Hydrodynamic Modeling Results

Financial District and Seaport Climate Resilience Master Plan

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Overview

This appendix supplements the Financial District and Seaport Climate Resilience Master Plan – Chapter 5: Flood Defense. This appendix provides additional detail on the hydrodynamic and wave modeling that the Project Team conducted to understand how water moves in and around the study area, determine the height of the flood defense, or design flood elevation, and predict future water velocity – or speeds – and tides in the East River with the new flood defense infrastructure in place.

1. Introduction

Hydrodynamic modeling was integral to developing the Financial District and Seaport Climate Resilience Master Plan. Modeling informed every phase of the Master Plan's development and served as an ongoing "check" that the proposed flood defense did not significantly alter hydrodynamics—or the movement of coastal waters—in the study area or regionally. While any in-water structure (e.g., a pier) will inevitably cause some nominal change to circulation patterns, *large* hydrodynamic changes can potentially increase flood elevations outside of the study area, scour coastal foundations, disrupt navigation, and harm aquatic habitats. These adverse impacts are not acceptable based on the guiding principles that shaped the development of the Master Plan, as well as from a regulatory perspective.

The complexity of the hydrodynamic modeling increased over the course of the project as the Project Team learned more about the study area and developed potential resilience solutions. First, the Project Team performed wave modeling and an overtopping analysis to determine the project design flood elevation (DFE) – or the required elevation of the flood defense to prevent overtopping by waves and storm surge. Once the Project Team concluded that it was not practicable to construct the flood defense entirely on land, the Project Team began by modeling concept-level shoreline extensions in a relatively coarse 2D model to understand the broad potential for hydrodynamic impacts, focusing primarily on identifying potential changes in tidal circulation patterns and velocities as well as ensuring that extending the shoreline of Lower Manhattan into the East River would not result in an increase in tidal elevation. Later, as the potential resilience solutions were further refined based on technical, community, and regulatory feedback, the Project Team developed a high-resolution model (both 2D and 3D) to understand the potential impacts different flood defense alignments could have on the environment. Lastly, a hydraulic impact analysis was performed to confirm that the Master Plan would not lead to increased flood elevations during a major storm event – either within or outside the study area.

This appendix provides an overview of the hydrodynamic modeling completed while developing the Master Plan. Where possible, the modeling approach is consistent with previous work completed for other coastal resiliency projects in the New York City Region, including the *Southern Manhattan Coastal Protection Study, the East Side Coastal Resiliency project, the New York City FEMA Region II Flood Insurance Rate Map Appeal*, and *New York State's Living Breakwaters project*.

2. Wave Modeling, Overtopping Analysis, and DFE

The height of the flood defense is determined by how tides and coastal storms currently impact the study area and, crucially, how this will change in the future with rising sea levels. Computer modeling helped the Project Team answer various "what if" scenarios to evaluate how future conditions would impact the performance of flood defense structures and ultimately determine the design flood elevation (DFE). The DFE is the elevation of the flood defense structures needed to protect the area from future coastal storms.

The flood elevation used to determine the DFE includes the stillwater elevation (SWEL), or the static water surface elevation (WSEL) from the coastal storm tide, plus any sea level rise (SLR), and any additional wave height above that. [1] The DFE was then computed based on an allowable amount of water that can spill over the top of the barrier during the design flood event as waves break against the structure (this is referred to as the "wave overtopping rate"). The DFE can also include additional freeboard to account for uncertainties in the determination of flood elevations, uncertainty in wave contributions, uncertainty in SLR projections, and/or additional safety.

For the Master Plan, the Project Team determined the DFE by first understanding the 100-year wave climate in the project area using several one-dimensional SWAN wave models. The Project Team then used the wave data from the modeling to perform an overtopping analysis. The results of the overtopping analysis were then used to set the DFE for different sub-geographies within the study area.

2.1 Stillwater Elevations

The Project Team obtained stillwater elevations from the FEMA Preliminary Flood Insurance Rate Maps (PFIRMs) for New York City. The Project Team used PFIRM transect number NY18, which is in Battery Park and has the highest stillwater elevation out of the transects in the study area. The present day 100-year SWEL at NY18 is 11.30 feet[2], which the Project Team then adjusted for projected sea level rise (SLR) in the 2050s and 2100 by adding 2.5 and 6.25 feet, respectively. These SLR values are the 90th percentile projections from the New York City Panel on Climate Change (NPCC) 2019 report. Adding the SLR projections to the present day 100-year SWEL results in SLR-adjusted future stillwater elevations of 13.8 feet and 17.55 feet for the 2050s and 2100, respectively. The stillwater elevations with projected SLR are summarized **Error! Reference source not found.** in Table 1**Error! Reference source not found.**.

Table 1 – Projected stillwater elevations for the study area.

Time Horizon	Present Day 100-yr SWEL (ft)	NPCC 90th Pct SLR Projection (ft)	SWEL with SLR (ft)
2050s	11.30	2.50	13.80
2100	11.30	6.25	17.55

2.2 SWAN Modeling

The Project Team used several one-dimensional SWAN (Simulating Waves Along Nearshore) models to understand the 100-year wave climate in the study area. The SWAN model is a third-generation wave model that has been used extensively to study waves in nearshore areas by researchers and consultants.[3] SWAN can simulate the following offshore and nearshore wave processes:

- Wind-wave generation (wave growth due to winds)
- Shoaling (increases in wave height as waves travel into shallow water)
- Refraction (changes in wave direction due to changes in water depth)
- Energy dissipation due to bottom-friction, depth-limited breaking, white capping, and wind-wave non-linear interactions

[1] Note that the SWEL used specifically when referring to the static component of storm surge from a coastal storm where wave action is present. WSEL is a more generic term used to describe the water surface in other situations.
[2] Based on the North American Vertical Datum of 1988 (NAVD 88). All other elevations in this appendix are referenced to NAVD 88, unless otherwise noted.
[3] Booij, N. R. R. C., Ris, R. C., & Holthuijsen, L. H. (1999). A third-generation wave model for coastal regions: 1. Model description and validation. *Journal of geophysical research: Oceans*, 104(C4), 7649-7666.

The Project Team used the 1D version of the code because the models are efficient and operate on the conservative assumption that the wind direction is parallel to the transect.

i. Wind Speed Selection

To determine the 100-year wave climate, SWAN requires an estimate of the 100-year wind speed as an input. The Project Team estimated the 100-year wind speed using the same approach as the East Side Coastal Resiliency (ESCR) project. The Project Team used data from the 2015 FEMA Preliminary Flood Insurance Study (PFIS) for New York City to plot peak wind speed against the modeled storm tide for the suite of 189 synthetic tropical and historical extratropical storms used to determine the coastal flood elevations. The Project Team fit a curve to the data and used it to associate a peak wind speed of 82 mph with the present-day 100-year storm tide elevation of 11.3 feet.

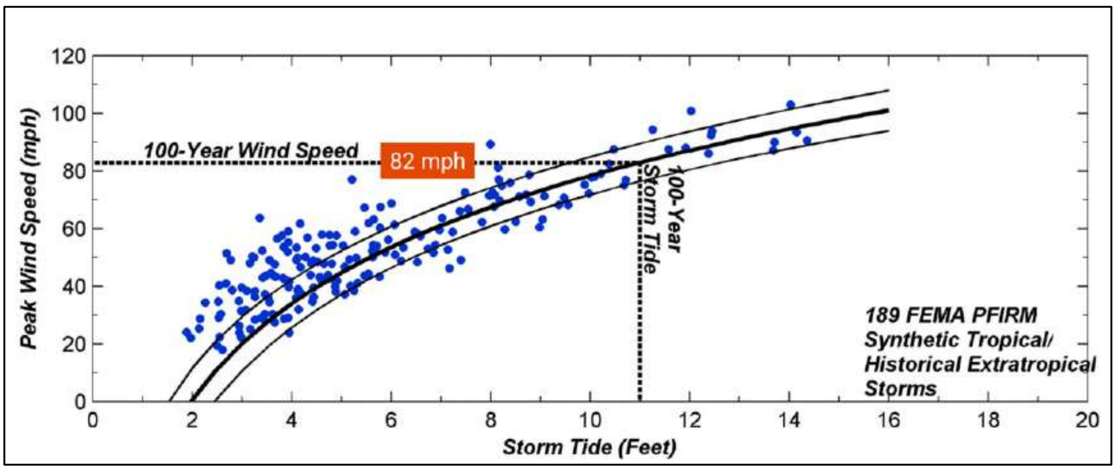


Figure 1 – Plot of peak wind speed vs. storm tide for the 189 storms in the FEMA PFIS study suite.

ii. SWAN Transects

The Project Team ran six SWAN models to characterize the 100-year wave climate in the study area. The transect alignments are shown **Error! Reference source not found.Error! Reference source not found.** (Figure 2). Each model was initialized using an offshore boundary condition based on the 100-year storm significant wave height in the PFIS at the nearest FEMA coastal transect. The SWAN models were run using both the 2050s and 2100 SLR stillwater elevations using the 100-year wind speed developed from the PFIS data.

iii. SWAN Model Results

Figure 3 is an example plot showing the SWAN output for Transect No. 1 with 2100 sea level rise. This output is representative of the results at the other transects. The other transects all shared the same pattern of having relatively constant offshore wave height that dropped significantly at the shoreline and continuing to decay moving onshore.

Table 2 is a summary of the computed significant wave height (H_s) at the six transects in the study area. As expected, the Project Team observed that wave heights are highest at the Battery transect (No. 1). This is because the area is directly exposed to New York Harbor, and there is a long fetch length (the distance the wind blows over open water) for waves to develop. The computed wave heights are lower in the East River, and they decrease moving up towards the Brooklyn and Manhattan bridges as the exposure to the more open areas of New York Harbor decreases.

As the Project Team was considering several inland protection alignments through the Battery the Project Team also looked at two inland locations, 300 and 600 feet from shore, on Transect No. 1. Wave heights at these locations were much lower than at the shoreline (also shown in Figure 3). This means that inland barrier alignments would require a lower DFE because of the reduced wave height.

Figure 2 – Map showing the six SWAN transects used to characterize the 100-year wave climate in the study area

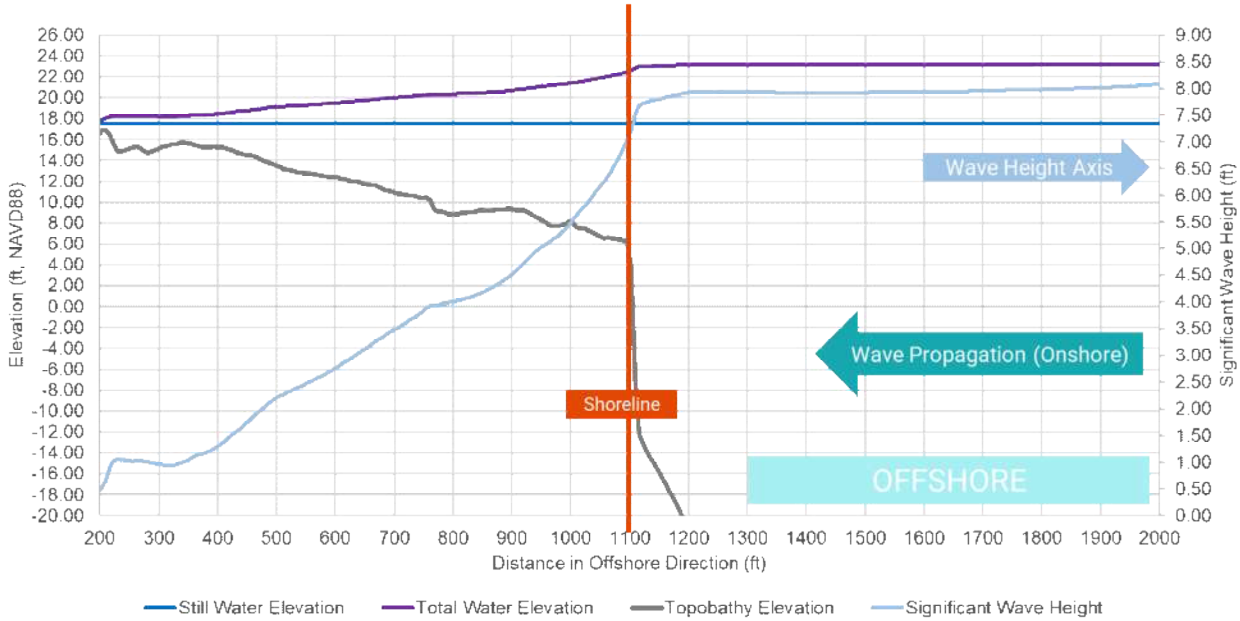


Figure 3 – Example SWAN output for Transect No. 1 with 2100 SLR.

Table 2 – SWAN model estimates of the 100-year significant wave heights at the six model transects in the study area

Transect No.	Location	2050s Significant Wave Height (H_s)	2100 Significant Wave Height (H_s)
1	Shoreline	5.43	5.52
1	300 ft Inland	2.06	3.70
1	600 ft Inland	0.28	2.09
2	Shoreline	4.05	4.04
3	Shoreline	3.77	3.73
4	Shoreline	3.86	3.86
5	Shoreline	3.72	3.81
6	Shoreline	3.67	3.68

2.3 Overtopping Analysis and DFE Selection

Having evaluated the 100-year wave climate in the study area for the 2050s and 2100, the Project Team performed a wave overtopping analysis as a final step before selecting a DFE. The overtopping analysis works backwards from an allowable overtopping rate to compute the height of the barrier necessary to achieve that rate. There are many overtopping criteria and different overtopping rates may be desired in different circumstances depending on what is immediately behind or on top of the barrier. The Project Team reviewed overtopping guidelines from the following sources: EurOtop,[4] the U.S. Army Corps of Engineers Coastal Engineering Manual (USACE CEM),[5]

[4] Pullen, T., Allsop, N.W.H., Bruce, T., Kortenhaus, A., Schüttrumpf, H. and Van der Meer, J.W., 2007. EurOtop wave overtopping of sea defences and related structures: assessment manual.

[5] USACE (US Army Corps of Engineers). Coastal engineering manual. 2002.

documentation for the New Orleans Hurricane and Storm Damage Risk Reduction System (HSDRRS),[6] as well as FEMA guidance for flood hazard delineation behind levees.[7]

Error! Reference source not found. shows a summary of the selected overtopping criteria from the sources the Project Team reviewed. In general, the sources did not recommend that the overtopping rate exceed 0.1 cfs/ft; and generally, the rate where most of the documentation agreed was permissible in most circumstances was 0.01 cfs/ft or less. [8]

The Project Team decided to use two approaches to perform the overtopping analysis and then compare the results from both methods when determining the DFE. The first approach was to use the Franco and Franco methodology outlined in the USACE CEM.[9] The Franco and Franco method is a deterministic approach that uses an empirical formula to compute the mean overtopping rate. The Project Team chose an allowable overtopping rate of 0.011 cfs/ft based on EurOtop guidelines for protecting building structural elements. The second approach is what was used for the New Orleans HSDRRS. The approach is based on the EurOtop implementation of van der Meer's stochastic approach and computes both the 50th and 90th percentile overtopping rates. The 50th percentile overtopping rate cannot exceed 0.03 cfs/ft and the 90th percentile value cannot exceed 0.1 cfs/ft.

Table 3 – Summary of overtopping criteria. Mean overtopping rates are presented in this table.

Overtopping Rate Q (cfs/ft)	For Structure Safety			When Property Behind Structure or People/vehicles on Structure		
	HSDRRS*	**EurOtop**	**CEM**	**FEMA**	**EurOtop**	**CEM**
0.0001	Limited to no Damages	Bare slope levee	Bare crest/slope levee	Zone X	Safe behind structure with a certain setback	Safe vehicles parking and pedestrians on structure crest
0.001				Zone AO		
0.01 (.011)						
0.03		Grass covered levee	Levee crest/Backslope needs be covered		People safe on seawall crest	Unsafe
0.1	Damages expected; not recommended					
1.0						

**HSDRRS criterion also involves a 90-percentile overtopping rate – i.e., 90% assurance OT rate < 0.1 cfs/ft*

[6] USACE (US Army Corps of Engineers) New Orleans District. Interim Hurricane and Storm Damage Risk Reduction System Design Guidelines. 2012.
[7] FEMA (Federal Emergency Management Agency). Guidance for Flood Risk Analysis and Mapping. Coastal Wave Runup and Overtopping. 2018.
[8] cfs/ft: Cubic feet per second of overtopping flow rate per foot of levee or flood wall
[9] Franco, C. and Franco, L., 1999. Overtopping formulas for caisson breakwaters with nonbreaking 3D waves. Journal of waterway, port, coastal, and ocean engineering, 125(2), pp.98-108.

