NEW YORK-NEW JERSEY HARBOR AND TRIBUTARIES NEW YORK DISTRICT

Interim Report Engineering Analyses

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US Army Corps of Engineers

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New York District

New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study

Overview of Engineering Work for the NYNJHATS Interim Report

February 2019

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1 OVERVIEW OF ENGINEERING WORK FOR THE NYNJHATS INTERIM REPORT

USACE is conducting a feasibility level study to evaluate a coastal storm damage risk reduction plan in the New York/New Jersey Harbor & Tributaries Area.

The study area encompasses the New York Metropolitan Area, including the most populous and densely populated city in the United States, and the six most populated cities in New Jersey. The shorelines of some of the NYNJHAT study area are characterized by low elevation areas, developed with residential and commercial infrastructure and are subject to tidal flooding during storms. The study area covers more than 2,150 square miles and comprises parts of 25 counties in New Jersey and New York.

During coastal storms, storm surges are generated on the open coast and propagate through New York Harbor or through the Long Island Sound and flood the extensive low-lying areas surrounding the metropolitan area. The feasibility study includes 5 alternatives to function as flood risk reduction systems, the Alternatives are listed in Table A. Each alternative consists out of various CSRM Measures (both structural and non-structural (TBD)).

Table A: The five (5) plan alternatives as summarized in the following table:

Alt.	Areas Benefited by Alternative	Areas Not Benefitting from Alternative	Features/Measures	Main Environmental Concerns	Notes
1	None	Entire HAT study area	None	Entire HAT study area remains as vulnerable as it will be with the currently ongoing efforts to coastal flooding impacts.	Assumes all ongoing studies/projects by Corps and funded efforts by others (e.g., RBD) are implemented to extent currently considered feasible/actionable.
2	Nearly all of the HAT study area	Part of Bronx & Westchester Counties eastern shore	SHBP Barrier: TN Barrier:	Tidal exchange in HRE, migration of all estuary resources to Bight and LIS, etc.	Fewest “moving parts”, areas of possible failure.
3.a	Much of the HAT study area	Shorelines around Raritan, Sandy Hook, Lower Bay, in addition to the areas identified in Alt. 2	AK Barrier: VN Barrier: TN Barrier: Jamaica Bay Barrier:	Tidal exchange in Hudson estuary, migration of all estuary resources to Bight and LIS, etc.	Only relatively higher risk areas in NY and NJ in Lower and Jamaica Bay have measures. Few “moving parts”.
3.b	Inland NJ areas (incl. port, oil terminals and Newark airport) and backside of SI by barrier, high risk areas of NJ & upstate NY along HR & NYC	Segments of NY (incl. NYC) and NJ (along HR) that do not have high risk/exposure	AK Barrier: KVK Barrier: Jamaica Bay Barrier: NJ Shore along HR: NYC West Side/East Harlem: Newtown: Gowanus: Upstate HR Towns:	Tidal exchange in Kills/Newark Bay, migration of all estuary resources to Newark Bay. Cultural impacts from perimeter measures in NJ along HR and NYC.	Only relatively higher risk areas in NY (incl. NYC) and NJ (along HR) have measures. Major port facilities (incl. oil terminals, etc.), Newark and LaGuardia airports still at risk.

Alt.	Areas Benefited by Alternative	Areas Not Benefitting from Alternative	Features/Measures	Main Environmental Concerns	Notes
4	Only relatively higher risk sections of shoreline or smaller tributary basins in HAT study area.	Relatively moderate and low risk areas.	Hack R Barrier NJ Shore along HR: NYC West Side/East Harlem: Newtown: Gowanus: Upstate HR Towns: Jamaica Bay Barrier:	Tidal exchange in Hackensack River, Gowanus and Newtown Creek. Cultural impacts from perimeter measures in NJ along HR and NYC.	Only relatively higher risk areas in NY (incl. NYC) and NJ have measures. Major port facilities (incl. oil terminals, etc.), Newark and LaGuardia airports still at risk. Many "moving parts" and most of shoreline left without any risk mgmt. measures.
5	Only relatively higher risk sections of shoreline or smaller tributary basins in HAT study area.	Relatively moderate and low risk areas.	Hack R Perimeter: NJ Shore along HR: NYC West Side/East Harlem: Newtown non-struct &/or perimeter: Gowanus non-struct &/or perimeter: Upstate HR Towns:	Cultural impacts from perimeter measures in NJ, upstate Hudson in NY, and NYC.	Only relatively higher risk areas in NY (incl. NYC) and NJ have measures. Major port facilities (incl. oil terminals, etc.), Newark and LaGuardia airports still at risk. Many "moving parts" and most of shoreline left without any risk mgmt. measures.

SHBP – Sandy Hook, NJ-Breezy Point, NY;
TN – Throgs Neck; AK – Arthur Kill; VN – Verrazano Narrows; KVK – Kill Van Kull; HR – Hudson River

A key component of the Study consists out of the task to prepare conceptual layout of various coastal storm risk management measures. Structural measures such as, levees, floodwalls, etc. and non-structural (TBD) and natural and nature-based features (TBD) are included in the initial focused array of alternatives. In addition, storm surge barriers are included within the focused array of alternatives. The purpose of the storm surge barriers is to be an integral part of the coastal storm risk management strategy of each alternative and impede storm surge and reduce the risk of flooding for the area behind it.

The engineering studies to support the development of the alternatives can be split out into a number tasks (additional task may be needed)

- Conceptual designs for structural CSRM measures
  - Conceptual design of perimeter flood risk reduction measures (shore-based measures)
  - Conceptual designs of storm surge barriers
- Numerical Modeling, and
- Cost Engineering

Additional detail on these tasks is provided below.

1.1 Conceptual Designs for Structural CSRM Measures

As part of the New York/New Jersey Harbor & Tributaries Focus Area Feasibility Study (HATS) several structural alternatives were proposed and evaluated in an effort to protect the region from storm surge induced flooding. The alternative designs include various combinations of storm surge barriers around the harbor that would close when high water levels from storm events are expected as well as perimeter flood risk reduction measures.

1.1.1 Conceptual Design of Perimeter Flood Risk Reduction Measures

For each of the alternatives a plan description (see Table A), general plan view layouts and typical sections for each component of the line of flood risk reduction was developed. Layouts were prepared in GIS format with bathymetry and/or topography contours from readily available digital sources top elevations for the flood risk reduction alignment were calculated and the type of measure (levee, floodwall, etc.) in each reach were assigned. As part of the study typical cross-sections and were developed for each type of protection measure. Typical sections were developed based on representative ground elevations, approximate top elevations, and typical design concepts for similar structures in previous USACE projects and studies. Coastal, structural, civil, or geotechnical engineering analyses were only minimally performed when existing databases/tools allowed for it.

Two overtopping requirements were used to determine the structural elevations for the SBMs. In general, for the conceptual development and design of the SBMs, an overtopping criterion of 1.08 x 10-2 cubic foot per second per foot (cfs per foot) or 1 liter per second per meter (L/s/m) is set to determine the structure height. This criterion is applied at the end of the project service life. The required freeboard for the given overtopping criteria was developed using the design 1% AEP (100-year) SWL (with intermediate rate Sea Level Change) and 1% AEP (100-year) waves, were used. In addition, a resiliency check was included where the design elevation should be set such that it is equal to or above the 0.2% AEP (500-year) SWL (with intermediate rate Sea Level Change) (this is just a resiliency check to assure that continuous weir flow would not create an even greater risk for a slightly more severe storm). It is recognized that further refinement is required as the design progresses, yet the criteria provide for a systematic equal approach throughout the Study Area and are similar to that used for the HSDRRS in New Orleans.

The goal of this task was to develop generalized SBMs that are comprehensive enough to be applicable and suitable for the entire study area yet not too detailed or site specific such that they could only be applied at one location. The development of the conceptual designs of the SBMs allows for quantity take-offs. Based on the alignments developed by USACE for the HAT Study Alternatives, the following SBMs were developed and conceptual designed were presented:

1. Floodwalls
2. Levees
3. Promenades with Flood Risk Reduction Function
4. Seawall,

5. Composite Seawall with Beach and Dune,
6. Deployable Flood Barriers

The completion of the conceptual designs allowed for an inventory of shoreline based measures (total length and number of measures) per HAT Study Alternative and for an estimated quantity take-off per SBM. This data set was then used by USACE-NAN as input to compare cost estimates for the study alternatives and allow for a relative comparison with parametric cost of the study alternatives.

1.1.2 Conceptual Crest Elevation Design for Surge Barriers and Shore-Based Measures

Wave overtopping may lead to failure of the crest and landward side of a protective structure. The damage may be shown during a single event, or it may be cumulative. Other damage mechanisms include wave erosion/scour induced undermining, wave forces on the structure face. Those latter damage mechanisms can be addressed during the later design phases. But the crest elevation is of primary concern at this stage of the study, as it affects public concern over lost view-shed. So wave overtopping was used to determine the preliminary crest elevations of the vertical surge barriers, the on land vertical structures (e.g., concrete seawalls), and the on land trapezoidal structures (e.g. rubble seawalls). The on land barriers can represent tie-back to prevent flanking at the surge barriers, or a line of protection of themselves. The design storm water surface elevation is the expected value for the 100-year in the year 2080.

Sea Level Change include SLC Gages

The crest elevation is intended to prevent damaging overtopping for a design event for the next 50 years at least. This means that any sea level change between the date the water surface elevation represents and 2080. The Center for Operational Oceanographic Products and Services (CO-OPS) operated three long term gages in our area of interest-Sandy Hook, The Battery, and Kings Point. The locations of these gages are shown in Figure 1. The most appropriate gage for each of the barrier locations was selected. USACE Sea Level Change Curve Calculator was used to estimate the intermediate curve of sea level change for each gage. The resulting sea level changes up till the year 2100 are shown in Table 1.

Design Impacts from Sea Level Change

Future Sea Level Changes are projections. No assumptions are made as to the likelihood of one rate over another rate occurring. A conservative decision was made by assuming an Intermediate Rate. Should a rate lower than the Intermediate occur, the proposed design would be sufficient. Should a rate higher than the Intermediate occur, the proposed design includes capability to be elevated and augmented in the future (at the time where the High Rate surpasses the Intermediate Rate).

Design Impacts of Sea Level Change

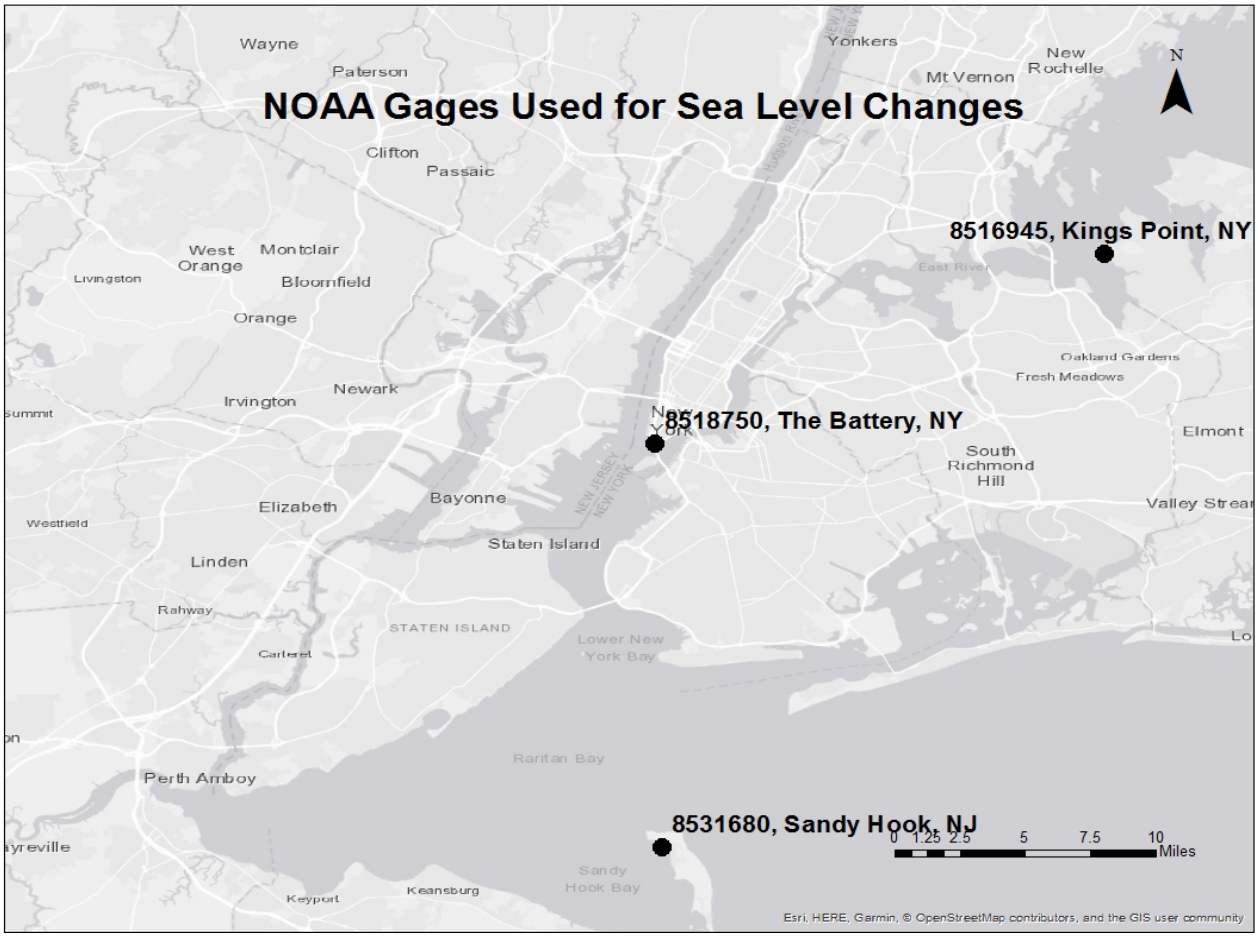


Figure 1: NOAA Gage Locations

Table 1: Estimated Relative Sea Level Change in Feet

	Estimated Relative Sea Level Change in ft.		
	8531680, Sandy Hook, NJ	8518750, The Battery, NY	8516945, Kings Point, NY
	NOAA's 2006 Published Rate: 0.01280 feet/yr	NOAA's 2006 Published Rate: 0.00909 feet/yr	NOAA's 2006 Published Rate: 0.00771 feet/yr
Year	USACE Intermediate Rate		
1992	0	0	0
1994	0.03	0.02	0.02
1996	0.05	0.04	0.03
1998	0.08	0.06	0.05
2000	0.11	0.08	0.07
2002	0.14	0.1	0.09
2004	0.17	0.12	0.11
2006	0.2	0.14	0.13
2008	0.23	0.17	0.15
2010	0.26	0.19	0.17
2012	0.29	0.22	0.19
2014	0.33	0.24	0.21
2016	0.36	0.27	0.24
2018	0.39	0.3	0.26
2020	0.43	0.32	0.29
2022	0.46	0.35	0.31
2024	0.5	0.38	0.34
2026	0.54	0.41	0.37
2028	0.58	0.44	0.39
2030	0.62	0.47	0.42
2032	0.65	0.51	0.45
2034	0.69	0.54	0.48
2036	0.74	0.57	0.51
2038	0.78	0.61	0.54
2040	0.82	0.64	0.57
2042	0.86	0.68	0.61
2044	0.91	0.71	0.64
2046	0.95	0.75	0.68
2048	1	0.79	0.71
2050	1.04	0.83	0.75
2052	1.09	0.87	0.78
2054	1.14	0.91	0.82
2056	1.18	0.95	0.86
2058	1.23	0.99	0.9
2060	1.28	1.03	0.94
2062	1.33	1.07	0.98
2064	1.38	1.12	1.02
2066	1.43	1.16	1.06
2068	1.49	1.2	1.1
2070	1.54	1.25	1.14
2072	1.59	1.3	1.19
2074	1.65	1.34	1.23
2076	1.7	1.39	1.27
2078	1.76	1.44	1.32
2080	1.82	1.49	1.37
2082	1.87	1.54	1.41
2084	1.93	1.59	1.46
2086	1.99	1.64	1.51
2088	2.05	1.69	1.56
2090	2.11	1.75	1.61
2092	2.17	1.8	1.66
2094	2.23	1.85	1.71
2096	2.29	1.91	1.76
2098	2.36	1.96	1.82
2100	2.42	2.02	1.87

Water Surface Elevation

USACE's North Atlantic Coast Comprehensive Study coastal storm water level modeling was utilized. Nodes of data recoding nearest to the center of each barrier was selected. Figure 2 shows the surge barrier locations and Figure 3 shows the shore-based measure northern locations and Figure 4 shows the southern locations. At each node, the expected value for the 100-year water surface elevation in 1992 MSL datum was converted to 2080 NAVD88 in feet and is shown in Table 2 for the surge barrier locations, and in Table 3 for the shore-based measures (with the exception of Hackensack Perimeter measures, for which existing studies were utilized).

Wave Heights

USACE's North Atlantic Coast Comprehensive Study coastal storm wave modeling was utilized at the same nodes nearest the center of the surge barrier. The 100-year expected value wave height at each node was converted from meters to feet. The wave heights at the surge barrier locations are shown in Table 2 and at the shore-based measure locations in Table 3 (with the exception of Hackensack Perimeter measures, for which existing studies were utilized).

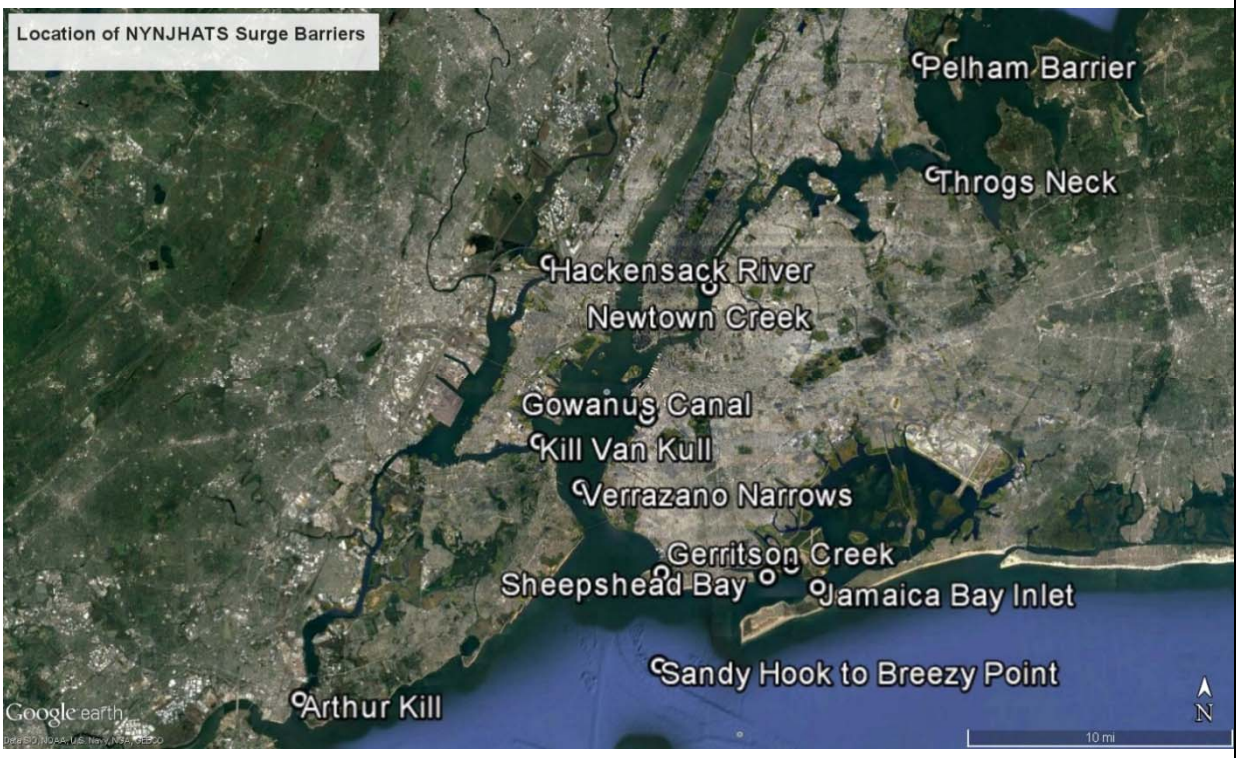


Figure 2: Surge Barrier Locations

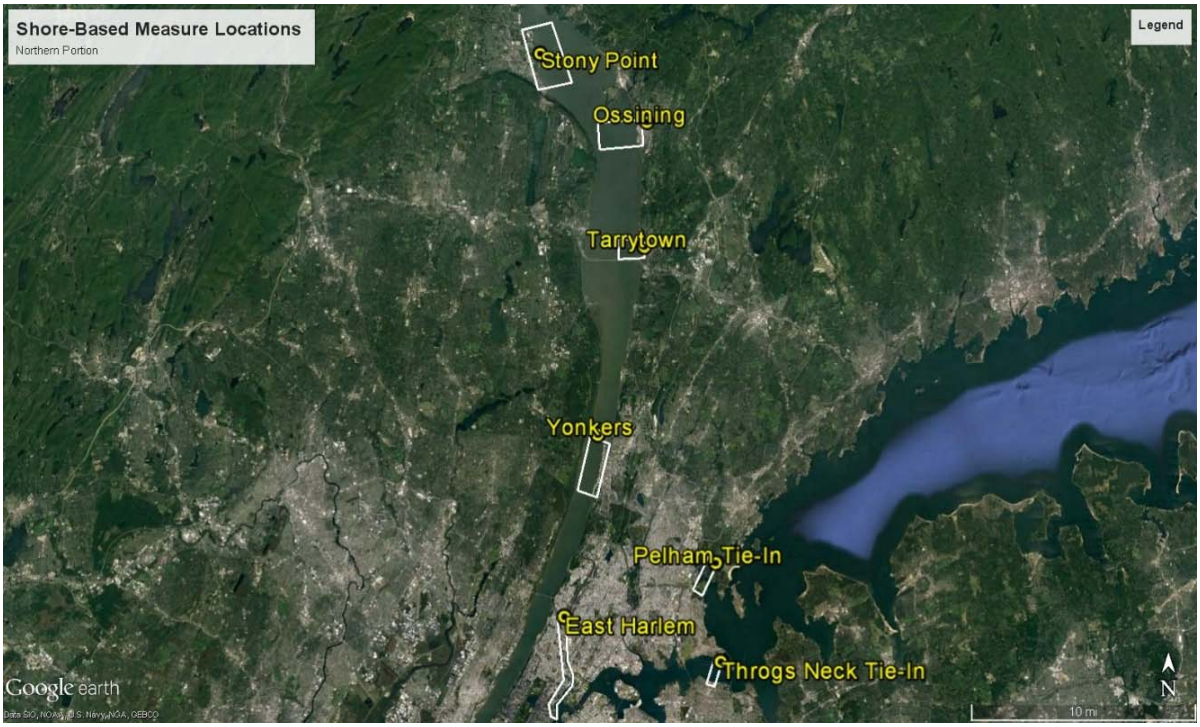


Figure 3: Shore-Based Measure Northern Locations

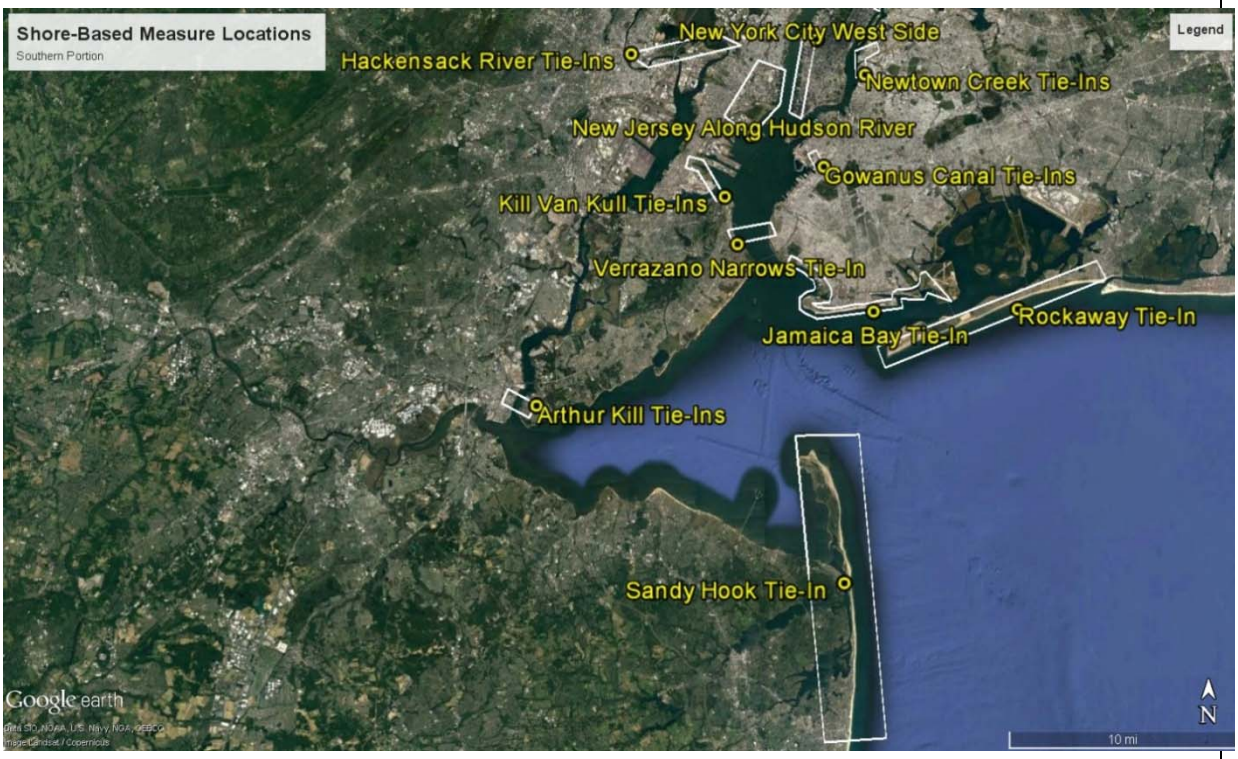


Figure 4: Shore Based Measures Southern Locations

Table 2: Water Surface Elevation and Wave Heights for Surge Barrier Locations

Reach	node	Assigned Gage for SLR	100yr Stage Elevation in 2080 ft. navd	100yr Significant Wave Height in ft
Newtown Creek	13898	the Battery	12.5	3.7
Gowanus Canal	11862	the Battery	12.8	3.7
Coney Island Creek	14067	Sandy Hook	12.7	3.3
Sheepshead Bay	11967	Sandy Hook	11.8	5.2
Throgs Neck	4347	Kings Point	14.4	4.3
Pelham Barrier	12997	Kings Point	14.5	3.9
Sandy Hook-Rockaway	3900	Sandy Hook	12.2	16.4
Verrazano Narrows	11781	Sandy Hook	12.9	5.9
Kill Van Kull	11766	the Battery	12.9	6.0
Hackensack River	11816	the Battery	12.1	3.3
Arthur Kill	11650	Sandy Hook	14.4	3.8
Gerritson Creek	14085	Sandy Hook	11.8	3.9
Jamaica Bay Inlet	3592	Sandy Hook	11.6	4.8

Table 3: Water Surface Elevations and Wave Heights for Shore-Based Measure Locations

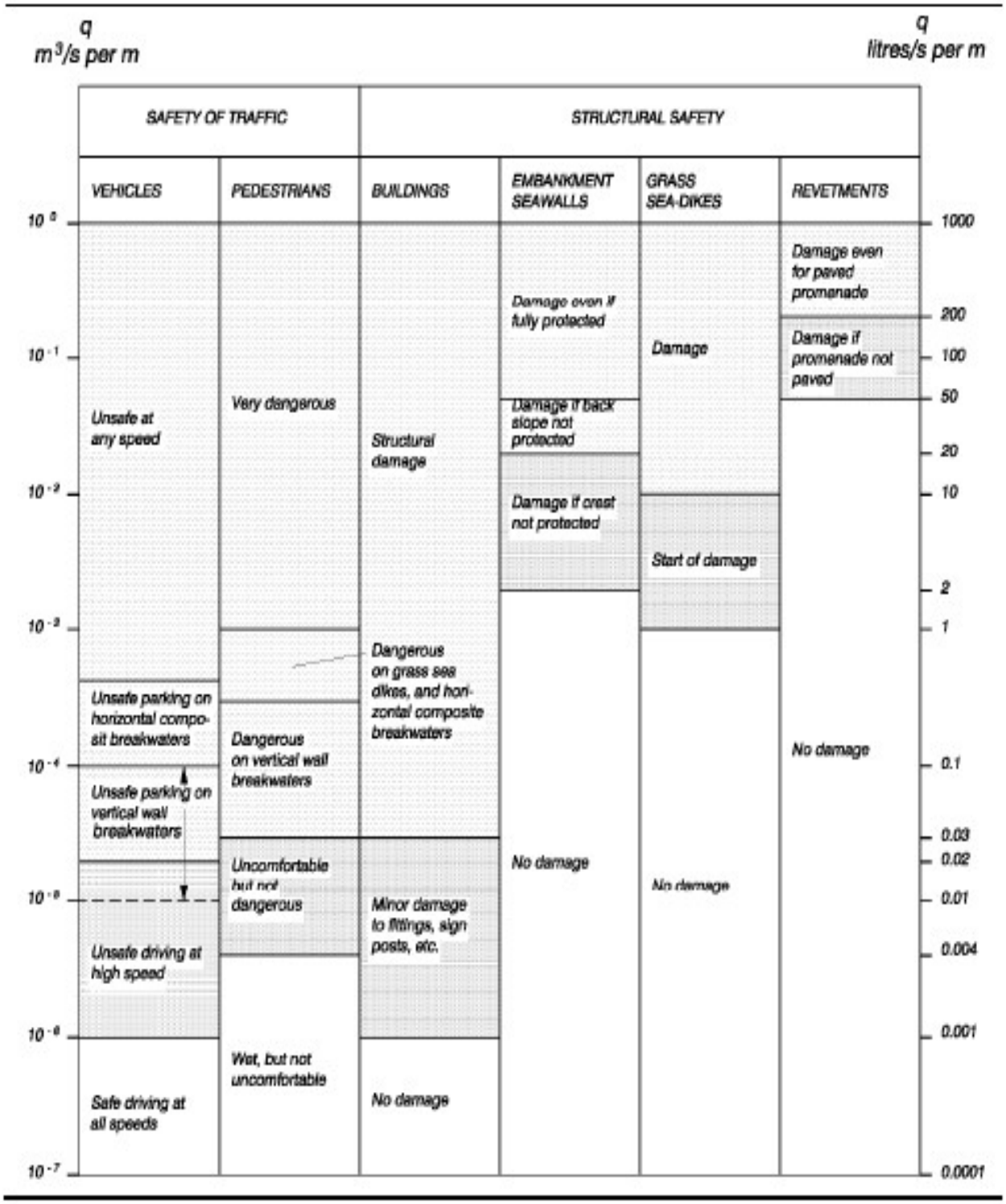
Study Shoreline and Tributary Segments	Segment Name	NACCS Node#	Latitude NAD83	Longitude NAD83	Sea Level Gage	0.01% Annual Chance of Occurrence 2080 Water Surface Elevation in ft. NAVD88	0.01% Annual Chance of Exceedence Significant Wave Height, Hs, in ft.
1	Sandy Hook-Rockaway Barrier Tie-in	3789	40.447835	-74.00394	Sandy Hook	13.0	4.9
6	Arthur Kill Barrier Tie-In	3967	40.577596	-74.211387	Sandy Hook	14.3	3.3
7	Verrazano Narrows Barrier	13818	40.65039	-74.11392	The Battery	12.9	3.6
7	Kill Van Kull Barrier Tie-In	13818	40.65039	-74.11392	The Battery	12.9	3.6
8	Arthur Kill Barrier Tie-In	3967	40.577596	-74.211387	Sandy Hook	14.3	3.3
11	Hackensack River Barrier Tie-in	4206	40.714432	-74.121093	The Battery	13.3	3.9
12	Hackensack River Barrier Tie-in	4284	40.7673	-74.0898	The Battery	11.7	2.6
13	Kill Van Kull Barrier Tie-In	13818	40.65039	-74.11392	The Battery	12.9	3.6
14	New Jersey along Hudson River	4176	40.695751	-74.050209	The Battery	12.9	4.9
16	Yonkers	3575	41.7833	-73.95	The Battery	10.1	0.0
16	Ossining	3600	42.253106	-73.801772	The Battery	12.0	0.0
16	Stony Point	7976	41.03464	-73.90854	The Battery	10.1	3.6
16	Yonkers	13872	40.99222	-73.90163	The Battery	10.0	0.0
16	Tarrytown	13872	40.99222	-73.90163	The Battery	10.0	0.0
18	New York City West Side SBM	13862	40.79536	-73.99269	The Battery	11.5	4.4
19	Gowanus Canal Barrier Tie-in	11875	40.75109	-73.96316	The Battery	12.5	4.4
20	East Harlem SBM	13888	40.80185	-73.92648	The Battery	12.7	3.0
22	Pelham Barrier	4349	40.797923	-73.811106	Kings Point	14.0	4.9
22	Throgs Neck Barrier Tie-in	4349	40.797923	-73.811106	Kings Point	14.0	4.9
26	Newtown Creek Barrier Tie-in	4321	40.787959	-73.91449	The Battery	8.1	3.9
28	Newtown Creek Barrier Tie-in	7673	40.7	-74.015	The Battery	12.8	4.3
29	Gowanus Canal Barrier Tie-in	11933	40.6344	-74.0399	The Battery	12.4	5.4
31	Jamaica Bay Barrier Tie-In	14070	40.57836	-74.01342	Sandy Hook	13.0	5.7
32	Jamaica Bay Barrier Tie-In	3963	40.574571	-73.939088	Sandy Hook	11.8	5.6
34	Jamaica Bay Barrier Tie-In	14196	40.57028	-73.85128	Sandy Hook	12.9	12.0
34	Sandy Hook-Rockaway Barrier Tie-in	14196	40.57028	-73.85128	Sandy Hook	12.9	12.0

Flowrate as Metric and Figure

The barrier crest elevation must be able to limit the wave flow rate to prevent damage to the gate itself and to limit the volume of water accumulated on the protected side. ER 1110-2-1100 Coastal Engineering Manual includes a figure showing critical overtopping discharges which is shown as Table 5. Embankment seawalls (vertical) was selected as the appropriate category and 50 liters/sec/m was chosen as the critical mean overtopping discharge, for the vertical surge barriers, which represents the condition "damage if back slope not protected". For the on shore-based measures, pedestrians were selected as the appropriate category and 1 liters/sec/m was chosen as the critical mean overtopping discharge, which represents the condition "damage if and grass sea dikes and horizontal composite breakwaters".

Table 4: Critical Values of Average Overtopping Discharges

Table VI-5-6
Critical Values of Average Overtopping Discharges



Methods Selected and Geometry and Equations

Two methods were selected to estimate mean overtopping discharges, one contained in EM 1110-2-1100 Coastal Engineering Manual (Franco and Franco 1999, and one referenced in EM 1110-2-1614 Design of Coastal Revetments, Seawalls, and Bulkheads-Ward and Ahrens (1992). Franco and Franco tested structures with nonbreaking waves. Ward and Ahrens tested structures with breaking waves. Vertical structures were modeled for the vertical surge and vertical on land barriers. Trapezoidal cross-sections were used where more land was available, and the land fronting the cross-section was assumed by the model to function as a berm.

