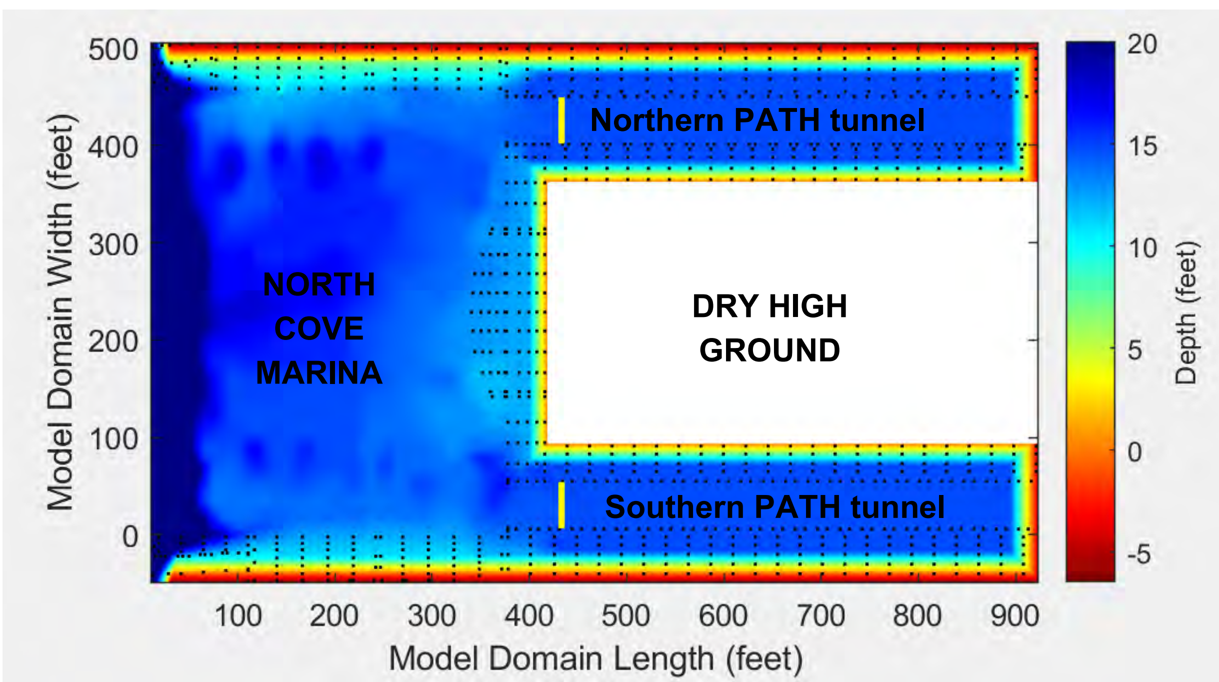


Figure 39: The Delft3D model representation of the study area. Contours indicate depth, the black dots represent pile support structures, and the yellow lines represent approximate proposed locations of the butterfly valves.

7.3 Modeling Results

The base condition Delft3D model result is shown in **Figure 40** with a close-up of the model configuration at the channel opening (left) and the computed flow field of the tunnel (right). The model computes a maximum instantaneous flow rate to be around 22 cfs and the depth averaged velocity less than 0.03 ft/s. Note that in the proposal phase prior to developing the hydrodynamic model, the design assumption was to limit butterfly valve velocities to be below 1 ft/s to reduce the risk of scour. The numerical modeling demonstrates that the flows in the channels are very low, even for existing conditions. The contours of maximum flow velocity for both flood and ebb tide are less than approximately 0.03 ft/s.

The Delft3D model is configured for several representations of potential gate structures. The preliminary concept proposed two gates with a diameter of 10 ft as shown in **Figure 41**. As can be seen in **Figure 41**, the distribution of the flow appears to have similar shape as the existing condition flow shown in **Figure 40**. The peak velocities are increased in the gate openings and approximately doubled to 0.06 ft/sec. Even with the flow constriction from two gates, the water movement is very slow. The Delft3D model was also configured to look at a constriction of a single gate with increasingly smaller opening using gate widths of 47.6 ft, 34.4 ft, 21.3 ft, and 8.2 ft. For each of the four openings, the existing elevations and shallowed elevations were configured for simulations. In total eight cases were set up, as listed in **Table 9**. The model result for the largest single gate opening of 47.6 ft is shown in **Figure 42** and the result for the smallest single gate of 8.2 ft is shown in **Figure 43**. As with the dual-gate configuration (**Figure 41**), the constrictions alter the spatial distribution of where the highest flows occur, but the magnitudes of the maximum velocity do not significantly increase. A plot of maximum velocity vs gate opening is shown in **Figure 44**. The low velocities predicted for this location should be considered with respect to sedimentation as the Hudson River is known to have a high sediment load and the estimated velocities may not be adequate to regularly flush sediment from gate openings. Maintenance associated with removal of debris and sediment should be included in future planning.