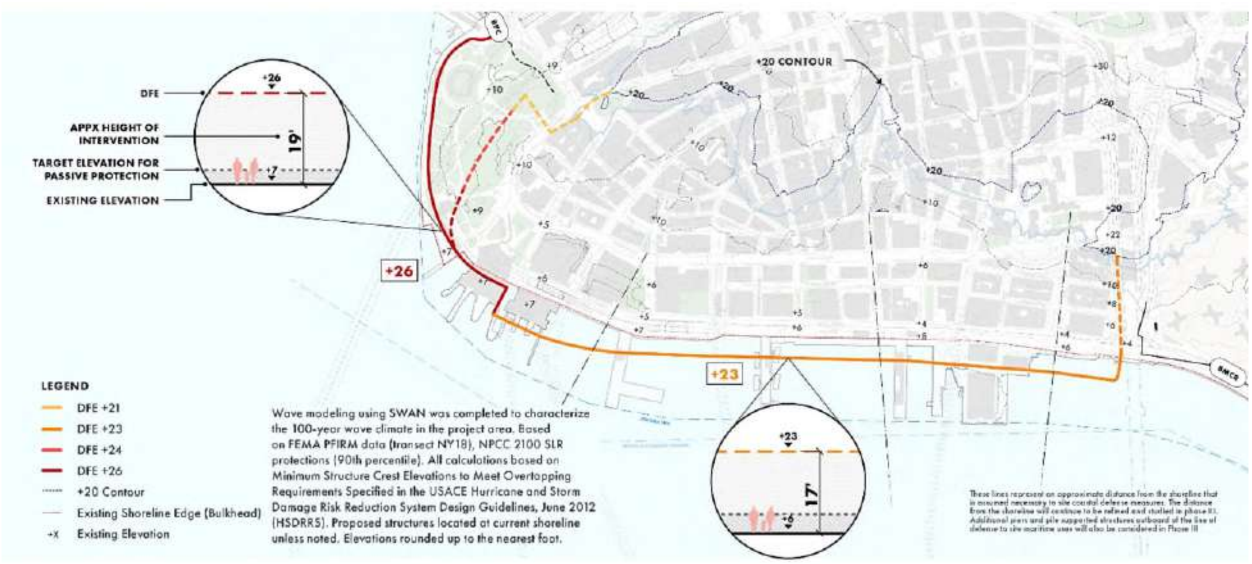
The Project Team ran both methods for the 100-year storm surge event with 2050s and 2100 sea level rise. Additionally, the Project Team calculated the minimum elevation that would be required for FEMA accreditation in the 2050s, which is defined as the maximum of either the wave runup elevation plus 1 foot or the wave crest elevation plus one foot. The results are summarized **Error! Reference source not found.** in **Error! Reference source not found.**.

Table 4 – Overtopping analysis results. Leftmost row labels correspond to the SWAN transect numbering

SWL	100-year SWL + 2050s 90th SLR	100-year SWL + 2050s 90th SLR	100-year SWL + 2050s 90th SLR	100-year SWL + 2050s 90th SLR	100-year SWL + 2100s 90th SLR	100-year SWL + 2100s 90th SLR	100-year SWL + 2050s 90th SLR
Criterion	Q50<0.03, Q90<0.1	Building Safety <0.011	Building Safety	Building Safety	Q50, Q90	Building Safety	**FEMA Accreditation**
Method	EuroTop(2016) Van de Meer and Bruce (2014) HSDRRS (2014): Monte Carlo	EuroTop(2016) Van de Meer and Bruce (2014)	Franco & Franco (1999)	Franco & Franco (1999) w/ minimum 19.6* ft	EuroTop(2016) Van de Meer and Bruce (2014) HSDRRS (2014): Monte Carlo	Franco & Franco (1999) w/ minimum 19.6 ft	Maximum of Runup elev +1ft and Wave crest elev +1ft
T1	21.5	24.0	24.0	24.0**	25.5	28.0	19.4
T1-300	16.5	17.5	17.0	19.6	23.5	24.0	23.6
T1-600	15.0	15.0	14.0	19.6	20.5	21.0	16.0
T2	19.5	21.0	21.0	21.0	23.0	25.0	18.2
T3	19.0	20.5	20.0	20.5	22.5	24.0	18.0
T4	19.0	20.5	20.0	20.5	23.0	24.0	18.1
T5	19.0	20.5	20.0	20.5	22.5	24.0	18.0
T6	19.0	20.5	20.0	20.5	22.5	24.0	17.9
Note: Minimum Crest Elevation			→→→→→→→→→		**Safer recommendation**		

*19.6 ft = 100-year SWEL + 2100 90pct SLR + 2 ft FEMA minimum freeboard

Based on the output in the table, the Project Team decided to use the HSDRSS methodology results for the 100-year 2100 scenario (green column) as the primary elevations for determining the DFE. These elevations were higher than the Franco and Franco results for the 2050s[10] and the minimum elevations for FEMA accreditation in the 2050s (light yellow column). These values were then rounded to the nearest foot to obtain the final DFE values shown below in Figure 4**Error! Reference source not found.**.



[10] With a minimum elevation of 19.6 ft in inland areas. This is based off the 100-year SWEL with SLR and the FEMA minimum freeboard of 2 feet.

Figure 4 – Map of the selected DFEs for the study area

3. Concept-Level Tidal Analysis

The Project Team's initial hydrodynamic modeling focused on understanding the present-day tidal conditions in the study area and understanding the sensitivity of the hydrodynamics to a variety of representative shoreline conditions. The purpose of this initial study was to develop a high-level understanding of the of the range of tidal current velocity impacts that may arise from a shoreline extension into the East River as well as a preliminary understanding of tidal elevation changes. Quantifying the magnitude and extent of these changes is important to determining the ecological, navigational and scour impact of potential flood defense strategies.

3.1 Model Selection

The Project Team chose the Advanced Circulation Model for Ocean, Coastal, and Estuarine Waters (ADCIRC) and the Simulating Waves Nearshore (SWAN) coupled wave and hydrodynamic model, ADCIRC+SWAN, as the primary hydrodynamic modeling system to model tides and storm surge in this project.[11] ADCIRC+SWAN is a state-of-the-art system that can accurately compute currents, wind-waves, storm surge, and wave-induced contributions to storm surge. The ADCIRC+SWAN modeling system has been used extensively in coastal regions of the United States and using the modeling system maintains continuity with the most recent preliminary FEMA floodplain mapping study of New York City[12,13] and recent coastal resiliency projects in the New York City Region.

3.2 ADCIRC+SWAN Mesh

The computational mesh is the core of a hydrodynamic analysis; it is how the physical world—e.g., ground elevation, land use—is represented and communicated to the computer model. ADCRIC+SWAN uses an unstructured triangular mesh, meaning the mesh elements vary in size. This allows for more detail to be included in the model where it is needed and less detail – and therefore more efficient computations – in areas where it is not.

ADCIRC+SWAN meshes need to span the entire western North Atlantic to accurately compute tides and to simulate the onshore propagation of storms surge. Developing a mesh for an ADCIRC+SWAN model a significant undertaking. Therefore, building upon previous work by using meshes from previous studies is the preferred strategy for model development. This approach is more efficient and maintains continuity with previous projects and mesh validation studies.

The Project Team used the same mesh as the East Side Coastal Resiliency (ESCR) project as the starting point for the analysis. The ESCR mesh is based on the mesh used to develop the FEMA Preliminary Flood Insurance Rate Maps for the NYC region, but with additional resolution added for simulations performed for NYC's Special Initiative for Rebuilding and Resiliency (SIRR) following Hurricane Sandy. The ESCR mesh was validated – shown to match observed data – to Hurricane Sandy during the SIRR study. The mesh includes 924 thousand nodes and 1.8 million elements and has a 100-ft element size in the study area. Figure 5 is a plot showing the entire ESCR mesh.

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[11] Dietrich, J.C., Zijlema, M., Westerink, J.J., Holthuijsen, L.H., Dawson, C., Luettich, R.A., Jensen, R., Smith, J.M., Stelling, G.S., and Stone, G.W. 2011. Modeling Hurricane Waves and Storm Surge Using Integrally-Coupled, Scalable Computations. Coastal Engineering, 58, 45-65

[12] FEMA. 2013. Flood Insurance Study: City of New York, New York (Preliminary). FEMA Region II.

[13] FEMA. 2014. Region II Storm Surge Project – Model Calibration and Validation.

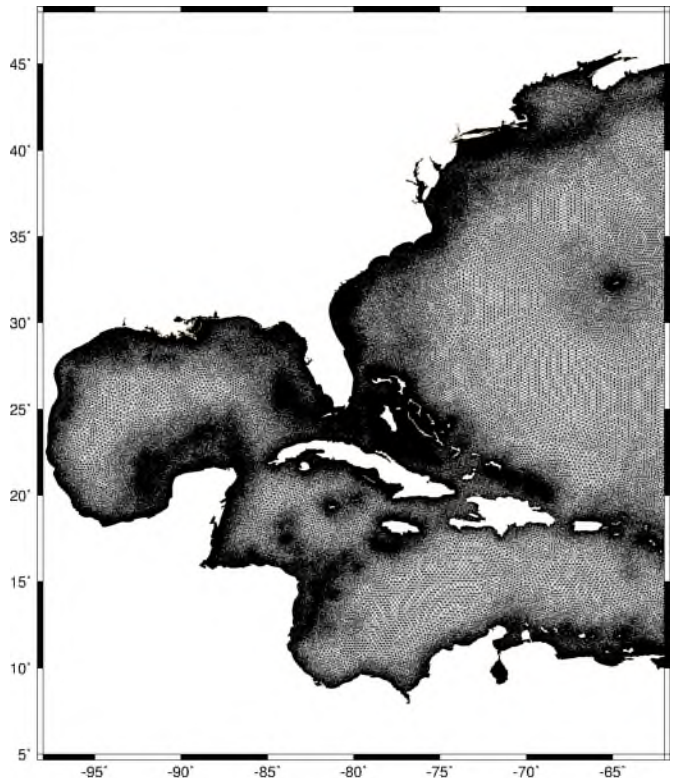

Figure 5 – Plot of ESCR mesh showing the full extent of the model, including the Gulf of Mexico and the western North Atlantic

3.3 Representative Tidal Time Series

The Project Team simulated a full lunar month for the tidal analysis to capture the Spring and Neap tidal cycles. The Project Team identified a tidal cycle in October 2019 that includes predicted water surface elevations that exceeded both Mean Higher High Water (MHHW) and Mean Lower Low Water (MLLW) at the NOAA tidal gauge at the Battery, NY (8515750) – including elevations as high as 3.38 feet above NAVD88. This tidal cycle was used as the representative tidal time series to perform tidal impact analyses for this study, including in the high-resolution modeling that are discussed in later sections. Figure 6 is a plot of the NOAA-predicted tides at the Battery, NY gauge for the tidal cycle.

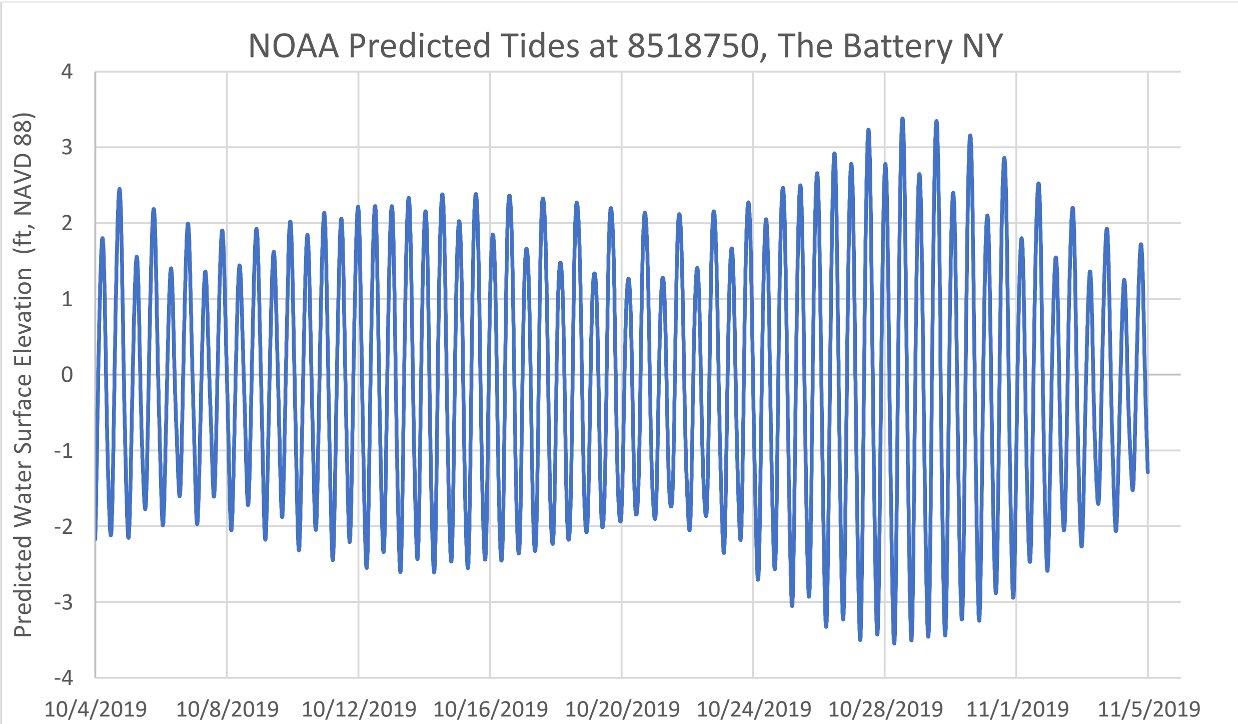

Figure 6 – Plot of NOAA-predicted tides at the Battery, NY for the typical tidal cycle used for the impact analyses

3.4 Present-day Condition Tidal Results

The Project Team ran ADCIRC over a 40-day simulation with an 8-day ramp-up to simulate the full lunar tidal cycle, keeping all model parameters besides the timeframe identical those used in ESCR. Figure 7 shows the maximum water surface elevation (WSEL) in feet above NAVD88 computed by the model over the entire representative tidal cycle under present-day conditions (Baseline). The maximum WSEL does not vary significantly across the study area. Figure 8 shows the Baseline maximum velocity in feet per second (ft/s) computed over the entire representative tidal cycle. Peak tidal velocity varies significantly across the project area. It is higher in the northern study area, where the East River constricts near the Brooklyn Bridge. In this region, the Project Team observed that the maximum velocity exceeds 8 ft/s (4.7 knots) in some locations, which is very high for purely tidal velocities. Further to the south, offshore from the Financial District, the Project Team computed a much lower maximum velocity—on the order of 3 to 4 ft/s (1.8 to 2.4 knots).