Resulting Freeboard

Freeboard is defined as the difference between the crest elevation of the surge barrier and the design still water surface elevation. The model equations were used iteratively, predicting overtopping discharge for each freeboard entered until the discharge was close enough to but not over 50 liters/sec/m for the surge barriers; and close enough to but not over 1 liter/sec/m for the on shore-based measures. Then the most conservative (greatest) freeboard of the two methods was designated as the design freeboard and when added to the 100-year in 2080 water surface elevation becomes the design crest elevation. The resulting design crest elevations for each surge barrier are shown in Table 6, in Table 7 for each shore-based measure.

Safety Check

As a safety check, the design crest elevation at each surge barrier was compared to the 500-year 2080 still water elevation to make sure that for that condition weir flow would not occur transmitting large volumes of water INTO the protected side of the line of protection (like filling a bathtub). In every case, the crest elevations of the structures exceeded the 500-yr water surface elevation. These are shown in Table 6.

Table 5: Design Crest Elevations for Vertical Surge Barriers

Reach	Freeboard Required for Flowrate of less than 50 liters/second/meter	100yr (1% Annual Exceedence Probability) Crest Elevation in ft. NAVD	Compare with 500yr (0.2% Annual Exceedence Probability) Still Water Elevation in ft. NAVD in ft. in 2080	FEMA 100yr (1% Annual Exceedence Probability) Floodplain Elevation in ft. NAVD
Newtown Creek	4.0	16.5	13.9	15
Gowanus Canal	3.9	16.7	14.3	14
Coney Island Creek	5.1	17.8	16.3	14
Sheepshead Bay	9.2	21.0	14.9	12
Throgs Neck	5.2	19.6	15.8	18
Pelham Barrier	4.2	18.7	16.0	18
Sandy Hook-Rockaway	33.7	45.9	13.6	17
Verrazano Narrows	8.6	21.5	14.4	14
Kill Van Kull	7.6	20.5	14.3	14
Hackensack River	5.3	17.4	14.5	10
Arthur Kill	4.3	18.7	16.0	12
Gerritson Creek	6.6	18.4	14.9	13
Jamaica Bay Inlet	6.0	17.6	12.7	12

Table 6: Design Crest Elevations for Shore-Based Measures

Study Shoreline and Tributary Segments	Segment Name	NACCS Node#	Latitude NAD83	Longitude NAD83	Sea Level Gage	0.01% Annual Chance of Occurrence 2080 Water Surface Elevation in ft. NAVD88	Freeboard in ft. (Difference between Crest and Water Surface Elevation)	0.01% Annual Chance of Occurrence 2080 Design Crest Elevation in ft. NAVD88
1	Sandy Hook-Rockaway Barrier Tie-in	3789	40.447835	-74.00394	Sandy Hook	13.0	0.8	13.8
6	Arthur Kill Barrier Tie-In	3967	40.577596	-74.211387	Sandy Hook	14.3	2.2	16.5
7	Verrazano Narrows Barrier Tie-in	13818	40.65039	-74.11392	The Battery	12.9	3.6	16.5
7	Kill Van Kull Barrier Tie-In	13818	40.65039	-74.11392	The Battery	12.9	17.2	30.1
8	Arthur Kill Barrier Tie-In	3967	40.577596	-74.211387	Sandy Hook	14.3	2.2	16.5
11	Hackensack River Barrier Tie-in	4206	40.714432	-74.121093	The Battery	13.3	3.0	16.3
12	Hackensack River Barrier Tie-in	4284	40.7673	-74.0898	The Battery	11.4	3.5	14.9
13	Kill Van Kull Barrier Tie-In	13818	40.65039	-74.11392	The Battery	12.9	4.9	17.8
14	New Jersey along Hudson River SBM	4176	40.695751	-74.050209	The Battery	12.9	4.9	17.8
16	Yonkers	3575	41.7833	-73.95	The Battery	10.1	12.3	22.4
16	Ossining	3600	42.253106	-73.801772	The Battery	12.0	9.7	21.7
16	Stony Point	7976	41.03464	-73.90854	The Battery	10.1	11.5	21.6
16	Yonkers	13872	40.99222	-73.90163	The Battery	10.0	0.8	10.8
16	Tarrytown	13872	40.99222	-73.90163	The Battery	10.0	11.2	21.2
18	New York City West Side SBM	13862	40.79536	-73.99269	The Battery	11.5	13.6	25.1
19	Gowanus Canal Barrier Tie-in	11875	40.75109	-73.96316	The Battery	12.5	4.3	16.8
20	East Harlem SBM	13888	40.80185	-73.92648	The Battery	12.7	7.0	19.7
22	Pelham Barrier Tie-In	4349	40.797923	-73.811106	Kings Point	14.0	8.6	22.6
22	Throgs Neck Barrier Tie-in	4349	40.797923	-73.811106	Kings Point	14.0	8.6	22.6
22	Bronx River Westchester Creek Barrier Tie-in	4349	40.797923	-73.811106	Kings Point	14.0	6.8	20.8
26	Newtown Creek Barrier Tie-in	4321	40.787959	-73.91449	The Battery	8.1	9.2	17.3
28	Newtown Creek Barrier Tie-in	7673	40.7	-74.015	The Battery	12.8	3.6	16.4
29	Gowanus Canal Barrier Tie-in	11933	40.6344	-74.0399	The Battery	12.4	4.4	16.8
31	Jamaica Bay Barrier Tie-In	14070	40.57836	-74.01342	Sandy Hook	13.0	38.3	51.3
32	Jamaica Bay Barrier Tie-In	3963	40.574571	-73.939088	Sandy Hook	11.8	8.7	20.5
34	Jamaica Bay Barrier Tie-In	14196	40.57028	-73.85128	Sandy Hook	12.9	4.1	17
34	Sandy Hook-Rockaway Barrier Tie-in	14196	40.57028	-73.85128	Sandy Hook	12.9	22.1	35

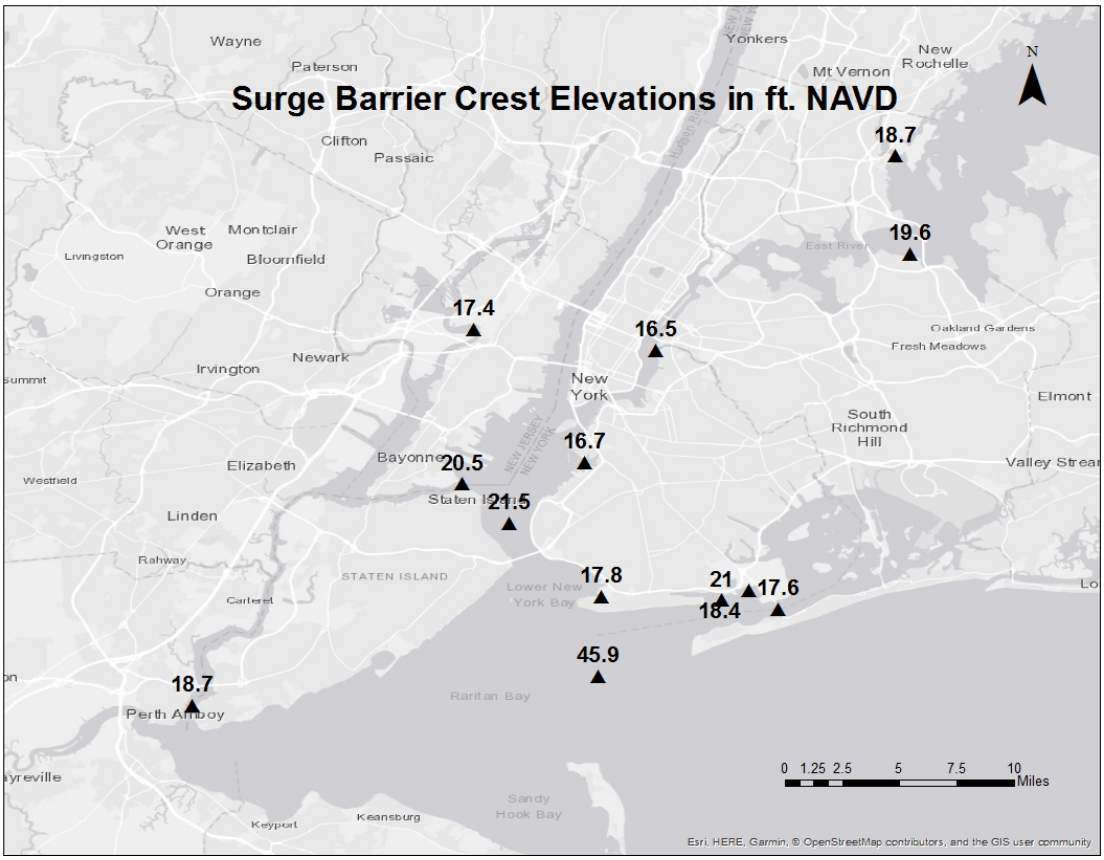


Figure 5: Design Crest Elevations for Vertical Surge Barriers

Discussion of Surge Barrier Crest Elevation Results

Of note is the extremely high design crest elevation for the Sandy Hook to Breezy Point Surge Barrier (45.9 ft. NAVD). The orientation of the adjacent land masses make this location very exposed to east and south fetches, and the fact that the many of the storms affecting this area come from these directions. This combined with the vertical nature of the gate face and the 16 ft. wave height leads to the conclusion that the 45.9 ft. NAVD crest elevation for this location is reasonable.

The next highest surge barriers in the 21 ft. NAVD range are, in decreasing order; Verrazano Narrows (21.5) and Sheepshead Bay (21.0). In the 20 ft. NAVD range is Kill Van Kull (KVK, 20.5). In the 19.0 ft. NAVD range is Throgs Neck (19.6). In the upper 18 ft. NAVD range are Pelham Barrier and Arthur Kill (both at 18.7). These for the most part also have funneling effects, though to a lesser degree than for Sandy Hook to Rockaway.

The last group of surge barriers in the lower 18 ft. NAVD range is Gerritsen Creek (18.4); in the 17 ft. NAVD range - Coney Island Creek (17.8), Jamaica Bay (17.6), and Hackensack River (17.4); and lastly in the 16 ft. NAVD range are Gowanus Canal (16.7) and Newtown Creek (16.5). These primarily have the lowest wave heights.

Discussion of Shore-Based Measure Crest Elevations

The largest crest elevations of the shore-based measures occur at Segment 31-the Jamaica Bay Barrier Tie-In at +51.3 ft. NAVD88. This is the shoreline with the longest south facing fetch (the primary directions of hurricane propagation. The 2nd largest is at Segment 34-the Sandy Hook-Rockaway Barrier Tie-In at +35 ft. NAVD88, with the SSE facing shoreline which is slightly exposed to hurricanes.

The next largest crest elevations occur at Segment 7-the Kill Van Kull Barrier south Tie-In at +30.1 ft. NAVD, for what reason isn't entirely clear, though wave-current interaction may have a part. Then is Segment 22-the Bronx River Westchester Creek Barrier Tie in at +28.5 ft. NAVD88-here also, wave current interaction may be occurring. Next is +25.1 ft. NAVD88 of Segment 18-New York City West Side-currents at work, likely, and then +24.8 ft. NAVD of Segment 25-Astoria, and +22.6 ft. NAVD of Segment 22-Pelham and Throgs Neck Barrier Tie-Ins (exposed wave fetch-influenced).

The next largest crest elevations (+21.2 to +21.7 ft. NAVD88) are for Segment 16-Tarrytown, Stony Point, Ossining and Yonkers along the Hudson River, for the directions facing the predominant currents. The next largest are +20.0 to +20.8 ft. NAVD88 for Segments 25 and 26 (Long Island City/Astoria), Segment 32 Jamaica Bay Barrier Tie-In, and Segment 22 Bronx River Westchester Creek Barrier Tie in, respectively.

Then follow +19.4 to +19.8 ft. NAVD88 (Segment 31-Jamaica Bay Barrier Tie-In, 28-Newtown Creek Barrier Tie-In, 20-East Harlem, and 25-Flushing Creek Barrier Tie-Ins, respectively). Jamaica Bay is due to wave fetch, Newtown Creek, East Harlem, and Flushing Creek are influenced by currents. After this are +17.0 to +17.8 ft. NAVD88 (Segment 34-Jamaica Bay Barrier Tie-In, 26-Newtown Creek, 14-New Jersey along the Hudson River, and 13-Kill Van Kull Barrier Tie-In, respectively). Jamaica Bay ocean facing is wave fetch-influenced; Newtown Creek, NJ/Hudson and KVK are current-influenced.

The next largest crest elevations range from +16.3 to +16.8 ft. NAVD (Segment 20-East Harlem, 18-New York City West Side, 11-Hackensack River Barrier Tie-In, 28-Newtown Creek, 8-Arthur Kill Barrier, 7-Verrazano Narrows Barrier Tie-In, 6-Arthur Kill Barrier Tie-In, and 29 and 19-Gowanus Canal Barrier Tie-Ins). Harlem, NY City West Side, Hackensack Newtown, Arthur Kill and Verrazano Narrows are current-influenced and Arthur Kill is also wave fetch-influenced. Then follow +13.8 to +14.9 ft. NAVD88 (Segment 1-Sandy Hook-Rockaway Barrier Tie-In (east-facing), 32-Jamaica Bay Barrier Tie-In and 12-Hackensack River Barrier Tie-In, respectively). Sandy Hook east facing shoreline has a long wave fetch, however the more severe storms do not propagate from the east, hence the crest elevation is significantly lower than the Tie in on the Rockaway side (Segment 34). Jamaica Bay is crest elevations face west (with a narrow-wave fetch), and south (though protected by the Rockaway peninsula), thus the lower crest elevations than Segment 31 (Jamaica Bay Barrier Tie-In). Hackensack River Barrier location is current-influenced.

The lowest crest elevations (+10.6 to +11.5 ft. NAVD88) are from Segments 16-Yonkers and Tarrytown and 12-Hackensack Perimeter. The Yonkers and Tarrytown crest elevations face obliquely to the predominant current directions of the Hudson River, and the Hackensack Perimeter crest elevations are in the Meadowlands marsh areas, on the river overflow areas and not facing the current directions. Furthermore the Hackensack Perimeter areas utilize previously developed design calculations from Meadowlands studies.

1.1.3 Conceptual Designs of Storm Surge Barriers

The storm surge barriers were only conceptually designed. The purpose of the tasks was to provide the preliminary engineering basis to support the evaluation and development of conceptual designs for the storm surge barriers that are part of the HAT Study Alternatives, specifically the minimum practical dimensions of the barrier openings and preliminarily selection of gate types. The barrier openings consist of a navigable passage and auxiliary flow openings. The objective at this stage of the study is to provide a description and first conceptual design which outline the basis for key geometric characteristics (e.g. height, width, and depth of openings) for each storm surge barrier. This will enable USACE-NAN Cost Engineering to use a parametric cost model to estimate the approximate cost of the storm surge barrier for this report.

Design criteria and dimensions are based on qualitative data, quantitative data when readily available and desktop analyses to a level of detail commensurate with a feasibility study. In instances where limited data was available, assumptions were made based on engineering judgment, previous experience and/or the partial data that has been collected over the course of the feasibility study phase. The implications of such assumptions along with recommendations for further data collection and refined analyses to support the design were documented.

The preliminary assessment of navigable passage widths and storm surge barrier configuration shall not be construed as recommendations or requirements for actual design for implementation. Significant additional study is required to substantiate the conceptual design of the storm surge barriers. Additional study is required to study the width, location, and configuration of the navigable passages and auxiliary flow gates, including a full evaluation of navigation, environmental, ecological, and cost considerations, amongst others.

Analyses and engineering studies were summarized in the reports listed below:

ID	Title	Contents/Subject
R001	Conceptual Design for HAT Study Storm Surge Barriers – Volume 1	Conceptual Design for the following list of Storm Surge Barriers: Outer Harbor, Verrazano Narrows, Arthur Kill, Kill van Kull, Jamaica Bay, Hackensack River and Throgs Neck.
R002	Conceptual Design for HAT Study Storm Surge Barriers – Volume 2	Conceptual Design for the following list of Storm Surge Barriers: East Chester Creek, Westchester Creek, Bronx River, Flushing Creek, Newtown Creek, Gowanus Canal, Sheepshead Bay and Gerritsen Creek
R010	Shore Based Measures	The Structural Coastal Storm Risk Management (CSRM) measures evaluated as part of the HAT Study other than the storm surge barriers

1.2 Numerical Modeling

The HATS alternative designs include various combinations of surge barriers around the harbor that would close when extreme high water levels from storm events are expected. Under normal tidal conditions and lower level surges the barriers would remain open for navigation and to allow for water exchange. Building and operating infrastructure of this scale, with varying barrier gate design and location, has the potential to impact hydrodynamics, salinity, sediment transport, etc. at both the location of the proposed infrastructure and more broadly throughout the harbor and estuary. Further, operation of the barrier during an impending coastal storm event also has the potential to 1) alter surge along the shoreline and flooding behind the barrier due to freshwater ponding and seiching within the enclosed basin, and 2) alter surge levels that would have occurred otherwise outside the barriers.

In order to quantify the magnitudes and geographic extents of these potential changes at a screening level, two modeling studies were conducted:

1) Regional changes to the normal progression of the tide, water levels, current velocities, and salt distribution were assessed using a previously calibrated existing Adaptive Hydraulics (AdH) three dimensional numerical model of this study area.
2) Closed barrier impacts on regional water levels were evaluated using The Coastal Storm Modeling System (CSTORM-MS).

The reader should note that both the AdH and CSTORM-MS numerical models were applied to this preliminary screening study based on existing available calibration without further calibration to specific local hydrodynamic and bathymetric site conditions, such as tidal current, river flow, local wetlands, and more detailed shoreline configuration. For this reason (among others), USACE anticipates that the characterizations of the extent and level of inundation will differ from what is presented in this document once more rigorous modeling is conducted. Models used for the next level analysis will likely include greater numerical Modules capable of hydrodynamic, coastal, environmental, and sedimentation evaluation and may not be limited to AdH or CSTORM-MS.

1.2.1 AdH Modeling Effort

The AdH model was applied to the study region in order to quantify changes in tide characteristics, water levels, current velocities and salinity associated with barrier open conditions. Year-long AdH three dimensional simulations were performed for 2 sets of the four conceptualized alternatives that include in-water structures, and a "without project" condition. The "without project" AdH mesh used and modified for each of the simulations, was previously developed as part of a larger sedimentation study of the harbor. The simulations were forced with inflow and tidal conditions from 1995, a representative "non-storm" year. The initial set of alignments was simulated using somewhat standard or assumed template surge gate structures that have been constructed in other estuaries with similar environmental conditions. The model output was utilized along with ship sizes/frequency/transit path information in an iterative manner to inform

and define the second, more refined set of structure configurations in terms of gate sizes, locations, and elevations (i.e., alternatives 2, 3A, 3B, and 4). The refined configurations were applied toward the CSTORM-MS modeling effort discussed below. For all alternatives, the barriers had similar broader flowfield changes, a localized increase in current velocities surrounding the structures, and a slight reduction in tidal prism. Reduced tidal mixing resulted in more local and regional stratification depending on the alternative. This process was clearly observed during higher flows (75th velocity percentile, spring tides) which experienced a more substantial reduction in the overall velocity field when compared to the "without project" conditions. Alternatives 2 and 3a had the largest reduction in tide ranges and tidal exchanges throughout the harbor. Alternatives 3b and 4 induced lesser change to water levels, regional flushing and stratification within the estuary. This general result is somewhat intuitive given the larger scale of shoreline/study area encompassed by Alternatives 2 and 3A versus Alternatives 3B and 4. The with- and without project flowfield results were also evaluated for engineering/navigation considerations described elsewhere in this report. These results may also be informative for environmental evaluations of any of these conceptual in-water measures included in these alternatives, should they be carried forward in the study.

1.2.2 CSTORM-MS Modeling Effort

This modeling task was designed to assess the potential increase in storm surge outside of the barriers along with the corresponding changes in water level within the protected basin due to freshwater ponding and wind setup during an event in which the barriers are closed. Twenty-one (21) storm events selected from the North Atlantic Coast Comprehensive Study (NACCS) model simulation database were used to evaluate "without project" to "with-project" conditions for Alternatives 2, 3a and 3b. Due to limitations in model resolution, Alternative 4 was not simulated. Base NACCS model grids were refined and modified to represent each alternative and the selected storms roughly correspond to the 10, 20, 50, 100, 200 and 500 year events for the study area. Hurricanes Sandy (2012) and Irene (2011) were included in the storm suite along with select extra tropical events. In general, water level response is sensitive to storm track, or the path that storms take. Therefore, a set of 21 historical storms were utilized to provide a larger sampling of potential storm tracks that could be expected for this region. For most of the events ocean surge was enhanced (i.e. larger) by the presence of the structures, or barriers. The geographic extent of influence was dependent on wind direction and strength of the simulated storm and the barrier configuration being tested. However, in a few events, ocean surge was reduced in some areas compared to the base condition. Alternative 2 produced the greatest regional change in ocean surge, elevating water levels above the base condition from Sandy Hook, NJ to Breezy Point, NY during mild storms, and from Manasquan Inlet, NJ through Jones Inlet, NY for moderate to severe events. Alternative 3a also produced an elevated ocean surge response for most of the simulated events. The impacted area ranged from lower New York Harbor throughout Raritan Bay.

In addition for select events, Alternatives 2 and 3a also experienced an increase in ocean surge in western Long Island Sound. Alternative 3b had the least impact on ocean surge for all events and

the best performance during the Hurricane Sandy simulation, in that it had the least induced flooding and ponding. Elevated water levels associated with Alternative 3b were more localized to the individual structures. Ponding behind the barriers was evident in three (3) of the twenty-one (21) simulated storms. This process was the result of wind setup (seiche) and fresh water inflow to the enclosed basin. However this result is erroneous as the simulations assumed closed gate structures throughout the two-week period that was simulated and used fixed freshwater inflows during the simulation. Actual gate closures are anticipated to be far shorter in duration (e.g., 2-3 days) as a coastal storm was approaching such that the ponding behind the structures would be less likely to occur.

To compare to these results, a simple set of AdH model simulations, which included statistically derived conservative river inflows but did not include wind setup effects (seiches), confirmed that water levels behind the barriers, with the exception of the Newtown Creek barrier, are likely to contain freshwater in-flow ponding for 4 days or more without causing flooding. Newtown Creek, given its large wastewater treatment plant freshwater inflow, did indicate flooding in less than 4 days which would necessitate rerouting of that input source should that measure be advanced further in the study. Should any in-water measure in any of the alternatives be carried further in the study, further refined CSTORM-MS simulations are planned to correct temporal effects associated with surge structure closure.

1.2.3 Conclusions

In summary, these modeling tasks indicate the use of fixed, in-water infrastructure will modify tide propagation, water levels, current velocities and salinity locally and regionally within the study area. The geographic extent of change for each of these characteristics is directly influenced by the size of the barrier alternative. The greater the structure footprint (i.e. alternative 2), the larger geographic influence is. During barrier open conditions, each alternative experienced a reduction in tidal prism and an increase in local fluid velocities. The reduced water exchange between ocean and estuary results in increased stratification and a change in salt distribution. The closure of the barriers appears to enhance ocean storm surge for most of the simulated events. With storage capacities on the order of several days, flooding potential associated with wind setup and ponding within the enclosed basins is expected to be minimal. More detailed modeling will be required to refine barrier designs and to determine sediment transport patterns, changes to tidal range and flows, scour around the proposed structures, as well as environmental impacts.

1.3 Typical conceptual design “Jamaica Bay Storm Surge Barrier”

Following all engineering best judgment and considerations presented within the study, a preliminary geometry for the storm surge barrier openings has been established and are displayed below.

The table and figure below summarizes the key hydraulic and geometric parameters for the conceptual design of the Jamaica Bay Storm Surge Barrier. The objective at this stage of the study

is a description and first conceptual design which outline the basis for key geometric characteristics (e.g. height, width, and depth of openings). This will enable cost engineering analysis to use a parametric cost model to estimate the approximate cost of the barrier.

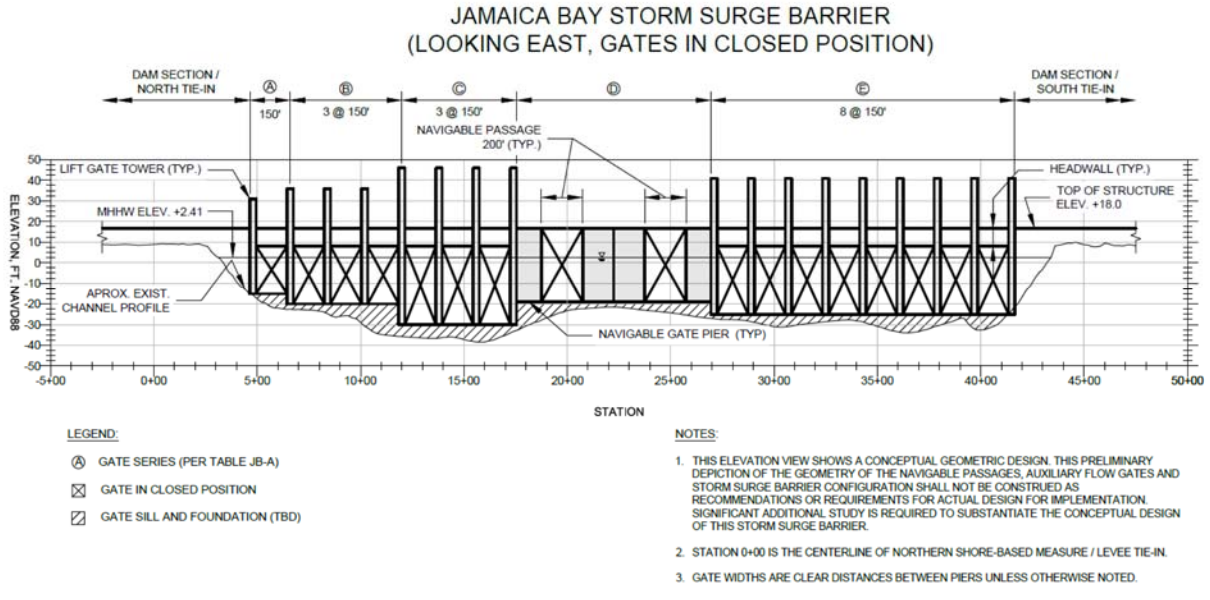
	Storm Surge Barrier Preliminary Geometry									
Title:	Summary for Parametric Cost Estimate									
Contract No.:	W912DS-15-D-0002							**Discipline:**	Civil Engineering	**Sheet:** 6 of 7
Task Order No.:	W912DS-18F0027							**Prepared By:**	M. Kluijver / A. Pearson	**Date:** 9/28/2018
Project Title:	New York/New Jersey Harbor & Tributaries CSRM Study							**Updated By:**	T. Dembia / A. Pearson	**Date:** 11/2/2018
Consultant Name:	Moffatt & Nichol									
Consultant Contact:	M. Kluijver									
Revision:	A									

Table JB-A
Jamaica Bay Storm Surge Barrier

Gate Series	# of Gates	Sill El.	Gate Width (Ea.)	Pier Width (Ea.)	Total Flow Area[3]	Total Pier Area[4]	Total Surface Area (Closed Gates)[5]	Headwall Area (Ea.)[6]	Total Gate Span	Notes	Top of Structure El.
(North) A	1	-15	150	30	2,250	990	5,940	1,650	180		18
B	3	-20	150	30	9,000	3,420	20,520	1,650	540		18
C	3	-30	150	30	13,500	4,320	25,920	1,650	540	Deepest Auxillary Gate	18
D	2	-19	200	150	7,600	22,200	37,000	0	700	Navigable Passage	18
(South) E	8	-25	150	30	30,000	10,320	61,920	1,650	1,440	Deepest Auxillary Gate	18
North Dam	1	-15	500	0	7,500	0	16,500	0	500	Dam Section - North	18
South Dam	1	-25	225	0	5,625	0	9,675	0	225	Dam Section - South	18
Total:	**19**				**75,475**	**41,250**	**177,475**	**6,600**	**4,125**		

Gate Structures	Area Breakdown				Length Breakdown
Navigable Structures:	7,600	22,200	37,000	0	700
Auxiliary Structures:	54,750	19,050	114,300	6,600	2,700
Dam Structures:	0	0	26,175	0	725
				Total Length:	4,125

Notes:
1. Elevations are based on the NAVD88 vertical datum.
2. Measurements are in feet and square feet unless otherwise noted.
3. Sum of all open flow areas in the S.S.B. gate series measured between elev. 0.0 and the sill elev and the clear distance between piers.
4. Sum of all pier surface areas in the S.S.B. gate series between the top of the structure elev. and the sill elev.
5. Sum of all S.S.B. surface areas (dams, piers, closed gates, headwalls) in the gate series between the top of the structure and the sill elev.
6. Area between top of structure and bottom of headwall at elevation +7.0.



1.3.1 Cost Summary

The NY-NJ Harbor & Tributaries Feasibility Study (HATS) for coastal protection against storm surge investigates four alternative flood risk mitigation systems ranging from large transect storm surge barriers (SSB's) paired with the necessary on-land tie-in features (alternative 2) to an entirely on-land, perimeter solution of shoreline-based measures (SBM's) throughout the study area (alternative 5) with additional alternatives with different combinations of intermediate SSB's and the corresponding SBM's for shoreline left exposed outside of the protection of the SSB's (alternatives 3A, 3B & 4).

For a project with such a sprawling geographic area under consideration, at this phase of the feasibility study, parametric cost estimates are required for plan selection. This is particularly the case on HATS because of the prominent role for SSB's in the various designs. The complexity and site-dependency of SSB design demands greater resources than available for traditional "bottom-up" cost estimating methods. While such an approach will be implemented in a future phase of the study, at this phase parametric cost engineering methods are employed.

Details of the parametric approach to cost engineering on this study are detailed in the Methodology section (COST APPENDIX), but to summarize: All proposed measures for the four alternatives under consideration fall within the two categories of Storm Surge Barriers (SSB's) and Shoreline-Based Measures (SBM's). Each measure is assigned a unit cost based on type while design dimensions are estimated based on preliminary layout informed by available topography/bathymetry and development data.

SSB's include a range of features (mostly dynamic such as navigable gates and auxiliary flow openings) constructed to transect existing bodies of water. Their design is meant to permit the maximum flow of / minimum interruption to existing fluvial and tidal flow while retaining the capacity to close and block the advance of a high surge associated with a large coastal storm.

SBM's include a range of features (mostly static ones like levees and floodwalls) constructed along the coast to impede rising waters associated with fluvial or storm surge flood events. Their design is meant to integrate as smoothly as possible with current coastal land use while providing elevated mitigation against flood risk.

For this study, the large size of—and the design uncertainties associated with—the SSB's under consideration drive the cost engineering approach. The proportional weight of these measures upon the ultimate construction costs and durations of whichever alternative is selected has meant that the bulk of the cost engineering effort thus far has focused on development of a cost model which is both sufficiently sensitive to the limited design decisions available at this phase and well-grounded in the limited reference data available.

Development of appropriate cost and duration models involved integrating the low level of design detail available (overall dimensions, not detailed quantities) and restricting the model to reasonable terms (positive correlation with increases in size, etc.). Compared to previously published SSB cost models, the inclusion of additional variables of interest produce models with improved

conceptual strength and usable cost models are selected which offer reasonable elasticity for each of the variables.

For construction cost and duration, the following models are recommended:

Cost = $19,000 x Navigable Area + $14,000 x Auxiliary Area + $3,000 Dam Area

Where, all areas are measured in Square Feet and Cost, in 2019 American Dollars.

Duration = 2 Years + 10 months / 100FT of Navigable Span + 7 weeks/ 100FT of Auxiliary Flow Span + 5 Days / 100FT of Dam

See below for a summary of the proposed barriers' construction cost[1] and durations.

Proposed Storm Surge Barrier	Estimated Initial Cost of Construction (w/o Contingency) [$, 2019Q1]	Estimated Duration of Construction [Years]
Throgs Neck	$ 4,027,000,000	10
Sandy Hook - Breezy Point	$ 42,911,000,000	25*
Verrazzano Narrows	$ 8,056,000,000	18
Arthur Kill	$ 1,739,000,000	7
Kill Van Kull	$ 2,981,000,000	8
Jamaica Bay	$ 2,783,000,000	9
Hackensack River	$ 676,000,000	4
Gowanus Canal	$ 85,000,000	2
Newtown Creek	$ 170,000,000	3
Flushing Creek	$ 200,000,000	3
Gerritson Creek	$ 98,000,000	2
Sheepshead Bay	$ 343,000,000	3
Coney Island Creek	$ 187,000,000	3
Bronx River	$ 150,000,000	3
Westchester Creek	$ 170,000,000	3
Pelham Barrier	$ 318,000,000	4
'Three Areas' model developed with regression analysis on a three parameters: the areas of navigable, auxiliary flow and dam features. Cost = $19000 x 'Navigable Area' + $14000 x 'Auxiliary Flow Area' + $3000 x 'Dam Area'		
*SH-BP Barrier construction duration is estimated with the same reference-based parametric duration model as the others, but assumes that the total span will be constructed in 3 parts, concurrently.		

Figure 6: Summary of estimated initial construction costs and durations for Storm Surge Barriers studied as part of NY/NJ Harbor & Tributaries Study

The Army Corps' greater experience with SBM's, paired with research into estimates compiled by study partners with experience in higher density applications, informs the unit costs applied for perimeter features. For both SSB's and SBM's the unit costs published in the NACCS report were taken as a starting point and refined based on the need for greater site-specific design dependency and improved modeling based on additional available information. The lengths of shoreline by measure were developed on a preliminary basis and will be refined with additional study and feedback from partnering agencies.

[1] These values are estimates of construction, not total, cost. See below for total estimated costs by alternative.

SHORELINE BASED MEASURE UNIT COST TABLE			
FEATURES \ ACCESS	Very Limited	Limited	Unlimited
Floodwall	$ 37,500	$ 11,250	$ 6,000
Levee	$ 9,000	$ 3,750	$ 2,000
Seawall	$ 11,500	$ 4,500	$ 3,000
Operable Flood Gate	$ 75,000	$ 30,000	$ 16,500
Elevated Promenade	$ 45,000	$ 15,000	$ 7,500
Buried Seawall/Dune	$ 15,000	$ 6,000	$ 3,000
Tide Gate	$ 75,000	$ 30,000	$ 16,500

Figure 7: Unit Costs by Shoreline Based Measure

Operations and Maintenance costs are treated as a function of construction costs and durations informed by known O&M costs of similar structures. Durations are estimated based on engineering judgment informed by experience and, in the case of SSB's, based on analysis of construction durations for reference barriers studied.

Alternative	Initial Construction (w/ contingency)	Environmental and Cultural Mitigation	Real Estate	Interest During Construction	Planning, Engineering & Design	Supervision & Administration	Operations, Maintenance, Repair and Rehabilitation	Total	Duration
2	$67.5	$0.27	$0.002	$39.99	$8.11	$6.76	$14.5	$137.1	25 Years
3a	$26.2	$0.24	$0.005	$8.26	$3.15	$2.63	$7.5	$48.0	18 Years
3b	$23.7	$1.27	$0.250	$4.04	$2.91	$2.46	$8.3	$42.9	9 Years
4	$18.1	$1.27	$0.175	$2.84	$2.23	$1.89	$6.5	$32.9	9 Years
5	$8.0	$1.38	$0.200	$0.91	$0.92	$0.79	$2.8	$15.0	9 Years

Figure 8: Total Estimated Costs by Alternative. All values are in Billions.