Table 9: Simulation Case ID and Corresponding Opening Dimensions

Case ID	Cross-section Area (ft^2)	Notes
Open	1,683	Existing Condition
Gate1	197	Conceptual design of two 10’ gate
O1D0	709	Opening (47.6’/14.5m) with existing bed elevation
O2D0	513	Opening (34.4’/10.5m) with existing bed elevation
O3D0	318	Opening (21.3’/6.5m) with existing bed elevation
O4D0	122	Opening (8.2’/2.5m) with existing bed elevation
O1D1	529	Opening (48’/14.5m) with shallower bed elevation
O2D1	379	Opening (34.4’/10.5m) with shallower bed elevation
O3D1	234	Opening (21.3’/6.5m) with shallower bed elevation
O4D1	90	Opening (8.2’/2.5m) with shallower bed elevation

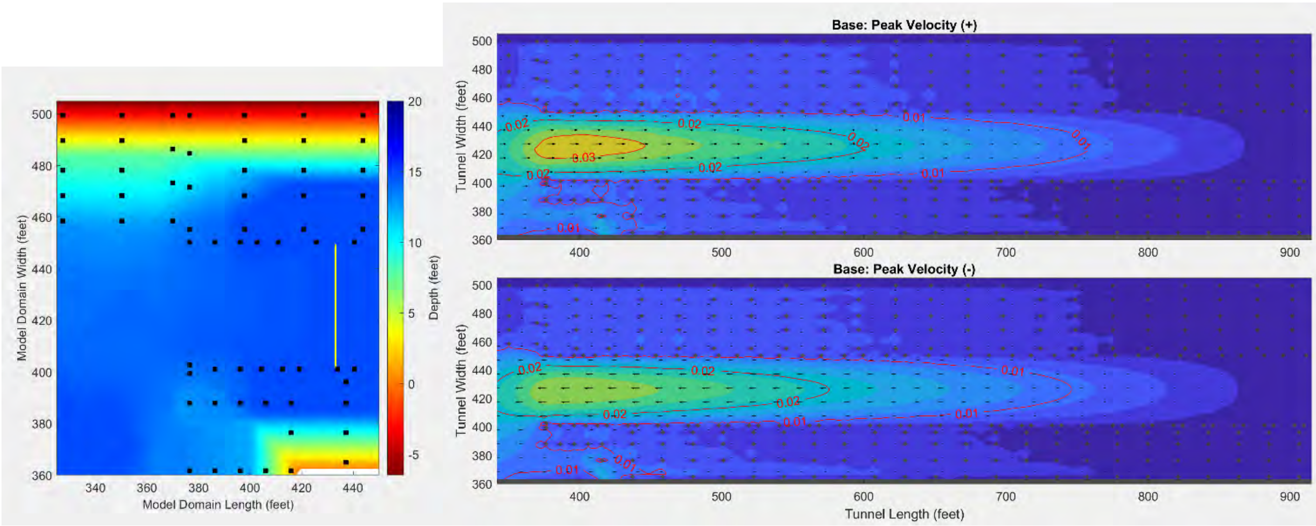


Figure 40: Delft3D Base Model Results (Existing Conditions; yellow line is the proposed location of the gate structure), model configuration (left) and flow field (feet/s) for flood tide (top right) and on ebb tide (bottom right).