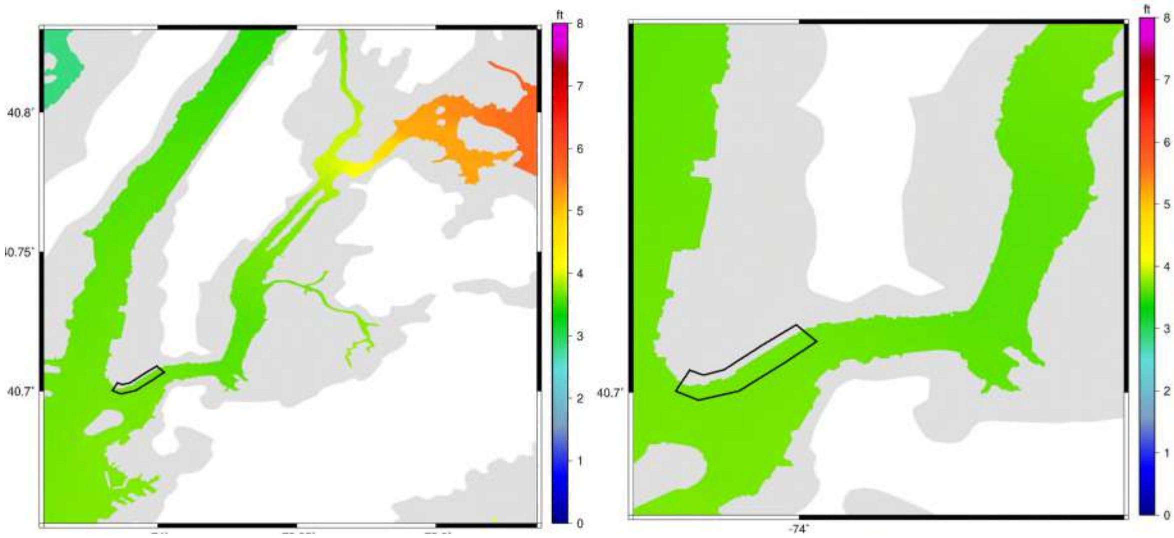

Figure 7 – Maximum WSEL (ft, NAVD88) over the full tidal cycle computed by ADCIRC. Left – Regional view. Right – Study area view. Approximate study area boundaries outlined in black.

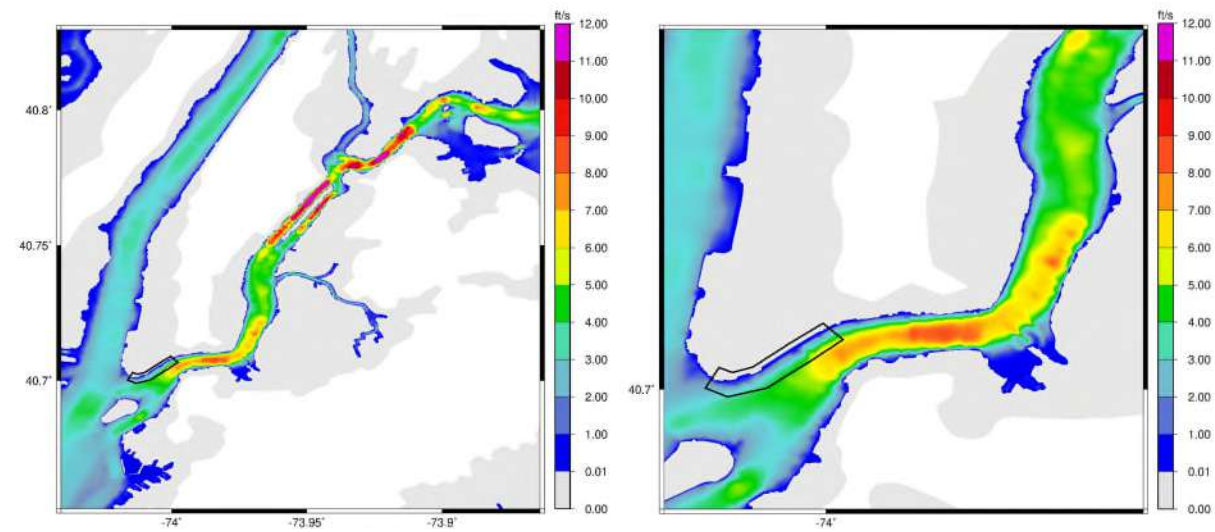

Figure 8 – Maximum depth-averaged water velocity (ft/sec) computed by ADCIRC over the full representative tidal cycle. Left – Regional View. Right – Study Area view. Approximate study area boundaries outlined in black.

3.5 Simple Shoreline Typologies

The Project Team evaluated several simple shoreline typologies to understand how different shoreline typologies may impact the hydrodynamics of the East River. The Project Team considered 50-, 100-, 250- and 500-feet shoreline extension options. 500 feet was considered as the maximum because it is the approximate location of the pierhead line and the start of the Federal Navigation Channel. Figure 9 is a map showing alignment of the 500-foot simple shoreline typology as an example. Note how that the shoreline extension is just a uniform offset from the bulkhead line; this alignment was only intended to assess the potential hydrodynamic impacts and was not a project alignment under consideration at any point.

Figure 9 – Example 500-ft simple shoreline typology. This alignment was only intended to assess the potential hydrodynamic impacts and was not a project alignment under consideration at any point.

3.6 Tidal Impact Analysis and Results

The Project Team modified the ADCIRC mesh to reflect the simple shoreline typologies. The Project Team then ran the model for each of the shoreline typologies over the representative tidal cycle, and then created a series of plots comparing the computed maximum water surface elevations and maximum tidal current velocity with the shoreline typologies to the existing conditions model results, without any project intervention.

iv. Maximum Water Surface Comparisons

The Project Team did not observe any increase in tidal water surface elevation across all the simple shoreline typologies for either the immediate project area or the NYC Region. This was expected based on the experience that the Project Team has had with previous projects: typically, in-water structures impact tidal velocity but not elevation. Figure 10 is a comparison plot showing the change in water surface elevation from the Baseline to post-project conditions with the 500-foot concept-level.

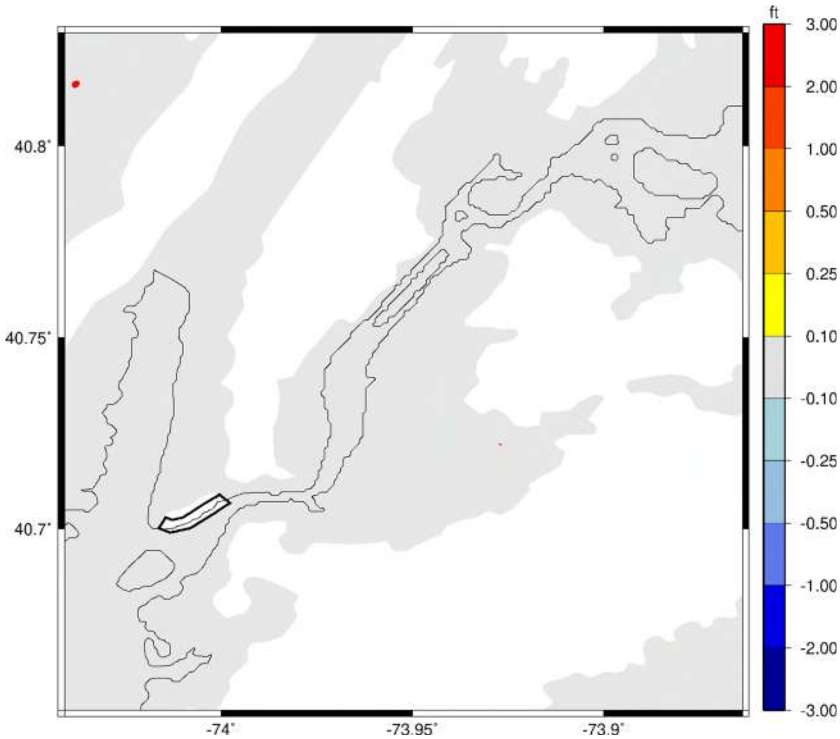

Figure 10 – Difference plot showing the change in maximum WSEL (ft) between Baseline and Proposed Conditions with the 500-ft simple shoreline typology. Warm colors represent increases in WSEL with the project and cool colors represent decreases in WSEL.

v. Maximum Velocity Comparisons

As expected, the Project Team observed changes in tidal velocities with the simple shoreline typologies relative to present-day conditions (Baseline). Figure 11 is a difference plot comparing the modeled velocities between present-day conditions, and with the 500-foot concept-level shoreline extension. As shown in the figure, the extent of tidal velocity increase is large, and spans across the entire East River to Brooklyn. Based on these results, the Project Team ruled out shoreline extension options with this large of a footprint because this level of impact would be untenable from a regulatory and navigational perspective. However, despite the large increase in the study area, it should be noted that the 500-foot typology did not cause any *regional* changes to tidal velocities – all impacts were local to the study area.

The Project Team found the results for the 50-, 100- and 250-foot typologies to be more promising. All three typologies caused much smaller changes to tidal velocities than the 500-foot alternative (e.g., Figure 12 and Figure 13). Based on this observation, the Project Team concluded that 250-foot and smaller footprints may be viable from a purely hydrodynamic perspective and therefore merited further study.

However, the ADCIRC model has a relatively coarse mesh resolution in the study area, and while its resolution was sufficient to draw conclusions between the 500-foot and the smaller typologies, it was insufficient to resolve the differences between the smaller typologies themselves. Figure 12 is a difference plot for the 50-fot typology and Figure 13 is a difference plot for the 250-foot typology. Both show very similar changes relative to baseline conditions, and it is difficult to draw any conclusions about their relative impacts in terms of tidal velocity. The 100-foot alternative, as expected, had similar results to these. To address this limitation, the Project Team developed a high-resolution model to evaluate future alternatives as concept-level typologies were further refined (see sections 4 and 5).

None of the studied typologies caused any regional changes to tidal velocity.

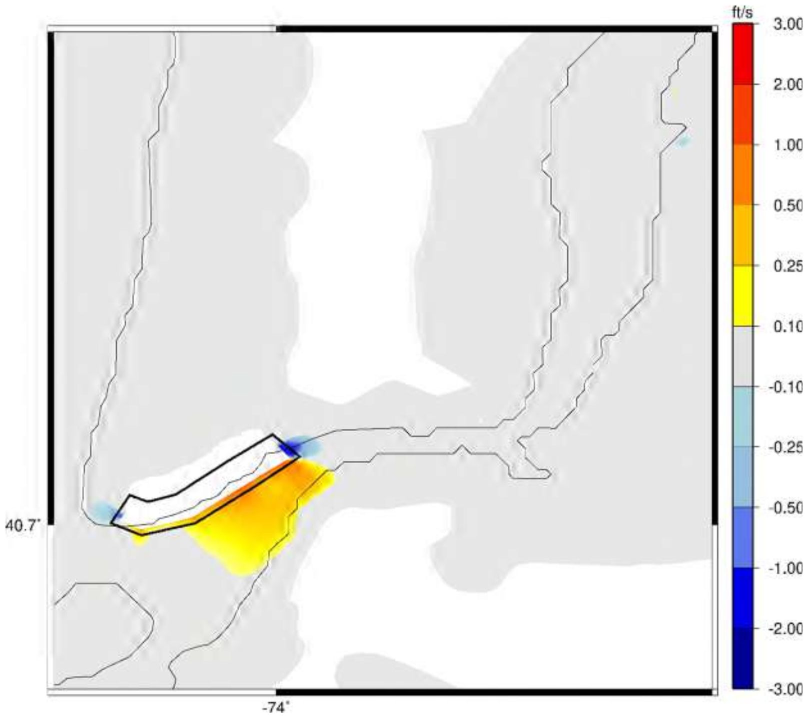

Figure 11 – Difference plot showing the change in maximum tidal velocity (ft/s) between Baseline and with the 500-foot simple shoreline typology. Warm colors represent increases in tidal velocity and cool colors represent decreases.

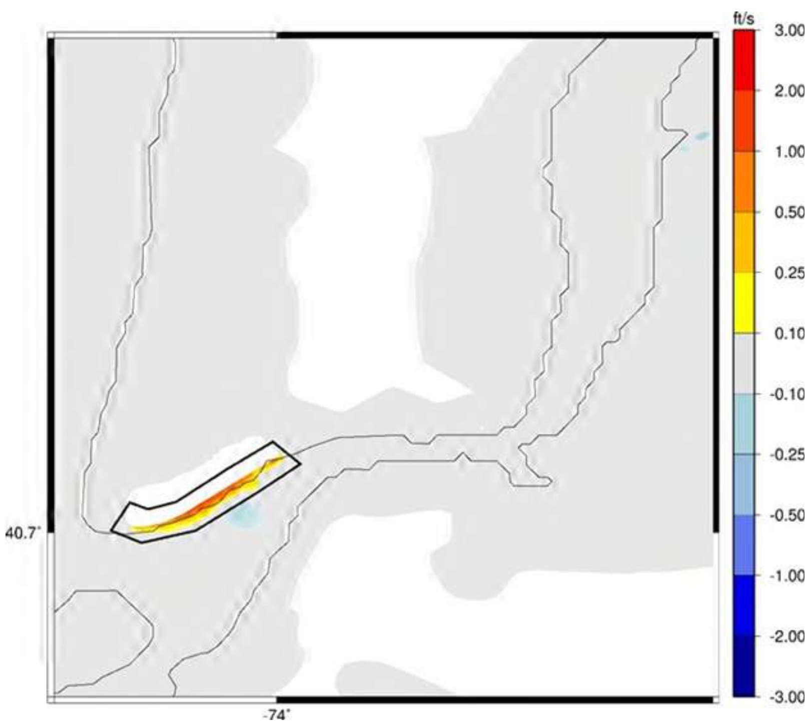

Figure 12 – Difference plot showing the change in maximum tidal velocity (ft/s) between Baseline and with the 50-foot simple shoreline typology. Warm colors represent increases in tidal velocity and cool colors represent decreases.

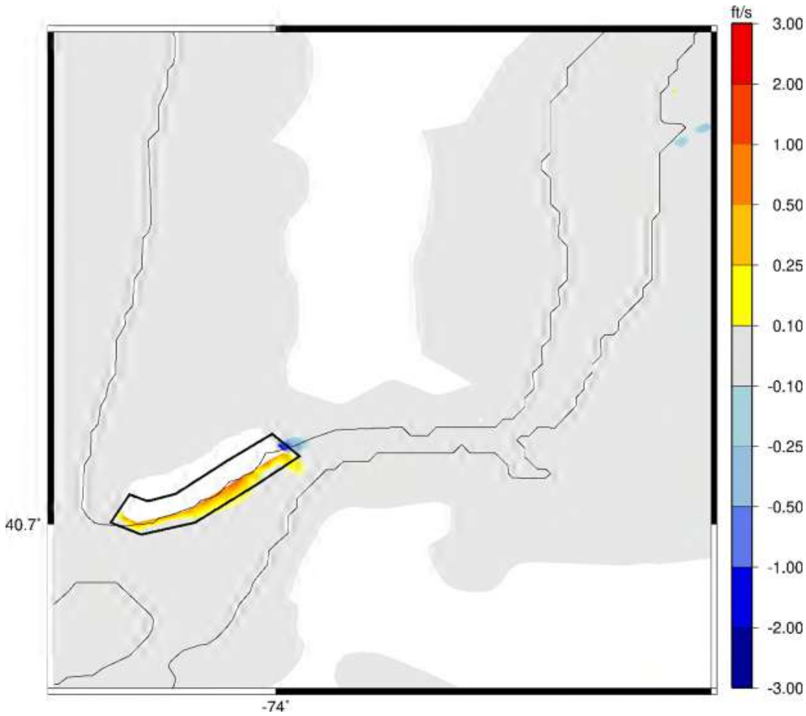

Figure 13 - Difference plot showing the change in maximum tidal velocity (ft/s) between Baseline and with the 250-ft simple shoreline typology. Warm colors represent increases in tidal velocity an\d cool colors represent decreases.

3.7 Key Findings

Based on the modeling of the initial simple shoreline typologies, the Project Team was able to conclude the following:

- The magnitude and extent of tidal velocity change with 500-foot typology relative to present-day conditions is unacceptable. Viable protection alignments will need to have a much smaller offshore footprint to be viable.
- The 250-foot and smaller typologies had much smaller velocity impacts than the 500-foot typology, both in terms of the magnitude of change and the change footprint. The Project Team believe footprints in this range may be viable from a purely hydrodynamic perspective.
- The limited resolution of the ADCIRC+SWAN model was insufficient to resolve differences between the velocity impacts of the 250-ft and smaller typologies. A high-resolution model is required to resolve the nuances between nearshore alignments.
- The Project Team did not observe a regional or localized increase in tidal water surface elevation for any of the simple typologies, including the 500-foot shoreline extension.