Total costs for the alternatives studied are expressed below in present (2019) value dollars to be compared against the corresponding benefits and against the no action Alternative 1. The table below summarizes total estimated costs by each alternative. These are the values used in the Cost-Benefit Analysis to inform plan selection or elimination.

US Army Corps of Engineers®
New York District

INTERIM REPORT

New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study

Conceptual Design for Storm Surge Barriers

Volume 1

February 2019

DOCUMENT VERIFICATION

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LIST OF ACRONYMS

AASHTO	American Association of State Highway Transportation Officials
ACES	Automated Coastal Engineering System
ADCIRC	Advanced Circulation Model
AdH	Advanced Hydraulics Model System
AEP	Annual Exceedance Probability
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
ASTM	American Society for Testing and Materials
BOD	Basis of Design
CFS	Cubic Feet Per Second
CO-OPS	Center for Operational Oceanographics Products and Services
CSRM	Coastal Storm Risk Management
CY	Cubic Yards
E&D	Engineering & Design
EL.	Elevation
EM	Engineering Manual
ETL	Engineering Technical Letter
ER	Engineering Regulation
ERDC	USACE Engineering Research and Development Center
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
FS	Factor of Safety
FWOP	Future Without Project
FWP	Future With Project
HAT	Highest Astronomical Tide
NYNJHAT Feasibility Study	New York-New Jersey Harbor and Tributaries Feasibility Study
HSDRRS	Hurricane Storm Damage Risk Reduction System
ICC	International Code Council
ISO	International Standards Organization
LAT	Lowest Astronomical Tide
LRFD	Load Resistance Factor Design
M&N	Moffatt & Nichol
MHHW	Mean Higher-High Water
MHW	Mean High Water
MSL	Mean Sea Level
MLW	Mean Low Water
MLLW	Mean Lower-Low Water
NAVD88	North American Vertical Datum of 1988
NJ	New Jersey

NOAA	National Oceanic and Atmospheric Administration
NNBFs	Natural and Nature-Based Features
NY	New York
NYC	New York City
PDT	Project Delivery Team
Rev	Revision
ROW	Right of Way
SBM	Shoreline Based Measure
SLR	Sea Level Rise
TM	Technical Memorandum
TOW	Top of Wall
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey

New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study

Navigable Storm Surge Barriers Conceptual Design
Volume 1

1 INTRODUCTION

1.1 The Study

The study purpose is to determine the feasibility of coastal storm risk management (CSRM) in the New York – New Jersey Harbor and Tributaries (NYNJHAT) study area, and to recommend a plan that will contribute to community and environmental resilience.

The study area encompasses the New York Metropolitan Area, including the most populous and densely populated city in the United States, and the six most populated cities in New Jersey. The shorelines of some of the NYNJHAT study area are characterized by low elevation areas, developed with residential and commercial infrastructure that are subject to tidal flooding during storms. The study area covers more than 2,150 square miles and comprises parts of 25 counties in New Jersey and New York.

During coastal storms, storm surges are generated on the open coast and propagate through New York Harbor or through the Long Island Sound and flood the extensive low-lying areas surrounding the metropolitan area.

The HAT Study alternatives encompass multiple CSRM measures such as flood walls, levees and storm surge barriers and all include, to some extent, raising existing coastal edges. Maps of the Study alternatives are included in Appendix A.

1.2 Perimeter Flood Risk Reduction vs. Coastal Barriers

1.2.1 General

A typically employed solution for reducing flood risk is to raise the level of existing perimeter flood risk reduction systems. This solution can be challenging to implement in geometrically constrained urbanized areas where waterfront spaces have multiple uses and serve a variety of stakeholders such that social and economic impacts could be considerable. In large bays, estuaries, natural harbors and port entrance channels, coastal barriers constructed as integral part of a flood risk reduction systems can be a cost-effective alternative to reduce the risk of flooding for the area. As such, the HAT Study includes evaluation of coastal barriers, in combination with other flood risk reduction systems.

Generally, four primary types of coastal barriers are identified: closure dams, storm surge barriers, tide gates and tidal locks. Closure dams permanently close off the connection between a coastal waterbody, such as an estuary or a bay, and the ocean. Closure dams eliminate the tidal connection and result in the formation of a man-made lake, effectively minimizing the chance of coastal storm surge induced floods for the area behind it. Due to the elimination of the tidal exchange, closure dams hinder navigation and introduce a steep gradient in environmental conditions across the structure, which are generally considered a substantial negative impact.

A storm surge barrier is a fully or partial movable barrier that includes operable elements (usually gates) that can be closed temporarily to impede storm surge generated by coastal storms and limit

water levels in the basin, thereby reducing flood risk for low lying coastal areas within the basin. Key characteristics of a storm surge barrier are that it allows for navigation to transit the barrier, and it maintains tidal exchange between the ocean and the newly created inner basin during normal hydrometeorological, i.e. non-storm, conditions. Some examples of storm surge barriers are the IHNC storm surge barrier (New Orleans, LA), the New Bedford storm surge barrier (New Bedford, MA) and the Eastern Scheldt storm surge barrier (The Netherlands). For more examples one is referred to Enclosure 2.

Tide gates[1] or tide gate complexes are generally considered to show some similarity to storm surge barriers, with the exception that they do not provide for navigation. These structures provide a barrier between the ocean and a waterbody at a location that is considered or designed to be non-navigable. In most instances, such locations are inherently shallower in depth and smaller in span compared to storm surge barriers. As such, without any clearances needed to accommodate vessels tide gates, or tide gate complexes are in general relatively smaller structures and allow for tidal flow exchange or discharge of stream flows during normal conditions. These structures include operable gates that can be closed temporarily during storm conditions to impede storm surge and limit water levels in the waterbody behind it and provide flood risk reduction. In some instances, tide gates are accompanied by a pump station that is operated in the event of gate closure to discharge streamflows from the upstream waterbody and maintain safe water levels. E.g. the 17th Street Canal Closure and Permanent Pump Station complex (New Orleans, LA).

Tidal locks or tidal lock complexes[2] are structures that have the primary function to allow for the passage of vessels between tidal and non-tidal water and where operation is affected by the state of the tides. Notwithstanding the fact that some water exchange occurs with every lock cycle, these structures are generally not designed to accommodate tidal flow exchange between the ocean and inland waterbody. When constructed in areas prone to storm surge and flood risk, coastal locks can be designed to tie in to, and be an integral part of the perimeter flood risk reduction system.

1.2.2 Structural CSRM Measures as Part of the HAT Study

For the alternatives currently being analyzed as part of the HAT Study, the structural CSRM measures do not include closure dams and tidal lock complexes. Due to the elimination of the tidal exchange, closure dams and tidal locks are considered not to be aligned with the planning objectives. Therefore, structural CSRM measures considered for this study include storm surge barriers, tide gate complexes, and more typical perimeter-based structural measures such as

[1] In some instances, tide gates are referred to as flood gates. However, flood gate is a more general term for a flood control structure that is not necessarily situated in the coastal plain. The term can be used to describe gated structures along rivers or structures on land that close off openings in flood risk reduction systems, hence tide gate is the preferred term here to refer to a coastal barrier as described herein.

[2] In some instances, tidal locks are included within closure dams or storm surge barriers and as such "hybrid" coastal barriers that include components or characteristics of the four primary types exist.

floodwalls, levees, seawalls, deployable flood barriers, etc. Non-structural measures are considered too but not discussed herein.

1.3 Organization

The engineering analysis and supporting documentation for the preliminary studies completed in support of the New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study are extensive. Therefore, the engineering work products that document the analyses of the alternative structural CSRM measures have been divided into three parts. Two reports discuss the navigable storm surge barriers that are part of the five HAT Study Alternatives and one report discusses the structural CSRM measures that are the essential "building blocks" of the perimeter-based risk reduction systems. The documentation on the storm surge barriers has been presented in two separate volumes, where the first volume describes navigable storm surge barriers that exceed a span of 1,500 ft; the second volume describes the barriers smaller than 1500 ft.

This report is R001 (Volume 1) of a series of reports, listed in Table 1-1, that document the preliminary engineering studies and analyses completed in support of the feasibility of the New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study.

Table 1-1: Reports in support of the Engineering Studies and Analyses for the HAT Study Alternatives.

ID	Title	Contents/Subject
R001	Conceptual Design for HAT Study Navigable Storm Surge Barriers – Volume 1	Conceptual Design for the following Storm Surge Barriers: Outer Harbor, Verrazano Narrows, Arthur Kill, Kill van Kull, Jamaica Bay, Hackensack River and Throgs Neck.
R002	Conceptual Design for HAT Study Navigable Storm Surge Barriers – Volume 2	Conceptual Design for the following Storm Surge Barriers: East Chester Creek, Westchester Creek, Bronx River, Flushing Creek, Newtown Creek, Gowanus Canal, Sheepshead Bay and Gerritsen Creek
R003	Shore Based Measures	The Structural Coastal Storm Risk Management (CSRM) measures evaluated as part of the HAT Study other than the Storm Surge Barriers

1.3.1 Report Purpose and Reader's Guide

This report documents the general design criteria and first conceptual designs for seven (7) navigable storm surge barriers, which are part of the storm surge barrier alternatives currently considered under the New York-New Jersey Harbor and Tributaries (NYNJHAT) Study for Coastal Storm Risk Management. The storm surge barriers are listed in Table 1-2 and presented on a map in Figure 1-1.

The purpose of this document is to provide the preliminary engineering basis to support the evaluation and development of conceptual designs for seven (7) navigable storm surge barriers, including the minimum practical dimensions of the barrier openings and preliminarily selection of gate types. The barrier openings consist of a navigable passage(s) and auxiliary flow openings. Five out of the seven storm surge barriers discussed in this report accommodate deep draft maritime traffic as well as shallow draft and/or small boats[3]. Deep-draft navigation refers to channel depths greater than 15 feet (ft) and channels that can accommodate and are frequented by larger, more heavily laden ocean-going ship traffic (EM 1110-2-1613). The investigations detailed within this report document the analyses completed for the barriers provided in Table 1-2. The remainder of the (smaller) storm surge barriers are discussed in Volume 2.

Table 1-2: Storm Surge Barriers - HAT Study Alternatives.

Location	HAT Study Alternative	Report Section providing Conceptual Design Summary
Verrazano Narrows Storm Surge Barrier	Alternative 3A	5
Throgs Neck Storm Surge Barrier	Alternative 2 and 3A	6
Arthur Kill Storm Surge Barrier	Alternative 3A and 3B	7
Outer Harbor Storm Surge Barrier	Alternative 2	8
Kill van Kull Storm Surge Barrier	Alternative 3B	9
Jamaica Bay Storm Surge Barrier	Alternative 3A, 3B and 4	10
Hackensack River Storm Surge Barrier	Alternative 4	11

[3] Per EM 1110-2-1206 small boats are classified as recreational craft, fishing boats, or other small commercial craft with lengths less than 100 ft (31 m).



Figure 1-1: Plan overview of all Storm Surge Barriers included within the HAT Study Area

Section 2 provides a brief overview of the Study and the locations of the storm surge barriers discussed herein. Section 3 provides a general overview of the preliminary design criteria and associated design considerations, which is subsequently used to elaborate on the conceptual design development for each storm surge barrier. Sections 5 through 11 provide a conceptual design summary of each storm surge barrier (see also Table 1-2). Recommendations for further study are documented in Section 12.

1.3.2 Scope and Limitations

The A/E has been tasked with establishing a conceptual layout and design of the storm surge barriers that are part of the Feasibility Study's alternatives. The A/E is to propose suitable gate types (e.g. flap gates, sector gates, vertical lift gates) that will accommodate navigation and other site-specific constraints. The level of detail of the proposed concepts is commensurate with that of a feasibility study. Given the size of the study area and potential number and locations of storm surge barriers, the study level is very conceptual. The A/E will define the approximate anticipated size of the gate openings, heights and general geometry of the overall storm surge barrier structures. The number and size of the navigable gates will be based on federal navigation channel

dimensions, or in accordance with USACE-NAN project-specific directions. The approximate number and size of flow gates will be based on tidal discharge requirements provided by USACE-NAN. The objective at this stage is to provide USACE-NAN a description and early conceptual design that identifies the basis for key geometric characteristics (e.g. height, width, and depth of openings). This will enable the USACE-NAN to use a parametric cost model to estimate the approximate cost of construction of the storm surge barriers.

2 HAT STUDY – STORM SURGE BARRIERS OVERVIEW

2.1 General Description of a Storm Surge Barrier with Navigable Passage

Mooyaart and Jonkman (2017) provide general design considerations and an overview of navigable storm surge barriers based on data and design documentation review of a select set of constructed storm surge barriers throughout the world. They also provide a general description of a storm surge barrier where a typical layout contains three elements; a gated section, a dam section and a navigable passage. The gated sections, as noted in section 1.2, are a key characteristic of a storm surge barrier as it allows for navigation to transit the barrier, and it maintains tidal exchange between the ocean and the inner basin. A navigable passage can either be established with a lock or with a gated navigable opening. The difference is that a lock passage is usually closed during normal operational conditions and only opens for the passage of vessels; a gated navigable passage is usually open for free navigation passage and only closed during the occurrence of a storm surge event. Figure 2-1 below provides a schematic plan view of a navigable storm surge barrier. The navigable passage is schematically shown as a gated navigable opening not as a lock, since the storm surge barriers studied under the HAT Study require minimal interruptions of maritime traffic except during storm surge events. Figure 2-1 schematically shows a total of three (3) auxiliary flow gates; however the storm surge barriers discussed herein may have fewer or many more. Both navigation and tidal flow exchange can be provided through the navigable passage opening.

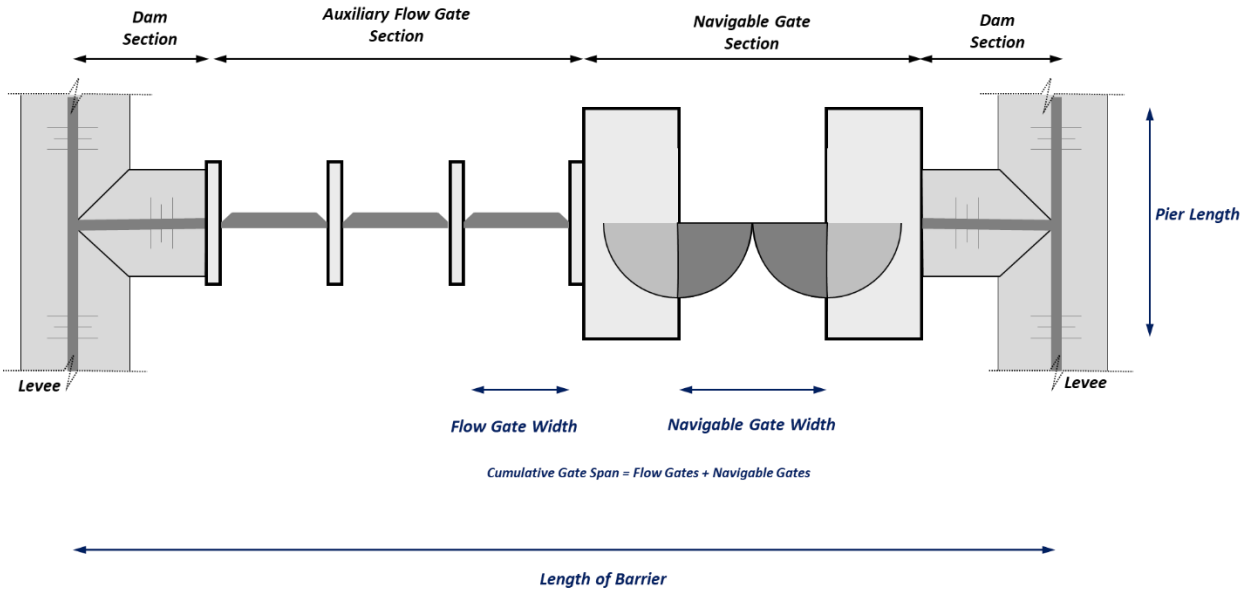


Figure 2-1: Schematic plan view of a Storm Surge Barrier. Modified after (Mooyaart and Jonkman 2017)

2.2 Storm Surge Barrier Locations

2.2.1 Siting

The proposed locations of the storm surge barriers have been informed by previous analysis completed by USACE-NAN. Although a full siting study has not been completed it is important to note that many of the storm surge barriers discussed herein are part of multiple HAT Study Alternatives (see Table 1-2) and that each alternative is evaluated amongst a total of five (5) alternatives. The overall goal is to compare the HAT Study Alternatives amongst each other and select and advance feasible alternatives for further study and design. During the next phase a more detailed evaluation and complete siting study of the selected storm surge barriers is recommended.

At this time, the siting of the storm surge barriers is based on the following assumptions and basic principles:

- Span across the waterbody to connect the perimeter-based flood risk reduction system from and cross the federal navigation channel(s),
- The geologic and geotechnical site conditions are assumed to be fairly uniform at each site and as such no specific area or corridor is preferred to minimize foundation cost,
- When practicable, the conceptual alignment should favor shallower portions of the waterbody/inlet to minimize foundation depths to the extent practical,
- When practicable, the start and end the storm surge barrier alignments should favor sheltered coastlines to reduce overall wave energy exposure, and tie-in locations are suitable for landward extension of the storm surge barrier structure to connect to the shore-based perimeter flood risk reduction system,
- When practicable, the alignment should minimize the number of conflicts with subaqueous or buried utilities,
- Provide a conceptual design that minimizes the total length and changes in orientation to the extent practical, since as a general assumption, it is presumed that additional length or changes in orientation would increase the overall cost of the storm surge barrier system[4].

Following the above general principles, the locations of the storm surge barriers discussed herein are for the most part, determined by the extent and location of the perimeter risk reduction systems and have been provided by USACE. The following sections provide a brief overview of the locations for the navigable storm surge barriers that all exceed 1,500 ft and arediscussed herein:

- Verrazano Narrows
- Throgs Neck
- Arthur Kill

[4] Without a complete siting study and evaluation of alternatives for the Verrazano Narrows storm surge barrier (inclusive of preliminary cost estimates) this can only be regarded as a preliminary assumption that will have to be verified at a later stage. Further analysis could prove that a longer barrier with less gates or with a lower design elevation could be more cost effective.

- Outer Harbor
- Kill van Kull
- Jamaica Bay
- Hackensack River

Additional maps that show the locations are provided in Appendix B.

2.2.2 Verrazano Narrows

The Verrazano Narrows storm surge barrier is situated where the Hudson River transitions into Raritan Bay. The proposed storm surge barrier spans from Staten Island to Brooklyn, NY just upriver from the Verrazano-Narrows Bridge. The distance between the two shores is approximately 4800 ft (0.9 miles). Water depths vary, with depths ranging from 30ft to 50ft outside the federal navigation channels, and depths exceeding 70ft at certain locations within the channel. Depths vary considerably across the Verrazano Narrows, with the deepest point exceeding 150ft (See Appendix B). Depths are shallower and more consistent at locations further north of the Verrazano Narrows Bridge, and these locations are preferred.

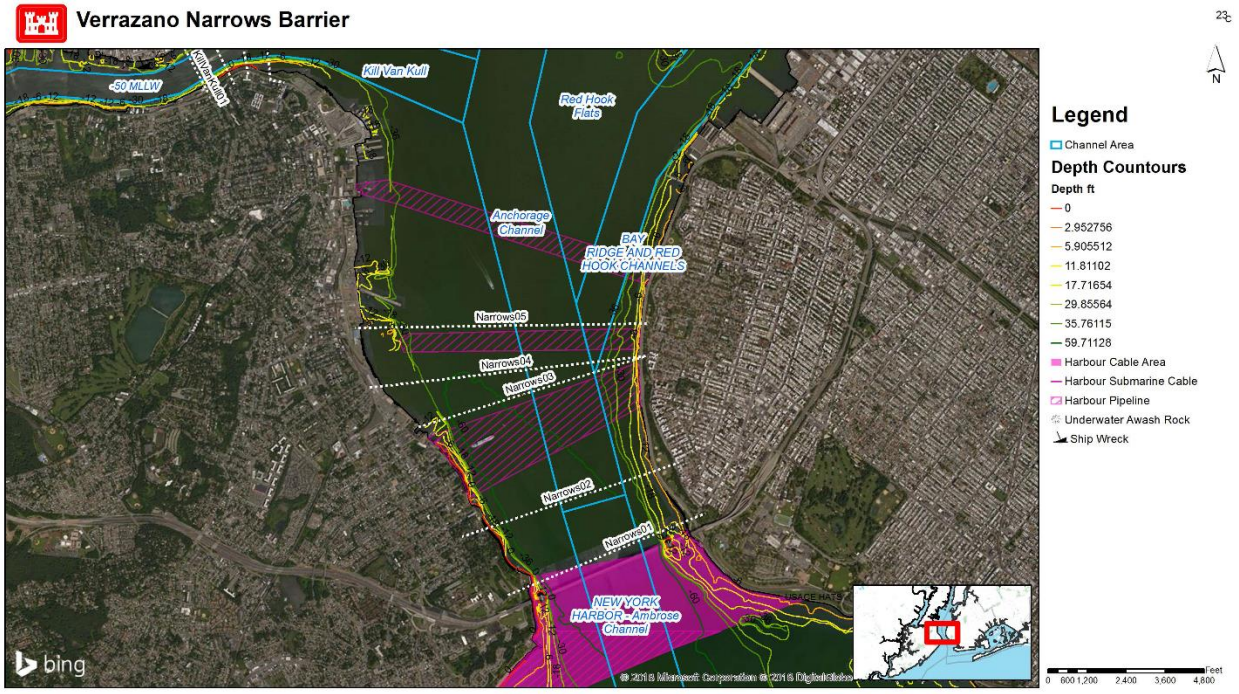


Figure 2-2: Area of interest for the Verrazano Narrows Storm Surge Barrier which spans from Staten Island (NY) to Brooklyn (NY).

2.2.3 Throgs Neck

The Throgs Neck storm surge barrier is located where Long Island Sound transitions into the East River. The proposed storm surge barrier would span the entrance to the East River at Throgs Neck, from Westchester, NY to Queens, NY. The distance between the land masses is approximately 4500 ft (0.85 miles); water depths vary from 40ft to 50ft (further site details and characteristics are provided in Section 3). Preliminarily, the proposed storm surge barrier is sited along Transect ThrogsNeck04, as shown in Figure 2-3. At this location water depths are considered most favorable compared to the other transects, while this transect provides for a relatively short tie-in to high ground on land.

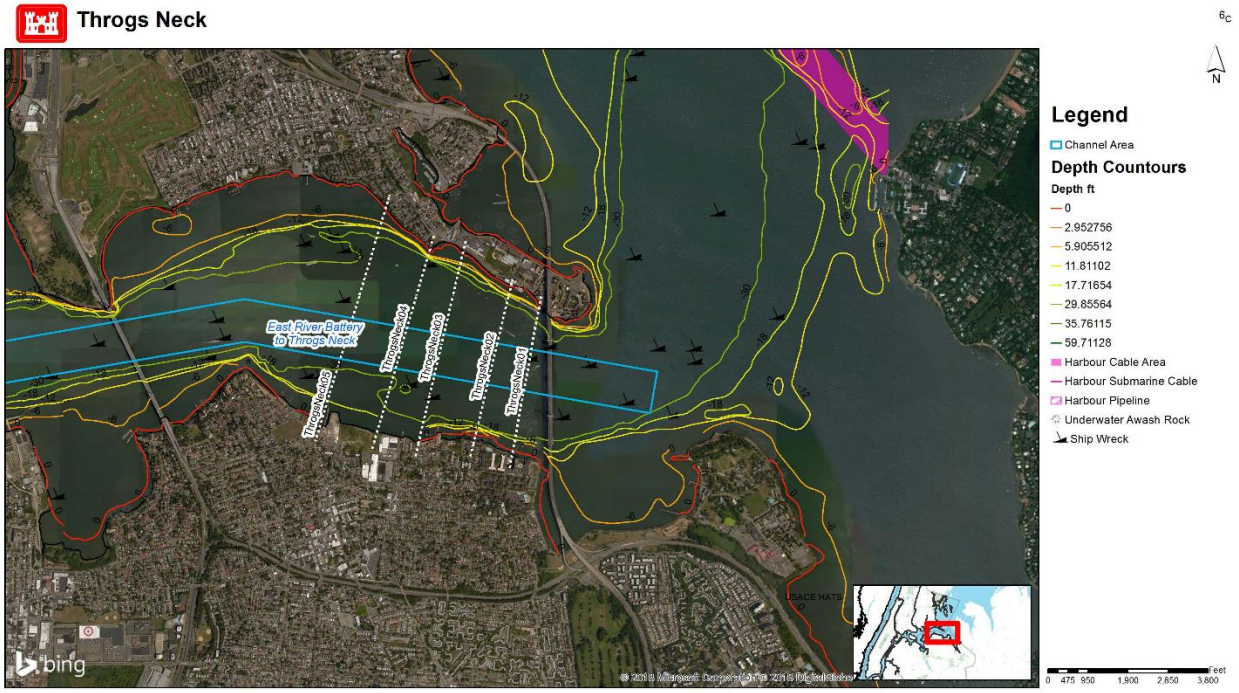


Figure 2-3: Area of interest for the Throgs Neck Storm Surge Barrier which spans from Westchester (NY) to Queens (NY).

2.2.4 Arthur Kill

The Arthur Kill storm surge barrier is located where the Arthur Kill transitions into Raritan Bay. The proposed storm surge barrier would span Arthur Kill from Woodbridge, NJ to Staten Island, NY. The distance between the barrier island land masses is approximately 1500ft (0.3 miles). Water depths vary from -20 to -45ft. The storm surge barrier is preliminarily sited along Transect ArthurKill03, as shown in Figure 2-4.

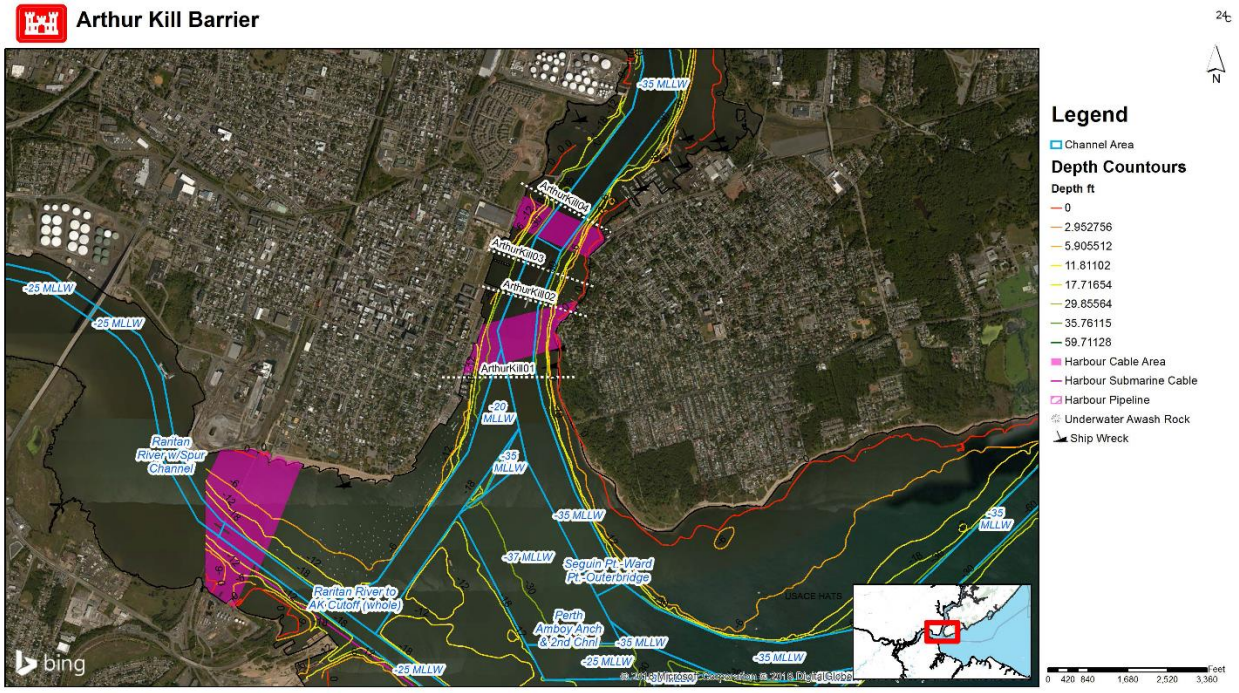


Figure 2-4: Area of interest for the Arthur Kill Storm Surge Barrier which spans from Woodbridge (NJ) to Staten Island (NY).

2.2.5 Outer Harbor

The Outer Harbor storm surge barrier is located where the Atlantic Ocean transitions into Raritan Bay, at the apex of the New York Bight. The proposed storm surge barrier would span the entrance to the New York and New Jersey Harbor, from Breezy Point on the Rockaway peninsula to Sandy Hook. The distance between the barrier island land masses is approximately 29,000 ft (5.5 miles). Water depths vary, with depths generally ranging from 20ft to 30ft outside the federal navigation channels. Within the Ambrose Channel and Sandy Hook channel depths exceed 60ft, with the deepest point in the Ambrose navigation channel exceeding 70ft. This storm surge barrier will cross three navigation channels (Sandy Hook Channel, Ambrose Channel and Rockaway Inlet). The Outer Harbor storm surge barrier is preliminarily sited along Transect Ambrose 03, as shown in Figure 2-5.

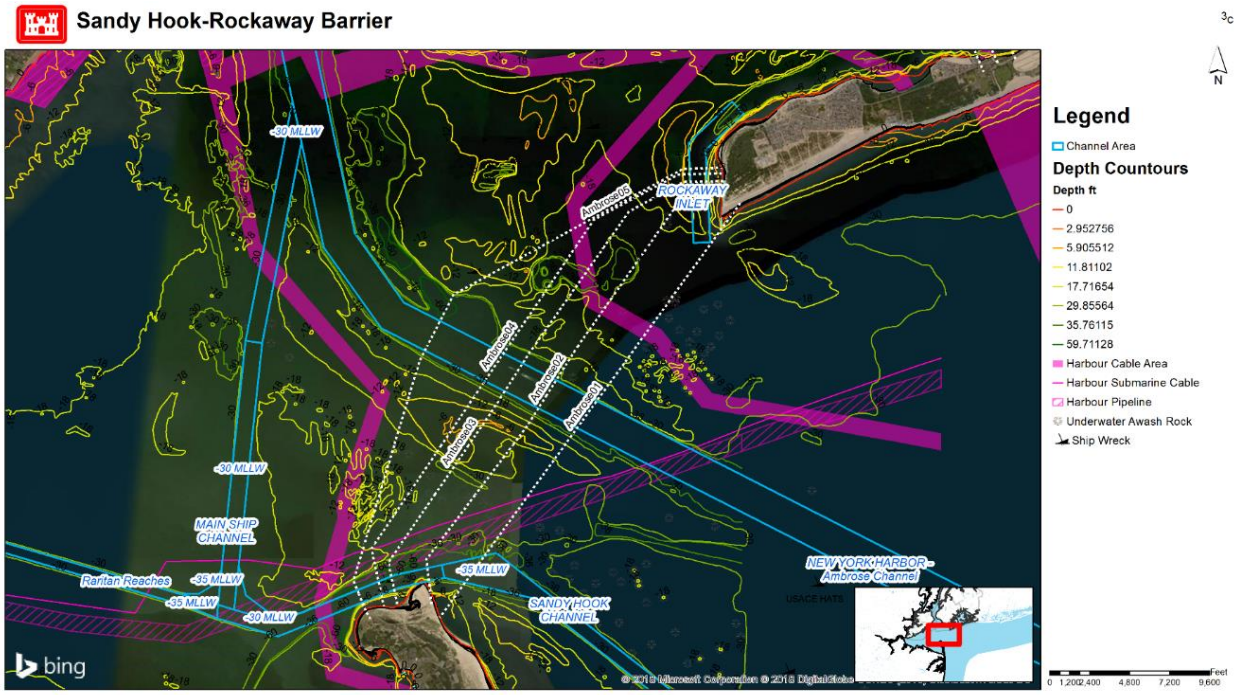


Figure 2-5: Area of interest for the Outer Harbor Storm Surge Barrier which spans from Sandy Hook (NJ) to Rockaway Point (NY).

2.2.6 Kill van Kull

Kill Van Kull Channel connects Newark Bay to the Upper Bay in New York Harbor, cutting between Bayonne, NJ and Staten Island, NY. The Kill Van Kull storm surge barrier is located where the Kill Van Kull transitions into Upper Bay. The distance between the land masses at this location is approximately 2400 ft (0.45 miles). Water depths vary from 40ft to 55ft. This storm surge barrier is preliminarily sited along Transect KillVanKull04, as shown in Figure 2-6. The 04transect avoids industrial development on the north side of the channel to the greatest extent possible, while also providing for the inclusion of additional auxiliary flow gates to maximize flow openings for tidal exchange.

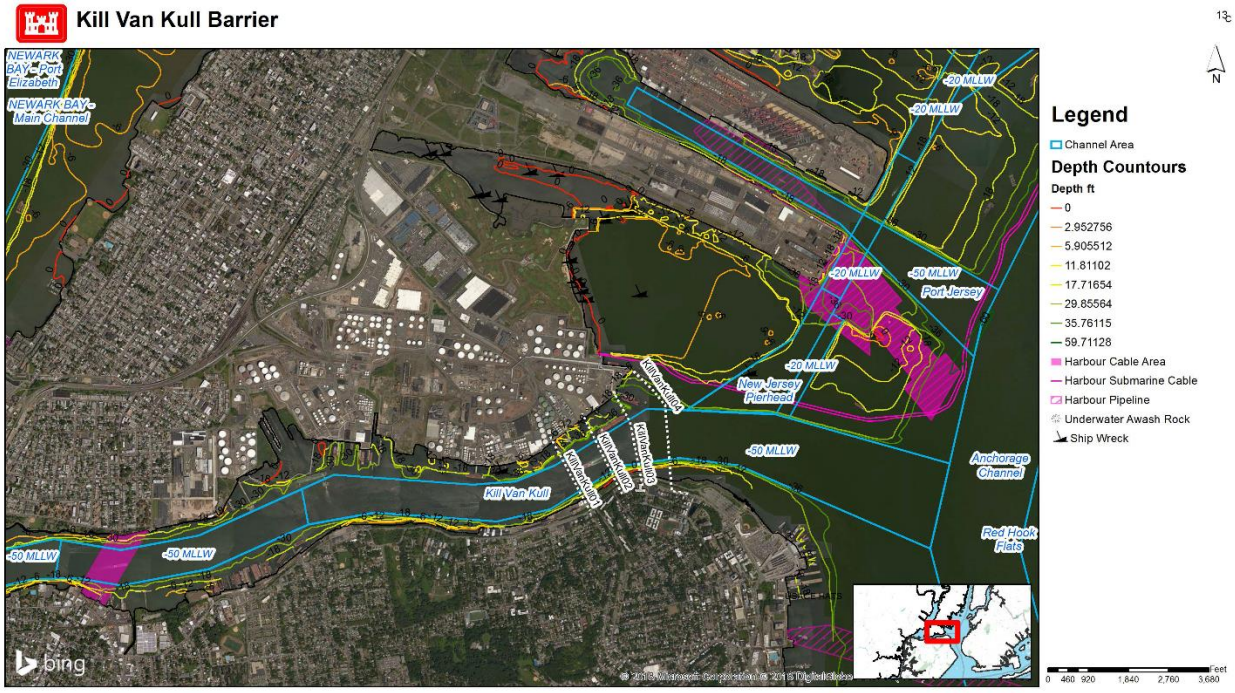


Figure 2-6: Area of interest for the Kill Van Kull Storm Surge Barrier which spans from Bayonne (NJ) to Staten Island (NY).