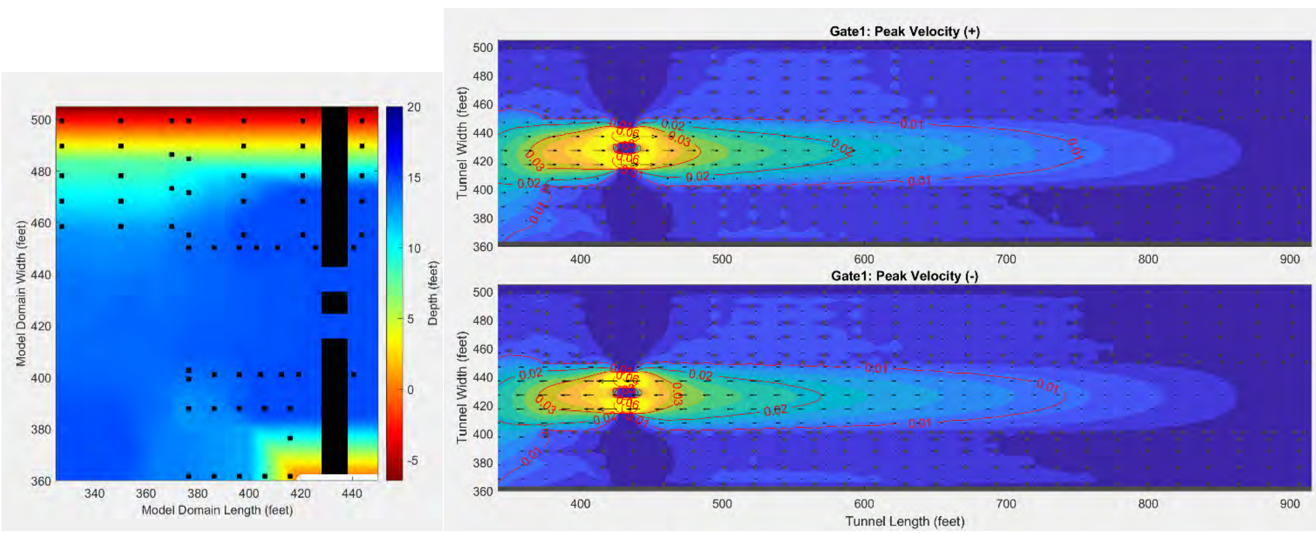


Figure 41: Delft3D Model Results for two 10’ diameter gate configurations (left) and flow field (feet/s) for flood tide (top right) and on ebb tide (bottom right).

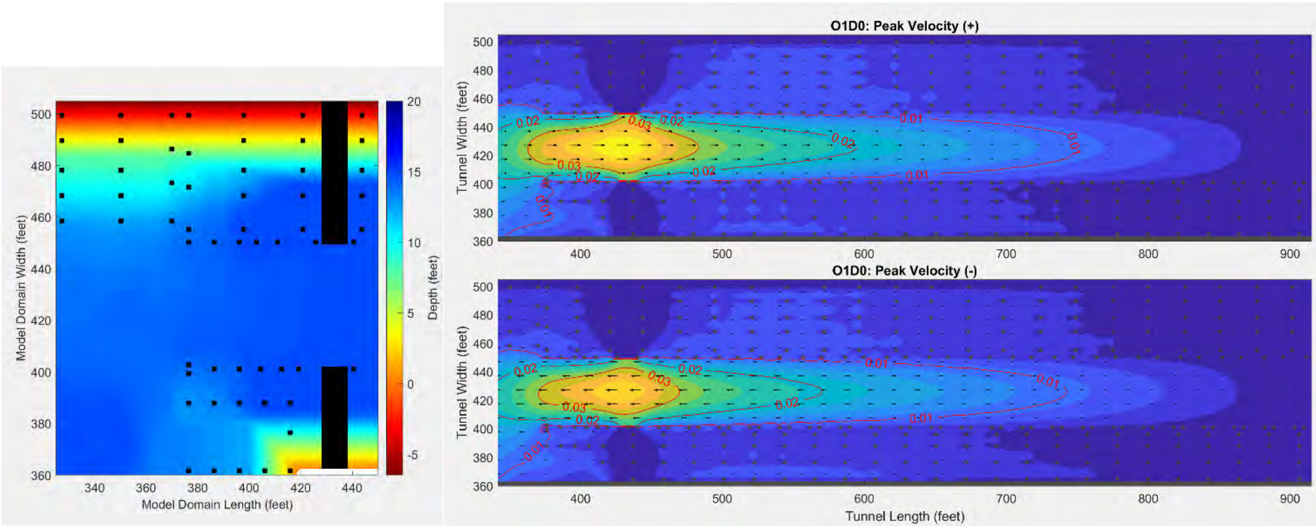


Figure 42: Delft3D model for largest 47.6' gate configuration (left), and model configuration (left) and flow field (feet/s) for flood tide (top right) and on ebb tide (bottom right).