4. 2D Depth-Averaged Tidal Impact Analysis

After the concept-level tidal analysis, the Project Team studied the potential project alignments in greater detail, focusing on other areas besides hydrodynamics. After the potential resilience solutions were narrowed and further developed based on technical feasibility and community and regulatory feedback, the next step in the hydrodynamic analysis was to perform a more detailed study of the potential hydrodynamic impacts of the refined concepts. The ADCIRC+SWAN model used for the initial assessment of the simple shoreline typologies had inadequate resolution to resolve the nuances between nearshore alternatives. The Project Team developed a high-resolution model to perform this analysis.

4.1 Model Selection

The Project Team selected the Delft3D Flexible Mesh (Delft3D-FM) modeling suite to perform the high-resolution modeling. The hydrodynamics module in the Delft3D-FM suite, D-Flow FM, is the successor to Deflt3D-Flow, which uses a structured curvilinear mesh for hydrodynamic modeling. D-Flow FM uses an unstructured mesh consisting of triangles, quadrilaterals, and pentagons – this allows for efficient representation of the complex coastal geometry of the NYC region in the model. Another strength of D-Flow FM is that it uses a variable time step, which allows the model to use a longer computation interval during low-velocity periods of the simulation and a shorter computation interval to maintain stability during high-velocity periods. This can result in shorter model run times than a model like ADCIRC+SWAN that requires a fixed time step.

D-Flow FM can run in both two and three dimensions ("2D" and "3D"). For the initial tidal impact analysis, the Project Team decided to run the model in 2D mode because it was more efficient, less data intensive, and would provide enough information to compare the relative impacts of alternatives. 3D modeling was performed later in the study with a refined project geometry – this will be discussed in a later section.

4.2 Mesh, Model Parameters, and Boundary Condition Configuration

Figure 14 shows the extents of the D-Flow FM model. It extends from approximately Bay Ridge to the south and north to Roosevelt Island on the East River and 80th street on the Hudson River. The D-Flow FM mesh has element sizes that range from hundreds of feet in the middle of New York Harbor and in the Hudson River to as small as 15 feet immediately along the shoreline in the study area. Figure 15 is a detailed plot of the D-Flow FM mesh near the study area that shows the variation in element size.

The Project Team ran the D-Flow FM model in a nested configuration with the ADCIRC+SWAN model. This allowed us to leverage the strength of both models: ADCIRC+SWAN was used to simulate the large-scale tidal circulation in the western North Atlantic, and D-Flow FM was used to resolve the small-scale flow details close to the study area. In the nested configuration, the ADCIRC+SWAN model was run first to generate boundary conditions that are used to drive the D-Flow FM model. A stage boundary from ADCIRC+SWAN was applied on the southern boundary of the D-Flow FM model and two velocity boundaries from ADCIRC+SWAN were applied to the northern D-Flow FM boundaries on the East and Hudson Rivers. The boundary locations are shown in Figure 14.

The Project Team obtained most of the D-Flow FM model parameters from the ADCIRC+SWAN model since that model was validated in previous studies. Manning's roughness values and bathymetric data were taken from the ADCIRC+SWAN model. The ADCIRC+SWAN bathymetry was supplemented with available high-resolution bathymetry near the study area.

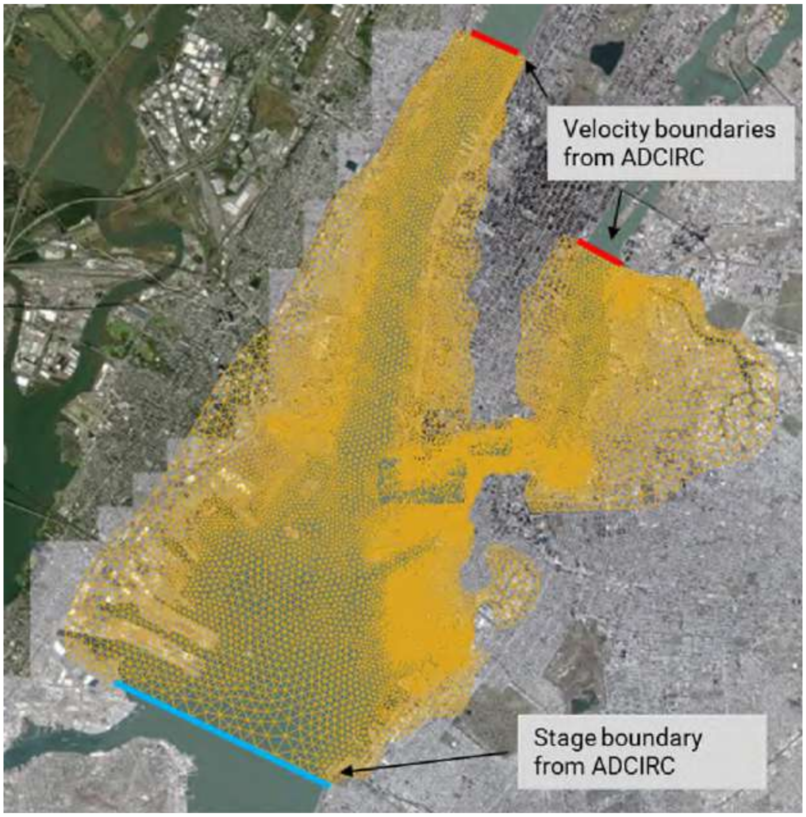

Figure 14 – Screenshot of D-Flow FM mesh with boundary condition configuration

4.3 Model Validation

To confirm that the D-Flow FM model accurately represented circulation patterns in the study area, the Project Team simulated an 18-day tidal cycle and compared the output to observed tidal elevations at the NOAA tidal gauge at the Battery and a USGS gauge at Pier 26 on the Hudson River. The Project Team used wind data from the North American Mesoscale Forecast System (NAM) to add wind forcing to both the ADCIRC and D-Flow FM models. Figure 16 is a plot showing the validation results at the Battery and Figure 17 shows the results at Pier 26. The D-Flow FM model (red dotted line) does an excellent job matching the observed data (solid black line) at both locations. Furthermore, the ADCIRC+SWAN (blue dotted line) and D-Flow FM output are almost identical, confirming that the boundary condition set-up is properly exchanging data from the ADCIRC+SWAN model to D-Flow FM.

Finally, the Project Team should note that while the D-Flow FM modeling framework was validated against observed stage data and performed well, the velocity output was not validated. The Project Team believes that this is acceptable for relative comparisons between project alternatives, but future studies should seek to validate the modeled velocity to observed data. This is particularly important if the output is to be used assess scour potential or evaluate potential ecological impacts. Furthermore, if parametric representations of flow around piers are required, they should be validated to observed velocity data. For this study, the Project Team's modeling approach assumes that pile-supported structures have a negligible impact on hydrodynamics.

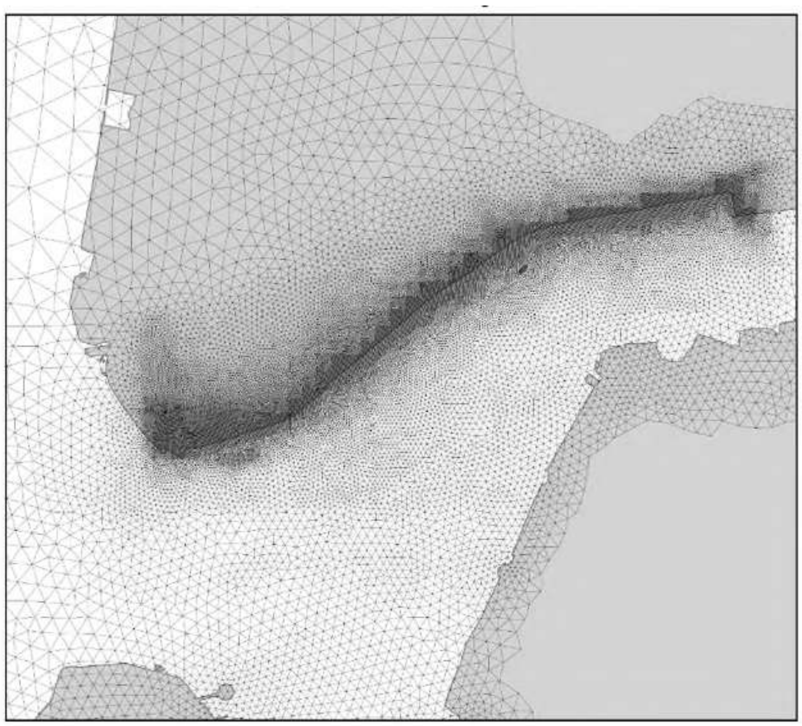

Figure 15 – Detailed view of mesh triangulation near the study area

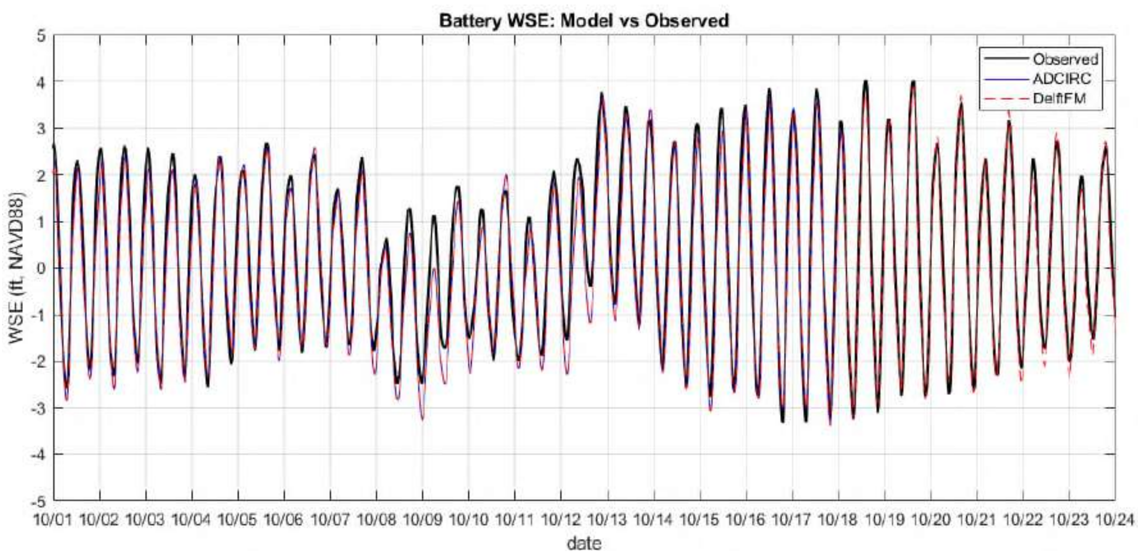

Figure 16 – D-Flow FM model validation results at NOAA tidal gauge at the Battery

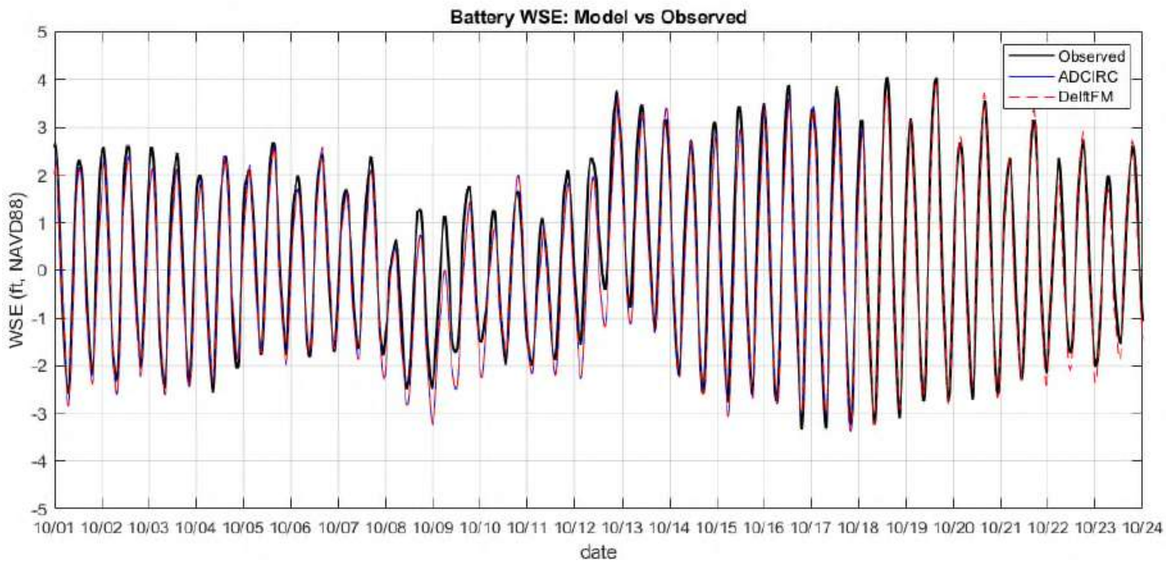

Figure 17- D-Flow FM model validation results at the Pier 26 USGS gauge on the Hudson River

4.4 D-Flow FM Baseline 2D Results

The Project Team used the same full-month tidal cycle for the D-Flow FM tidal impact analysis as the ADCIRC+SWAN modeling (Figure 6.) Figure 18 shows the present-day conditions D-Flow FM output showing the maximum depth-average tidal velocity observed over the course of the month-long tidal simulation. The D-Flow FM modeled velocity is similar to the ADCIRC output in terms of magnitude and spatial variability.

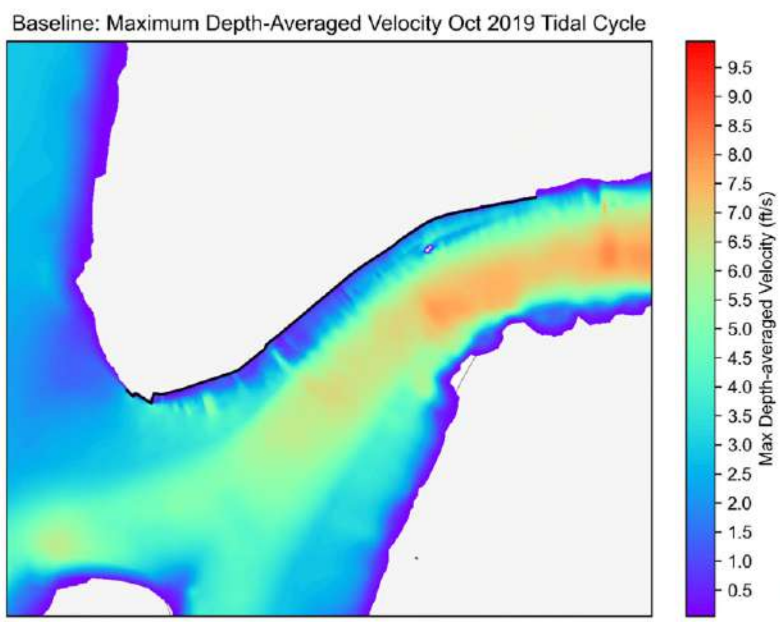

Figure 18 – D-Flow FM model output plot showing maximum depth-averaged tidal velocity over the full-month tidal cycle

4.5 2D D-Flow FM Tidal Impact Analysis

The Project Team used the D-Flow FM model to compare the relative tidal impacts of two refined project concepts: one was a narrower shoreline extension with less offshore footprint and the other was a wider shoreline extension with more offshore footprint. Figure 19 is a map showing the narrower shoreline extension concept, and Figure 20 shows the wider concept. The offshore footprint of the narrower shoreline extension concept varies from being entirely onshore at Whitehall Terminal, to 60 feet near the heliport, to 30 feet throughout the middle of the study area, to 150 feet near Pier 15, and back onshore near the Brooklyn Bridge. For the wider shoreline extension concept, the offshore footprint varies from 125 feet near Whitehall Terminal, 110 feet through the middle of the study area, to 150 feet near Pier 15, and then onshore near the Brooklyn Bridge.