2.2.7 Jamaica Bay

The Jamaica Bay storm surge barrier is located where Rockaway Inlet enters Jamaica Bay. The proposed storm surge barrier would span the entrance to Jamaica Bay, from Barren Island, NY, to Rockaway, NY in the vicinity of the Gil Hodges Memorial bridge, as shown in Figure 2-7. At this location, the distance between land masses is approximately 3,500ft. (0.65 miles). Water depths vary from 20ft to 40ft. A cross section is provided in Section 3.

The preliminary location of the Jamaica Bay storm surge barrier has been informed by previous analysis completed by USACE-NAN, where the storm surge barrier location matches the preferred alternative from (USACE 2016). It is further noted that this proposed alignment also corresponds approximately to the alignments for Barrier Plans A, B, C-1, C-2, and C-3 investigated in the previous 1976 USACE-WES studies (USACE-WES 1976).

It is assumed that previous analyses and evaluations provide sufficient basis to site the storm surge barrier at this location and as such eliminates the need to revisit such analyses. The storm surge barrier is preliminarily sited along Transect JamBay03, as shown in Figure 2-7.

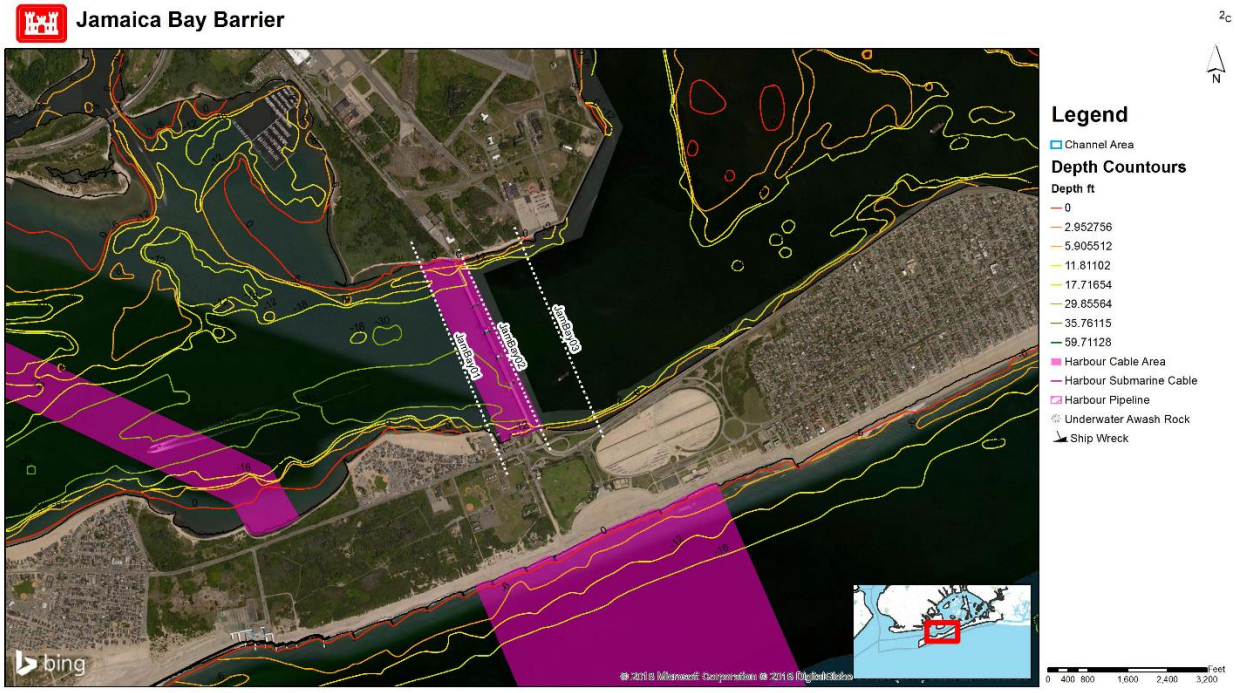


Figure 2-7: Area of Interest for Jamaica Bay Storm Surge Barrier

2.2.8 Hackensack River

The Hackensack storm surge barrier location is on the Hackensack River in New Jersey, just downstream and to the south of the Newark-Jersey City Turnpike. The distance between the banks is approximately 1600 ft (0.3 miles) at the proposed barrier location. Water depths vary from 20ft to 30ft. The proposed alignment corresponds closely to the alignment investigated in the previous USACE-NAN Hackensack River Basin Reconnaissance Report (USACE February 1989). In addition, dimensions, features and gate types proposed herein (see section 11) closely correspond to the recommended plan in 1989. It can also be noted that the proposed alignment is somewhat similar to that investigated in the previous Rebuild-by-Design Study (AECOM 2018) yet the alignment studied therein is situated approximately 0.65 miles downstream from the location proposed in this report. The Hackensack River storm surge barrier is preliminarily sited along the westernmost transect (Hackensak01) depicted in Figure 2-8.

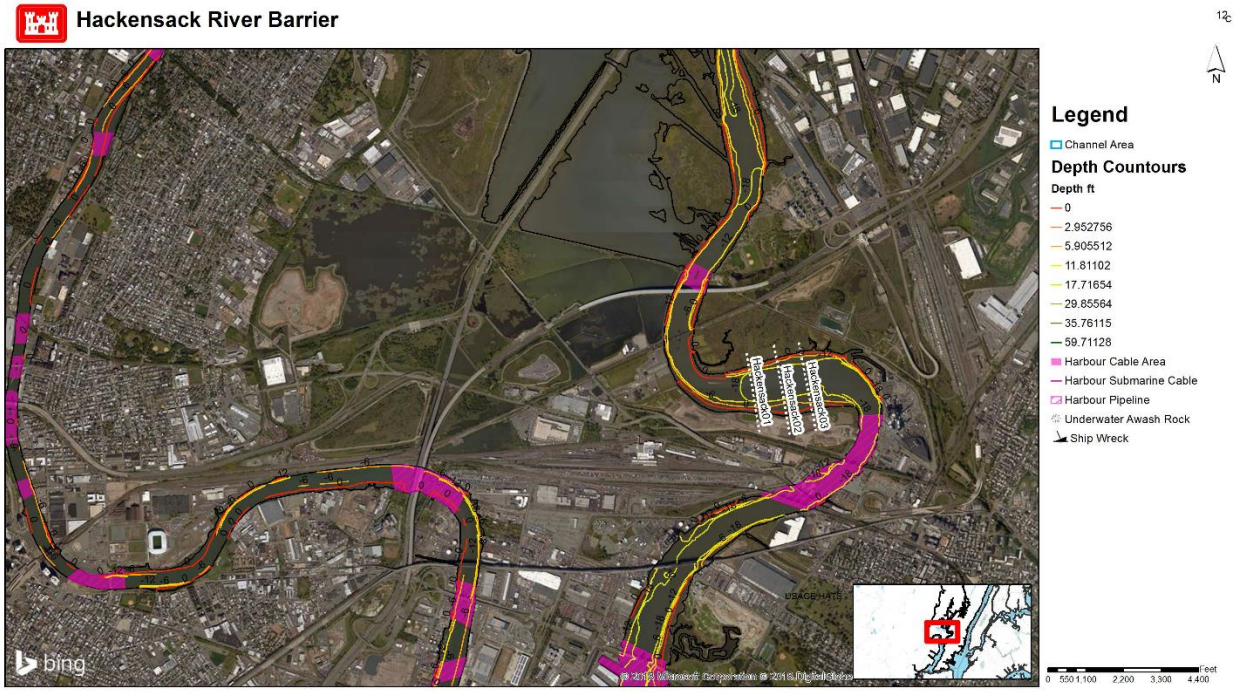


Figure 2-8: Area of interest for the Hackensack Storm Surge Barrier which spans the Hackensack river in New Jersey

2.3 Prior Studies and Reports

There are several visioning studies, reports and presentations that have addressed the concept of storm surge barriers for the larger New York Metropolitan Area and these have been used to inform the data presented herein. Amongst other relevant reports and publications include the numbered items below.

1) L. Smith (2005), Closing the Doors on Storm Surge, Coastlines, vol. 34, no. 1, pp. 6-7. (Smith 2005)
2) Bowman et al. (2004) Hydrologic Feasibility of Storm Surge Barriers to Protect the Metropolitan New York – New Jersey Region – Summary Report. (Bowman, et al. 2004)
3) Bowman, M., Hill, D., Buonaiuto, F., Colle, B., Flood, R., Wilson, R., Hunter, R. and Wang, J. (2008) 'Threats and responses associated with rapid climate change in Metropolitan New York', in M. McCracken, F. Moore, J. C. Topping (Jr.) (eds) *Sudden and Disruptive Climate Change: Exploring the Real Risks and How We Can Avoid Them*, Earthscan, London pp119–142 (Bowman, et al. 2008)

Concepts for the storm surge barrier at the Verrazano Narrows, Arthur Kill, Outer Harbor and Throgs Neck location in particular were presented at a seminar titled: "Against the Deluge: storm

surge barriers to Protect New York City", which was held on March 30th and 31st 2009 at the Polytechnic Institute of NYU:

4) Hill, D. (ed) (2011) *Against the Deluge: Storm Surge Barriers to Protect New York City,* Conference Proceedings, Polytechnic Institute of New York University, Brooklyn, NY, 30–31 March 2009, Annals of the New York Academy of Sciences, New York, NY (Against the Deluge: Storm Surge Barrier to Protect New York City 2013)

Other relevant papers and reports that address the topic more broadly for the New York metropolitan area include:

5) Dircke, P. T. M., T. H. G. Jongeling, and P. L. M. Jansen. 2012. "Navigable Storm Surge Barriers for coastal cities: An overview and comparison." In *Climate Adaptation and Flood Risk in Coastal Cities*, by J Aerts, W Botzen, M Bowman, P Ward and P Dircke, 201-223. New York: Earthscan. (Dircke, Jongeling and Jansen 2012)
6) Aerts, J.C.J.H., W.J. Botzen, and H De Moel. 2013. *Cost Estimates for Flood Resilience and Protection Strategies in New York City.* New York: Annals of the New York Acadamy of Sciences. (Aerts, Botzen and De Moel 2013)

Papers and reports that provide an overview of gate types utilized in storm surge barriers and/or address the general approach to assess the feasibility of storm surge barriers in particular settings include:

7) PIANC. 2006. *Design of Movable Weirs and Storm Surge Barriers.* Brussels, Belgium: PIANC. (PIANC 2006)

8) Mooyaart, Leslie F, Sebastiaan N Jonkman, Peter A. L. de Vries, Ad van der Toorn, and Mathijs van Ledden. 2014. "Storm Surge Barrier: Overview And Design Considerations." Edited by Patrick J. Lynett. *Coastal Engineering Proceedings.* Seoul, Korea: Coastal Engineering Research Council. 808. (L. F. Mooyaart, et al. 2014)

9) Mooyaart L.F., Jonkman S.N., Overview and Design Considerations of Storm Surge Barriers. ASCE Journal of Waterway, Port, Coastal, and Ocean Engineering, Vol. 143, Issue 4. (Mooyaart and Jonkman, Overview and Design Considerations of Storm Surge Barriers 2017)

10) van Ledden, Mathijs, A.J. Lansen, H.J. de Ridder, and B. Edge. 2012. "Reconnaissance level study Mississippi Storm Surge Barrier." Edited by Patrick Lynett and Jane McKee Smith. *Proceedings of 33rd Conference on Coastal Engineering.* Santander, Spain: Coastal Engineering Research Council. (van Ledden, et al. 2012)

For both the Jamaica Bay and Hackensack River storm surge barrier detailed feasibility studies were previously completed, as described below:

There are two previous studies that investigated storm surge barriers for Jamaica Bay:

- USACE-WES (1976). Technical Report H-76-14 Effects of Hurricane Surge Barrier on Hydraulic Environment, Jamaica Bay, New York - Hydraulic Model Investigation. Hydraulics Laboratory, US Army Engineer Waterways Experiment Station, September 1976. (USACE-WES 1976)

- USACE-NAN (2016). Atlantic Coast of New York, East Rockaway Inlet to Rockaway Inlet and Jamaica Bay. Draft Hurricane Sandy General Reevaluation Report and Environmental Impact Statement. Engineering Appendix A – 2, August 2016. (USACE 2016)

These documents have been used to inform the data presented herein and are referenced in the relevant sections. Albeit the 1976 USACE-WES report contains very useful data and is a valuable study in many regards, the storm surge barrier alternatives, all in very close vicinity to the Gil Hodges bridge, studied within that report had only a limited number of openings and would restrict the inlet to about one-third of the existing condition. Reduction in the cross-sectional area resulted in increased velocities within the navigation channel and a reduction in tidal amplitude. The HAT Study seeks to minimize the impact on tidal flow exchange and will investigate maximizing the number of additional flow openings.

Table 2-1: Jamaica Bay Barrier Plans from USACE 197 study – Openings and Gate Dimensions (USACE-WES 1976)

	Navigable passage (ungated)	Sill Elevation (MSL)	Aux. Flow Gate	Sill Elevation (MSL)	Number of Aux. Flow Gates	Flow area through barrier (sq. ft)
Plan 3	300ft	-33ft	75ft	-26ft	12	33,300
Plan 6	110ft	-33ft	75ft	-26ft	16	34,830

The USACE 2016 report included an evaluation of a number of barrier alignments and barrier alternatives. This document relies in large part on the considerations, data and findings from USACE 2016. One such finding is the storm surge barrier siting, discussed in the following section. An overview of the gates included within the C1-E alternative from USACE 2016 is presented below in Table 2-2. The conceptual design presented herein will include additional auxiliary flow gates to minimize the impact on flow exchange between the bay and the ocean.

Table 2-2: Preferred Storm Surge Barrier Plan from (USACE 2016) – Openings and Gate Dimensions

	Gated Navigable passage (2 total)	Sill Elevation (NAVD88)	Aux. Flow Gate	Sill Elevation (NAVD88)	Number of Aux. Flow Gates	Flow area through barrier (sq. ft)
C1-E	200	-30ft	100ft	-15ft	7	22,500

For the Hackensack River there is a USACE Reconnaissance Study from 1989 that addressed the concept of a storm surge barrier. In addition, the Draft Environmental Impact Statement (DEIS) and Feasibility Study for the Rebuild By Design – Meadowlands Project also included an alternative in the early stages that included a storm surge barrier.

These two documents have been used to inform the data presented herein.

- USACE-NAN, 1989, Reconnaissance Report – Hackensack River Basis, New Jersey. (USACE February 1989)
- AECOM, 2018, Subappendix F1 – Alternative 1 Development and Screening, For the Feasibility Study of Rebuild by Design Meadowlands Flood Protection Project. Report submitted to State of New Jersey Department of Environmental Protection, April 2018. (AECOM 2018)

3 GENERAL DESIGN CRITERIA – ABBREVIATED

As part of advancing a conceptual design for a storm surge barrier a number of criteria need to be established and defined, quantitatively where possible or qualitatively otherwise. Due to the preliminary nature of this feasibility study the following criteria should not be seen as comprehensive, all-encompassing or complete. Instead the requirements and criteria form the basis for an iterative design approach of which the feasibility analysis and conceptual design are the first phase. Decisions are highlighted that impact the conceptual design and where possible a discussion is included for issues that need to be addressed as the design advances.

The listed design criteria are based on qualitative data and desktop analysis to a level of detail commensurate with a feasibility study. In instances where limited data was available, assumptions were made based on engineering judgement, previous experience and/or the partial data that has been collected over the course of the feasibility study phase. The implications of such assumptions along with recommendations for further data collection and refined analyses to support the design are described at the end of this report.

3.1 Available Data

A review of meteorological, oceanographic, and vessel traffic conditions at the study site was performed to provide a basis for the evaluation and selection of the gate types and structures. The following were investigated:

- Existing navigation requirements: vessel traffic patterns, and vessel sizes, from which minimum practical channel widths for the gated navigable passage were preliminarily established (see also Enclosure 1)
- Storm surge elevations
- Wave climatology
- Local wind conditions
- Inlet Hydraulics and/or discharge regimes
- Bathymetry

Existing conditions for the study site were developed primarily from available data supplied by USACE. These data included:

- Coastal Hazards System (CHS) that includes NACCS storm surge and wave modeling
- Numerical modeling of normal hydrometeorological conditions with the use of the AdH model (in prep by ERDC)
- Geological maps and profiles for the project vicinity and or existing borings when readily available

3.2 System of Units and Reference

3.2.1 System of Units

US customary units shall be used.

3.2.2 Vertical and Horizontal Reference

The vertical datum for the project shall be NAVD88, Geoid 12B. All elevations throughout the report are referenced to NAVD88 Geoid12B unless otherwise stated. The horizontal datum shall be the North American Datum of 1983 (NAD83) State Plane.

3.3 Service Life

The storm surge barriers have various project components for which Life Cycle Design should be considered (ER 1110-2-8159). At this stage of the project (feasibility study) no such analysis has been performed. A minimum project service life of 100 years is preliminarily recommended as a result of the size and nature of the project. For certain project elements a shorter service life may be feasible.

For the storm surge barrier to meet the functional requirements regarding flood risk reduction (see Section 3.4) a period of analysis has been established that is shorter than the recommended project service life. The project will perform to meet the design criteria related to flood risk reduction in this document for a 50-year period spanning the years between 2030 and 2080. The project is to be designed for sea level rise, regional subsidence, and local settlement occurring for a 50-year planning horizon (to the year 2080). After such a time, to achieve the same level of risk reduction, the structures may have to be modified or improved (i.e. adaptive management may be necessary or structural improvements may be needed if the observed sea level rise exceeds the planning criteria). Design provisions will be required to accommodate such improvements if and when needed.

3.4 Basic Functional Requirements

The following basic functional requirements have been identified for the conceptual design of the storm surge barrier consistent with the overall objectives of the HAT Study:

- The storm surge barrier will provide a reliable structural measure as part of the HAT Study Alternatives (see Appendix A) to reduce the risk of coastal storm damage to coastal region behind it;
- The storm surge barrier will minimize impact to navigation and waterborne commerce;
- The storm surge barriers will minimize impact on the water exchange through the opening during normal operation (non-storm conditions) in order to minimize impacts on the inner basin environmental conditions; and
- The storm surge barrier will minimize the impact on upstream water levels during operation.

As a result of the above requirements the storm surge barrier concept will at its simplest consist of two principal parts; 1) moveable gates (both navigable gates and flow control gates) with associated gate support structures and 2) tie-in structures. For any system of movable gates, including navigable gates and flow control gates, there are three basic functional requirements:

- The moveable gates of the storm surge barrier will be able to open and close with a high degree of reliability;
- The gates will be sized and provide a range of motion suitable for purpose (e.g. navigation, flow conveyance); and
- The moveable gates and gate structures will be an integral part of the overall storm surge barrier structure and be designed such that they impede the coastal storm flood levels and minimize the risk of coastal storm damage to the region behind it.

3.5 Site Conditions

3.5.1 Federal Navigation Projects

All the proposed storm surge barriers discussed in this report cross federally authorized navigation channels. Channel dimensions are derived from the following resources.

- Controlling Depth Reports and Surveys (USACE 2018)
- Project Maps, Rivers & Harbors, Navigation Projects (USACE 1986).
- Nautical Charts (NOAA 2017).

The Verrazano Narrows, between Staten Island and Brooklyn, is the principal entrance to the New York and New Jersey Harbor, and is one of the busiest waterways in the United States. The proposed storm surge barrier at the Verrazano Narrows would intersect with Ambrose Channel Federal Navigation Channel. The proposed Outer Harbor storm surge barrier would intersect with three federal navigation channels, including Ambrose, Sandy Hook, and Rockaway Inlet. The authorized channel dimensions for all federal channels that intersect with the proposed storm surge barrier locations are provided in Table 3-1. For those locations where a varying authorized width is documented, the approximate width at the proposed storm surge barrier location is provided in the last column.

Table 3-1: Federal Navigation Channel Intersecting with the Proposed Storm Surge Barrier

Federal Navigation Project						
Storm Surge Barrier Location	**Name**	**Reach of Federal Channel**	**Authorized Project**			**Approx. Width at proposed location (ft)**
			Width (ft)	**Length (nautical miles)**	**Depth (feet MLLW)**	
Verrazano Narrows	Ambrose Channel	Reach D	2000	2.9	53	
Throgs Neck	East River	Reach K	175 to 1040	1.45	35	1,000
Arthur Kill	Arthur Kill	Outerbridge Reach	600 to 840	1.60	35	600
Outer Harbor	Ambrose Channel	Reach B	2000	4.2	53	
	Sandy Hook Bayside Reach	Partial Reach B - Bayside Reach	800	2.4	35	
	Rockaway Inlet	Jamaica Bay Reach A	1000	1.5	20	
Kill van Kull	Kill Van Kull Channel	Constable Hook Reach	2000 to 800	2.52	50	800
Jamaica Bay	Jamaica Bay Federal Navigation Channel	Reach B	1000 to 500	0.71	18	500
Hackensack River	Newark Bay, Hackensack & Passaic Rivers; Hackensack River, New Jersey	Partial Reach D - Route #3 Highway:	200	0.34	15	

For the Hackensack River it should be noted that the constructed portion of the federally authorized navigation channel terminates at the end of the Marion Reach and Turning Basin (Reach C); this is located downriver of the Newark-Jersey Turnpike Bridge and downriver of the proposed storm surge barrier. Reach D of the federal channel continues upstream at a width of 200ft and an authorized depth of 15ft below MLLW. The authorized depth of 15ft was never constructed; however, it is used as a design criterion.

Additionally, it is noted that no deepening of the federal channels is assumed to occur within the study area during the project service life. This assumption is based on the plan formulation and the description of the Future Without Project (FWOP) conditions of the HAT Study.

3.5.2 Subaqueous Utilities

Buried utilities, including pipelines and cables, cross the project areas for all storm surge barrier discussed herein, except for the Throgs Neck storm surge barrier[5]. Appendix B provides plan views of approximate utility locations based on NOAA chart information. Storm surge barriers are sited with consideration of avoiding utility conflicts to the extent practical. Further exploration will be needed to identify marked and unmarked utilities; for selected storm surge barrier locations, utilities will have to be located precisely and potentially relocated prior to construction. Further site investigations are recommended at later stages of the study and future phases of the design process.

3.5.3 Bathymetry and Topography

Bathymetric Data was obtained from the NOAA DEM as well as from USACE-NAN channel survey conditions. The bathymetric data was used to generate channel cross-section profiles and assess existing flow areas and flow area restrictions as a result of the conceptual design and geometry of the storm surge barrier openings. Appendix B bathymetric maps and cross-sectional bathymetric profiles are presented in the conceptual design summary sections for each storm surge barrier.

3.6 Operational Hydrodynamic Conditions

3.6.1 Hydrological Characteristics

The characteristics of the hydrodynamic circulation of the New York Harbor area, Newark, Hudson River, East River and Throgs Neck's connection to the Long Island Sound are well documented in previous studies. Aerts et al. provides a brief but clear description, of the hydrological characteristics of the area of interest (Aerts, Botzen and De Moel 2013). The description is provided hereafter but shortened for brevity, with parameters converted to US customary units.

Numerous descriptive and modeling studies have described the hydrology and hydrodynamic circulation of the Hudson Estuary, the NY Harbor area and the NY Bight (for an overview, see Blumberg, Khan and St. John 1999). The New York Harbor is located at the mouth of the Hudson River, which discharges to the ocean via New York Bay and the Verrazano Narrows. This area is bounded by Brooklyn in the east and Staten Island in the west. The second connection of the

[5] Based on preliminary desktop analyses only

Hudson River/ New York Bay to the Atlantic Ocean is via the East River and Long Island Sound. Long Island Sound is an estuary of about 100 miles with a mean depth of 65ft.

The highest freshwater inflow to the NY Bay area is provided by the Hudson River. The river has a length of 315 miles that originates at 'Lake Tear of the Clouds' in the Adirondack Mountains, and drains a watershed of about 14,000 square miles. The long-term annual mean discharge is about 21,900 cfs., with a peak discharge in April (mean monthly flow ~42,400 cfs). Minimum flows occur in August (discharge ~6,700 cfs.). The Hudson River has an average depth of 30-50 ft (Geyer and Chant, 2006), and is influenced by the ocean tide, which can propagate upstream about 180 miles. Other fresh water sources are from water treatment plants and storm-water runoff. (Rozenzweig, et al. 2007). Blumberg et al. estimated a runoff of 4,025 cfs., from 110 wastewater treatment plants in their hydrodynamic modeling framework (Blumberg, Khan and St. John 1999). Additional runoff can be produced by rainfall and storm water discharges.

The harbor receives a significant sediment load from the Hudson River with an average of 1 million tons per year (HydroQual 2007). Siltation problems occur in the lower Hudson estuary where the river widens as it empties into New York Harbor and the Lower New York Bay. Furthermore, on the southern coast of Long Island, a westward migration of sand and a northward migration along the New Jersey coast contribute further sedimentation problems to the NY Bay area, which requires periodic dredging to maintain the depth of navigation channels.

The following hydrologic description of the Hackensack River is based on the report titled *Hackensack River Basin, New Jersey - Reconnaissance Report* (USACE February 1989).

The Hackensack River Basin is situated in the northeasterly part of the State of New Jersey and the most southerly section of New York State, west of the Hudson River. The Hackensack River and its tributaries are located primarily in Bergen County, New Jersey with portions in Hudson County, New Jersey and Rockland County, New York. Tidal flooding occurs along the Hackensack River and its tidal tributaries, specifically in the Hackensack Meadowlands 1ocated in Bergen County, New Jersey. The Hackensack River Basin drains 197 square miles. The river originates in the northern Palisades in Rockland County, NY and runs 50 miles to its mouth in Newark Bay. The river is tidal and navigable from the mouth for 21.5 miles upstream; at this point there is a tidal barrier.

The Hackensack River estuary is of the coastal plain type, formed when rising ocean levels inundated a former glacial lake bed and the river that fed it. The depth is shallow when compared to the width, and the river depth increases gradually going downstream towards Newark Bay. The Hackensack is well mixed vertically and laterally but has a horizontal salinity gradient from its mouth to the upstream areas. The ratio of tidal prism to fresh water inflow is high. The river is tidal as far upstream as river mile 21.5.

The same report also provides a peak discharge vs. frequency curve for USGS gaging station #01378500: Hackensack River at New Milford, N.J. This station is approximately 15 miles upstream, yet the discharge in the Hackensack River is largely correlated to water release from the

Oradell reservoir. 10%, 2% and 1% Annual Exceedance Probability (AEP) river discharges correspond to 3800 cfs., 5800 cfs. and 6870 cfs. respectively.

The Passaic River formed as a result of drainage from a massive proglacial lake that formed in Northern New Jersey at the end of the last ice age, approximately 13,000 years ago. The Passaic River is approximately 80 miles long and located in northern New Jersey. The river in its upper course flows in a highly circuitous route, meandering through the swamp lowlands between the ridge hills of rural and suburban northern New Jersey, in its lower portion, it flows through the most urbanized and industrialized areas of the state, including along downtown Newark. Annual exceedance probability of river flows have been determined using USGS *StreamStats* tool (USGS 2018). 10%, 2% and 1% Annual Exceedance Probability (AEP) peak flow river discharges correspond to 19,100 cfs., 27,400 cfs. and 31,500 cfs. respectively.

3.6.2 Astronomical Tides

Information on tidal water levels is obtained from NOAA's Center for Operational Oceanographics Products and Services (CO-OPS) website (National Oceanic and Atmospheric Administration n.d.). Tidal data for each location are derived from the NOAA gauges provided in Table 3-2 and used for the conceptual design.

Table 3-2: Storm Surge Barrier Locations and Tidal Gauges

Location	Closest Tide Gauge	Station Number
Verrazano Narrows	The Battery	8518751
Throgs Neck	Kings Point	8516946
Arthur Kill	Sandy Hook	8531680
Outer Harbor	Sandy Hook	8531680
Kill van Kull	Bergen Reach West	8519483
Jamaica Bay	Sandy Hook	8531680
Hackensack River	Amtrak RR Swingbridge, Hackensack River, NJ	853069

The tidal datums for each of these gauges are provided in the tables included within Appendix C.

3.6.3 Sea Level Change

Sea level change values are based on the USACE moderate scenario (ER 1100-2-8162) and were provided by USACE-NAN for three tidal gauge stations: Sandy Hook, The Battery, and Kings Point. These values are provided in Table 3-3.

Table 3-3: Sea Level Change per USACE's Moderate Scenario, in feet.

Year	Station 8561680, Sandy Hook	Station 8518750, The Battery	Station 8516945, Kings Point
1992 (base year)	0	0	0
2030	0.62	0.47	0.42
2080	1.82	1.49	1.37

Sea level change values from the station nearest to the proposed storm surge barriers will be applied to obtain design water levels for both operational and extreme conditions. The Sandy Hook Station data will be utilized for the Outer Harbor, Arthur Kill, Verrazano Narrows and Jamaica Bay storm surge barrier. The Battery station data will be utilized for the Kill van Kull and Hackensack River storm surge barrier and the Kings Point station data will be utilized for the Throgs Neck storm surge barrier.

3.6.4 Tidal Flows and Current Magnitudes

Tidal flow characteristics are obtained from modeling work performed by ERDC (USACE ERDC 2019). USACE-ERDC analyzed the HAT Study Alternatives and the impacts on normal tidal conditions and circulation. This modeling effort focused mainly on the impacts of the larger storm surge barriers discussed herein. A data summary is provided here and is based on the statistical analysis of model output for the 1995 calendar year for a cross-section spanning the storm surge barrier sites. The values presented in Table 3-4 show tidal fluxes and are model results rounded to the nearest thousand cfs and averaged between ebb and flood flows for base conditions, i.e. without the storm surge barriers in place.

Table 3-4: Tidal Flow

Location	Mean Tidal Flow		Maximum Tidal Flow	
	(m3/s)	(cfs)	(m3/s)	(cfs)
Verrazano Narrows	16,100	568,000	36,400	1,286,000
Throgs Neck	5,800	204,000	13,000	459,000
Arthur Kill	1,200	43,000	3,200	113,000
Outer Harbor	39,400	1,391,000	115,900	4,092,000
Kill van Kull	2,400	83,000	7,700	274,000
Jamaica Bay	3,700	129,000	10,300	363,000
Hackensack River	900	32,000	2,500	87,000

The values presented in Table 3-5 show tidal surface currents at predetermined output locations and are model results averaged between ebb and flood flows for base conditions, i.e. without the storm surge barriers in place. Values are also provided in knots for ease of reference and in support of interpretation by the navigation community and marine engineering discipline.

Table 3-5: Tidal Currents

Location	Output point	Mean Tidal Current Magnitude		Maximum Tidal Current Magnitude	
		(knts)	(ft/s)	(knts)	(ft/s)
Verrazano Narrows	S2	1.3	2.2	2.7	4.6
Throgs Neck	V4	1.0	1.8	2.6	4.4
Arthur Kill	S1	0.7	1.2	1.5	2.5
Outer Harbor (Sandy Hook Channel)	V1	1.5	2.6	3.6	6.0
Outer Harbor (Ambrose Channel)	V2	1.2	2.0	2.8	4.8
Outer Harbor (Rockaway Inlet)	V3	1.0	1.8	2.5	4.2
Kill van Kull	T1	0.7	1.2	2.2	3.6
Jamaica Bay	S3	0.8	1.3	1.9	3.2
Hackensack River	R1	0.9	1.5	2.1	3.6

3.7 Extreme Hydraulic Conditions

Extreme Hydraulic conditions, both water level and waves, are based upon storm surge modeling results completed for the NACCS Study (USACE 2015). Annual exceedance probability statistics for water level and wave characteristics were supplied by the USACE North Atlantic Division - New York District (NAN).

3.7.1 Extreme Water Levels

Table 3.6 provides an overview of the extreme water levels at the storm surge barrier location. The utilized Advanced Circulation Model (ADCIRC) nodes/output stations per storm surge barrier location are listed in the second column.

Table 3-6: AEP Still Water Levels (50% confidence limit) from NACCS ADCIRC output, inclusive of Sea Level Rise in 2080 per the site specific stations as presented in 3.6.3. Specific output nodes referenced in table.

AEP	ADCIRC Output Station	Return Period	Still Water Level (1992 MSL)	Still Water Level (2030 Sea Levels)	Still Water Level (2080 Sea Levels)
			ft, NAVD88	ft, NAVD88	ft, NAVD88
			Verrazano Narrows		
1%	11781	100	11.2	11.8	13.0
0.2%	11781	500	14.7	15.3	16.5
0.1%	11781	1000	16.4	17.0	18.2
			Throgs Neck		
1%	4347	100	13.0	13.5	14.4
0.2%	4347	500	16.6	17.0	18.0
0.1%	4347	1000	18.3	18.7	19.7
			Arthur Kill		
1%	11650	100	12.8	13.4	14.6
0.2%	11650	500	16.7	17.3	18.5
0.1%	11650	1000	18.4	19.0	20.2
			Outer Harbor		
1%	3900	100	11.4	12.0	12.2
0.2%	3900	500	13.9	15.1	15.7
0.1%	3900	1000	15.5	16.1	18.2
			Kill van Kull		
1%	11766	100	11.4	11.9	12.9
0.2%	11766	500	15.0	15.5	16.5
0.1%	11766	1000	16.7	17.2	18.2
			Jamaica Bay		
1%	3592	100	9.8	10.4	11.6
0.2%	3592	500	12.7	13.3	14.5
0.1%	3592	1000	13.9	14.5	15.7
			Hackensack River		
1%	11816	100	10.7	11.3	12.5
0.2%	11816	500	13.1	13.7	14.9
0.1%	11816	1000	14.1	14.7	15.9

3.7.2 Extreme Wave Heights and Period

Table 3.7 shows the wave height and period from that of a 100 year and 500-year storm. The ADCIRC nodes/output stations per storm surge barrier location are listed in the second column.

Table 3-7: AEP Wave Characteristics (50% confidence limit)

AEP	ADCIRC/STWAVE output Station	Return Period	Wave Height (Hs)	Peak Period (Tp)
		year	Ft.	sec
		Verrazano Narrows		
1%	11781	100	5.9	6.0
0.2%	11781	500	6.9	7.3
		Throgs Neck		
1%	4347	100	4.3	4.0
0.2%	4347	500	4.5	4.3
		Arthur Kill		
1%	11650	100	3.8	3.3
0.2%	11650	500	4.4	4.1
		Outer Harbor		
1%	3900	100	16.1	15.1
0.2%	3900	500	16.8	15.5
		Kill van Kull		
1%	11766	100	6.0	6.2
0.2%	11766	500	6.2	6.5
		Jamaica Bay		
1%	3592	100	4.8	4.6
0.2%	3592	500	5.1	5.0
		Hackensack River		
1%	11816	100	3.2	2.6
0.2%	11816	500	3.6	3.0

3.7.3 Wave Overtopping Criteria

For the storm surge barriers in the HAT Study an overtopping criterion of 50 liter per second per meter (l/s/m) is set to determine the structure height. This equates to 0538 cfs/ft and is based on ER 1110-2-1100 (USACE 2002) and guidance from USACE-NAN. This criterion is applied at the end of the project service life.

3.7.4 Design Crest Elevation

The storm surge barriers are conceptually designed to meet the functional criteria (impede storm surge) over the entire planning horizon (2030 – 2080). Based on the provided 1% AEP hydrodynamic characteristics for the year 2080 (extreme water levels and waves) and given the provided overtopping criterion, USACE-NAN has set the storm surge barrier design crest elevation for the storm surge barriers discussed herein. Crest Elevations are summarized in Table 3-8.