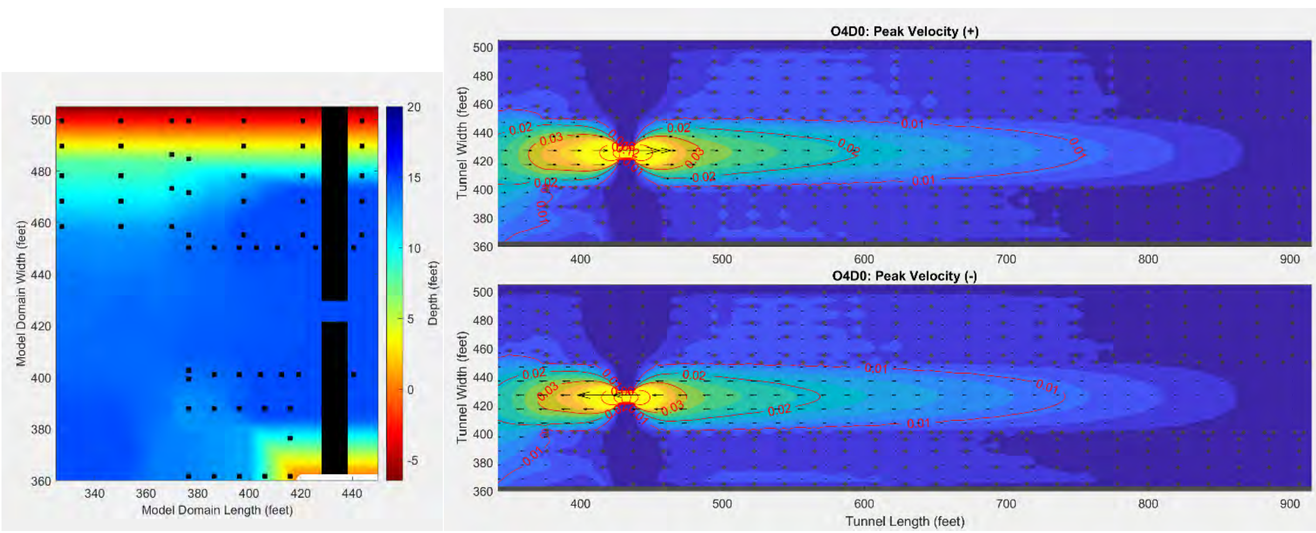


Figure 43: Delft3D model for smallest 8.2' gate configuration (left), and model configuration (left) and flow field (feet/s) for flood tide (top right) and on ebb tide (bottom right).