Both concepts were run for the tidal-month simulation. Difference plots were made of the maximum depth-averaged velocity relative to present-day conditions so that the relative impacts of the shoreline extensions could be assessed. Figure 21 shows the velocity change for the narrow alignment concept and Figure 22 shows the velocity change for the wider velocity alignment. As expected, the narrow alignment has less of change on circulation patterns than the wider alternative. For the narrow alignment, there are some minor increases in velocity outboard of the filled areas as well as some areas where velocity decreases, as stagnation points develop upstream and downstream of the fill areas—particularly near Pier 17.

The wider alignment has a more pronounced impact on the maximum tidal velocity. Large stagnation areas develop upstream and downstream of the fill area at the Battery Maritime Building (BMB) – and offshore of the BMB fill area, velocities increased by as much 0.5 ft/sec (0.3 knots), although typical increases were on the order of 0.25 ft/sec (0.15 knots). These changes are relatively minor, especially compared to the peak tidal velocity in this area, which is on the order of 4-5 ft/sec (2.4-3 knots). However, additional analysis and regulator feedback is required to confirm that the changes are acceptable – this will be completed in future studies, as discussed in the future work section at the end of this document. The Project Team should note that the observed velocity changes are limited to the study area and do not have regional implications. Furthermore, there is little change in velocity near the Brooklyn Bridge.

Also, comparing the output for the narrow and wider conceptual alignments, the Project Team was able to observe the benefits of the increased model resolution provided by the D-Flow FM model. The differences between the two offshore footprints in terms of their hydrodynamic impact were readily apparent, including the impact of small-scale features in each alignment.

Lastly, the Project Team should also note that no local or regional increase in WSEL for either alignment was observed.

As the design advances into later stages, additional design, engineering, and hydrodynamic analysis is recommended to understand how potential changes in the configuration of the shoreline extension can minimize areas of stagnation and potential velocity increases near the BMB.

Figure 19 - Narrow shoreline extension concept alignment. Filled areas are highlighted in orange.

Figure 20 - Wider shoreline extension concept alignment. Filled areas are highlighted in orange.

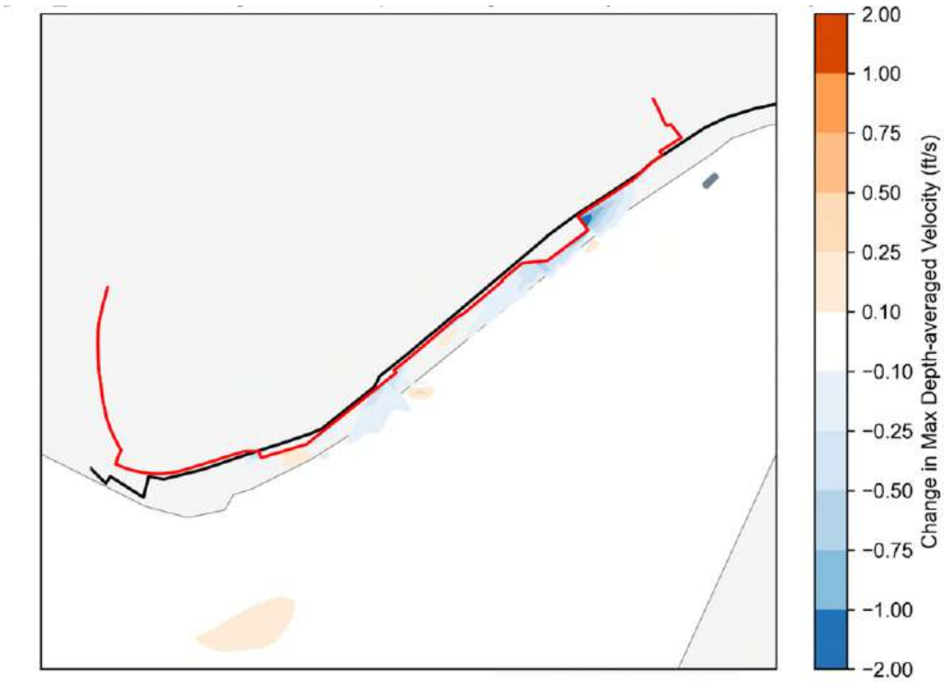

Figure 21 - Change in maximum depth average velocity with the narrow extension concept relative to present-day conditions. Warm colors show areas of velocity increase and cool colors show areas of decrease. The concept alignment is shown as a red line; the approximate bulkhead alignment is plotted in black.

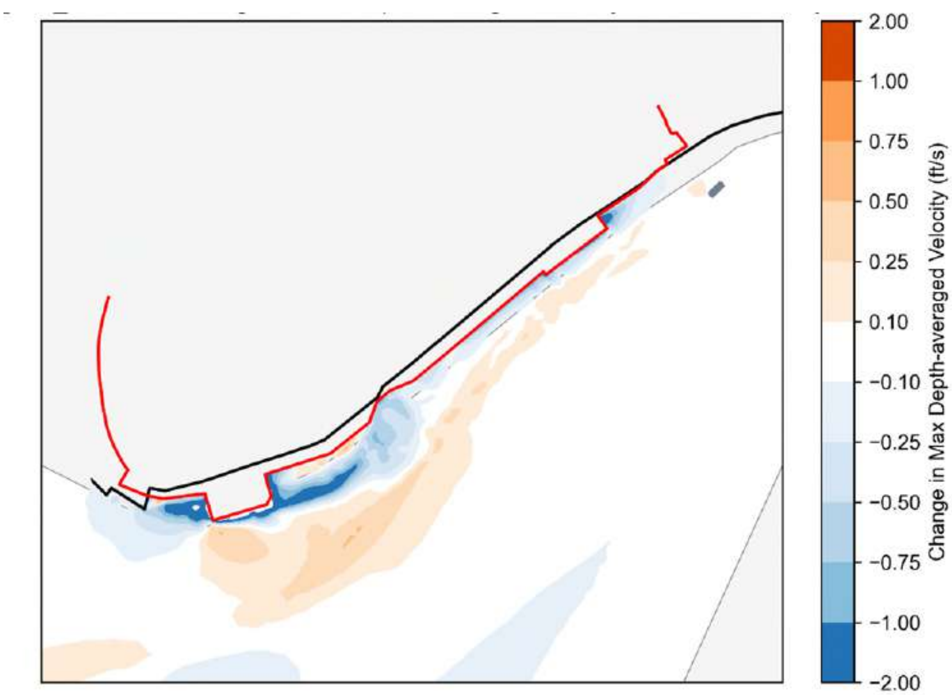

Figure 22 - Change in maximum depth average velocity with the wider extension concept relative to present-day conditions

4.6 Sensitivity Analysis near Brooklyn Bridge

After the initial comparison between the narrow and wider concepts, a sensitivity analysis was performed in the northern portion of the study area between Pier 17 and the Brooklyn Bridge. The Project Team wanted to better understand how the offshore footprint in this region affected tidal velocities near Brooklyn Bridge, given its criticality and historic importance, and to also ensure that the risk of scour would be minimized at the bridge foundation. To do so, the Project Team evaluated four extension alternatives: three uniform shoreline extensions (SLEs) of 30, 80 and 160 feet, and a tapered version of the 160-foot extension that started with a 160-foot width near Pier 17 and contracts to 80-foot, midway to the bridge. The 30-foot extension option is the same width as what was used in this area for the narrow and wider alternative analysis. The Project Team used the wider alternative alignment for the rest of the study area. The four alignments are shown below in Figure 23.

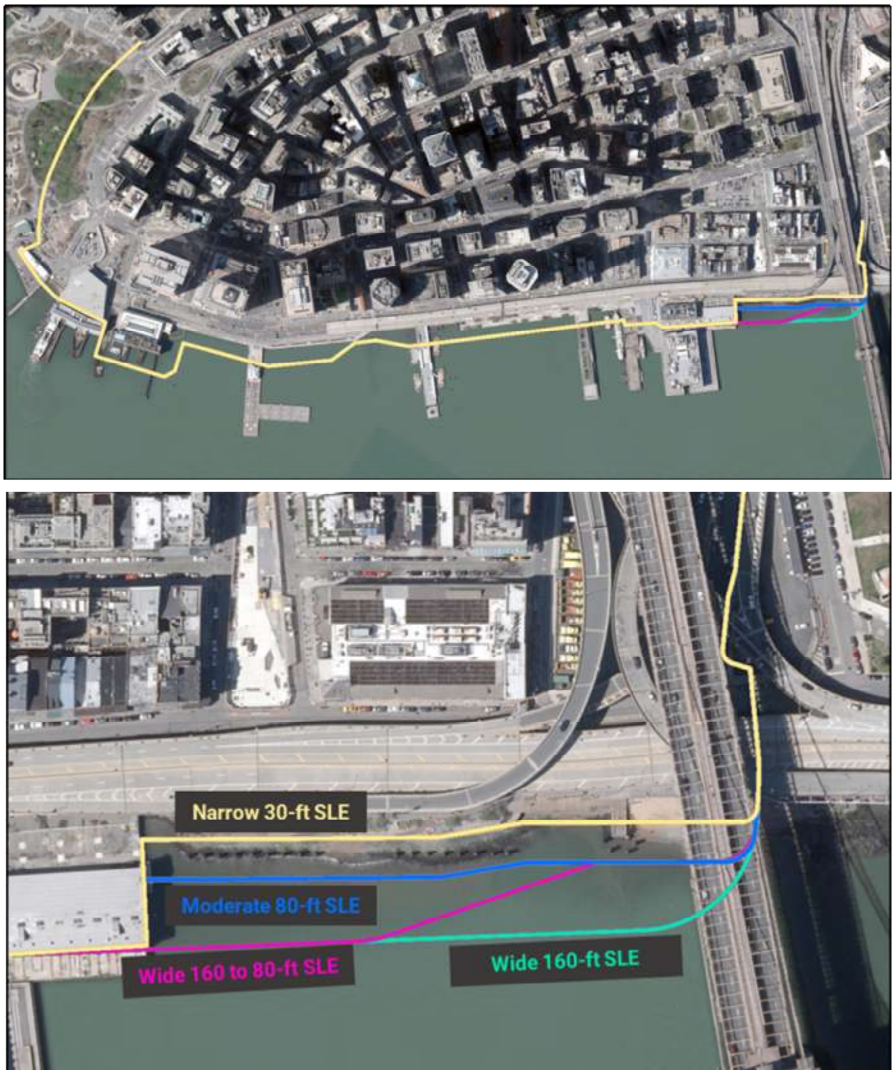

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Figure 23 – Map showing the four alignments modeled in the Reach D sensitivity analysis. The "Passive Flood defense Alignment 'A2' is equivalent to the wider alternative in the previous section.

The Project Team ran the four scenarios in D-Flow FM and then compared the computed velocities to present-day velocities. The Project Team used the same monthly tidal cycle as the other velocity comparisons for consistency. Comparing the 30- and 80-foot SLEs (Figure 24 and Figure 25), the Project Team observed minor changes in terms of the magnitude and extents of the velocity increase relative to present-day conditions. Comparing the 30- and 160-foot SLEs (Figure 24 and Figure 26), the Project Team observed a much larger velocity increase both in terms of the extent and magnitude of the changes. However, when the Project Team compared the 30-foot SLE with the tapered version of the 160-foot SLE, the velocity changes were lessened, suggesting that a tapered alternative would be preferrable near the Brooklyn Bridge from a hydrodynamics perspective if a smaller offshore footprint was not viable due to other project constraints.

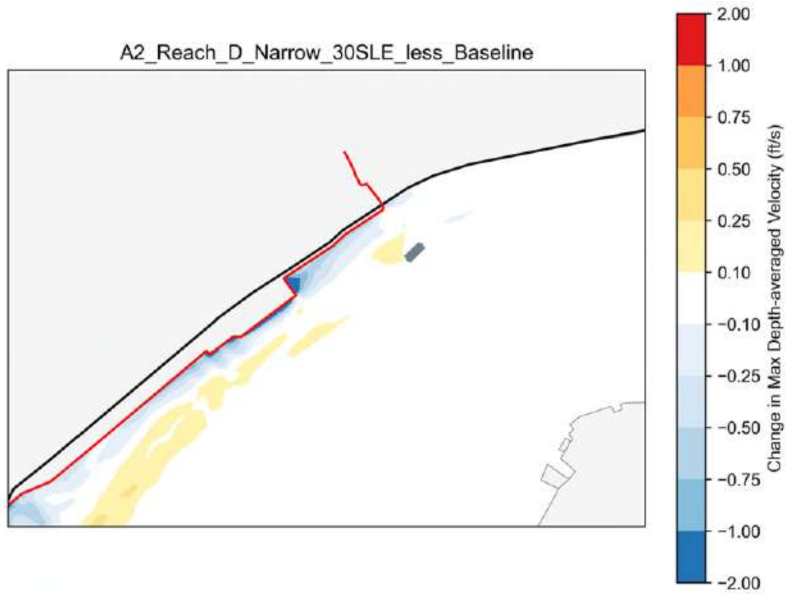

Figure 24 – Change in maximum depth averaged velocity from present-day conditions with the 30-ft shoreline extension concept

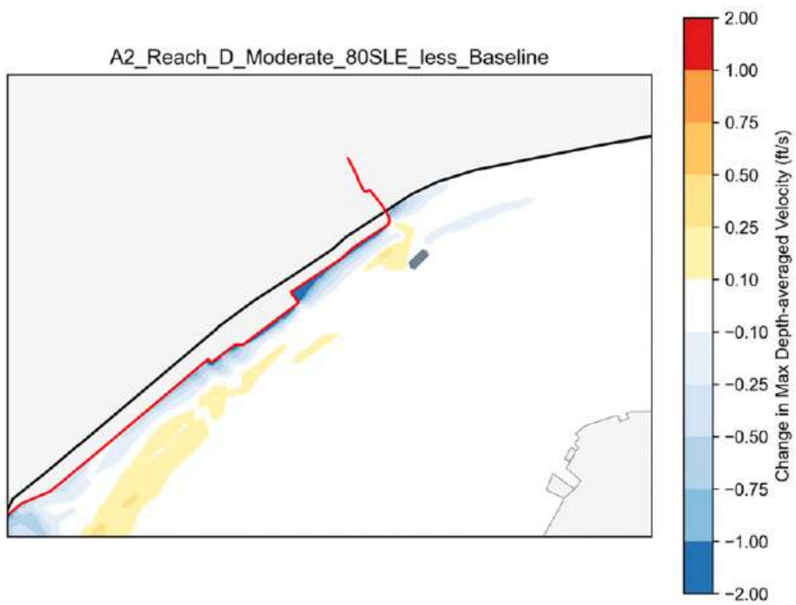

Figure 25 – Change in maximum depth-averaged velocity from present-day conditions with the 80-ft shoreline extension concept

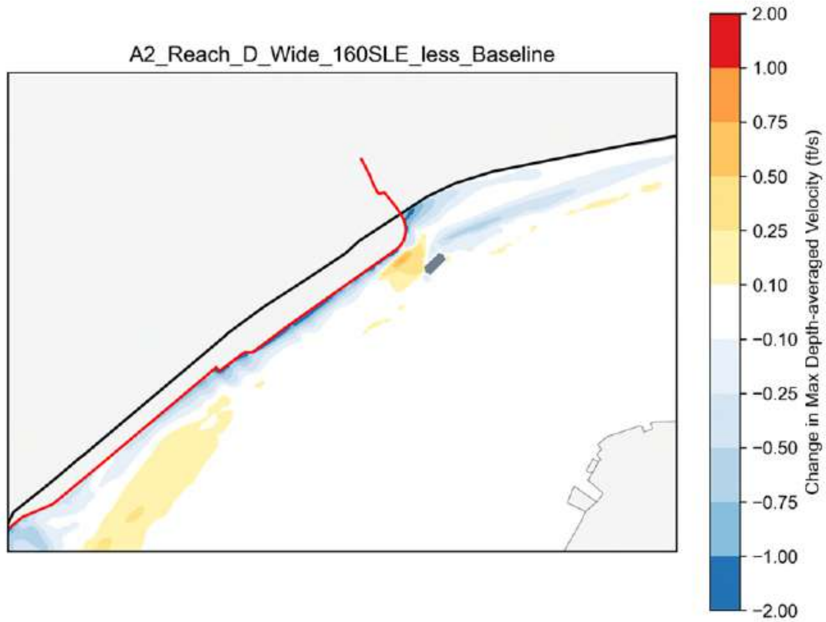

Figure 26 - Change in maximum depth-averaged velocity from present-day conditions with the 160-ft shoreline extension concept