Table 3-8: Crest Elevations of Storm Surge Barriers

Storm Surge Barrier	Design Crest Elevation (ft NAVD88)
Verrazano Narrows	+22
Throgs Neck	+20
Arthur Kill	+19
Outer Harbor	+46
Kill van Kull	+21
Jamaica Bay	+18
Hackensack River	+18

3.7.5 Rising Water Levels after Storm Surge Barrier Closure

3.7.5.1 Rise in Protected Side Water Levels

During storm conditions, when the storm surge barrier is closed, the water levels on the protected side could rise as a result of wave overtopping, river discharge, rainfall and leakage (permissible flow through the storm surge barrier when closed). ERDC is analyzing this scenario and results are pending. At this time a preliminary analysis of a select set of ADCIRC model results has been completed. ADCIRC model results for historical and synthetic storms were used to evaluate head difference at the barriers. Time series of surface water elevation at locations on either side of the storm surge barriers were analyzed.

To evaluate the head differences at the storm surge barriers, maximum differences in water levels on both sides of the barriers were computed in both directions (i.e. direct head and reverse head conditions). The largest head differences were observed during two conditions:

- High storm surge on the ocean side of the barrier; and
- High water level on the bay side forced by winds or due to river discharges.

It should be noted, that river discharges in some simulations caused a significant increase in water levels behind the closed barriers. This effect was magnified by the model configuration, when barriers were modeled as closed throughout the entire simulation. Such high water levels in combination with effects of winds, caused significant head differences. For this analysis, only storms with a maximum storm surge near the 1% AEP (100-year return period) water level on the flood side of the storm surge barrier were selected. Only synthetic storms were used in this analysis with a few exceptions which are noted in the tables below. The storms, during which the maximum head differences occurred, were identified and presented for each alternative in Table 3-9. For the Hackensack River storm surge barrier no good coverage of model data is available and data for this location was not processed.

Table 3-9: Direct and Reverse Head Conditions for 1% AEP conditions for the storm surge barriers

Parameter	Head	Flood Side Water Level	Protected Side Water Level	Notes
	ft.	ft, NAVD88	ft, NAVD88	
Verrazano Narrows				
Direct head (1% AEP)	7.3	12.5	5.2	(storm 445 - max flood side WL 12.5ft)
Reverse Head (1% AEP)	-3.9	-2.5	1.4	(storm 445 - max flood side WL 12.5ft)
Throgs Neck (HAT Study Alternative 2)				
Direct head (1% AEP)	16.8	14.2	-2.6	(storm 403 - max flood side WL 14.2)[2]
Reverse Head (1% AEP)	-10.6	-5.2	5.4	(Storm 60 - max flood side WL 12.5)[1]
Throgs Neck (HAT Study Alternative 3A)				
Direct head (1% AEP)	13.0	13.9	0.9	(storm 403 - max Flood side WL 14.2ft)[2]
Reverse Head (1% AEP)	-10.9	-5.2	5.7	(Storm 60 - max flood side WL 12.5ft) [1]
Arthur Kill (HAT Study Alternative 3A)				
Direct head (1% AEP)	9.4	13.8	4.5	(storm 400 - max flood side WL 13.9ft)[2]
Reverse Head (1% AEP)	-5.0	-6.6	-1.6	(storm 525 - max flood side WL 9.9ft)[2]

Parameter	Head	Flood Side Water Level	Protected Side Water Level	Notes
	ft.	ft, NAVD88	ft, NAVD88	
Arthur Kill (HAT Study Alternative 3B)				
Direct head (1% AEP)	9.5	13.1	3.6	(storm 536 - max Flood side WL 13.1ft)
Reverse Head (1% AEP)	-7.7	-5.1	2.6	(storm 536 - max Flood side WL 13.1ft)
Outer Harbor				
Direct head (1% AEP)	9.9	10.4	0.5	(storm 525 - max flood side WL 10.6ft)
Reverse Head (1% AEP)	-5.3	-5.5	-0.3	(storm 525 - max flood side WL 10.6ft)
Kill van Kull				
Direct head (1% AEP)	9.3	11.2	1.9	(storm 525 - max flood side WL 11.2ft)
Reverse Head (1% AEP)	-5.4	-3.3	2.0	(storm 525 - max flood side WL 11.2ft)
Jamaica Bay (HAT Study Alternative 3A)				
Direct head (1% AEP)	12.5	12.0	-0.5	(storm 445 - max flood side WL 12.1ft)[2]
Reverse Head (1% AEP)	-1.6	-1.4	0.1	(storm 445 - max flood side WL 12.1ft)[2]
Jamaica Bay (HAT Study Alternative 3B)				
Direct head (1% AEP)	11.8	11.3	-0.6	(storm 184 - max flood side WL 10.7ft)
Reverse Head (1% AEP)	-1.7	-1.4	0.3	(storm 445 - max flood side WL 11.5ft)[2]

Notes:

1. No good match was available to determine the 1% AEP reverse head conditions and historical storm 60 was used as indication for reverse head conditions.
2. Difference between flood side water level for selected storm and 1% AEP water level is more than 1ft (see also Table 3-6)

3.7.5.2 Rise in Flood Side Water Levels

In addition to the above, it has been well documented that the water levels on the flood side could increase, compared to the situation without the storm surge barrier in place, as a result of the storm surge barrier closure. ERDC is analyzing this scenario, results are pending and will be documented herein at a later date.

3.8 Wind Data

Wind data for each location are derived from the wind gauges in closest vicinity to the storm surge barrier location and are provided in Table 3-10. The wind roses and statistical data for each of these gauges are provided in Appendix D.

Table 3-10: Wind Gauges

Location	Closest Wind Gauge	Station Identification
Verrazano Narrows	John F. Kennedy International Airport	KJFK
Throgs Neck	LaGuardia Airport	KLGA
Arthur Kill	Linden Airport, NJ	KLDJ
Outer Harbor	John F. Kennedy International Airport	KJFK
Kill van Kull	Linden Airport, NJ	KLDJ
Jamaica Bay	John F. Kennedy International Airport	KJFK
Hackensack River	Linden Airport, NJ	KLDJ

Wind load on structures shall be based on the wind load requirements specified in ASCE 7-10 and the USACE HSDRRS Design Guidelines. Per ASCE 7-10, the structures are Risk Category IV and the design wind speed is 130mph (3-second gust at 33 feet above ground, Exposure Category C). This corresponds to a .0588% annual exceedance probability, or a 1700 year mean recurrence interval.

3.9 Geotechnical Conditions

The selection of foundation types for the storm surge barriers will depend on the soil conditions and allowable capacities of geological formations at each location. In addition, the design loads from the gated structure, including overturning moment, uplift, sliding, and global stability will influence the final design of each storm surge barrier.

Limited site-specific data are available at this time and it is strongly recommended that through coordination with the NFS geotechnical data would be obtained for any (unrelated) projects that are in close vicinity to the proposed storm surge barrier sites. Previous studies in the vicinity of the Jamaica Bay and Kill van Kull storm surge barrier provide some more detailed insights for the subsurface conditions while for the other storm surge barriers the Project Delivery Team (PDT) had to rely on publicly available geological maps to characterize the general subsurface conditions.

At this stage, due to the limited available data and given the fact that the parameteric cost model used to estimate the construction cost of the storm surge barriers does not rely on foundation characteristics, all geotechnical evaluations are at a very high level and of a cursory nature. For any of the storm surge barriers that are selected for further study, a more detailed analysis is recommended for the next phase. Such analyses should investigate the existing geotechnical

conditions and any available existing borings at the site. This would help further inform the conceptual design of the storm surge barrier. Details and a synopsis of available geotechnical data for the storm surge barrier sites is presented in Appendix E.

3.10 Navigation Criteria

3.10.1 Navigation Safety

The proposed storm surge barriers will cross one or more federal navigation channels, and cross currents or excessive head-on and helping currents at the navigable passage could adversely impact navigation. The storm surge barrier and gate layout should be optimized to minimize adverse influence on vessels transiting the storm surge barrier. Additionally, the provision of vessel guides, aids-to-navigation, and protective structures at the navigable channel entrance and exits will be considered to increase navigational safety and potentially protect the storm surge barrier from aberrant vessel impact.

3.10.2 Design Vessels

The design of each navigable passage within the storm surge barriers must accommodate the volume of vessel traffic transiting the respective navigation channels. To do so, an understanding of the range of vessel types and configurations that comprise the marine traffic and the frequency of their passage through these channels was established through the analysis of AIS data. The Maritime Traffic Analysis (Enclosure 1) presents vessel categories that capture the range of vessels that transit the channels. A summary of the findings documented in Enclosure 1 is provided in the table below.

Table 3-11: Design Vessels for the Navigable Passages

Location	Design Vessel	Vessel Category	LOA	Beam	Draft	Air Draft
Verrazano Narrows	OOCL Hong Kong	Container (21.4k TEU)	400m 1310ft	58.8m 193ft	16.8m 55ft	Not available
Throgs Neck	Asphalt Splendor	Tanker	179.9m 590ft	30.6m 100ft	7.9m 26.2ft	Not available
Arthur Kill	Australian Spirit	Tanker	256m 840ft	44.8m 147ft	14.1m 46ft[1]	Not available
Outer Harbor (Sandy Hook Channel)	Cape Bonney	Tanker	274.5m 901ft	48m 158ft	10.7m 35ft	Not available
Outer Harbor (Ambrose Channel)	OOCL Hong Kong	Container (21.4k TEU)	400m 1310ft	58.8m 193ft	16.8m 55ft	Not available
Outer Harbor (Rockaway Inlet)	American Princess	Passenger Ship	45m 148ft	11m 35ft		Not available

Location	Design Vessel	Vessel Category	LOA	Beam	Draft	Air Draft
Kill van Kull	OOCL Hong Kong	Container (21.4k TEU)	400m 1310ft	58.8m 193ft	16.8m 55ft	Not available
Jamaica Bay	American Princess	Passenger Ship	45m 148ft	11m 35ft	N/A	Not available
Hackensack River						

The Verrazano Narrows, Kill van Kull and the Outer Harbor – Ambrose Channel navigable passage list the same design vessel as all these storm surge barriers cross the channels that lead to and from to the Container Terminals in Newark Bay. The American Princess (Passenger Vessel) is selected as design vessel for the approach into and out of Jamaica Bay.

3.11 Design Criteria Yet to be Specified

3.11.1 Design Criteria

For the feasibility study, design criteria are limited to those elaborated in the previous sections. The scope of this report is too broad to evaluate and establish the minimum criteria requirements for the gated portion of the structure. The documentation provided within the previous sections is the first step in an iterative design approach. The preliminary storm surge barrier geometry presented in later sections is to first test the hydrodynamic performance through modeling and provide a first estimated for construction costs using a parametric cost model. Results from the hydrodynamic modeling, findings and further refinements can be incorporated in the next phase to further the conceptual design.

The general design criteria within the previous sections are not complete and many criteria are yet to be specified. Such criteria can include but are not limited to:

- Constructability Criteria
- Geotechnical and Foundation Criteria
- Seismic Design Criteria
- Structural Loading and Associated Criteria
- Mechanical Design Criteria
- Electrical and Control Systems Design Criteria
- Civil Design Criteria
- Dredging and Earthwork Design Criteria
- Infrastructure and Transportation Criteria
- Operational and Maintenance Assessments
- Materials
- Aesthetics criteria

- Risk (assessments would occur across a broad range of structures, environmental, and operations evaluations)
- Extreme event loadings (seismic, vessel collision, force protection)
- Serviceability and design life assessments

In addition, environmental criteria other than those identified in the preceding sections are only qualitatively considered when deemed appropriate and will need to be further evaluated during subsequent design phases. Such criteria, amongst others, include:

- Criteria related to the impacts on bed morphology and sediment transport
- Criteria related to water quality
- Criteria related to marine habitats, flora and fauna (e.g. fish passage) - ecology
- Criteria related to environmental impacts during construction an upon completion of the project)

3.11.2 Operational Criteria

For the feasibility study design criteria related to the operation of the storm surge barrier are limited to functional requirements as listed in Section 3.4. However more detailed requirements with respect to many aspects of the structure, its function and its operation will need to be addressed in later phases of the study. Such criteria include, but are not limited to:

- Machinery and controls requirements and criteria,
- Maintenance requirements (access, inspection, and maintenance for the gates, machinery controls, abutments, piers, dam section, bulkheads, stoplogs, etc.),
- Operational requirements (optimum closure frequency, timescale for deployment and recovery, operational forecast and triggers),
- Passive or active flow control to assist navigable gate transits,
- Redundancy requirements,
- Navigation criteria (see list as documented in Enclosure 1 – vessel traffic service, advisory, control, tug assist, limitations on passage, restrictions, etc.).

4 CONCEPTUAL DESIGN DEVELOPMENT

4.1 Storm Surge Barriers – Principle Elements

The purpose of a storm surge barrier is to impede storm surge when closed yet maintain tidal flow exchange between the ocean and the upstream water body, e.g. bay, basin or river, during normal conditions when the gates are open. The storm surge barriers will have a navigable passage to allow for vessels to pass and auxiliary flow gates to maintain tidal flow exchange.

Based on the functional requirements presented in Section 3.4 it is proposed that the first iteration of the conceptual design for the storm surge barriers will maximize the number of flow gate openings and minimize the portion of "dam section" of the storm surge barrier. The auxiliary flow gates serve to maximize the water exchange through the opening and minimize impacts on the inner basin environmental conditions during normal hydrodynamic and meteorological conditions.

At the tie-in locations of the storm surge barrier to the shore-based system, i.e. shallow waters, a dam section on the order of 100 ft to 500 ft long, depending on the location, would be needed. This dam section will be the transition between the operable storm surge barrier gate structure and land-based flood risk reduction measures.

As discussed in section 2.2, the locations of the storm surge barriers discussed herein have been provided by USACE and are, for the most part, determined by the extent and location of the perimeter risk reduction systems that are part of the HAT Study Alternatives. Site location maps are provided in Appendix A. Once a location has been set the overall geometry is then dictated by the existing bathymetry, geometry of the navigable passage(s) and the auxiliary flow gates given the existing bathymetric profile and the design criteria as summarized in Section 3. Sections 4.2 and 4.3 provide additional detail on the navigable passage and auxiliary flow gates, respectively.

4.2 Navigable Passage

This section documents the preliminary assessment of the minimum required dimensions for the navigable passages and auxiliary flow gates of the storm surge barriers. It summarizes the design development of the minimum practical dimensions of the navigable passages of the storm surge barriers. The design vessel specifications and channel dimensions were established following PIANC guidance. Enclosure 1 details the maritime traffic study and the required minimum geometry for the navigable passages. The following sections provide a brief synopsis of the conceptual design development for the navigable passages. A summary overview is presented at the end of this section in Table 4-1.

4.2.1 Navigable Passage Dimensions Verrazano Narrows

The storm surge barrier from Staten Island to Brooklyn would at least require two navigable passages:

1. Ambrose Channel Navigable Passage Opening: minimum 1400ft wide

- PIANC guidance to set navigational opening: to a minimum of 968ft and 1428ft for one-way and two-way traffic respectively. Rounded to the nearest hundred, 1400ft is considered a conservatively appropriate value for use as preliminary estimate in the overall storm surge barrier feasibility study.
- Set sill of Navigable Passage at -55ft MLLW (Authorized Channel Depth equals -53ft MLLW and additional 2ft clearance below design channel is included to account for the hard bottom structure). -55ft MLLW equals -58ft NAVD88,
- Air Draft is controlled by the Verrazano Bridge at 228ft. This clearance is adopted as a conservative assumption which in practicality for the design of moveable gates translates to unrestricted air clearance.

In addition to the main navigation channel, the AIS data suggests that there is a high volume of smaller traffic. It would be advisable to include at least one smaller, navigable gate in the storm surge barrier to pass smaller vessels, to keep the smaller vessels out of the main navigation channel. Enclosure 1 details the selection of the design vessel and analyses to establish the minimum practical width for the secondary navigable passage.

2. Secondary Navigable Passage on East Side of Main Channel: 200ft wide (one-way vessel traffic)
   - PIANC guidance to set navigational opening: minimum 178ft
   - Set sill of Navigable Passage at -42ft MLLW. Authorized Channel Depth is -40ft MLLW and additional 2ft clearance below the design channel is included to account for the hard bottom structure. -42ft MLLW equals -45ft NAVD88,
   - Air Draft: 50ft

It should be noted that the dimensions of the navigable passage for the Ambrose Channel is larger than any gated opening in constructed storm surge barriers (Maeslant Barrier in The Netherlands spans 1200ft). The findings presented herein are a preliminary assessment and further refinement of the gate dimensions and gate configurations (including layout, number and width) will need to occur during later stages in the design.

4.2.2 Navigable Passage Dimensions Throgs Neck

The storm surge barrier which would span from Westchester, NY to Queens, NY would at least require one navigable passage:

1. Throgs Neck navigable passage opening: 450ft wide (two-way vessel traffic)
   - PIANC guidance to set navigational opening for one-way traffic to a minimum of 447ft.
   - Set sill at -37ft MLLW (2ft clearance below the authorized channel depth of -35ft MLLW). -37ft MLLW equals -40ft NAVD88,

- Air Draft is controlled by the Throgs Neck Bridge at 142ft. This clearance is adopted as a conservative assumption which in practicality for the design of moveable gates translates to unrestricted air clearance.

4.2.3 Navigable Passage Dimensions Arthur Kill

The Arthur Kill storm surge barrier from Woodbridge, NJ to Staten Island, NY would at least require one navigable passage:

1. Arthur Kill Navigable Passage Opening: 600ft wide (width of existing channel).

    - PIANC guidance to set navigational opening for one-way traffic to a minimum of 661ft, yet the Authorized Channel Width is 600ft.

    - Set sill at -37ft MLLW. Authorized Channel Depth is -35ft MLLW and additional 2ft clearance below the design channel is included to account for the hard bottom structure. -37 ft MLLW equals -40ft NAVD88

    - Unrestricted air draft to the Outerbridge Crossing, after that it is controlled by the Outerbridge Crossing at 143ft. This clearance is adopted as a conservative assumption which in practicality for the design of storm surge barriers translates to unrestricted air clearance.

4.2.4 Navigable Passage Dimensions Outer Harbor

The storm surge barrier from Sandy Hook to Breezy Point would at least require three navigable passages:

1. Sandy Hook Navigable Passage Opening: 800ft wide.

    - PIANC guidance to set the navigational opening for one-way traffic to a minimum of 835ft, but the Authorized Channel Width is 800 ft as such the width of the existing channel is used to set the opening width

    - Set sill at -37ft MLLW. Authorized Channel Depth is -35ft MLLW and additional 2ft clearance below the design channel is included to account for the hard bottom structure. -37 ft MLLW equals -40ft NAVD88,

    - Air draft: 143ft (assume up to 43.6m for this vessel size class). This clearance is adopted as a conservative assumption which in practicality for the design of storm surge barriers translates to unrestricted air clearance.

2. Ambrose Channel Navigable Passage Opening: 1500ft wide

    - PIANC guidance to set Navigational Opening: to a minimum of 1026ft and 1543ft for one-way and two-way traffic respectively. Rounded to the nearest hundred, 1500ft is considered

appropriate for use as preliminary estimate in the overall storm surge barrier feasibility study.

- Set sill of Navigable Passage at -55ft MLLW. Authorized Channel Depth is -53ft MLLW and an additional 2ft is included to account for hard bottom structure. -55ft MLLW equals -58ft NAVD88.

- Air Draft is controlled by the Verrazano Bridge at 228ft. This clearance is adopted as a conservative assumption which in practicality for the design of storm surge barriers translates to unrestricted air clearance.

3. Rockaway Inlet Navigable Passage Opening: 200ft wide

    - PIANC guidance to set navigational opening: minimum 198ft (one-way vessel traffic)

    - Set sill at a minimum at -22ft MLLW. Authorized Channel Depth is -20ft MLLW and an additional 2ft is included to account for hard bottom structure. -22ft MLLW equals -25ft NAVD88,

    - Air Draft: 50ft

The difference in navigable passage opening for the Ambrose Channel in the Verrazano Narrows and Outer Harbor storm surge barrier is a resulted of the more exposed conditions of the Ambrose Channel (see Enclosure 1 for more details). Again, it can be noted that the dimensions of the navigable passage at the Ambrose Channel as part of the Outer Harbor storm surge barrier is larger than any gated opening in constructed storm surge barriers (Maeslant Barrier in The Netherlands spans 1200ft).

4.2.5 Navigable Passage Dimensions Kill van Kull

The storm surge barrier from Bayonne, NJ to Staten Island, NY will require one navigable passage:

1. Kill Van Kull Navigable Passage Opening: 800ft wide (width of existing channel).

    - PIANC guidance to set navigational opening for one-way traffic to a minimum of 856ft, yet the Authorized Channel Width is 800ft.

    - Set sill at -52ft MLLW. Authorized Channel Depth is -50ft MLLW and an additional 2ft is included to account for hard bottom structure. -52 ft MLLW equals -55ft NAVD88,

    - Air Draft is controlled by the Verrazano Bridge at 228ft. This clearance is adopted as a conservative assumption which in practicality for the design of storm surge barriers translates to unrestricted air clearance.

4.2.6 Navigable Passage Dimensions Jamaica Bay

The siting of the storm surge barrier is discussed in section 2 and is based on the analysis completed in 2016 (USACE, 2016) and equal to the concept proposed in that study, the storm surge barrier from Barren Island, NY, to Rockaway, NY will have two openings to allow for maritime traffic to traverse:

1. Rockaway Inlet Navigable Passages Openings: 200ft wide

    - PIANC guidance to set navigational opening: minimum 198ft
    - Set sill at a minimum at -22ft MLLW. The Authorized Channel Depth is -18ft MLLW and a 2ft clearance is included to account for the hard bottom structure and an additional 2ft is included to increase flow conveyance. -22ft MLLW equals -25ft NAVD88
    - Air Draft is controlled by the Gil Hodges Bridge at 152ft. This clearance is adopted as a conservative assumption which in practicality for the design of storm surge barriers translates to unrestricted air clearance.

4.2.7 Navigable Passage Dimensions Hackensack

The storm surge barrier is located just upstream of the terminus of the constructed federal channel (see also section 3.5.1). However, the river is still navigable beyond that point. The storm surge barrier on the Hackensack River would at least require one navigable passage to accommodate vessel traffic. The most restricting horizontal dimension on the waterway is the swing bride upstream of the storm surge barrier, which has a horizontal clearance of 100 ft. The existing channel thalweg depths upstream of the Marion Reach, where the storm surge barrier is proposed, vary between -20ft and -25ft NAVD88 (-17ft MLLW and -22ft MLLW). At this stage of design, a sill elevation of -20ft is proposed.

1. Hackensack River Navigable Passage Opening: 100ft wide. (one-way vessel passage)

    - Navigation opening is based on upstream bridge restriction
    - Set sill at -20ft MLLW (equal to -23ft NAVD88)
    - Air draft is controlled by the upstream bridge at 103 ft. This clearance is adopted as a conservative assumption which in practicality for the design of moveable gates translates to unrestricted air clearance.

4.2.8 Summary of Navigable Passage Dimensions

The navigable passage dimensions and findings from the previous sections are summarized in Table 4-1. Note that the width of the navigable passage for the same federal channel can differ

amongst storm surge barriers due to a difference in exposure to environmental conditions. Analyses for navigable passage dimensions are included within Enclosure 1.

Table 4-1: Minimum Practical Dimensions for the Navigable Passages

Location	Federal Channel	Existing Depth (ft)	Minimum Practical Width of Opening [ft]	Authorized Channel Depth [ft, NAVD88]	Minimum Depth of Opening [ft NAVD88]	Air Clearance [ft NAVD88]
Verrazano Narrows	Ambrose Channel	70-75	1400[2]	-56	-58	Un-restricted
Verrazano Narrows	Secondary Navigation Channel	20-25	200[1]	-43	-45	50 ft
Throgs Neck	Throgs Neck	40-55	450[2]	-38	-40	Un-restricted
Arthur Kill	Arthur Kill Channel	35	600[1,3]	-38	-40	Un-restricted
Outer Harbor	Ambrose Channel	75-80	1500[2]	-56	-58	Un-restricted
Outer Harbor	Sandy Hook	60-70	800[1,3]	-38	-40	Un-restricted
Outer Harbor	Rockaway Inlet	20-30	200[1]	-23	-25	50 ft
Kill Van Kull	Kill Van Kull	40-55	800[1,3]	-53	-55	Un-restricted
Jamaica Bay	Rockaway Inlet	20-30	200[1]	-21	-25	Un-restricted
Hackensack River	Hackensack River	20-25	100[1,3]	-18	-23	Un-restricted

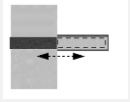
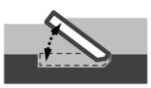
Notes:
1. Practical width of navigable passage based on one-way traffic
2. Practical width of navigable passage based on two-way traffic
3. Practical width of navigable passage limited based on existing authorized channel dimensions

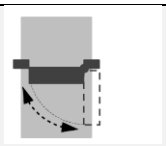
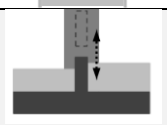
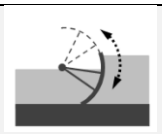
4.2.9 Navigable Gate Type Selection

Following the minimum design dimensions outlined in Table 4-1 a suitable gate type will have to be preliminarily selected. Enclosure 2 includes an overview of the supplemental data from

Mooyaart and Jonkman (2017), which provides an overview of characteristics of constructed storm surge barriers. In addition, Enclosure 2 includes an overview of hydraulic gate types used in navigable storm surge barriers and lists general advantages and disadvantages of each gate type. Using the data set and the listed advantages and disadvantages, this report provides a cursory review of the suitability of each gate type for the navigable passage. Based on the evaluation that is provided in Appendix B the sector gate (vertical axis) and floating sector gate are preliminarily selected for the conceptual design of the navigable passages. All gate types and a generalized overview of the evaluation in Appendix B is summarized in Table 4-2.

Table 4-2: Summary of Gate Type Selection for the Navigable Passage (from Appendix B)

	GATE TYPE	Generalization of Notes and Evaluation from Appendix F	Navigable Passage where Gate Type is Selected
	Horizontal Rolling Gate	Impractical due to area needed to dock the gate	
	Sector Gate (Vertical Axis)	**Suitable**, proven concept for similar spans (spans up to 200 ft in these instances) and sill elevations.	• Verrazano Secondary Navigation Gate • Outer Harbor Rockaway Inlet Gate • Jamaica Bay Navigation Gates • Hackensack River Navigation Gate
	Floating Sector Gate	**Suitable**, proven concept for wide and deep navigable opening.	• Verrazano Main Navigation Gate • Throgs Neck Main Navigation Gate • Outer Harbor Ambrose Channel Gate • Outer Harbor Sandy Hook Gate • Arthur Kill Main Navigation Gate • Kill van Kull Main Navigation Gate
	Rotating Segment Gate	**Possible for some locations**, but generally deemed relatively expensive and complex system and not suitable for deep and wide spans	
	Inflatable Gate or Dam	Generally, no proven concepts for for a large deep spans	
	Flap Gate	Not suitable for reverse head conditions and generally considered	

	GATE TYPE	Generalization of Notes and Evaluation from Appendix F	Navigable Passage where Gate Type is Selected
		too challenging from a maintenance perspective	
	Barge Gate	Generally, not suitable, too complex and challenging to operate under wave and current conditions	
	Miter Gate	Not suitable for large spans or large reverse head conditions.	
	Vertical Lift Gate	Generally, not suitable since either unrestricted air clearance is required or the required air clearance would prohibitively increase gate costs	
	Vertical Rising Gate	Generally assumed to be not suitable, related to the challenges from a maintenance perspective. In addition, no proven concept for large gate spans	
	Tainter Gate	Not suitable since either unrestricted air clearance is required or the required air clearance would prohibitively increase gate costs	

4.3 Auxiliary Flow Gates

This section provides the design development of the dimensions of the auxiliary flow gates and gate type selections.

4.3.1 Auxiliary Flow Gate Dimensions

For the auxiliary flow gates, a standard gate span of 150ft is preliminarily selected based on the review of gate characteristics as presented in Mooyaart and Jonkman, 2017 and in Enclosure 2. 150ft is considered to be a reasonable assumption, where this width falls within the gate spans for constructed storm surge barriers. Some storm surge barrier locations have spatial constraints and

smaller gate spans may be needed. Exceptions and special considerations for each location are provided in Table 4-4. Due to the variations in depth along the storm surge barrier alignment it is expected that varying gate sizes will be needed. Varying gate sizes will allow the design to follow the natural bathymetric contours of the area while maintaining a large open cross-section for flow. To minimize construction complexity and allow for optimization through the economies of scale the gate sill elevations are preliminarily assumed to vary in increments of 5ft. The sill elevation of the auxiliary flow gates is to be above the existing bed elevation such that the potential for sedimentation or siltation at the bottom of the sill is minimized.

When the gates are in the open position during normal day-to-day conditions, the bottom of the gates shall provide 3ft of clearance above MHHW at the end of the project service life. Positioning the bottom of the gate, when open, above this elevation will ensure that the gate will not be inundated more than needed and that flow will not be impeded by sluicing action under the gate for typical water level and operating conditions. The additional 3ft is used to account for potential wave action during normal conditions and allow for clear sight lines underneath the gate which may be needed during visual inspections. This elevation varies per storm surge barrier location. An example is provided here for the Verrazano Narrows storm surge barrier where this elevation is equal to +8ft NAVD88 which is the rounded sum of +2.41ft (MHHW), Sea level Change (1.82) and the additional 3ft clearance. Headwall elevations are set according to the table below.

Table 4-3: Headwall Elevations

Location	MHHW (ft, NAVD88)	Sea Level Rise (ft)	Clearance (ft)	Headwall Elevation (ft, NAVD88)[1]
Verrazano Narrows Auxiliary Flow Gates	2.28	1.49	3	+7
Throgs Neck Auxiliary Flow Gates	3.64	1.37	3	+8
Arthur Kill Auxiliary Flow Gates	2.41	1.82	3	+7
Outer Harbor Auxiliary Flow Gates	2.41	1.82	3	+7
Kill Van Kull Auxiliary Flow Gates	2.63	1.82	3	+7
Jamaica Bay Auxiliary Flow Gates	2.41	1.82	3	+7
Hackensack River Auxiliary Flow Gates	2.97	1.82	3	+8

Note:
1. Elevation = MHHW + SLR + Clearance, rounded to nearest foot.

The auxiliary flow gate only passes flow and as such the gate does not need to be raised above the elevation stated above. The design elevation of the storm surge barriers is provided in section 3.7.4. To reduce the gate size, weight and overall complexity of the hoisting mechanisms, it is proposed to include a solid, non-moveable, wall between the elevation +8ft and the top of the structure. For water control structures, this is commonly referred to as a headwall. While a headwall requires a fourth seal between structure and gate, since all four sides need to be sealed instead of three, the headwall reduces the overall height of the gates substantially[6]. For example, for the Verrazano Narrows storm surge barrier, a gate height of 52ft would be needed to close of an opening between a sill elevation of -30ft to a design elevation of +22ft, while with the use of a headwall the gate height would be 38ft (sill elevation at -30ft and top of gate at elevation +8ft).

Table 4-4 summarizes the dimensions for the Auxiliary flow gates discussed herein.

[6] E.g. the Eastern Scheldt storm surge barrier (The Netherlands) includes a headwall type feature.

Table 4-4: Design Dimensions for the Auxiliary Flow gates

Location	Existing Depth (ft)	Width of Flow Gate Opening (ft)	Depth of Flow Gate Opening (ft, NAVD88)	Required Bottom of the gate in raised position (ft, NAVD88)
Verrazano Narrows	Varying between 70ft to 30ft	150	-60, -25, and -20	+7
Throgs Neck	Varying between 55ft to 40ft	150	-45	+8
Arthur Kill	Varying between 20ft to 10ft	70[1]	-10	+7
Outer Harbor	Varying between 50ft to 20ft	150	varying	+7
Kill Van Kull	Varying between 40ft to 60ft	150	-28 to -30	+7
Jamaica Bay	Varying between 300ft to 20ft	150	varying	+7
Hackensack	Varying between 20 to 25	50[2]	varying	+8

Notes:

1. A gate span of 70ft is preliminarily selected based on the limited width and depth available between the navigable gate and gate housing and the shoreline
2. A gate span of 50ft is preliminarily selected based the previous USACE-NAN study **(USACE February 1989)**

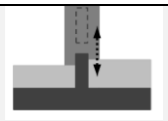
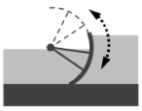
Finally, the sill elevation of the auxiliary flow gates is to be above the existing bed elevation such that the potential for sedimentation or siltation at the bottom of the sill is minimized. Future data collection will be needed to obtain bathymetric profiles and additional analyses are needed to evaluate the effect the storm surge barrier has on the hydrodynamics and morphology of the estuarine system.

4.3.1 Auxiliary Flow Gate Type Selection

Following the specifications in Table 4-4 a suitable gate type has preliminarily been selected. Appendix G provides a cursory review of the suitability of gate types for the auxiliary flow gates. Based on the evaluation that is provided in Appendix G the vertical lift gate is preliminarily selected for the majority of the conceptual designs of the storm surge barrier discussed herein. For the Hackensack River storm surge barrier, the tainter gate type selection is informed by the previous USACE-NAN study (USACE February 1989).

Gate types and a generalized overview of the evaluation in Appendix G is summarized in Table 4-5. The horizontal rolling gate, barge gate and miter gate are omitted from the table since these are the least suitable hydraulic gate options for auxiliary flow. Flap gates and vertical rising gates could potentially be utilized in some locations but are in general considered to be relatively expensive and complex systems to be used as auxiliary flow gate.

Table 4-5: Summary of Gate Type Selection for the Auxiliary Flow gates (from Appendix G)

	Gate Type	Generalization of Notes and Evaluation from Appendix G	Storm Surge Barrier where Conceptual Design Includes Gate Type
	Vertical Lift Gate	**Suitable**, proven concept	• Verrazano • Throgs Neck • Arthur Kill • Kill van Kull • Outer Harbor • Jamaica Bay
	Tainter Gate	**Suitable**, proven concept, however reverse head conditions will need to be investigated	• Hackensack River (per previous study - USACE, February 1989)
	Inflatable Gate or Dam	**Possible**, but no proven concept for tidal inlet applications. Furthermore, deemed relatively complex system.	
	Rotating Segment Gate	**Possible**, but no need for unrestricted air clearance, furthermore deemed relatively expensive and too complex system	

4.4 Piers and Gate Housing

As stated in Section 4.1; to minimize the environmental impacts and maintain as much tidal exchange as possible, the first concept will maximize the number of auxiliary flow gates to the extent considered reasonable. The storm surge barrier locations have been identified (see 2.2.1) and the dimensions of both the navigable opening and the flow gate openings have been established in the previous sections. In order to provide a conceptual geometry an assumption regarding the pier widths is be made. The sector gate abutment is assumed to be, at a minimum, two-thirds of half of the gate opening. For the vertical lift gate, a pier width of 30ft was conservatively assumed

based on the review of the characteristics of vertical lift gates constructed as part of storm surge barriers (Enclosure 2).

4.5 Foundation Considerations

4.5.1 Foundation Types

To evaluate potentially viable foundation concepts the potential gate types have been considered. Following the discussion and elaboration in section 4.2 and 4.3, four types of flood gate structures are being evaluated to provide flood risk reduction. These four types include sector gates and floating sector gates (for navigable passages) and vertical lift gates and tainter gates (for auxiliary flow). Using the generic geologic and soils data presented in the previous section and in Appendix E, a cursory evaluation of potential alternative foundation systems has been carried out. It should be noted that this is a high-level evaluation as no site-specific borings are available and designs for the gates are still conceptual in nature.