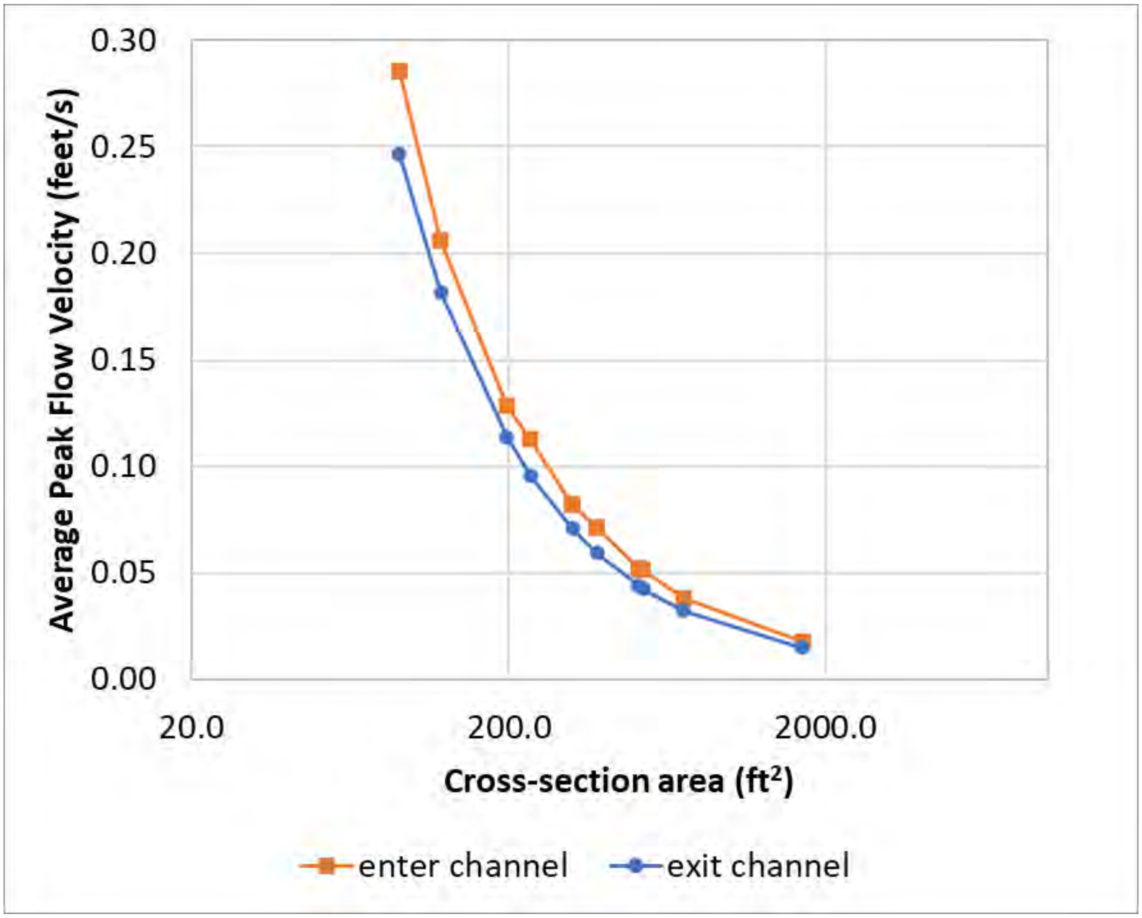


Figure 44: Maximum velocity at the gate opening vs gate opening size as computed with Delft3D.

An additional consideration with respect to the size of the gates is the impact on the tidal prism volume, the amount of water that flows into and out of an estuary or bay with the flood and ebb of the tide, and residence time, the time during which water remains within a water body. The tidal prism volume (blue bars) and residence time (orange line) for each of the modeled gate geometries are shown in **Figure 45**. The change in residence time is negligible. The conclusion from the modeling is that the size of the gate opening has limited impact on flushing and residence time due to the low velocities that exist even for existing conditions.

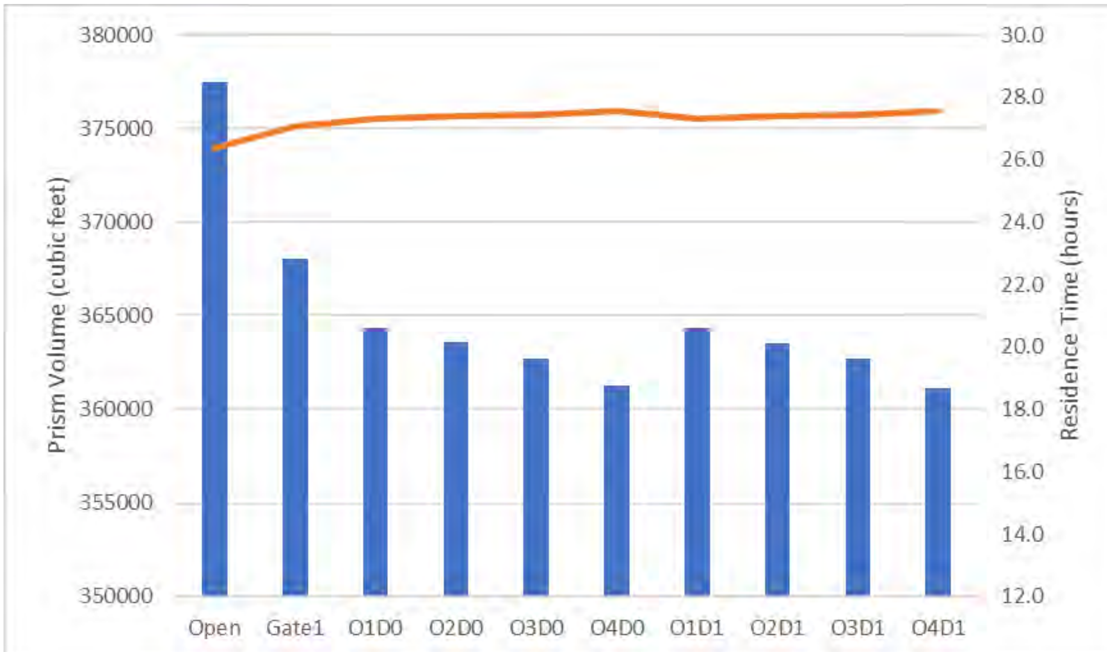


Figure 45: Prism Volume (blue bars) and residence time (orange line) for each modeled gate geometry.

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