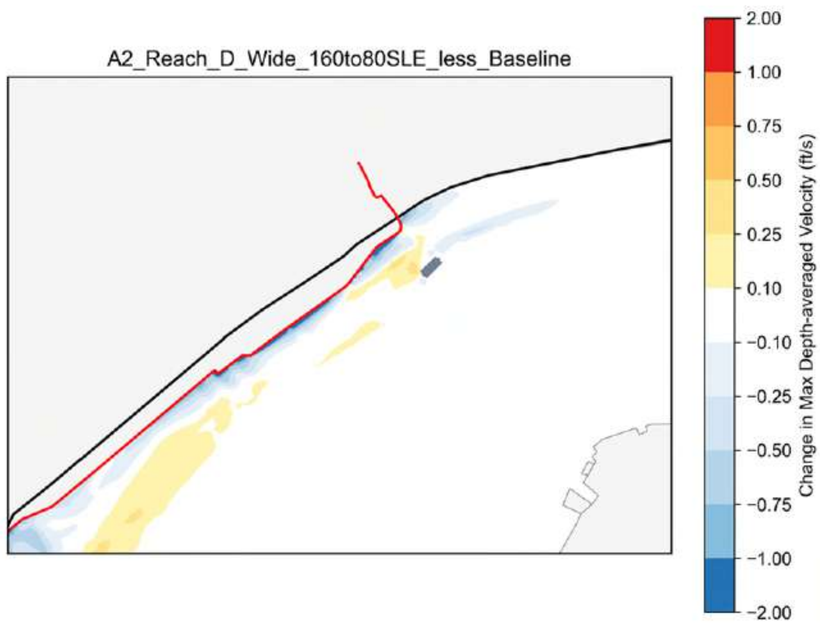

Figure 27 – Change in maximum depth-averaged velocity from present-day conditions with the tapered 160-ft shoreline extension concept

4.7 Key Findings

Based on the detailed hydrodynamic modeling using the high-resolution D-Flow FM model, the Project Team were able to draw the following conclusions:

- The D-Flow FM model accurately simulate tidal conditions in the study area, based on the Project Team's validation to observed tidal data.

- The additional resolution of the D-Flow FM model can resolve small-scale differences between flood defense alternatives and is a major improvement over the ADCIRC+SWAN model capabilities.
- The narrow and wider shoreline extension concept tidal velocity impact analysis demonstrated that both alternatives result in some localized increases in tidal velocities, and – as expected – the wider shoreline extension option did result in a larger extent and footprint of velocity change. However, the magnitude of the velocity increase for the wider alignment was not untenable and could be acceptable if other, non-hydrodynamic study constraints necessitated a larger offshore footprint. This would require additional analysis and feedback from maritime operators and regulators.
- No local or regional increase in WSEL was observed for the narrow and wider shoreline extension concepts.
- The sensitivity analysis near the Brooklyn Bridge confirmed that minimizing the outboard footprint in the northern portion of the study area reduces the velocity impact near the Brooklyn Bridge pier. However, there is only a small difference between the 30- and 80-foot SLEs, so there may be some outboard flexibility if a larger footprint is necessary.
- The 160-foot Reach D option causes too great of a velocity impact near the bridge. However, the tapered 160-foot alternative shows less of a velocity impact, showing that tapering is very affective and can allow for a larger offshore footprint near Pier 17 because it is the primary source of nearby velocity changes.

These results directly informed the refinement of the flood defense alignment that was ultimately recommended as part of the conceptual design of the Master Plan.

5. 3D Tidal Impact Analysis

After completing the 2D tidal velocity impact analysis, the Project Team completed a 3D analysis for the preferred project alignment that was developed by the rest of the Project Team. The preferred alignment incorporates the findings of the 2D tidal impact analysis and balances changes to tidal circulation patterns with constructability requirements as well as the protection goals of the project. Using a 3D model allowed the Project Team to see potential changes to circulation patterns throughout the water column, including at the water surface and the sea bottom. These regions are particularly important because changes to surface velocity could potentially affect vessel navigation and changes to bottom velocity – the speed of the tidal current at the seabed – have the possibility of causing scour/erosion or adversely impacting aquatic habitat.

5.1 3D Model Changes

D-Flow FM models can typically be run in both 2D and 3D mode with little change besides minor modifications to the model control file. However, to address model instability, the Project Team eliminated the velocity boundary on the Hudson River, extending D-Flow FM domain to the northern boundary of the ADCIRC+SWAN model. The Project Team used the ADCIRC+SWAN model bathymetry and Manning's roughness value in the extended mesh domain. The Project Team also compared the model results from the 2D model and the 3D model with this change to the mesh and the results were almost identical. This comparison ensures that the model is still performing to the same standard as when the Project Team validated it to observed data (see previous section). Figure 28 is a comparison of the 2D and 3D D-Flow FM domains.

The Project Team ran the 3D D-Flow FM model using a σ-grid topology with ten layers. A schematic view of the σ-grid layers is shown in Figure 29. The σ-grid topology divides the space between the ocean bottom and the free surface into a set number of layers. These layers adapt and change over time with changes to the free surface or the seabed.

Figure 28 - Comparison plot of 2D D-Flow FM model mesh (left) to the 3D D-Flow FM model mesh (right) that shows the Hudson River extension to the northern boundary of the ADCIRC+SWAN model domain

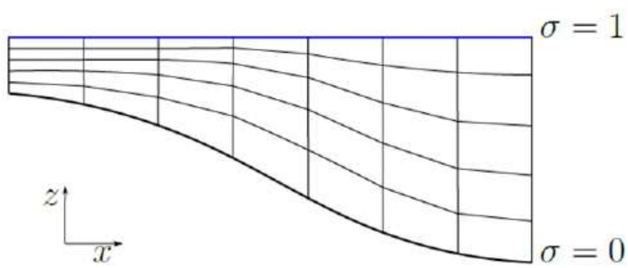

Figure 29 – Schematic view of the σ-layers in a 3D D-Flow FM model

5.2 3D Baseline 3D Results

The Project Team ran the 3D D-Flow FM model with the same monthly tidal cycle used for the other tidal modeling in this study. Figure 30 shows the maximum depth-averaged velocity (average across the ten σ-layers) for present-day conditions modeled over the whole tidal cycle, Figure 31 shows the maximum modeled surface velocity, and Figure 32 shows the maximum modeled bottom velocity. The maximum depth averaged velocity is almost identical to the 2D model output. The surface and bottom velocity output shows similar spatial variability as the depth-averaged results. As the Project Team expected, the surface velocities are higher than the depth-averaged values, and the bottom velocities are lower than the depth-averaged values.

Finally, as mentioned in the discussion of the 2D modeling, the Project Team was unable to perform a velocity validation for this study due to a lack of observed data in the study area. Using unvalidated results—while not ideal—is acceptable when comparing the relative impacts between in-water alignments in a preliminary assessment. Validation is recommended before the absolute modeled velocities can be used for design or regulatory compliance purposes. This is especially true for the 3D model output; modeled vertical velocity profiles will need to be validated in future studies against observed velocity profiles to ensure that the modeled profiles are accurate.

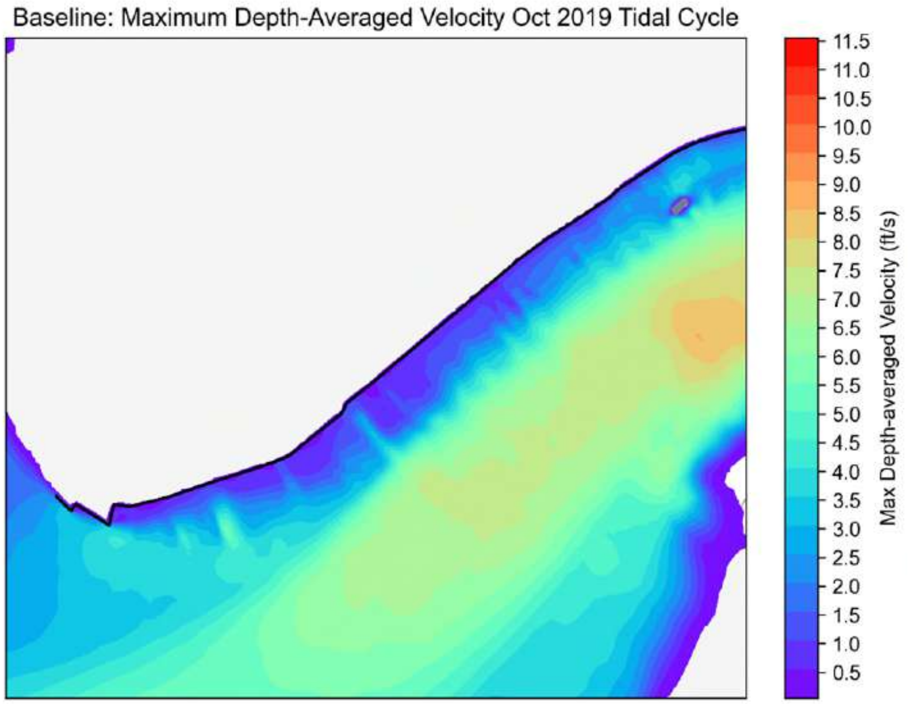

Figure 30 - D-Flow FM 3D model output plot showing maximum depth-averaged tidal velocity over the full-month tidal cycle

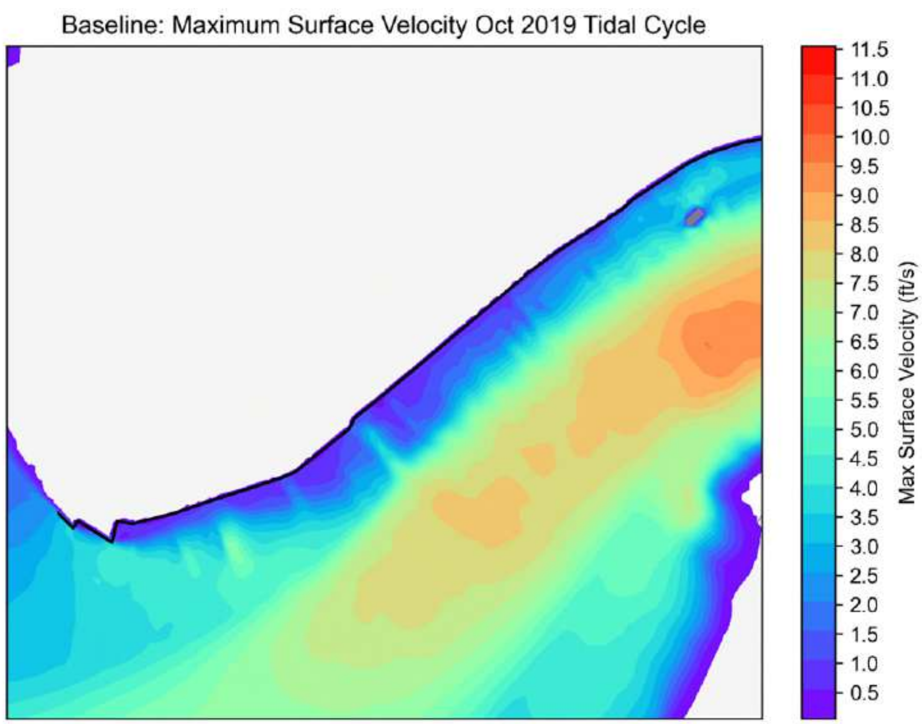

Figure 31 - D-Flow FM model output plot showing maximum surface velocity over the full-month tidal cycle

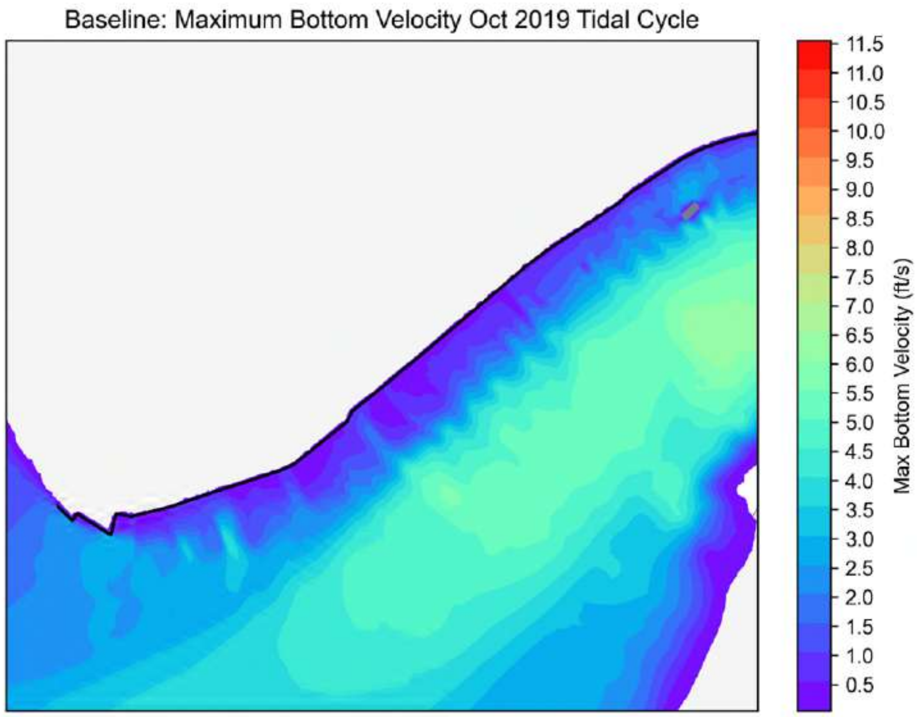

Figure 32 – D-Flow FM model output plot showing maximum bottom velocity over the full-month tidal cycle

5.3 3D D-Flow FM Tidal Impact Analysis

The Project Team then used the 3D D-Flow FM model to evaluate the tidal impacts of the preferred project alignment. As mentioned previously, the preferred alignment was developed using the information gained from the 2D modeling and was designed to accomplish the project objectives and accommodate the constraints of the study area—all while minimizing changes to circulation patterns. The preferred project alignment is shown below in Figure 34.

The preferred project alignment was run for the tidal-month simulation. Difference plots were made of the maximum surface, bottom, and depth-average velocity to show the alternative's impact on the tidal circulation. Figure 34 shows the change in maximum depth-average velocity, Figure 35 shows the change in maximum surface velocity, and Figure 36 shows the change in the maximum bottom velocity.

As expected by the Project Team, the preferred alignment changed tidal velocities in the study area. Similar to the wider shoreline extension concept in the 2D analysis, stagnation areas develop upstream and downstream of Whitehall Terminal and the Battery Maritime Building due to the in-water construction. Further offshore in this section of the study area, there is an area of minor velocity increase from present-day conditions. For the most part, the change in the depth-averaged velocity is on the order of 0.1 to 0.25 ft/sec (0.06 to 0.15 knots). However, there is a localized area where the change in velocity ranges from 0.25 to 0.75 ft/sec (0.15 to 0.44 knots). As with the 2D velocity results, the Project Team believes these changes are relatively minor, especially compared to the peak tidal velocity in this area, which is on the order of 4-5 ft/sec (2.4-3 knots). Additional analysis and stakeholder feedback will be required to confirm that the changes are acceptable – this will be completed in future studies, as discussed in the future work section at the end of this document.

To the northeast of Whitehall Terminal and the Battery Maritime Building, there is limited change from present-day conditions. In this sense, the preferred project results in this region are similar to narrow extension concept in the 2D analysis.

In general, a similar spatial pattern of change is observed for the surface and bottom velocity as the depth-averaged results. However, the surface velocity has a slightly higher change magnitude than the depth-averaged velocity, and the bottom velocity has a slightly lower magnitude of change than the depth-averaged velocity.