The following foundation types are discussed:

1. Gravity foundations with in-situ soils
2. Gravity foundations with soil improvements
3. Gravity foundations w/ rock anchors
4. Deep foundations - piles or drilled shafts
5. Cellular cofferdams and combi-walls

4.5.1.1 Gravity Foundation with In-Situ Soils

For study areas possessing in-situ soil conditions conducive to supporting large mat foundation structures, a gravity only foundation system may be suitable. In general, gravity type foundation systems derive their stability from the applied weight of the structure to resist sliding, bearing and overturning forces resulting from the deadweight of the structure and the applied environmental forces. In this scenario, the soils must be suitable to provide resistance to sliding and bearing failure within the soil, a function of the combined effect of both shear and compressive strength of the soil matrix. In general, the most suitable strata for gravity foundations is medium to dense sands and rock. If less desirable materials such as clay or organic silts exist at lower layers within the soil, and within the influence zone of the soil bearing pressures, then the capacity of the foundation could be significantly reduced.

A gravity foundation system is designed to address the following:

- Bearing capacity of the soils and rock
- Sliding resistance
- Overturning stability
- Settlement

Typically, for structures such as storm surge barriers which are expected to resist significant lateral loading, such as large hydrostatic loads and dynamic wave loads, the required mass of the structure will be driven by achieving the required sliding resistance between the foundation system and the underlying soils. To this effect, different strategies may be employed to further increase the frictional resistance on the base of the structure by keying the structure into the soil to drive down the shear failure plane to within the soil matrix.

The weight of the structure must also be sufficient to ensure the structure does not overturn due to the driving lateral loads, especially significant when the driving loads are focused on the top of the structure as is the case with storm surge barriers. Large overturning forces also contribute to increasing the soil bearing pressures under the toe of the structure, which must be checked in conjunction with the simultaneous application of the shear stresses in the soil.

Both near term and long-term settlements of the foundations need to be addressed in the design, with non-uniform long-term settlements due to consolidation of the weaker lower soil layers posing a significant challenge to the long term operational and maintenance requirements of the gates.

4.5.1.2 Gravity Foundation

There are instances where the desired soil conditions for a gravity-based foundation exists near the surface but are overlain by a thin layer of undesirable organic silt or clay. In these cases, the upper weak layer of soil would be removed by an appropriate dredging method and the gravity foundation placed directly on the lower firm layer.

When substantially weak strata exist at lower levels in the soil profile, and within the influence zone of the soil bearing pressures, it becomes uneconomical to try to replace these soils and the foundation design reverts to a pile or drilled shaft deep foundation system, discussed hereafter.

Another option uses gravity-based foundation elements, such as cellular structures, where weaker material is removed from within, and replaced with competent material. Capacities within the cells can be supplemented with deep foundation elements, such as piles.

4.5.1.3 Gravity Foundation with Rock Anchors

When rock exists at or near the surface of the gravity foundation base, the size and weight of the required gravity structure can be significantly reduced by employing rock anchors to assist in the sliding and overturning stability of the structure. In these cases, any thin layers of surface sediments are removed, and inclined rock anchors can be installed to supplement resistance to sliding and overturning forces.

4.5.1.4 Deep Foundations

At locations where the subsurface soils are not adequately strong to support a gravity foundation structure, or for structures which are highly dependent on controlled settlements or movements for

their proper function, a deep foundation system in the form of piles or drilled shafts will be required.

Deep foundations in the form of piles or drilled shafts are designed to transfer the applied loads from the gate structure and environment to deeper soil strata, often mobilizing the distributed and additive soil strengths of multiple layers of soil into the capacity of the pile or shaft element. The difference between piles and drilled shafts are in the method of installation. Piles are structural elements driven in place, either fully displacing the surrounding soil (solid or closed ended piles) or partially displacing the soil (open ended or hollow piles). Shafts are essentially drilled piers, used commonly in bridge construction. They are deep concrete foundations installed by excavating or augering into the soil within a steel permanent or temporary casing to create a deep hole into which reinforced concrete is placed. When installed in the water, a casing is required to segregate concrete for the water; steel pipe is most often used for the casing.

Piles and shafts are designed to resist compressive, tensile and lateral forces. Depending on the subsurface soil profile, the compressive resistance is either primarily resisted by the end bearing of the pile or shaft in a deeper very hard stratum (rock or glacial till), or through the combined frictional resistance of deep layers of sand and/or clays and silts. Tensile capacity of a pile or shaft element is always a function of its frictional resistance in the underlying soils. Lateral capacity at the head of the pile is a function of the relative stiffness of the soil layers and pile structural capacity, with larger and stiffer pile elements able to mobilize deeper layers of lateral soil resistance.

The use of deep foundation systems will reduce the reliance on the weight of the substructure to maintain stability of the structure, and as a result the substructure will not be as massive. However, the cost savings in structure mass may be outweighed by the added cost of the pile foundation system and the general construction method required. Some types of gravity-based structures may be able to be prefabricated offsite and transported and set into position at the gate site (following suitable seabed preparation) in the form of a float-in caisson. Structures that rely on deep foundations may be constructed in-the-dry (within a dewatered cofferdam) or may use more innovative techniques using float-in or lift-in segments placed on pre-installed deep foundation units.

Hybrid systems may also be used where the foundation base is designed to key into the soil to resist lateral loads but is supported by piles to resist bearing and overturning forces. In these cases, the base may also be partially embedded below the seabed to mobilize the shear strength of the lower strata, and also the soil bearing resistance on the face of the foundation structure. In all cases, sufficient and reliable scour protection will be required to protect against significant seepage and undermining below the base of the structures which will be critical to the operation and safety of the facility.

4.5.1.5 Cellular Structures and Combi-Walls

Where large sector gates or floating sector gates are to be installed, the "gate housing structures" are more akin to the construction of small islands than to construction of a stand-alone structures, as would be the case for a vertical lift gate.

In these cases, cellular structures or high-modulus sheet pile wall systems (combi-walls) may be used to construct the retaining structures for the "island" fill. Cellular structures are gravity structures constructed of steel sheet piles installed around a circular template and filled with granular material to maintain their stability. Their design is based on a combination of gravity-based stability, frictional resistance of the sheet piles embedded in the subsurface soils, tensile capacity generated between interlocks of adjacent sheets, and frictional resistance within interlocks. The circular sheet pile cells may be employed as the permanent perimeter walls of the "island," or as temporary structures to allow "in the dry" construction of the gate structures.

Combi-walls are large diameter steel or concrete pipe piles driven closely together with interconnecting elements to form a continuous retaining wall. The wall may be independently braced with batter piles, connected across and tied back against an opposing face, provided with a pile or sheet pile deadman anchor system, or may be free standing and cantilevered from the seabed to resist the lateral retained soil loading, depending on the water depth and loading conditions. Large capacity steel sheet piling sections may also be used for shallower water depths in lieu of combi-wall sections. As toe embedment of the retaining wall systems is critical to their stability, proper attention must be given to providing scour protection.

4.6 Foundation Concepts for the Storm Surge Barriers

These are high-level concepts based on preliminary data and require significant further study to fully vet and select an appropriate solution. Areas of further study include, but are not limited to, actual subsurface conditions, material sources, construction means and methods, construction impact on navigation and the environment, and cost. These topics are expanded upon in the final chapter of this report.

4.6.1 Preliminary Foundation Concepts for Verrazano Narrows, Throgs Neck, Outer Harbor and Arthur Kill Storm Surge Barrier Foundations

The general geologic setting varies across the sites for the Verrazano, Throgs Neck, Outer Harbor and Arthur Kill storm surge barrier; however, since very limited subsurface data is available, generalizations are made regarding the preliminary foundation types. As a result, recommendations for these four (4) storm surge barriers are similar.

The assumed general geologic conditions are as follows:

- Verrazano Narrows: 2-5ft of loose sand and silt underlain by medium dense sand and gravels.

- Throgs Neck: 25ft or more of soft organic silts and fine sand underlain by medium dense sands and clays.
- Arthur Kill: up to 25ft of organic silts and fine sand underlain by medium dense to dense sands and clays.
- Outer Harbor: is 2-5ft of loose sand and silt underlain by medium dense sand and gravels.

Based on this limited geotechnical data, the conceptual foundation types listed below are identified as being potentially viable for the Verrazano Narrows, Throgs Neck, Arthur Kill and Outer Harbor storm surge barriers:

- Floating sector gate housing (recess and abutments or in all practicality due to the size of the structure a small man-made island to house the floating sector gate)
    - Option 1: Cellular structures or combi-wall systems for construction of the "island" type facilities, coupled with pile-supported concrete gate recesses configured as "dry docks." The gate recesses may be constructed conventionally within dewatered cofferdams or may involve float-in or lift-in construction techniques.
    - Option 2: Pre-installed drilled shaft foundations on the channel bottom, mated with prefabricated pier / abutment and gate recess elements which are either floated or lifted into place.
- Floating sector gate sills:
    - deep foundation elements, either piles or drilled shaft.
- Auxiliary flow gate superstructures:
    - deep foundation elements, either piles or drilled shafts
    - alternatively (for Verrazano Narrows) gravity foundation (e.g. cellular structures supporting base foundations, with local soil improvements);
- Dam sections:
    - deep foundation elements, either piles or drilled shafts.
    - alternatively (for Verrazano Narrows) gravity foundation with local soil improvements;

4.6.2 Preliminary Foundation Concepts for Kill van Kull Storm Surge Barrier Foundations

The general geologic setting for the Kill van Kull area is 2 to 5ft of loose sands and silts underlain by serpentine bedrock. Based on this limited geotechnical data, the conceptual foundation types listed below are identified as being potentially viable for the Kill Van Kull storm surge barrier.

Recommended Foundation Systems for the entire storm surge barrier complex:

- All structures: gravity-based foundations with the removal of the loose surface sediments, weathered and weak rock, and founding of the structure on competent bedrock.

4.6.3 Preliminary Concepts for Jamaica Bay Storm Surge Barrier Foundations

For the Jamaica Bay area, the general geologic setting is 5-40ft of loose sand and silt underlain by medium dense sand and gravels. Foundation design for gate piers was investigated in detail in the USACE-GRR (2016). The study evaluated horizontal loads, vertical loads, and overturning moments and modeled the performance of a base slab supported by steel piles, both pipe and HP sections. The modeling software used was USACE's pile group analysis software Combined Pile Group Analysis (CPGA). The concept proposed by the study was a thick concrete base slab supported by closely spaced 65 to 70 ft long battered steel pipe piles of 24 to 36 inch diameter, with a steel sheet pile cutoff wall.

A pile supported deep foundation is a viable option for this location, yet ultimately the foundation will be dependent on the selection of the gate type and the geometry of the gate and piers. Based on the limited geotechnical data and the previously provided conceptual design, the conceptual foundation types listed below are identified as being potentially viable for the Jamaica Bay storm surge barrier.

- Sector gate housing;
    - Option 1: Deep foundation elements, either piles or drilled shafts.
    - Option 2: Pre-installed drilled shaft foundations on the channel bottom, mated with prefabricated pier / abutment and gate recess elements which are either floated or lifted into place.
- Sector gate sills:
    - deep foundation elements, either piles or drilled shafts.
- Auxiliary flow gate superstructures:
    - deep foundation elements, either piles or drilled shafts.
- Dam sections:
    - deep foundation elements, either piles or drilled shafts.

4.6.4 Preliminary Foundation Concepts for Hackensack River Storm Surge Barrier Foundations

The general geologic setting for the Hackensack River location is as follows:

- 5-10ft of salt-marsh deposits along the banks, 5-10ft of alluvial deposits in the channel
- Underlain by soft to stiff varved silts, clays, and sand
- Sedimentary rock (shale or Arkosic sandstone) at 60-80ft below MLW.

Based on this limited geotechnical data, the conceptual foundation types listed below are identified as being potentially viable for the Hackensack River storm surge barrier.

- Sector gate housing;
    - Option 1: The gate recesses and foundation may be constructed conventionally within dewatered cofferdams and include a pile supported deep foundations

connected to a reinforced concrete base slab or may involve float-in or lift-in construction techniques.
    - Option 2: Pre-installed drilled shaft foundations mated with prefabricated pier / abutment and gate recess elements that are either floated or lifted into place.
- Sector gate sills:
    - deep foundation elements, either piles or drilled shafts.
- Auxiliary flow gate superstructures:
    - deep foundation elements, either piles or drilled shafts, coupled with mass footing (base slab) or seal to resist uplift forces, as required.
- Dam sections:
    - deep foundation elements, either piles or drilled shafts, coupled with mass footing (base slab) or seal to resist uplift forces, as required.

4.7 Synopsis Conceptual Design Development

The purpose of this report is to assess and document a preliminary assessment of the minimum required dimensions for the navigable passages and axillary flow gates of the storm surge barriers. A description and first conceptual design that outline the basis for key geometric characteristics (e.g. height, width, and depth of openings) will be provided to USACE-NAN Cost Engineering. This will enable the USACE-NAN to use a parametric cost model to estimate the approximate cost of construction of the storm surge barriers. This section provided the rationale and the development of the conceptual designs of the minimum dimensions of the navigable passages, auxiliary flow gates and the selection of gate types. The following sections (Section 5 through 11) provide a summary of the conceptual design and dimensions for all storm surge barriers.

5 VERRAZANO NARROWS CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Verrazano Narrows storm surge barrier.

5.1 Hydraulic Design Parameters

Table 5-1 below summarizes the key environmental criteria to which the storm surge barrier elevation is designed.

Table 5-1: Summary of Environmental Design Criteria.

Parameter	Head (ft)	Flood Side Water Level (ft, NAVD88)	Protected Side Water Level (ft, NAVD88)
Direct head	7.3	12.5	5.2
Reverse Head	-3.9	-2.5	1.4

Parameter	Value
SLR for design life through 2080 (ft)	1.8
1% AEP Design Water Level (ft NAVD88) year 2080	13.0
1% AEP Significant Wave Height (ft)	5.9
1% AEP Peak Wave Period (s)	6.0
1% AEP Overtopping Criterion (l/s/m)	50

5.2 Preliminary Geometric Design Verrazano Narrows Storm Surge Barrier

Following the considerations presented within section 4, preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 5-1. The existing cross-sectional flow area at the storm surge barrier location is approximately 355,000 ft2. The aggregated flow opening area provided within the conceptual design is 207,900 ft2 (approximately 59% of the existing flow area at that location). The sill elevation for the auxiliary flow lift gates is set to -60 ft NAVD88. This elevation exceeds the sill elevation of lift gates for constructed storm surge barriers (see Enclosure 2), but the particular conditions at this site will most likely require a design that will test conventional limits for both the navigable passage and the auxiliary flow gate structures. A sill elevation of -60ft NAVD88 is assumed to be progressive but still constructible and practicable at this stage to be used for the conceptual design.

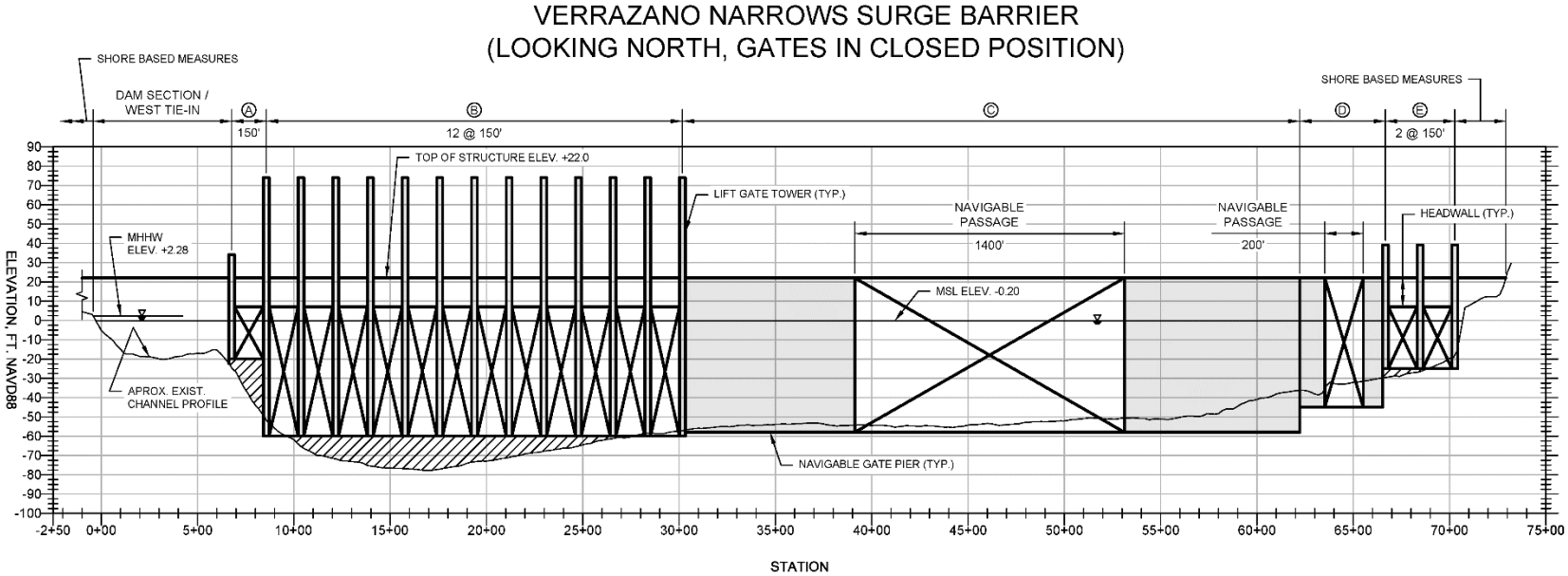


Figure 5-1: Conceptual Geometry of the Verrazano Narrows Storm Surge Barrier.

Table 5-2 lists the gate series and show that a total of 17 gates are preliminarily included within this design. Two (2) gated navigable passages and 15 auxiliary flow gates. The gate series are provided a lettered ID to clearly distinguish between the various sill elevations.

Table 5-2: Summary of Verrazano Narrows Storm Surge Barrier Geometry – Navigable Passages and Auxiliary Flow Gates ("Table VN-C")

Element[1] (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Notes	Top of Structure (ft, NAVD88)	Gate Height (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
A	1	-20	150	2,970	180	Auxiliary Flow (Vertical Lift Gate)	+22	27	34
B	12	-60	150	107,640	2,190	Deepest Auxiliary Flow Gate (Vertical Lift Gate)	+22	67	74
C	1	-58	1400	80,920	3,220	Navigable Passage (Floating Sector Gate)	+22	80	N/A
D	1	-45	200	8,960	460	Secondary Navigable Passage (Sector Gate)	+22	67	N/A
E	2	-25	150	7,440	390	Auxiliary Flow (Vertical Lift Gate)	+22	32	39
West Dam	0			0	720	Dam Section - Staten Island	+22	N/A	N/A
Total	**17**			**207,930**	**7,160**				

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths equal 65% of gate width for sector gates and the span for auxiliary structures equals gate width plus pier widths (pier widths equal 30 ft).
3. Top of single leaf lift gate with headwall at +7ft

6 THROGS NECK CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Throgs Neck storm surge barrier.

6.1 Hydraulic Design Parameters

Table 6-1 summarizes the key environmental criteria to which the storm surge barrier elevation is designed.

Table 6-1: Summary of Environmental Design Criteria

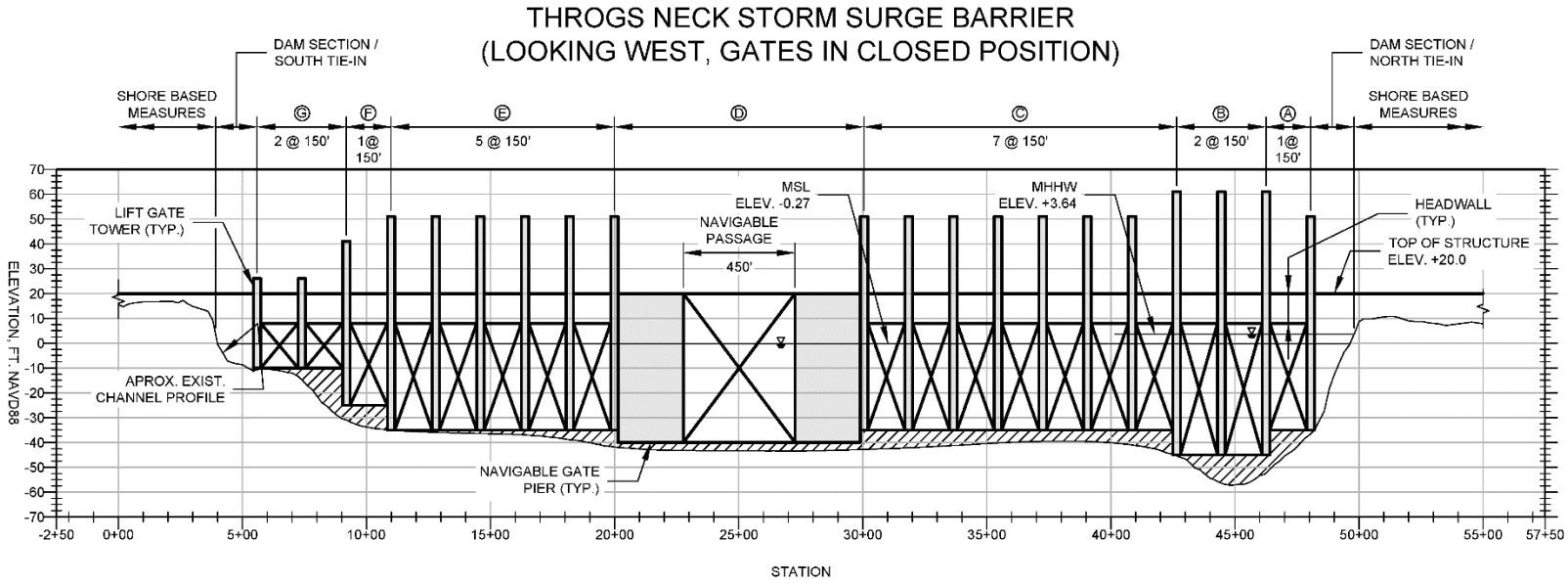
Parameter	Head (ft)	Flood Side Water Level (ft, NAVD88)	Protected Side Water Level (ft, NAVD88)
Direct head	16.8	14.2	-2.6
Reverse Head	-10.9	-5.2	5.7

Parameter	Value
SLR for design life through 2080 (ft)	1.8
1% AEP Design Water Level (ft NAVD88) year 2080	14.4
1% AEP Significant Wave Height (ft)	4.3
1% AEP Peak Wave Period (s)	4.0
1% AEP Overtopping Criterion (l/s/m)	50

6.2 Preliminary Geometric Design Throgs Neck Storm Surge Barrier

Following the considerations presented within section 4, a preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 6-1. The existing cross-sectional flow area at the proposed storm surge barrier location is approximately 171,400 ft^2. The aggregated flow opening area provided within the conceptual design is 106,500 ft2 (approximately 62% of the existing flow area at that location).

It is proposed that this geometry will be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design.



LEGEND:

Ⓐ GATE STRUCTURE SERIES (PER TABLE TN-C) - SEE NOTE 2.

☒ GATE IN CLOSED POSITION

▨ GATE SILL AND FOUNDATION (TBD)

NOTES:

1. THIS ELEVATION VIEW SHOWS A CONCEPTUAL GEOMETRIC DESIGN. THIS PRELIMINARY DEPICTION OF THE GEOMETRY OF THE NAVIGABLE PASSAGES, AUXILIARY FLOW GATES AND STORM SURGE BARRIER CONFIGURATION SHALL NOT BE CONSTRUED AS RECOMMENDATIONS OR REQUIREMENTS FOR ACTUAL DESIGN FOR IMPLEMENTATION. SIGNIFICANT ADDITIONAL STUDY IS REQUIRED TO SUBSTANTIATE THE CONCEPTUAL DESIGN OF THIS STORM SURGE BARRIER.
2. GATE SERIES ARE LABELED AS N@X FT, WHERE N IS NUMBER OF GATES AND X IS CLEAR DISTANCES BETWEEN PIERS (UNLESS OTHERWISE NOTED).
3. EXISTING FLOW AREA WITHOUT STORM SURGE BARRIER EQUALS 171,400 SF. WITH THE STRUCTURE IN PLACE THE FLOW AREA EQUALS 105,600 SF. (62% OF EXISTING).

Figure 6-1: Conceptual Geometry of the Throgs Neck Storm Surge Barrier.

Table 6-2 lists the gate series and show that a total of 19 gates are preliminarily included within this design. One (1) navigation gate and 18 auxiliary flow gates. The gate series are provided a lettered ID to clearly distinguish between the various sill elevations. The vertical lift gate sill elevations vary between -10ft NAVD88 and -45ft NAVD88.

Table 6-2: Summary of Throgs Neck Storm Surge Barrier Geometry – Navigable Passages and Auxiliary Flow Gates ("Table TN-C")

Element[1] (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Notes	Top of Structure (ft, NAVD88)	Gate Height (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
A	1	-35	150	5,210	180	Auxiliary Gate (vertical lift gate)	+20	43	51
B	2	-45	150	13,419	360	Auxiliary Gate (vertical lift gate)	+20	53	61
C	7	-35	150	36,467	1,290	Auxiliary Gate (vertical lift gate)	+20	43	51
D	1	-40	450	17,879	1,036	Navigable Passage (floating sector gate)	+20	60	N/A
E	5	-35	150	26,048	900	Auxiliary Gate (vertical lift gate)	+20	43	51
F	1	-25	150	3,710	180	Auxiliary Gate (vertical lift gate)	+20	33	41
G	2	-10	150	2,919	390	Auxiliary Gate (vertical lift gate)	+20	18	26
North Dam	0	N/A	N/A	0	175	Dam Section - Westchester	+20	N/A	N/A
South Dam	0	N/A	N/A	0	165	Dam Section - Queens	+20	N/A	N/A
Total	**19**			**105,650**	**4,676**				

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths equal 65% of gate width for sector gates and the span for auxiliary structures equals gate width plus pier widths (pier widths equal 30 ft).
3. Top of single leaf lift gate with headwall at +8ft

7 ARTHUR KILL CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Arthur Kill storm surge barrier.

7.1 Hydraulic Design Parameters

Table 7-1 summarizes the key environmental criteria to which the storm surge barrier elevation is designed.

Table 7-1: Summary of Environmental Design Criteria.

Parameter	Head (ft)	Flood Side Water Level (ft, NAVD88)	Protected Side Water Level (ft, NAVD88)
Direct head	9.5	13.1	3.6
Reverse Head	-7.7	-5.1	2.6

Parameter	Value
SLR for design life through 2080 (ft)	1.8
1% AEP Design Water Level (ft NAVD88) year 2080	14.6
1% AEP Significant Wave Height (ft)	3.8
1% AEP Peak Wave Period (s)	3.3
1% AEP Overtopping Criterion (l/s/m)	50

7.2 Preliminary Geometric Design for the Arthur Kill Storm Surge Barrier

Following the considerations presented within the chapter 4, a preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 7-1. The existing cross-sectional flow area at the proposed storm surge barrier location is approximately 53,900 ft^2. The aggregated flow opening area provided within the conceptual design is 25,200 ft2 (approximately 47% of the existing flow area at that location). The auxiliary flow gates are relatively small compared to the navigable passage and it is recommended that further analyses investigate the need, positioning and size of auxiliary flow gates for this storm surge barrier. It is proposed that this geometry will be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design.

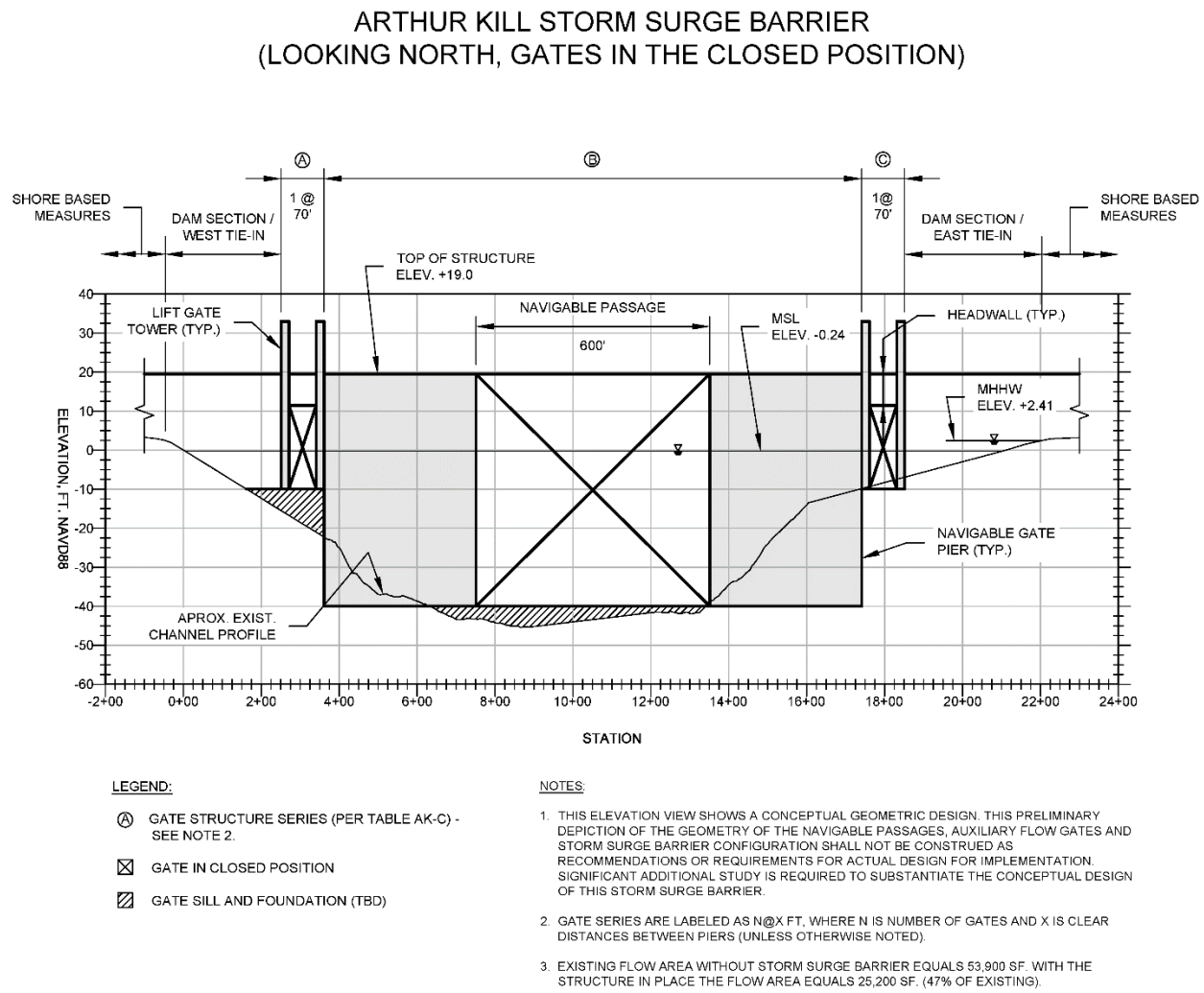


Figure 7-1: Conceptual Geometry of the Arthur Kill Storm Surge Barrier.

Table 7-2 lists the gate series and show that a total of three (3) gates are preliminarily included within this design. One (1) navigation gate and two (2) auxiliary flow gates. The gate series are provided a lettered ID to clearly distinguish between the various sill elevations. The vertical lift gate sill elevations are at -10ft NAVD88.

Table 7-2: Summary of Storm Surge Barrier Geometry – Navigable Passages and Auxiliary Flow Gates ("Table AK-C")

Element[1] (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Notes	Top of Structure (ft, NAVD88)	Gate Height (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
A	1	-10	70	683	110	Auxiliary Gate (vertical lift gate)	+19	17	+24
B	1	-40	600	23,856	1,380	Navigable Passage (floating sector gate)	+19	59	N/A
C	1	-10	70	683	110	Auxiliary Gate (vertical lift gate)	+19	18	+24
West Dam	0	N/A	N/A	0	295	Dam Section West Bank (New Jersey)	+19	N/A	N/A
East Dam	0	N/A	N/A	0	350	Dam Section East Bank (Staten Island)	+19	N/A	N/A
Total	**3**			**25,222**	**2,245**				

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths equal 65% of gate width and Span for auxiliary structures equals gate width plus pier widths (pier widths equal 30 ft).
3. Top of single leaf lift gate with headwall at +7ft

8 OUTER HARBOR CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Outer Harbor storm surge barrier.

8.1 Hydraulic Design Parameters

Table 8-1 summarizes the key environmental criteria to which the storm surge barrier elevation is designed.

Table 8-1: Summary of Environmental Design Criteria.

Parameter	Head (ft)	Flood Side Water Level (ft, NAVD88)	Protected Side Water Level (ft, NAVD88)
Direct head	9.9	10.4	0.5
Reverse Head	-5.3	-5.5	-0.3

Parameter	Value
SLR for design life through 2080 (ft)	1.8
1% AEP Design Water Level (ft NAVD88) year 2080	12.2
1% AEP Significant Wave Height (ft)	16.1
1% AEP Peak Wave Period (s)	15.1
1% AEP Overtopping Criterion (l/s/m)	50

8.2 Preliminary Geometric Design for the Outer Harbor Storm Surge Barrier

Following the considerations presented within section 4, a preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 8-1 through Figure 8-5. The existing cross-sectional flow area at the proposed storm surge barrier location is approximately 1,027,300 ft^2. The aggregated flow opening area provided within the conceptual design is 570,000 ft^2 (approximately 56% of the existing flow area at that location). This preliminary geometry should be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design.

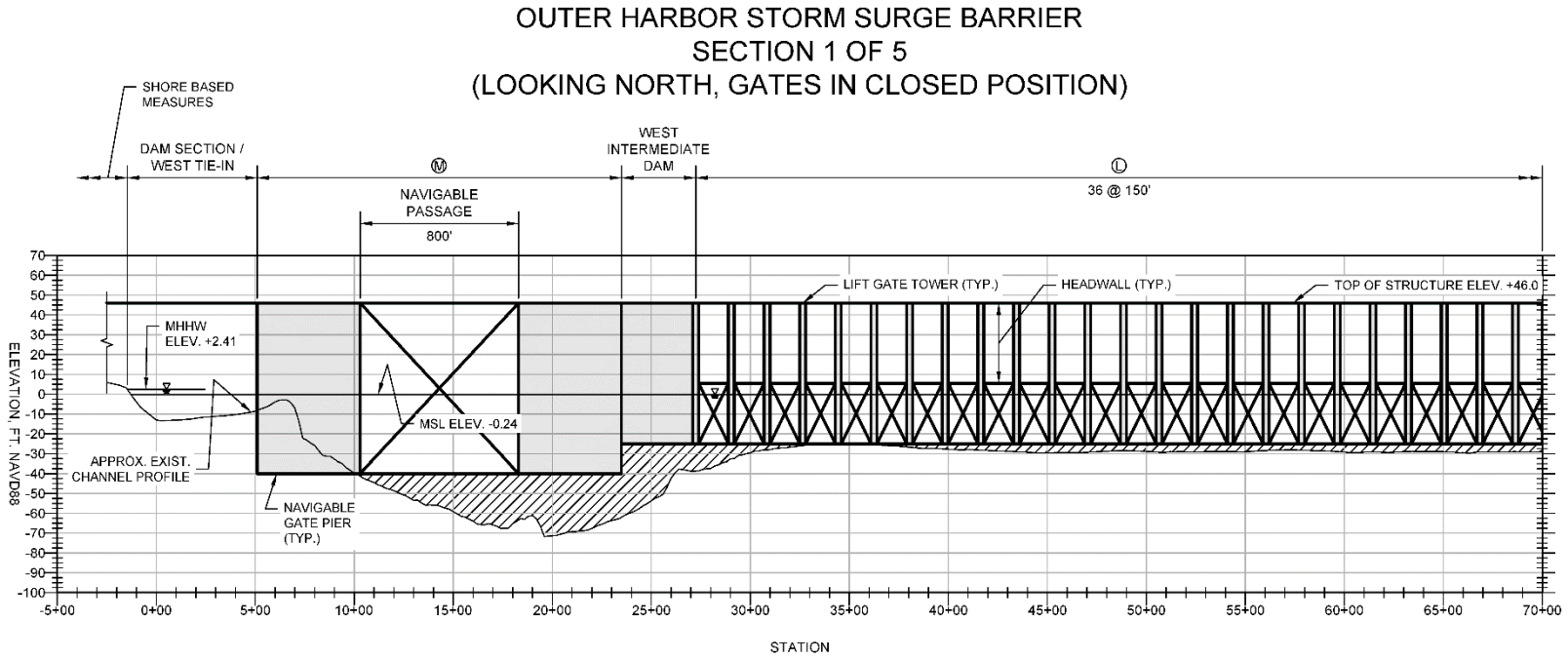


Figure 8-1: Conceptual Geometry of the Outer Harbor Storm Surge Barrier – section 1 of 5.