The velocity increases are confined to the offshore region near the preferred project alignment. Lastly, no local or regional increases in WSEL relative to present-day were observed with the preferred project alignment.

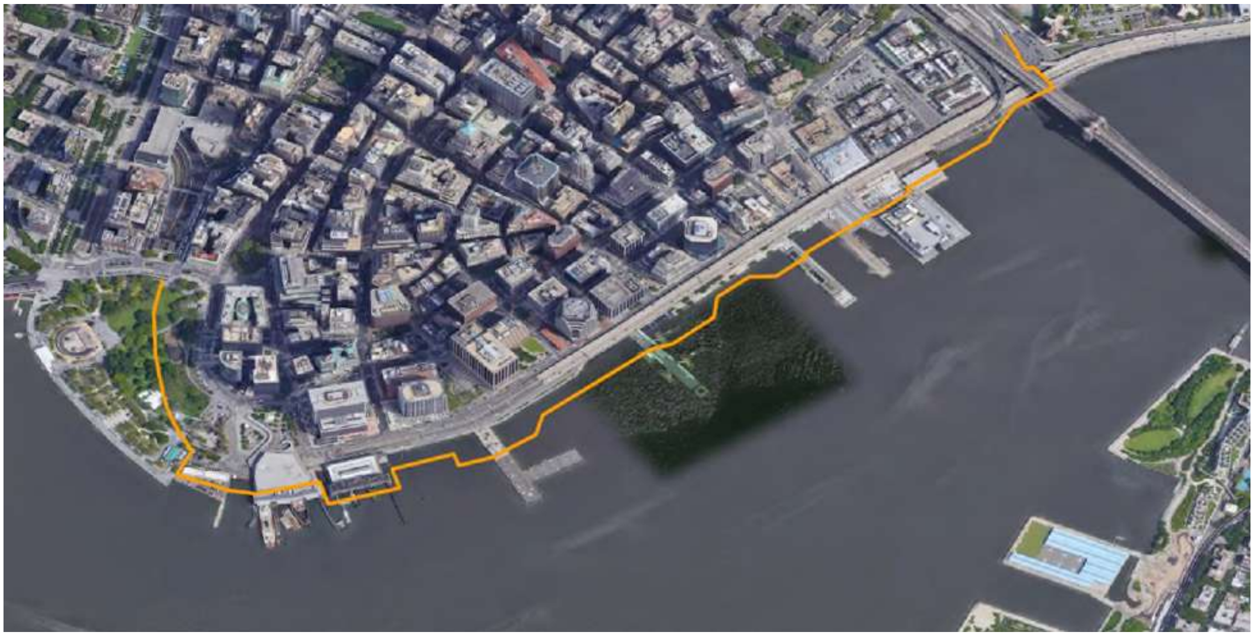

Figure 33 – Map showing the preferred project alignment

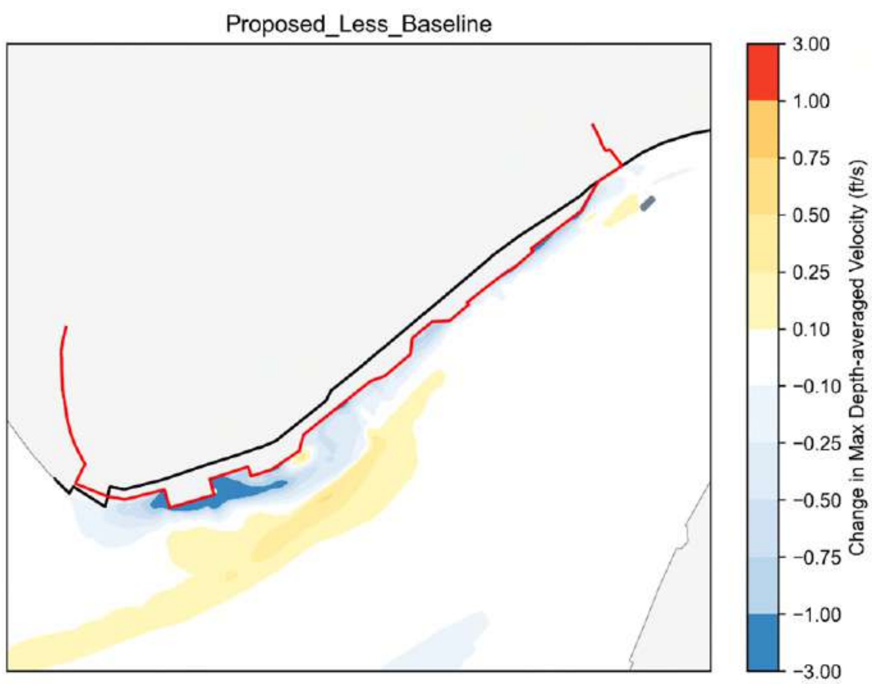

Figure 34 – Change in maximum depth-averaged velocity from present-day conditions with the preferred project alignment

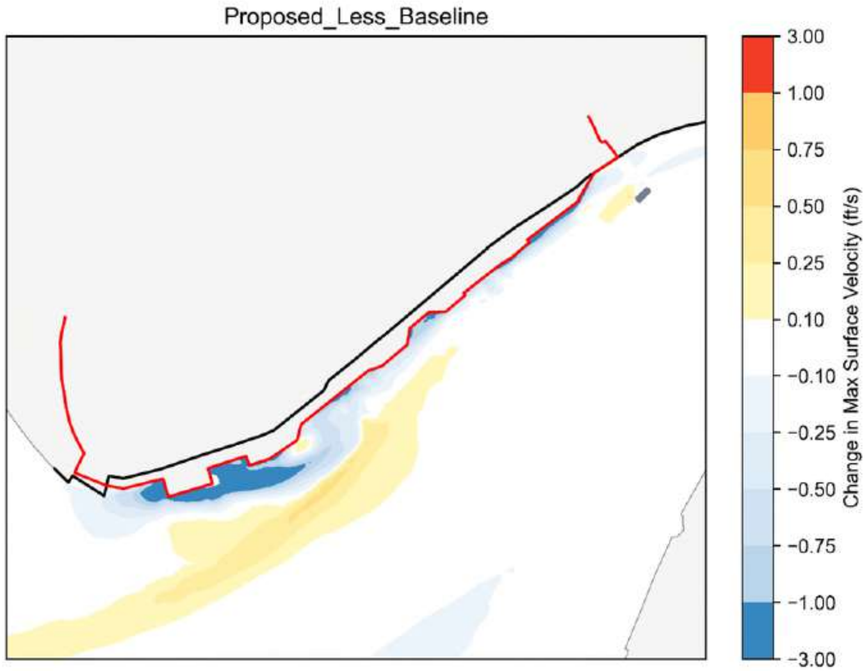

Figure 35 – Change in maximum surface velocity from present-day conditions with the preferred project alignment

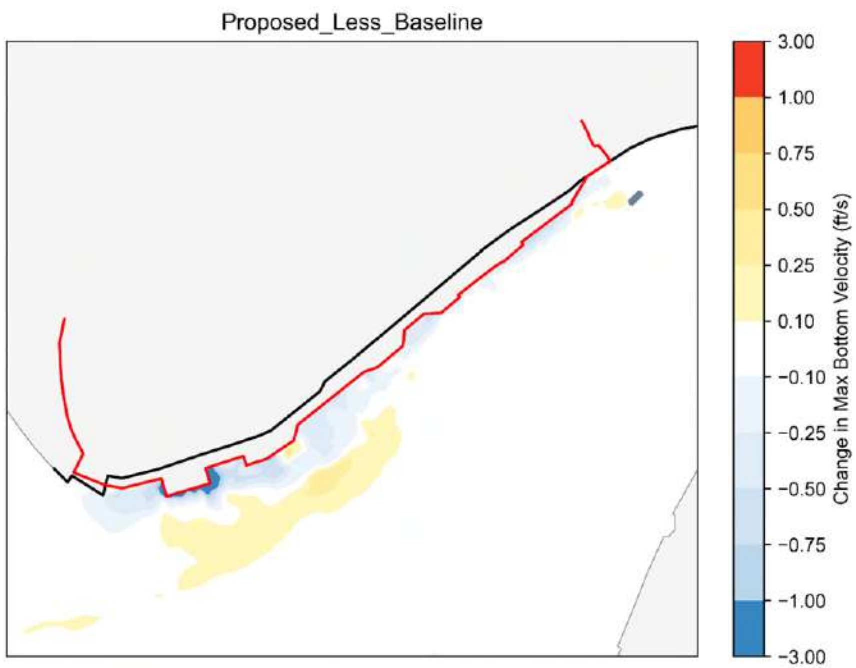

Figure 36 – Change in maximum bottom velocity from present-day conditions with the preferred project alignment

5.4 Key Findings

Based on the high-resolution, 3D modeling with the DFlow-FM model, the Project Team were able to draw the following conclusions:

- The maximum depth-averaged velocity computed using the 3D model is almost identical to the 2D depth-average model. The surface and bottom velocity output shows similar spatial variability as the depth-averaged results, with a higher magnitude for the surface velocity and a lower magnitude for the bottom velocity.
- As expected by the Project Team, the preferred project alignment results in some localized increases in tidal velocities. This was the case for the bottom, surface, and depth-averaged velocities. Additional hydrodynamic modeling and review by stakeholders/regulators will be needed in future phases in the project to confirm that any circulation changes caused by the flood defense alignment are tolerable. Furthermore, these preliminary findings will need to be confirmed in a model that has been validated to observed velocity profile data obtained in the immediate study area.
- No local or regional increase in WSEL was observed for the preferred project alignment.

6. Storm Surge Impact Analysis

The Project Team performed an impact analysis to determine the effects of the flood defense alignment on the 100-year storm tide elevations adjacent to and outside of preferred flood defense alignment (Figure 33), including all coastal areas of NYC. The Project Team simulated a representative 100-year storm event using the ADCIRC+SWAN model with and without the preferred flood defense alignment in place. An additional set of simulations were performed that considered the 100-year storm tide event with the 90th percentile SLR projection in 2100 to evaluate any potential impacts in the future.

6.1 Representative 100-Year Storm Tide Event

The Project Team used the same representative 100-year storm event as ESCR for consistency. The ESCR representative storm event was selected by reviewing the full suite of tropical and extratropical storm events used by FEMA to develop the NYC PFIRMs. Storm events that produced a peak storm tide close to the 100-year storm tide were tide were identified. Figure 37 shows storm tide hydrographs for all the FEMA PFIRM storms, two potential representative storm events (NJB_0003_010, NJA_0007_006) and a parametric fit used to represent the 100-year storm event using the USACE parametric methodology.[14] Out of the two potential representative 100-year storm events, the ESCR study selected NJB_0007_006 because it better fit the parametric hydrograph.

[14] USACE (US Army Corps of Engineers). Louisiana Coastal Protection and Restoration Final Technical Report Hydrology and Hydraulics Appendix. 2009.

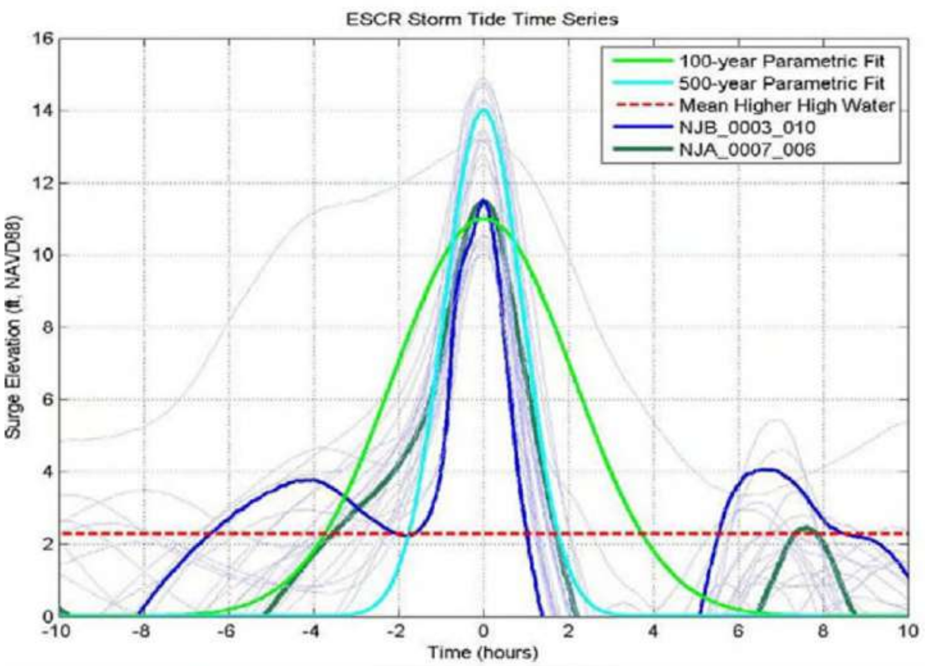

Figure 37 – Storm tide time series for all FEMA PFIRM events with the parametric fit time series and two potential 100-year storm tide events

6.2 Water Level Comparisons with Present-Day SLR Conditions

The Project Team simulated NBJ_0007_006 in the ADCIRC+SWAN model using the same mesh and parameters that were used for the concept-level tidal analysis. The model was run with and without the preferred alignment. Figure 38 shows the maximum SWEL computed by ADCIRC+SWAN for the storm with present-day SLR conditions without the preferred project alignment, and Figure 39 shows the maximum computed SWEL with the preferred project alignment under the same SLR condition. Figure 40 is a difference plot showing the change in computed SWEL with and without the preferred project alignment with present-day SLR. As shown in the figure, no increase in SWEL was observed in the immediate study area or the greater NYC region with the preferred project alignment relative to current conditions.

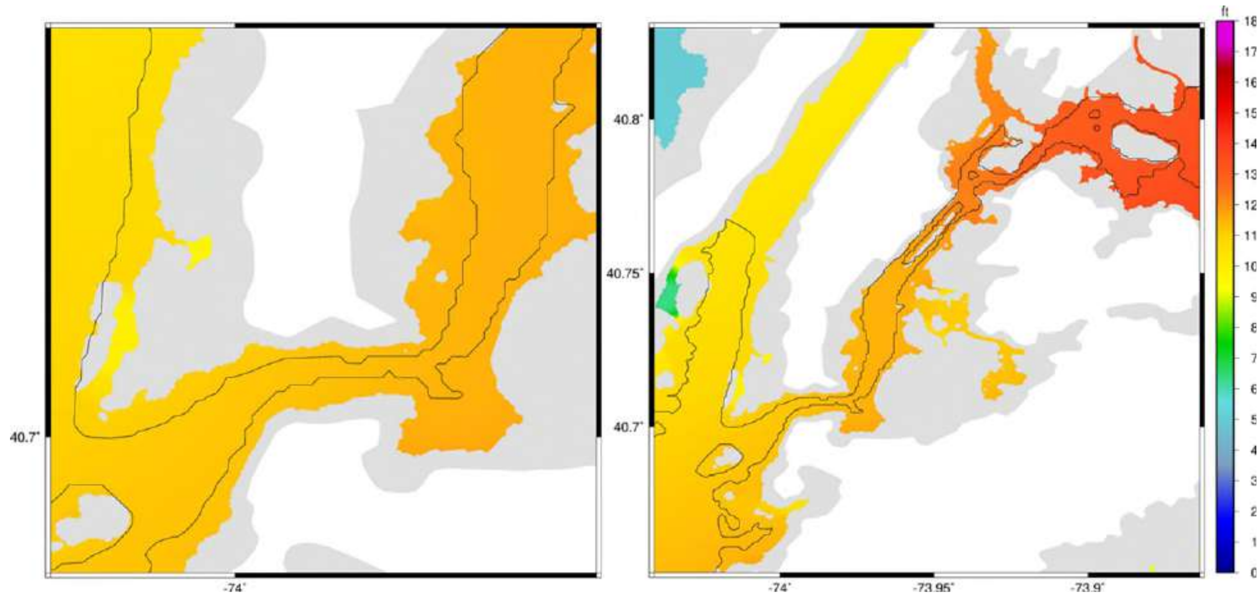

Figure 38 – Maximum SWEL computed by ADCIRC+SWAN for the 100-year storm tide event for present-day sea level conditions, without the proposed project. Left – zoomed into lower Manhattan. Right – broader view showing New York Bay, the Hudson River, and the East River.