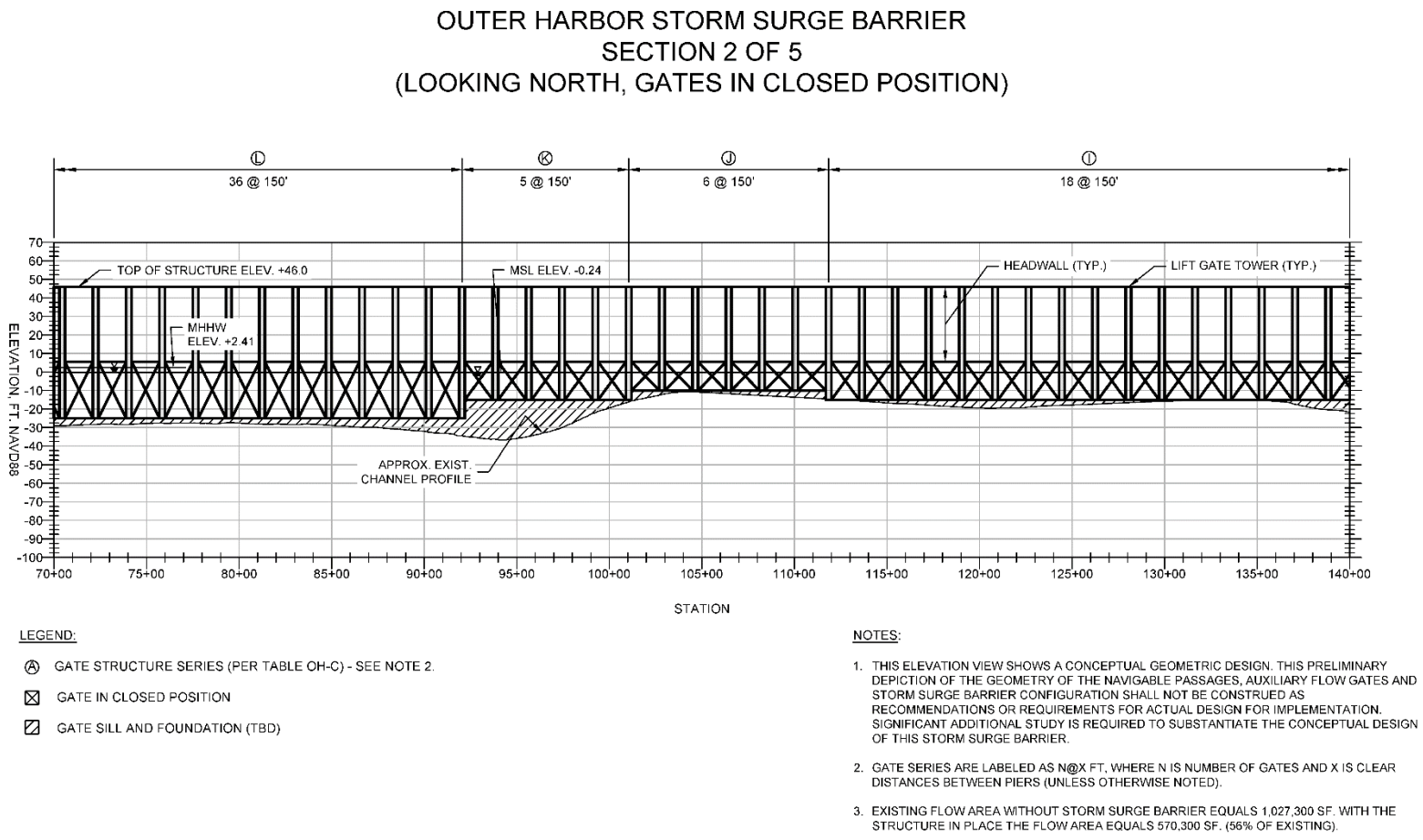


Figure 8-2: Conceptual Geometry of the Outer Harbor Storm Surge Barrier – section 2 of 5

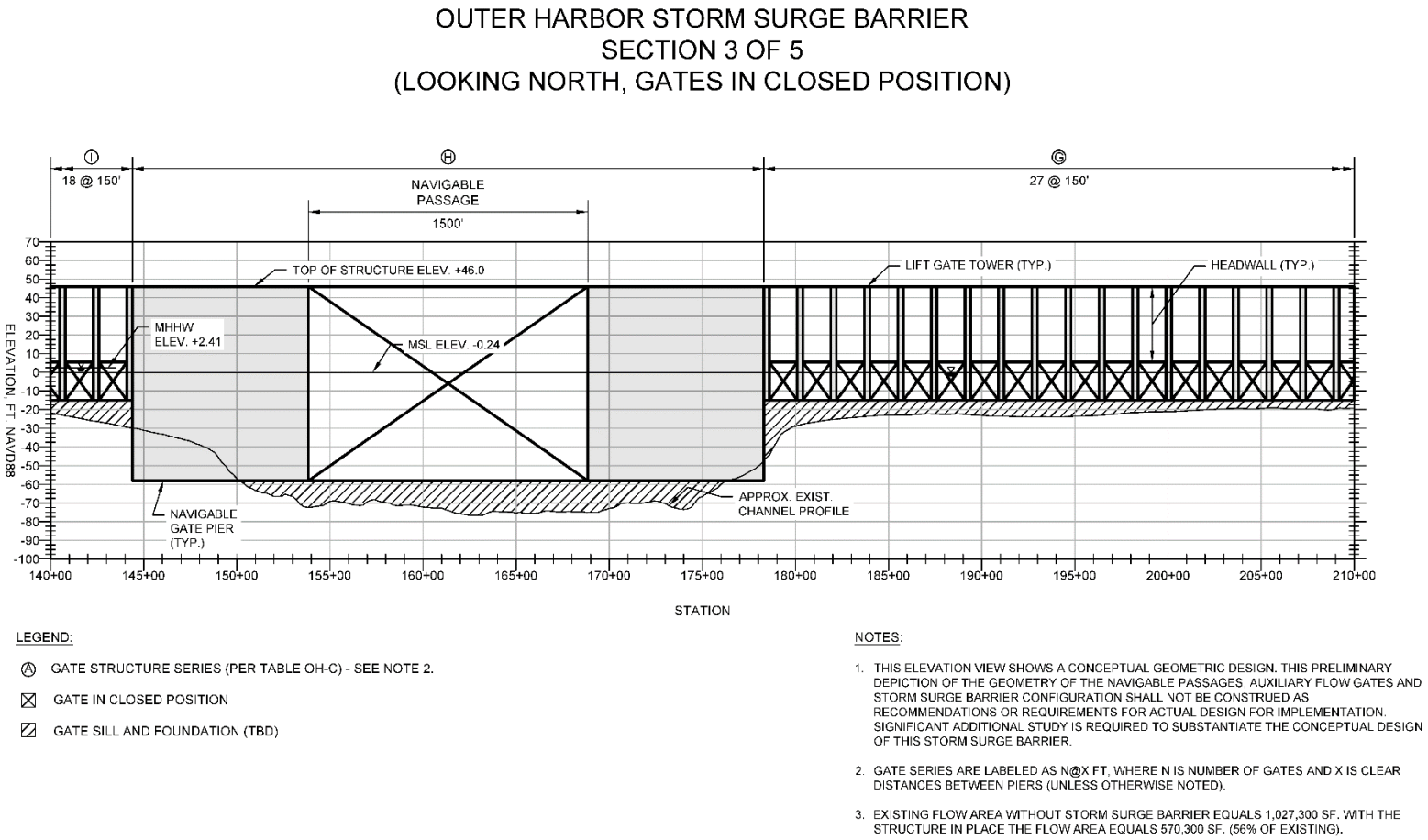


Figure 8-3: Conceptual Geometry of the Outer Harbor Storm Surge Barrier – section 3 of 5

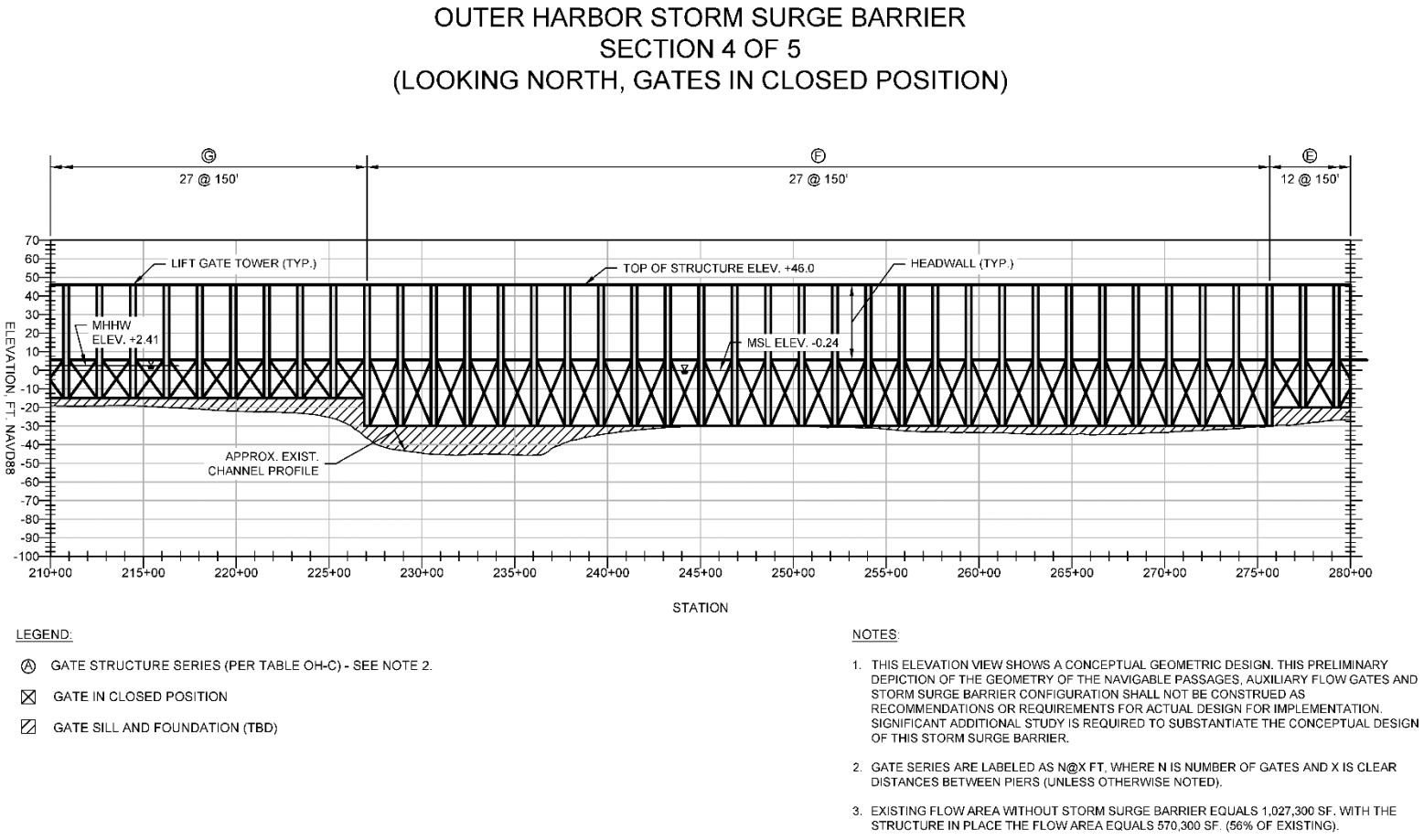


Figure 8-4: Conceptual Geometry of the Outer Harbor Storm Surge Barrier – section 4 of 5

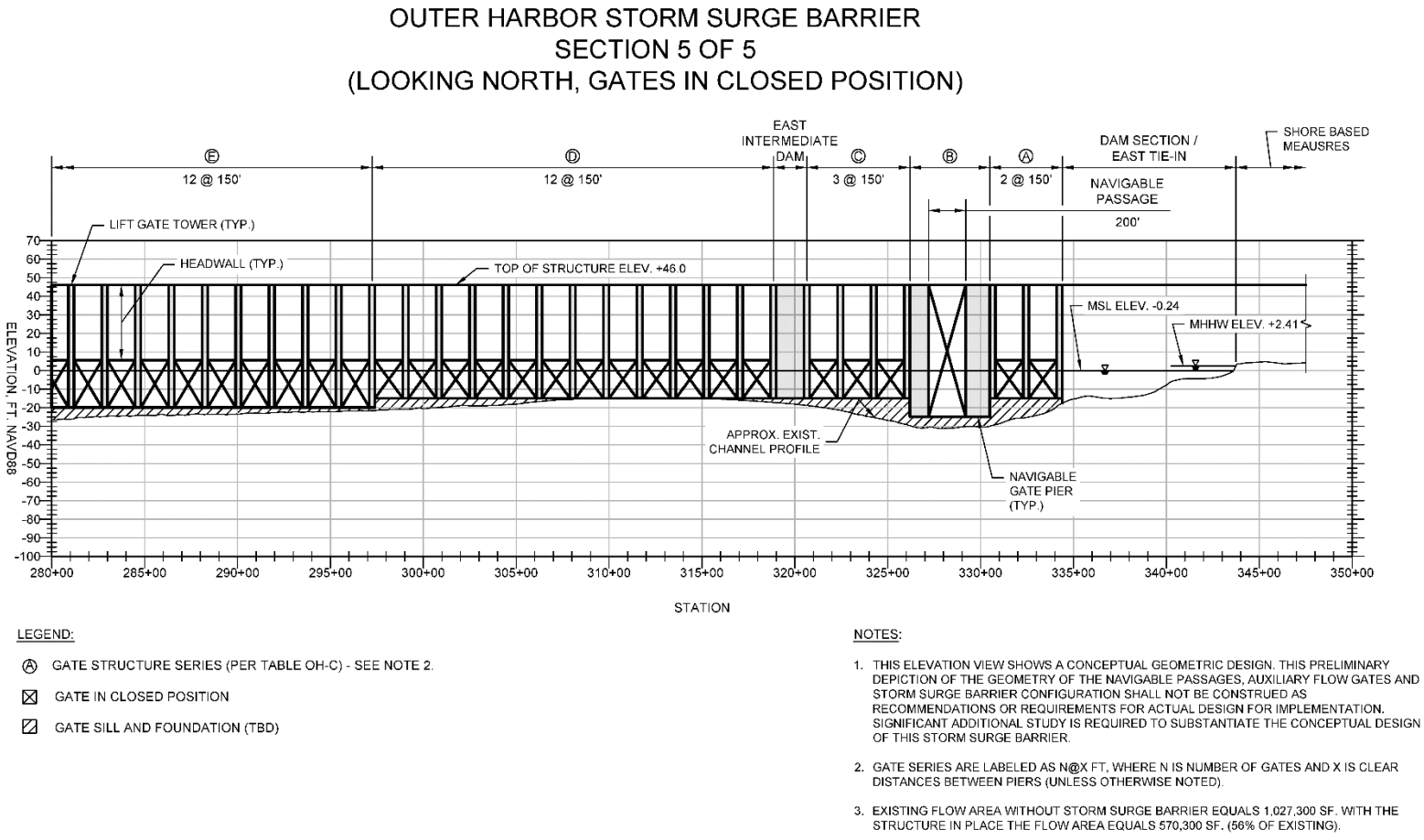


Figure 8-5: Conceptual Geometry of the Outer Harbor Storm Surge Barrier – section 5 of 5

Table 8-2 lists the gate series and show that a total of 151 gates are preliminarily included within this design. Three (3) navigation gates and 148 auxiliary flow gates. The gate series are provided a lettered ID to clearly distinguish between the various sill elevations. The vertical lift gate sill elevations vary between -10ft NAVD88 and -30ft NAVD88 in increments of 5ft.

Table 8-2: Summary of the Outer Harbor Storm Surge Barrier Geometry – Navigable Passages and Auxiliary Flow Gates ("Table OH-C")

Element[1] (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Notes	Top of Structure (ft, NAVD88)	Gate Height (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
A	2	-15	150	4,428	390	Auxiliary Flow Gate (Vertical Lift Gate)	+46	22	29
B	1	-25	200	4,952	460	Navigable Passage Rockaway Inlet (sector gate, vertical axes)	+46	71	N/A
C	3	-15	150	6,642	570	Auxiliary Flow Gate (Vertical Lift Gate)	+46	22	29
D	12	-15	150	26,568	2,190	Auxiliary Flow Gate (Vertical Lift Gate)	+46	22	29
E	12	-20	150	35,568	2,160	Auxiliary Flow Gate (Vertical Lift Gate)	+46	27	34
F	27	-30	150	120,528	4,860	Deepest Auxiliary Flow Gate (Vertical Lift Gate)	+46	37	44
G	27	-15	150	59,778	4,890	Auxiliary Flow Gate (Vertical Lift Gate)	+46	22	29
H	1	-58	1,500	86,640	3,450	Navigable Passage (floating sector gate)	+46	104	N/A
I	18	-15	150	39,852	3,240	Auxiliary Flow Gate (Vertical Lift Gate)	+46	22	29

Element[1] (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Notes	Top of Structure (ft, NAVD88)	Gate Height (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
J	6	-10	150	8,784	1,080	Auxiliary Flow Gate (Vertical Lift Gate)	+46	17	24
K	5	-15	150	11,070	900	Auxiliary Flow Gate (Vertical Lift Gate)	+46	22	29
L	36	-25	150	133,704	6,510	Auxiliary Flow Gate (Vertical Lift Gate)	+46	32	39
M	1	-40	800	31,808	1,840	Navigable Passage Sandy Hook (floating sector gate)	+46	86	N/A
East Intermediate Dam	0	N/A	N/A	0	150	Intermediate Dam	+46	N/A	N/A
West Intermediate Dam	0	N/A	N/A	0	360	Intermediate Dam	+46	N/A	N/A
East Dam	0	N/A	N/A	0	935	Dam Section - Breezy Point, NY	+46	N/A	N/A
West Dam	0	N/A	N/A	0	655	Dam Section - Sandy Hook, NJ	+46	N/A	N/A
Total	**151**			**570,322**	**34,640**				

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths equal 65% of gate width and Span for auxiliary structures equals gate width plus pier widths (pier widths equal 30 ft).
3. Top of single leaf lift gate with headwall at +7ft

9 KILL VAN KULL CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Kill Van Kull storm surge barrier.

9.1 Hydraulic Design Parameters

Table 9-1 below summarizes the key environmental criteria from Chapter 3 that determine the elevation of the conceptual storm surge barrier design.

Table 9-1: Summary of Environmental Design Criteria.

Parameter	Head (ft)	Flood Side Water Level (ft, NAVD88)	Protected Side Water Level (ft, NAVD88)
Direct head	9.3	11.2	1.9
Reverse Head	-5.4	-3.3	2.0

Parameter	Value
SLR for design life through 2080 (ft)	1.8
1% AEP Design Water Level (ft NAVD88) year 2080	12.9
1% AEP Significant Wave Height (ft)	6.0
1% AEP Peak Wave Period (s)	6.2
1% AEP Overtopping Criterion (l/s/m)	50

9.2 Preliminary Geometric Design for the Kill van Kull Storm Surge Barrier

Following the considerations presented within this chapter, a preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 9-1. The existing cross-sectional flow area at the proposed storm surge barrier location is approximately 121,000 ft^2. The aggregated flow opening area provided within the conceptual design is 66,000 ft2 (approximately 55% of the existing flow area at that location).

It is proposed that this geometry will be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design. Table 9-2 provides a summary of the overall storm surge barrier geometry presented herein.

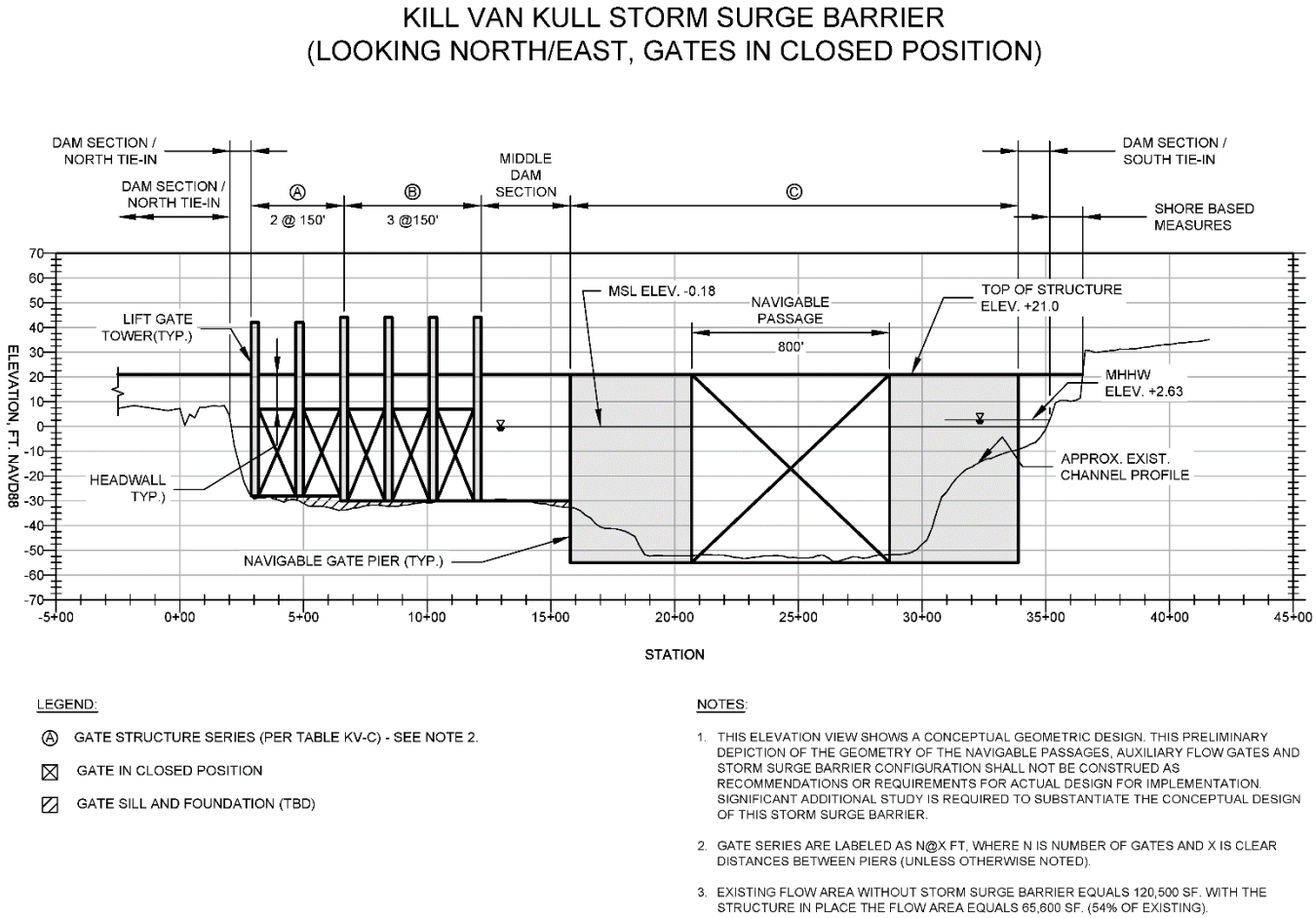


Figure 9-1: Conceptual Geometry of the Kill Van Kull Storm Surge Barrier

Table 9-2 lists the gate series and show that a total of 6 gates are preliminarily included within this design. One (1) navigation gate and 5 auxiliary flow gates. The gate series are provided a lettered ID to clearly distinguish between the various sill elevations. The vertical lift gate sill elevations vary from -28ft to -30ft NAVD88 and the navigable gate sill is set at -55ft NAVD88.

Table 9-2: Summary of Kill van Kull Storm Surge Barrier Geometry – Navigable Passage and Auxiliary Flow Gates (Table KVK-C)

Element[1] (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Notes	Top of Structure (ft, NAVD88)	Gate Height (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
A	2	-28	150	8,346	360	Auxiliary Gate (vertical lift gate)	+21	35	42
B	3	-30	150	13,419	570	Auxiliary Gate (vertical lift gate)	+21	37	44
C	1	-55	800	43,856	1,840	Navigable Passage (floating sector gate)	+21	76	N/A
North Dam	0	N/A	N/A	0	85	Bayonne Dam	+21	N/A	N/A
South Dam	0	N/A	N/A	0	125	State Island Dam	+21	N/A	N/A
Intermediate Dam	0	N/A	N/A	0	300	Intermediate Dam	+21	N/A	N/A
Total	**6**			**65,621**	**3,280**				

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths equal 65% of gate width and Span for auxiliary structures equals gate width plus pier widths (pier widths equal 30 ft).
3. Top of single leaf lift gate with headwall at +7ft

10 JAMAICA BAY CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Jamaica Bay storm surge barrier.

10.1 Hydraulic Design Parameters

Table 10-1 below summarizes the key environmental criteria from Chapter 3 that determine the elevation of the conceptual storm surge barrier design

Table 10-1: Summary of Environmental Design Criteria.

Parameter	Head (ft)	Flood Side Water Level (ft, NAVD88)	Protected Side Water Level (ft, NAVD88)
Direct head	11.8	11.3	-0.6
Reverse Head	-1.7	-1.4	0.3

Parameter	Value
SLR for design life through 2080 (ft)	1.8
1% AEP Design Water Level (ft NAVD88) year 2080	11.6
1% AEP Significant Wave Height (ft)	4.8
1% AEP Peak Wave Period (s)	4.6
1% AEP Overtopping Criterion (l/s/m)	50

10.2 Preliminary Geometric Design for the Jamaica Bay Storm Surge Barrier

Following the considerations presented within the previous chapters, a preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 10-1. The geometry and gate type selection relied in large part on the design as presented in USACE-NAN 2016, including the two sector gates for the navigable passages. In addition the flow area was further increased with the inclusion of additional flow gates to maximize the flow area. The existing cross-sectional flow area at the proposed storm surge barrier location is approximately 108,000 ft^2. The aggregated flow opening area provided within the conceptual design is 61,700 ft2 (approximately 57% of the existing flow area at that location). This preliminary geometry should be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design.

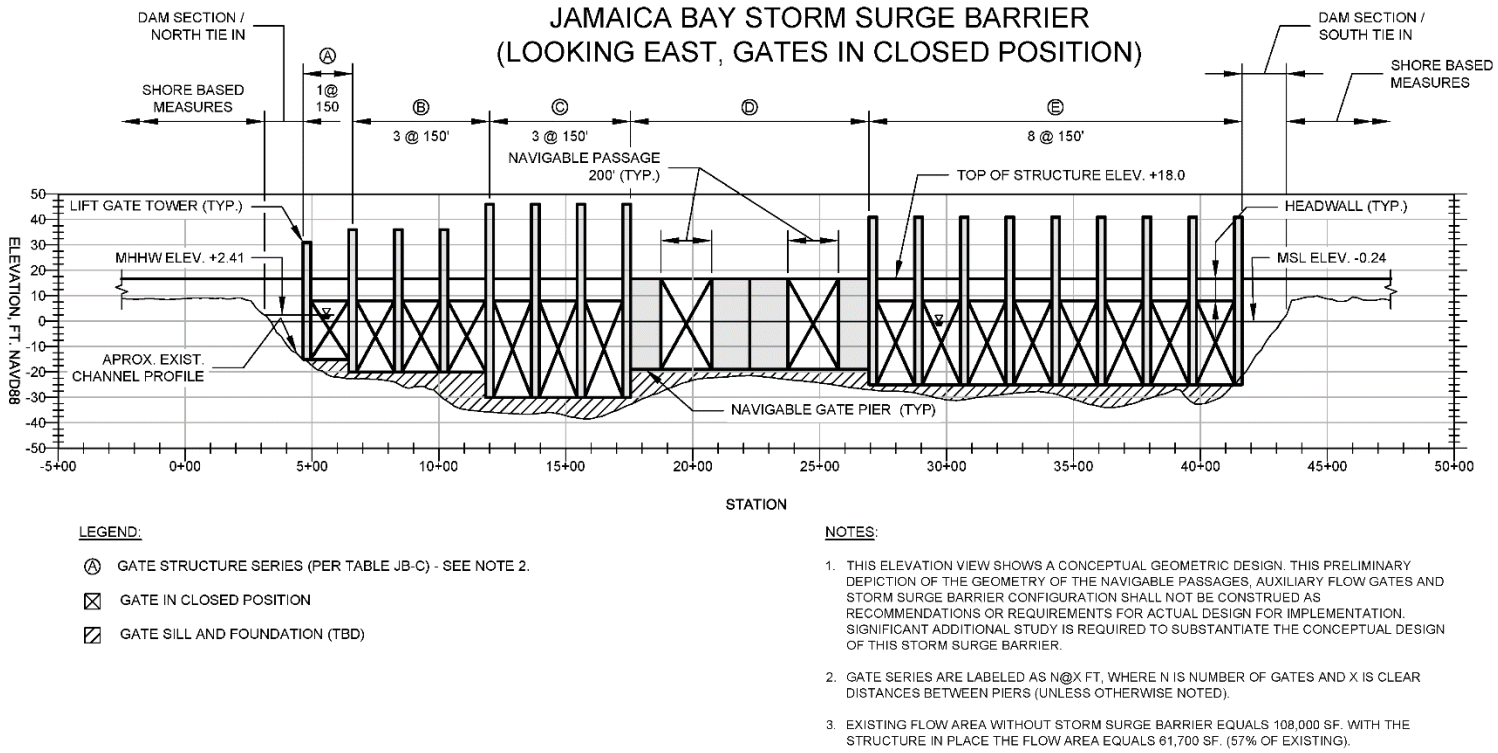


Figure 10-1: Conceptual Geometry of the Jamaica Bay Storm Surge Barrier

Table 10-2 lists the gate series and show that a total of 17 gates are preliminarily included within this design. Two (2) navigation gates and 15 auxiliary flow gates. The gate series are provided a lettered ID to clearly distinguish between the various sill elevations.

Table 10-2: Summary of Jamaica Bay Storm Surge Barrier Geometry – Navigable Passages and Auxiliary Flow Gates ("Table JB-C")

Element[1] (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Notes	Top of Structure (ft, NAVD88)	Gate Height (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
A	1	-15	150	2,214	180	Auxiliary Flow (Lift Gate)	+18	22	29
B	3	-20	150	8,892	540	Auxiliary Flow (Lift Gate)	+18	27	34
C	3	-30	150	13,392	570	Auxiliary Flow (Lift Gate)	+18	37	44
D	2	-19	200	7,504	1,000	Navigable Passage (Sector Gate)	+18	37	N/A
E	8	-25	150	29,712	1,470	Auxiliary Flow (Lift Gate)	+18	32	39
North Dam	0	N/A	N/A	0	150	Dam Section - North	+18	N/A	N/A
South Dam	0	N/A	N/A	0	175	Dam Section - South	+18	N/A	N/A
Total	**17**			**61,714**	**4,085**				

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths equal 65% of gate width and Span for auxiliary structures equals gate width plus pier widths (pier widths equal 30 ft).
3. Top of single leaf lift gate with headwall at +7ft

11 HACKENSACK CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Hackensack River storm surge barrier. The objective at this stage of the study is to provide USACE-NAN a description and first conceptual design which outline the basis for key geometric characteristics (e.g. height, width, and depth of openings). This will enable the USACE-NAN to use a parametric cost model to estimate the approximate cost of construction of the storm surge barrier.

11.1 Hydraulic Design Parameters

Table 11-1 below summarizes the key environmental criteria from Chapter 3 that determine the elevation of the conceptual storm surge barrier design.

Table 11-1: Summary of Environmental Design Criteria.

Parameter	Head (ft)	Upstream Water Level (ft, NAVD88)	Downstream Water Level (ft, NAVD88)
Direct head	TBD	12.2	TBD
Reverse Head	TBD	TBD	TBD

Parameter	Value
SLR for design life through 2080 (ft)	1.8
1% AEP Design Water Level (ft NAVD88) year 2080	12.2
1% AEP Significant Wave Height (ft)	3.2
1% AEP Peak Wave Period (s)	2.6
1% AEP Overtopping Criterion (l/s/m)	50

11.2 Preliminary Geometric Design

Following the considerations presented within the section 4, a preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 8-1, and as noted in section 4, tainter gates are selected for the auxiliary flow openings. The existing cross-sectional flow area at the proposed storm surge barrier location is approximately 26,400 ft^2. The aggregated flow opening area provided within the conceptual design is approximately 17,000 ft2 (approximately 64% of the existing flow area at that location). This preliminary geometry should be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design. Table 11-2 provides a summary of the overall storm surge barrier geometry presented herein.