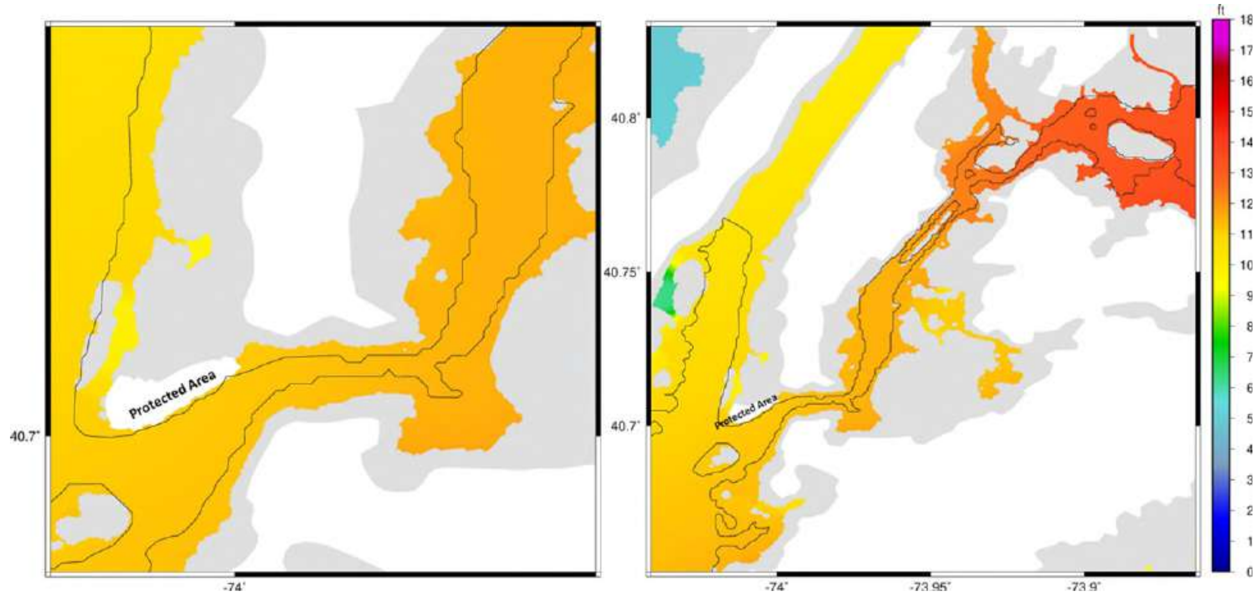

Figure 39 – Maximum SWEL computed by ADCIRC+SWAN for the 100-year storm tide event for present-day sea level conditions, with the proposed protection. Left – zoomed into lower Manhattan. Right – broader view showing New York Bay, the Hudson River, and the East River.

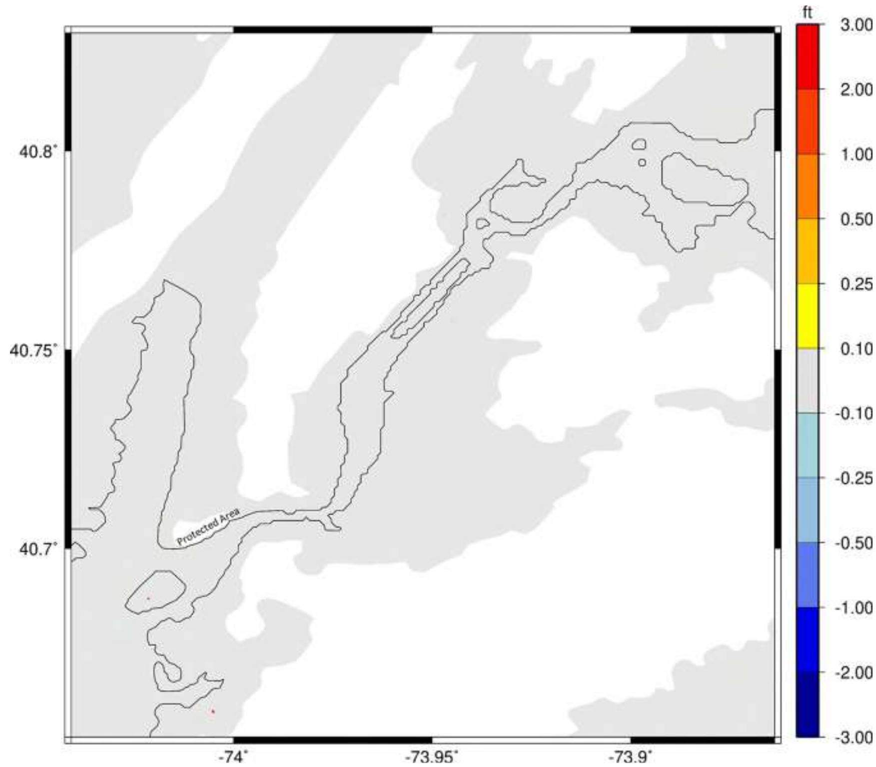

Figure 40 – SWEL difference plot comparing the SWEL computed by ADCIRC+SWAN with and without the proposed protection for present-day sea level conditions

6.3 Water Level Comparisons with 2100 SLR Conditions

The Project Team also modeled NJB_0007_006 with 2100 SLR conditions. 2100 SLR was represented in ADCIRC+SWAN by increasing the starting water surface by 6.25 feet from the present-day simulation. The 2100 SLR version of the model was run with and without the preferred alignment. Figure 41 shows the maximum SWEL computed by ADCIRC+SWAN for the storm with 2100 SLR conditions, and Figure 42 shows the maximum computed SWEL with the preferred project alignment under the same SLR conditions. Figure 43 is a difference plot showing the change in computed SWEL with and without the preferred project alignment with 2100 SLR. As shown in the figure, no increase in SWEL was observed in the immediate study area or the greater NYC region with the preferred project alignment relative to existing conditions.

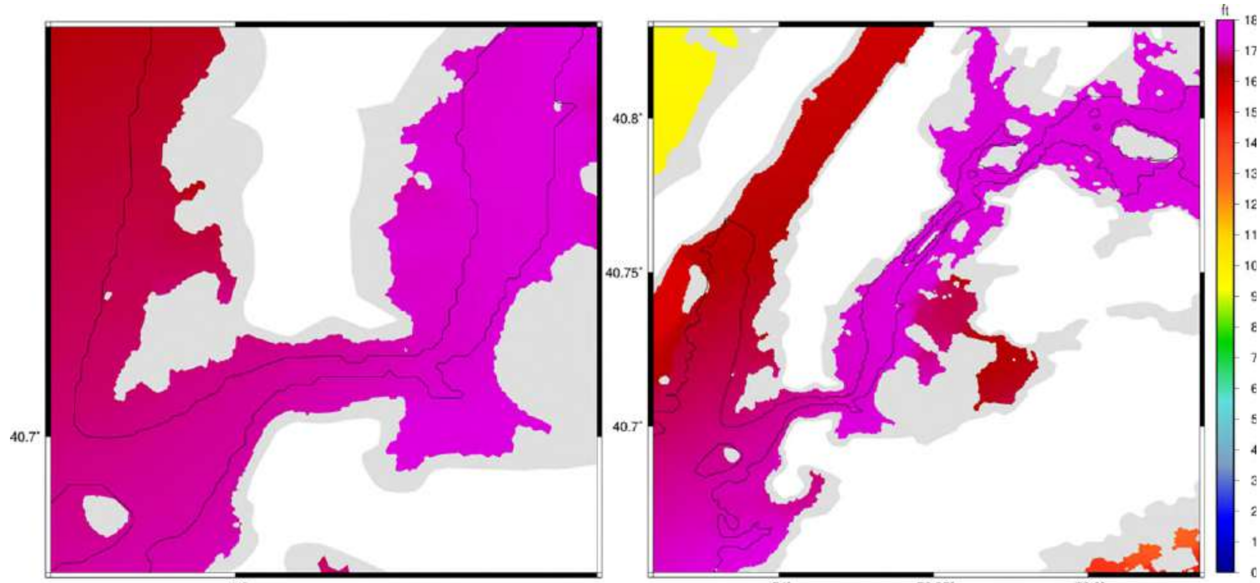

Figure 41 – Maximum SWEL computed by ADCIRC+SWAN for the 100-year storm tide event with 2100 sea level conditions, without the proposed project. Left – zoomed into lower Manhattan. Right – broader view showing New York Bay, the Hudson River, and the East River.

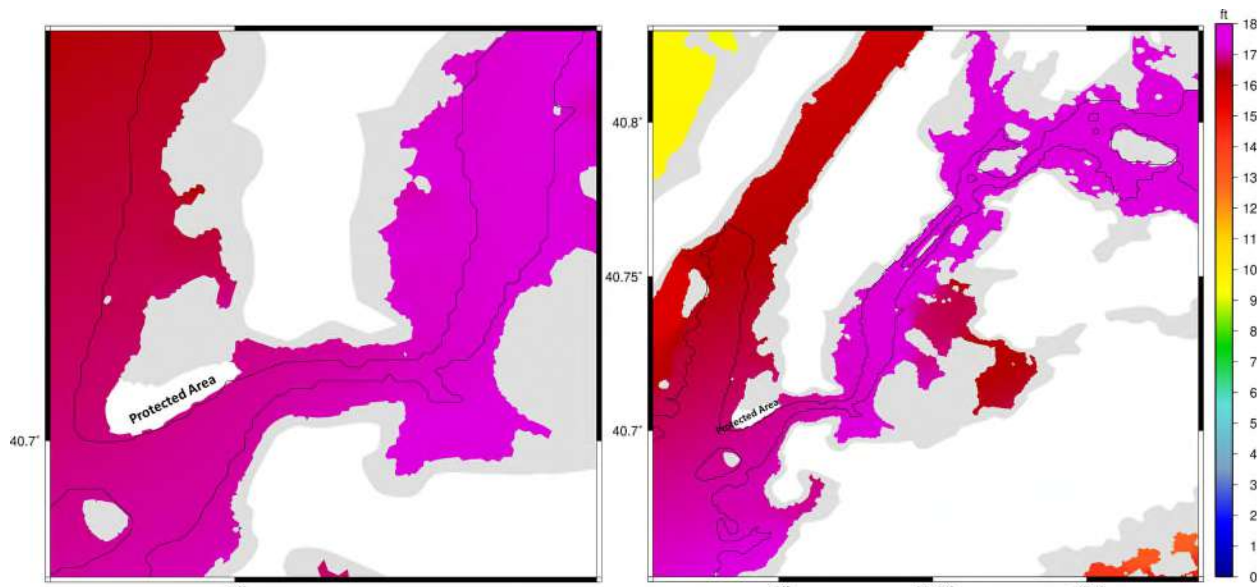

Figure 42 – Maximum SWEL computed by ADCIRC+SWAN for the 100-year storm tide event with 2100 sea level conditions, with the proposed protection. Left – zoomed into lower Manhattan. Right – broader view showing New York Bay, the Hudson River, and the East River.

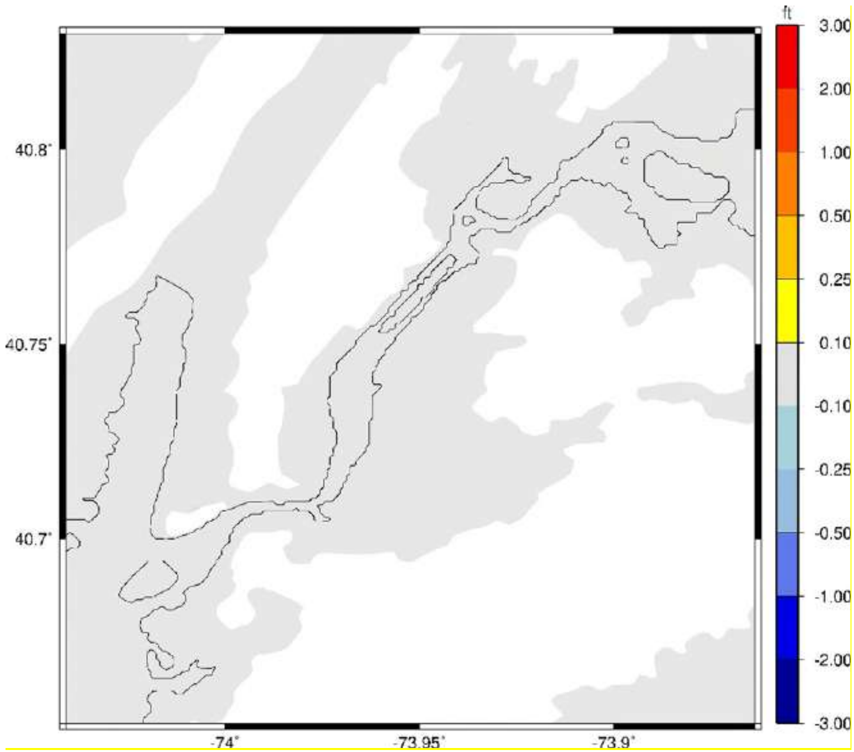

Figure 43 – SWEL difference plot comparing the SWEL computed by ADCIRC+SWAN with and without the proposed protection for 2100 sea level conditions

6.4 Key Findings

Based on the findings of the storm-surge impact analysis, no increase in local or regional storm surge stillwater elevation is expected for the 100-year storm with the preferred project alignment relative to existing conditions, for both present-day and future (2100) SLR conditions.

7. General Conclusions and Future Work

7.1 General Conclusions & Future Work

Several phases of hydrodynamic modeling were performed by the Project Team while developing the Master Plan. Modeling was an integral part of development of the Master Plan, informing every phase of its development, including determining project design flood elevation, studying simple shoreline extension geometries' impacts on tidal circulation patterns, evaluating tidal circulation with more complex project options using a high-resolution model in 2D and 3D, and performing a hydraulic impact analysis for large storm events. Through this effort, the Project Team was able to draw the following general conclusions:

- The proposed project is not likely result in an increase in tidal WSELs within the study area or regionally. No increase in WSEL was observed even with the most extreme, 500-ft concept-level shoreline typology (Section 3).
- The modeled changes in depth-averaged tidal velocity with the preferred project alignment results in some localized increases in tidal velocities. These changes are only found in the immediate study area and were not observed on a regional basis. The observed increases in tidal velocity are relatively minor, especially compared to the relatively high tidal velocities observed in the study area. However, additional analysis and stakeholder feedback will be required to confirm that the changes are acceptable – this will be completed as part of future studies (Section 5).
- A similar spatial pattern of change was observed by the Project Team for the surface and bottom velocity as the depth-averaged velocity; however, the magnitude of change was greater for the surface velocity (Section 5).
- The storm surge impact analysis showed that no increase in local or regional storm surge stillwater elevation is expected with the preferred project alignment relative to existing conditions for both present-day and future (2100) SLR conditions (Section 6).

While the Project Team made significant progress in understanding the potential hydrodynamic impact of the Master Plan, there are still several items that should be evaluated in future studies. The Project Team suggests the following for future work:

- The D-Flow FM model should be developed further, including using updated bathymetry data in the immediate study region and validating the model to observed velocity data. The velocity validation should include comparisons to observed vertical velocity profiles to confirm that the modeled surface and bottom velocities area accurate.
- The D-Flow FM model should also be updated with a better representation of pile-supported structures for both present-day conditions and proposed alternatives. The model's representation of flow around piles should be calibrated and validated to observed velocity data near existing pile-supported structures in the study area.
- The updated model should be used to re-assess the hydrodynamic impacts of the preferred project alignment and any future alignments, including additional study in the vicinity of Whitehall Ferry Terminal and the Battery Maritime Building.
- Further coordination is needed with project stakeholders across navigation and regulatory considerations to confirm an acceptable level of tidal velocity increase.

The Project Team notes that this is just a subset of the likely future work that will be required. Much depends on the feedback from project stakeholders and will be determined as the project advances forward.