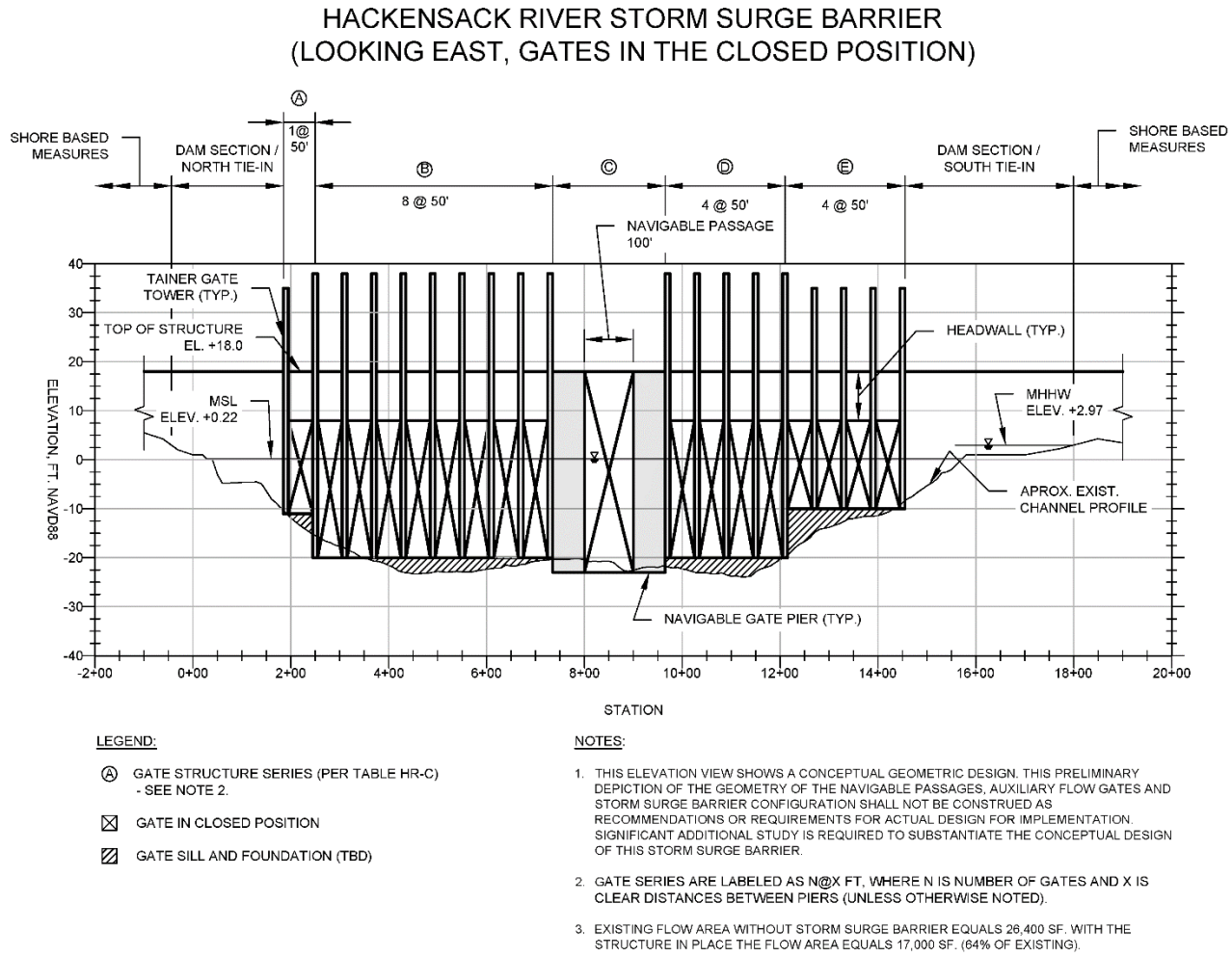


Figure 11-1: Conceptual Geometry of the Hackensack River Storm Surge Barrier.

Table 11-2 lists the gate series and show that a total of 18 gates are preliminarily included within this design. One (1) navigation gate and 17 auxiliary flow gates. The gate series are provided a lettered ID to clearly distinguish between the various sill elevations.

Table 11-2: Summary of the Hackensack River Storm Surge Barrier Geometry – Navigable Passage and Auxiliary Flow Gates (Table HR-C)

Element[1] (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Notes	Top of Structure (ft, NAVD88)	Gate Height (ft)	Gate Radius (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
A	1	-10	50	511	60	Auxiliary Flow (Tainter Gate)	+18	27	35	35
B	8	-20	50	8,088	490	Auxiliary Flow (Tainter Gate)	+18	17	45	38
C	1	-23	100	2,322	250	Navigable Passage	+18	41	55	38
D	4	-20	50	4,044	240	Auxiliary Flow (Tainter Gate)	+18	27	45	38
E	4	-10	50	2,044	250	Auxiliary Flow (Tainter Gate)	+18	17	35	35
North Dam	0	N/A	N/A	0	230	Dam Section - North Bank	+18	N/A	N/A	N/A
South Dam	0	N/A	N/A	0	345	Dam Section - South Bank	+18	N/A	N/A	N/A
Total	**18**			**17,009**	**1,865**					

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths equal 65% of gate width and Span for auxiliary structures equals gate width plus pier widths (pier widths equal 10 ft).
3. Tainter Gate with headwall at +8ft and a trunnion positioned at an elevation of +7ft NAVD88

12 RECOMMENDATIONS FOR FURTHER STUDY

12.1 Introduction

This study is the first where a suite of storm surge barriers is evaluated for the New York and New Jersey Harbor. The conceptual designs for the storm surge barriers as part of the HAT Study Alternatives, are based upon a broad yet comprehensive data analysis for the entire study area with equal level of detail for each storm surge barrier. The basis of design, albeit preliminary, are analogously and consistently prepared for all storm surge barrier locations and include amongst other items; the latest hydrodynamic storm surge modeling results to establish boundary conditions for design, AIS traffic data analyses and a basis for the minimum required dimensions of the navigable passages. Most importantly the conceptual designs and geometries of the storm surge barriers are evaluated using hydrodynamic models and are not solely analyzed on an individual basis but analyzed using a systems approach (USACE ERDC 2019). This document is the first step in an iterative design process using a systems approach, whilst previous completed studies in large part only provided singular concepts and did not assess impacts to the regional hydrodynamics, nor did those studies use such assessments to further the conceptual designs.

Furthermore, the gate types are selected based on a high-level but full review of hydraulic gate types and the applicability of such gates based on the review of constructed storm surge barriers. The conceptual design as presented herein is in part informed by the data and characteristics of other storm surge barriers that have been constructed throughout the world. As such the conceptual design is built upon proven concepts and principles which in turn improve the reliability of the overall concept; reliability is a key notion during the concept development of these storm surge barrier designs.

However, in some instances, the concepts considered are larger in scope and scale than those that currently exist in practice. Nonetheless, although some elements are proportionally larger, there is good confidence that the concepts presented are both constructible and feasible in their implementation.

Despite the depth and breadth of preliminary evaluation, this assessment of navigable passage widths and storm surge barrier configurations shall not be construed as definitive recommendations or requirements for actual design for implementation. Significant additional study is required to substantiate the width, location, and configuration of the navigable passages, and auxiliary flow gates, including a full evaluation of navigation, environmental, ecological, and cost considerations, amongst others.

The next sections provide a framework of additional studies and engineering analyses that should be considered, and what those efforts should, at a minimum, entail[7]. Certain topics can be expanded

[7] These sections are geared towards engineering analyses and studies, while it is recognized that environmental, economic, socio economic and other studies would be required similarly.

upon if there is a need to prioritize or expedite the refinement of a selected set of storm surge barrier designs.

12.2 Iterative Design - Next Steps

Following the analyses described within this report there are several overarching topics that warrant further investigation and are considered logical next steps as part of an iterative design process. For completeness these topics are summarized here. One of the main tasks to be evaluated for each storm surge barriers is a siting study. A complete siting study for the storm surge barriers would evaluate pros and cons of various alignment and conceptual design alternatives for each storm surge barrier. Such analyses should consider the following topics:

- Navigable passage dimensions:
    - The required width of the navigable passage
    - Requirements for one-way versus two-way traffic for the navigable passage (further detailed below under Navigation – Section 12.5 below)
- The impact of current velocities on navigation and the required dimensions of openings within the storm surge barrier to minimize impacts to navigation conditions. In particular;
    - For the Outer Harbor storm surge barrier preliminary modeling results indicate that under the configuration of HAT Study Alternative 2, flow velocities through the navigable passages of the Sandy hook Channel and Ambrose Channel could exceed 3knts during normal hydrometeorological conditions more than 26% and 18% of the time respectively (USACE ERDC 2019);
    - For the Verrazano Narrows storm surge barrier preliminary modeling results indicate that under the configuration of HAT Study Alternative 3A flow velocities through the navigable passage could exceed 3knts more than 16% of the time during normal hydrometeorological conditions (USACE ERDC 2019). The design dimensions proposed herein exceed the modeled dimensions and it is recommended to reassess the change in impacts on flow velocities and tidal amplitude through numerical modeling for this particular storm surge barrier.
    - For the Jamaica Bay storm surge barrier similar conditions were observed for the HAT Study Alternatives 3A, 3B and 4, yet exceedance of 3knt current velocity was around 10% of the time for all alternatives (USACE ERDC 2019).
    - Alternate storm surge barrier alignments with a longer span and relatively lower percentage of flow impediment may alleviate such concerns.
- Alternative gate types for the navigable passage;
    - It should be noted that this is a high-level evaluation as no site-specific borings are available and designs for the gates are still conceptual in nature. Further recommendations regarding geotechnical evaluations are provided below.
    - For the Outer Harbor and Verrazano Narrows storm surge barrier the floating sector gate span for the Ambrose Channel is beyond the limits of previously constructed comparable gates (>1,400ft wide opening vs. 1,190ft and 660ft for Maeslant and

St. Petersburg Storm Surge Barrier respectively). The gate housing of the floating sector gates occupies a relative substantial portion of the cross-section of the existing waterway and reduces the existing flow area. Following the current velocity concerns raised above, alternate gate configurations which occupy a smaller percentage of the existing cross-section should be investigated further.

- For the Throgs Neck storm surge barrier, the floating sector gate has been preliminarily selected for the navigable passage. The floating sector gate span (450 ft) is within the limits of previously constructed comparable gates yet, alternate gate configurations which occupy a smaller percentage of the existing cross-section, should be investigated further to limit impacts on the tidal flow exchange. One such an example for this location could be flap gates or a barge gate;
  - The largest barge gate constructed as part of a storm surge barrier spans an opening of 150 ft (IHNC storm surge barrier). A barge gate with the proposed span for Throgs Neck has not yet been constructed but has been considered elsewhere (van Ledden, et al. 2012)
  - Flap gates can be placed in series to provide a large navigable opening, but the reverse head conditions for this site and the complexity of such structures may make it a less favorable candidate.
- For those locations were air clearances are restricted, or the option exists to make air clearances restricted, lift gates may be suitable alternates.
- Apart from the examples provided here, it is recommended to analyze, cost and compare alternate gate types for all selected storm surge barriers. Potential alternate gate types for each barrier location are indicated in Appendix F.

- Alternative Gate types for the auxiliary flow gates,
  - For the majority of the storm surge barriers, the lift gate was selected for the auxiliary flow structures. At those locations where reverse head conditions are of limited concern, a tainter gate is most likely a viable option too. For example, the USACE 70ies study for Jamaica Bay (USACE-WES 1976) considered tainter gates. Applicability of tainter gates will, in large part depend on reverse head conditions and the potential for relatively high load concentration on the trunnion bearings. In addition, for locations that are shallower with a fairly even bathymetric profile, rotating segment gates or inflatable gates could be considered.
  - For the Verrazano Narrows lift gates were preliminarily selected and the sill elevation was set at -60ft NAVD88 which exceeds sill elevations of constructed lift gate structures in storm surge barriers. Additional flow area to alleviate current magnitude concerns may be realized if the sill elevation is lowered even further. It would come at additional cost and a gate type study is recommended to quantitatively assess and compare different gate types.
  - The conceptual designs presented maximized the number of auxiliary flow gates to the extent practicable, to minimize impacts on flow exchange. Tor some storm

surge barriers the number exceeded those from previous studies, e.g. for the Jamaica Bay and Hackensack River storm surge barrier. At some locations a different configuration (less openings or slightly less total flow area) may result in no appreciable difference in tidal flow exchange but could potentially be more economical.

  - Apart from the examples provided here, it is recommended to analyze, cost and compare alternate auxiliary flow gate types for all selected storm surge barriers. Potential alternate gate types for each barrier location are indicated in Appendix G.

- Geotechnical site conditions,
  - Foundation concepts were based on a high-level evaluation as no site-specific borings are available and the designs for the gates are still very conceptual. It is recommended that a geotechnical data gap analyses be completed, site specific geotechnical data gathered, and a generic geotechnical profile established for each storm surge barrier location.

12.3 Constructability

The siting and eventual construction of a storm surge barrier is a complex undertaking, and practical constraints may influence the eventual design based on constructability considerations. Constructability will influence design considerations, structure type, project costs, phasing requirements, and schedule. Large civil works projects involving marine-based construction are represented by significant complexity and cost factors. These factors are generally exacerbated as water depth, flow velocities, and proximity to navigation channels are considered. Likewise, structure configurations used to overcome the spatial and loading criteria for which the structure must perform also heavily influence complexity and cost. Hence, basic constructability assessments must be performed to consider viability and provide for proof of concept for foundations and structure types under consideration.

Constructability evaluations are an inherent part of any major civil works undertaking. Among the many considerations when considering constructability, the following should be considered:

- Maintenance of navigation and navigational impacts during construction
- General method of construction (e.g. in-the-dry, in-the-wet)
- Temporary works (e.g. cofferdams)
- Site access (e.g. barge-based work versus land-based access via temporary trestle)
- Site staging and laydown areas
- Material deliveries to the work site (e.g. floating concrete plant)
- Contractor capabilities, and the availability of both specialized contractors and equipment needed to perform the work
- Feasibility, availability, and locations of off-site fabrication areas for modular elements (e.g. graving dock for float-in elements)

- Variability of subsurface conditions, and methods used to address same t provide adequate foundations
- Impact from tides, current, weather, and other environmental factors on construction activities
- Extreme event scenarios, preparedness provisions, and similar risk considerations
- Potential availability of construction materials, including quality and quantity
- Waste and recycled materials considerations, including beneficial use
- Environmental considerations affecting construction activities (e.g. relocations, noise, work period restrictions)
- Construction schedule, including a variety of phasing and funding scenarios

12.4 Hydraulics, Hydrology, and the Aquatic Environment

The complexity of the regional hydraulics and hydrology warrants further study in the following topics.

- Permissible overtopping quantities and permissible leakage through the storm surge barrier to optimize structure elevation.
    - Currently an overtopping criterion of 50 l/s/m is applied, which could still be considered a conservative criterion as some coastal structures can accommodate higher overtopping discharges if properly designed for (USACE 2002).
    - Besides the proposed conventional option one alternate option that is recommended to be considered is a gated weir structure that allows for both flow through it during normal hydraulic and meteorological conditions while allowing for flow over the crest during severe storm surge conditions. The purpose of the storm surge barrier is to impede storm surge, which does not equate to complete blockage of the flow.
- Analyses of impacts to the tidal flow exchange and impacts to the tidal amplitude as a result of the proposed geometry. Such analyses should further the work completed by ERDC (USACE ERDC 2019) and continue the iterative design process to refine the storm surge barrier geometry, and include:
    - Assessment of the impact on water surface elevations, discharges and average velocities in the openings, and
    - Assessment of local hydraulic changes in the inner basin, harbor or bay such as local velocities and currents, salinities, tidal levels and circulation which are essential to pollution, fish and wildlife, and other environmental and ecological considerations
- Analyses of potential changes in tidal flow exchange and the impacts on both local and far field morphology

    - The net longshore sediment transport at both Sandy Hook and Rockaway Inlet are directed towards the New York Bight. Future analysis will need to evaluate the potential for erosion and sedimentation in the region of the storm surge barrier.
- Sea level rise sensitivity and adaptability analyses
    - Perform tests with different SLC scenarios and investigate changes in hydrostatic and dynamic loading as well as changes in overtopping discharge and identify options and project features that can provide for an adaptable design
    - Adaptive management may be necessary or structural improvements may be needed if the observed sea level rise exceeds the planning criteria; such provisions would be included in the design to accommodate improvements if and when needed.
- Impacts to water levels on the protected side during gate closures (reverse head conditions),
    - Analysis of inflows and potential for a rise in water levels on the protected side of the storm surge barrier after gate closure. This holds for all storm surge barriers discussed herein, but of particular note is the conceptual design for the Hackensack River storm surge barrier (USACE February 1989) that included a pump stations in line with the gated barrier.
    - Analysis of joint probability of river discharge (flood levels) and storm surge levels. This is in particular of interest for the Hackensack River storm surge barrier.
- Impacts to water levels to adjacent areas on the flood side of the storm surge barriers
- Analysis of impacts to water quality during and post gate closure.
- Analyses of potential changes in tidal flow exchange and impacts on salinity, water quality and ecology.

12.5 Navigation

The New York Bight, between Sandy Hook and the Rockaway Peninsula, is the principal entrance to the New York and New Jersey Harbor, and is one of the busiest waterways in the USA. Constructing a storm surge barrier across a navigation channel will require further study in the following areas.

- Waterway traffic
    - One-way versus two-way vessel passage, including meeting, passing, and overtaking
    - Number, frequency, and intensity of vessel passage
    - Vessel wait areas, queuing, and wait times
    - Storm surge barrier positioning and fairway lengths for maneuvering
    - Trends for future vessel traffic, including vessel size, frequency
    - Passage of recreational vessels
- Currents, cross currents, wind, tides, surge, weather, night, visibility, and other environmental considerations for vessel passage

- Navigation evaluations, including pilot and navigation industry input, and real-time simulations to assess, amongst others:
    - Flow and cross-current considerations
    - Gate approach and departure
    - Passing vessel assessments
- Requirements for Aids-to-Navigation, guide structures, and protective structures
- Requirements for vessel traffic service, including advisory / control / restrictions on navigation
- National security considerations.

12.6 Operations and Maintenance

Considerations for operations and maintenance affect the overall design philosophy. Operation and maintenance cost are a substantial part of the life cycle cost of storm surge barriers. Important factors that determine operation and maintenance costs are: 1) maintenance of the movable parts of the structure, 2) painting (steel) parts of the structure, 3) operations and maintenance personnel cost, 4) cost of an operational data and decision support network, 5) inspection of parts including submerged parts (after van Ledden, et al. 2012), and 6) Control systems, remote operation, emergency operation, redundant systems, 7) Size, scope, equipment, and location of facilities to support Operations and Maintenance. The following topics will require further study.

- Operational criteria for gate closure and the expected frequency of gate closures.
- Time scales for deployment, reliability and operation of gate and warning systems.
- Reliable operation of the storm surge barrier (gate closures) to obtain a reduction in flood risk.
- Reliable operation of the storm surge barrier gates to minimize the impacts of gate closures on navigation and the aquatic environment.
- Reduce to the extent practicable the complexity of Operation and Maintenance, Repair, Replacement and Rehabilitation (OMRR&R).

12.7 Multi-functionality of a Storm Surge Barrier Complex

Several the storm surge barriers discussed herein would be constructed in close vicinity to industrial and residential development areas. The tie-in to the shore-based perimeter flood risk reduction system and the integration of the form and function of the storm surge barrier would require further study. There may be opportunities to further blend form and function and assess shared uses and multi-functionality of this civil works complex such that it provides additional benefits to the community. Topics that require further study are:

- Inclusion of transportation infrastructure (roadways, bridges and tunnels).
- Potential for connections to existing transportation infrastructure.
- Inclusion of recreational, educational areas, and other considerations for public access, and,
- An assessment of aesthetics.

This report is part of a series of reports that document the preliminary analysis completed in support of the feasibility of the New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study.

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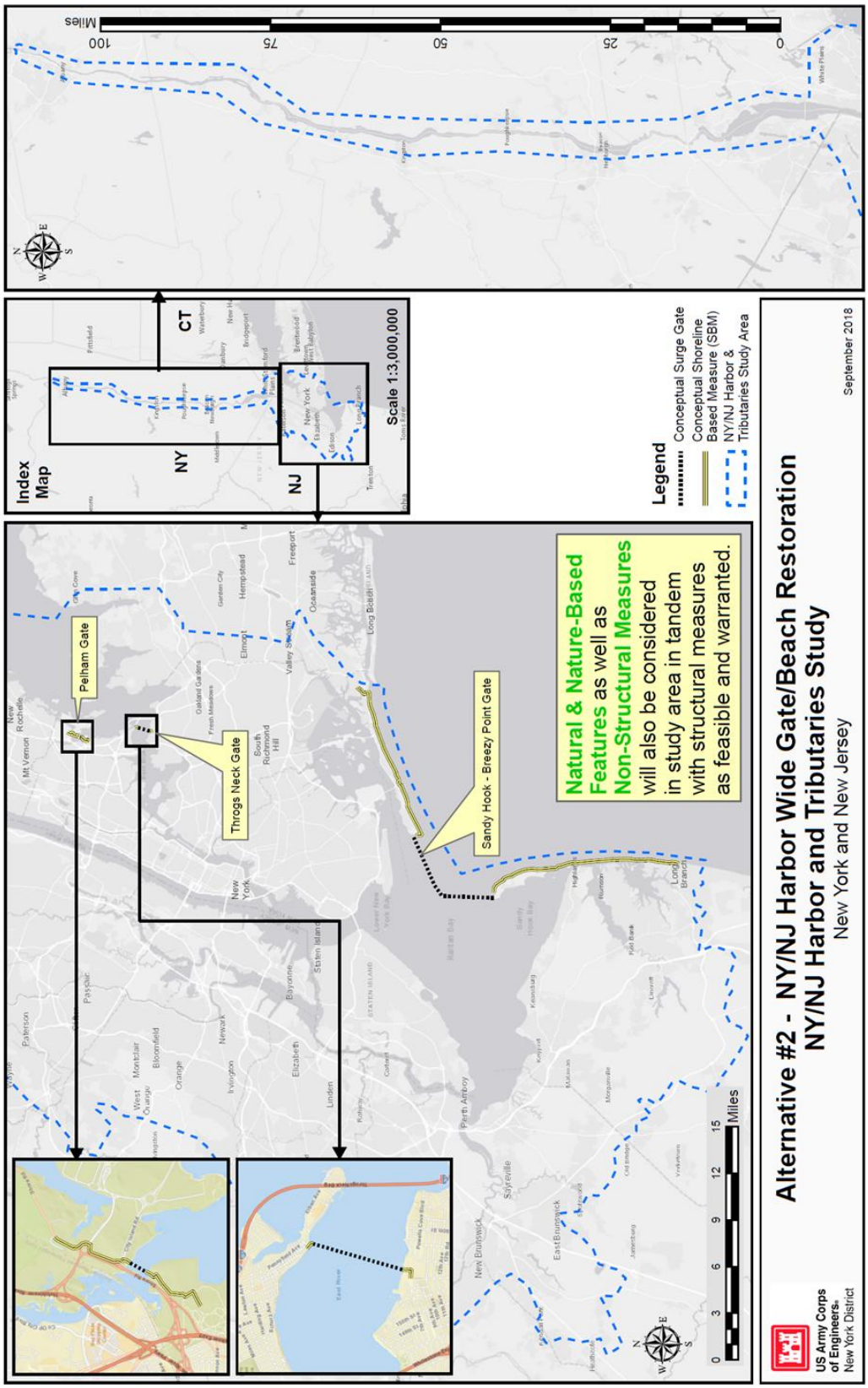
A. APPENDIX A – HAT STUDY ALTERNATIVES MAPS

Five scales of initial alternative concepts, in addition to No Action, were developed and constitute the HAT Study Alternatives:

1. No Action/FWOPC (baseline)
2. Outer Harbor Barrier system (Alternative 2)
3. Multiple Barriers, Floodwalls, and Levee systems
    a. Three Barrier Plan (Alternative 3A)
    b. Two Barrier Plan (Alternative 3B)
4. Single water body barriers, Floodwalls, and Levees (Alternative 4)
5. Perimeter only (Alternative 5)

The following pages show the overall plan for the HAT Study Alternatives 2, 3A, 3B, 4 and 5:


Alternative #2 - NY/NJ Harbor Wide Gate/Beach Restoration
NY/NJ Harbor and Tributaries Study
New York and New Jersey
September 2018
US Army Corps of Engineers
New York District
Legend
Conceptual Surge Gate
Conceptual Shoreline Based Measure (SBM)
NY/NJ Harbor & Tributaries Study Area
Index Map
NY
NJ
CT
Scale 1:3,000,000
Pelham Gate
Throgs Neck Gate
Sandy Hook - Breezy Point Gate
Natural & Nature-Based Features as well as Non-Structural Measures will also be considered in study area in tandem with structural measures as feasible and warranted.
Miles
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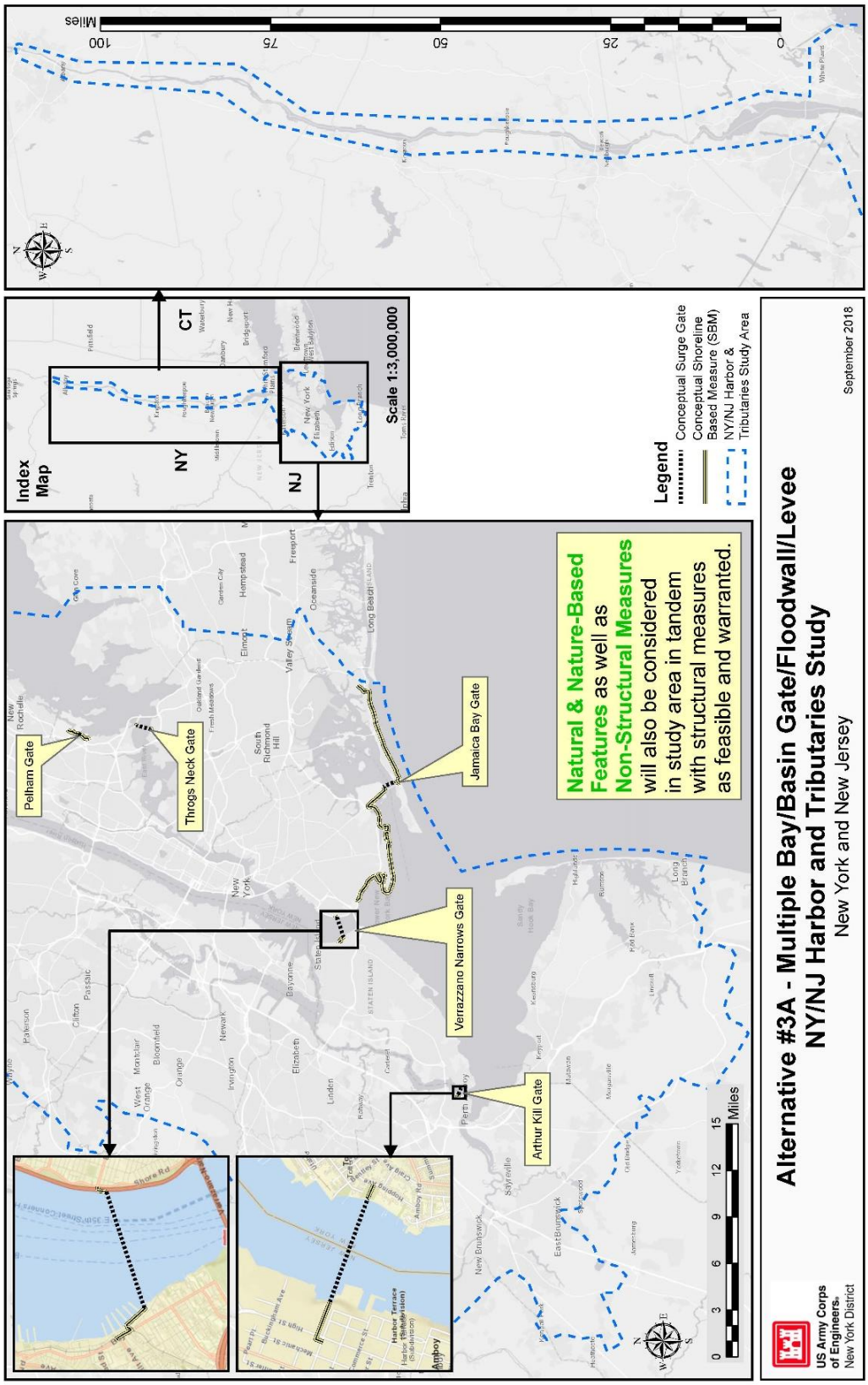
3
6
9
12
15
0
25
50
75
100
Miles


Alternative #3A - Multiple Bay/Basin Gate/Floodwall/Levee
NY/NJ Harbor and Tributaries Study
New York and New Jersey
September 2018
US Army Corps of Engineers.
New York District
Legend
Conceptual Surge Gate
Conceptual Shoreline Based Measure (SBM)
NY/NJ Harbor & Tributaries Study Area
Index Map
NY
NJ
CT
Scale 1:3,000,000
Natural & Nature-Based Features as well as Non-Structural Measures will also be considered in study area in tandem with structural measures as feasible and warranted.
Pelham Gate
Throgs Neck Gate
Jamaica Bay Gate
Verrazzano Narrows Gate
Arthur Kill Gate
Miles
0
3
6
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12
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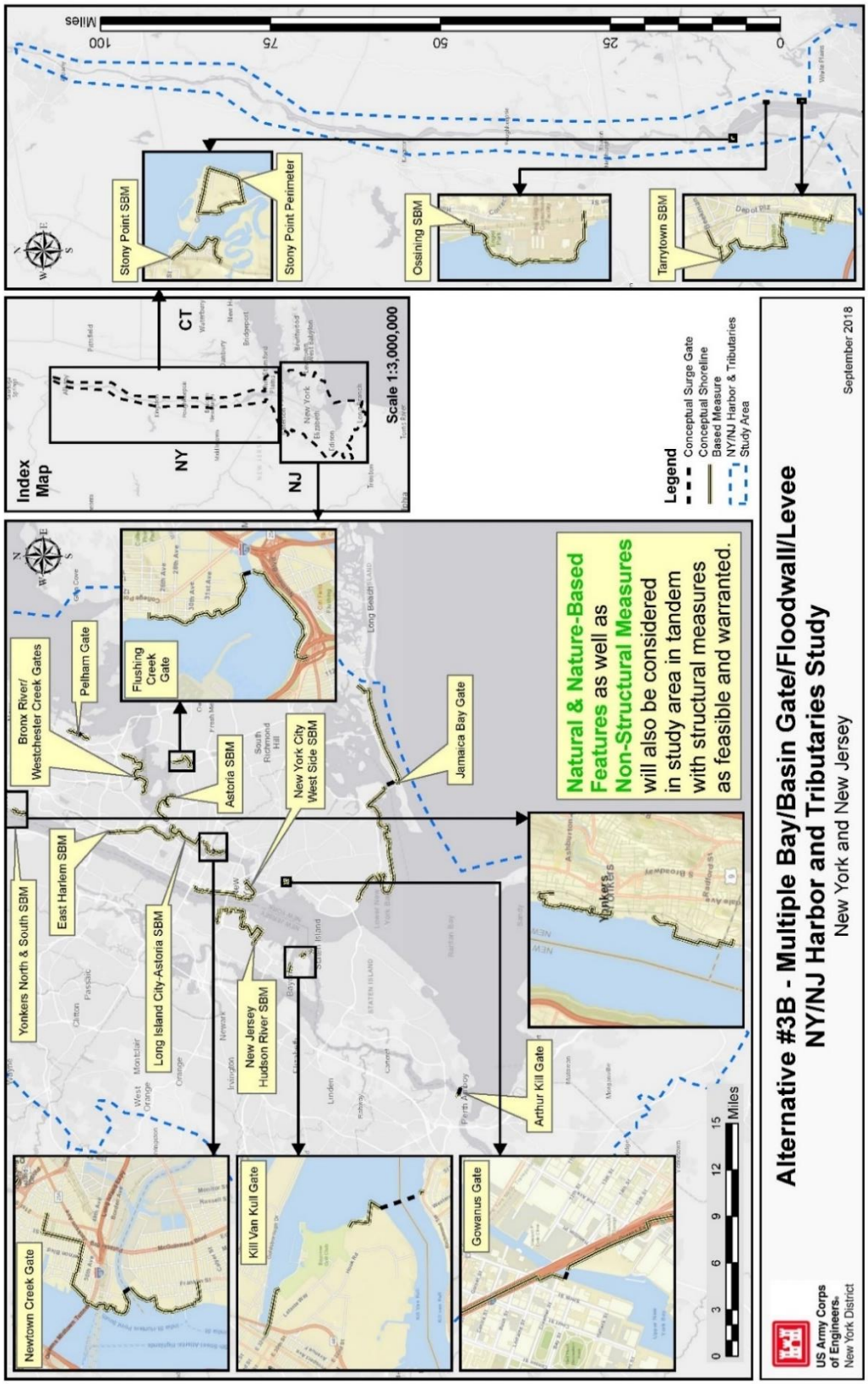
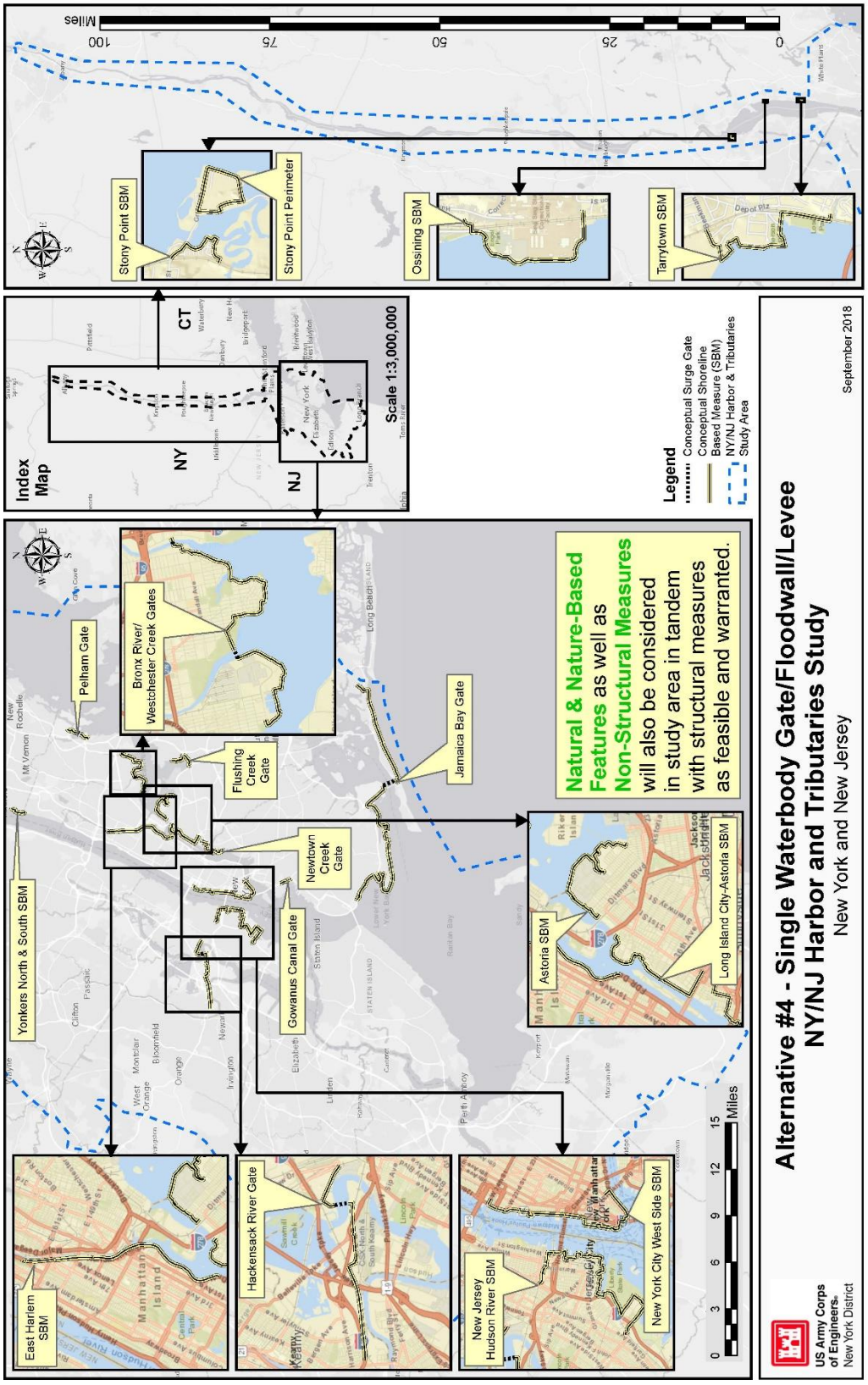
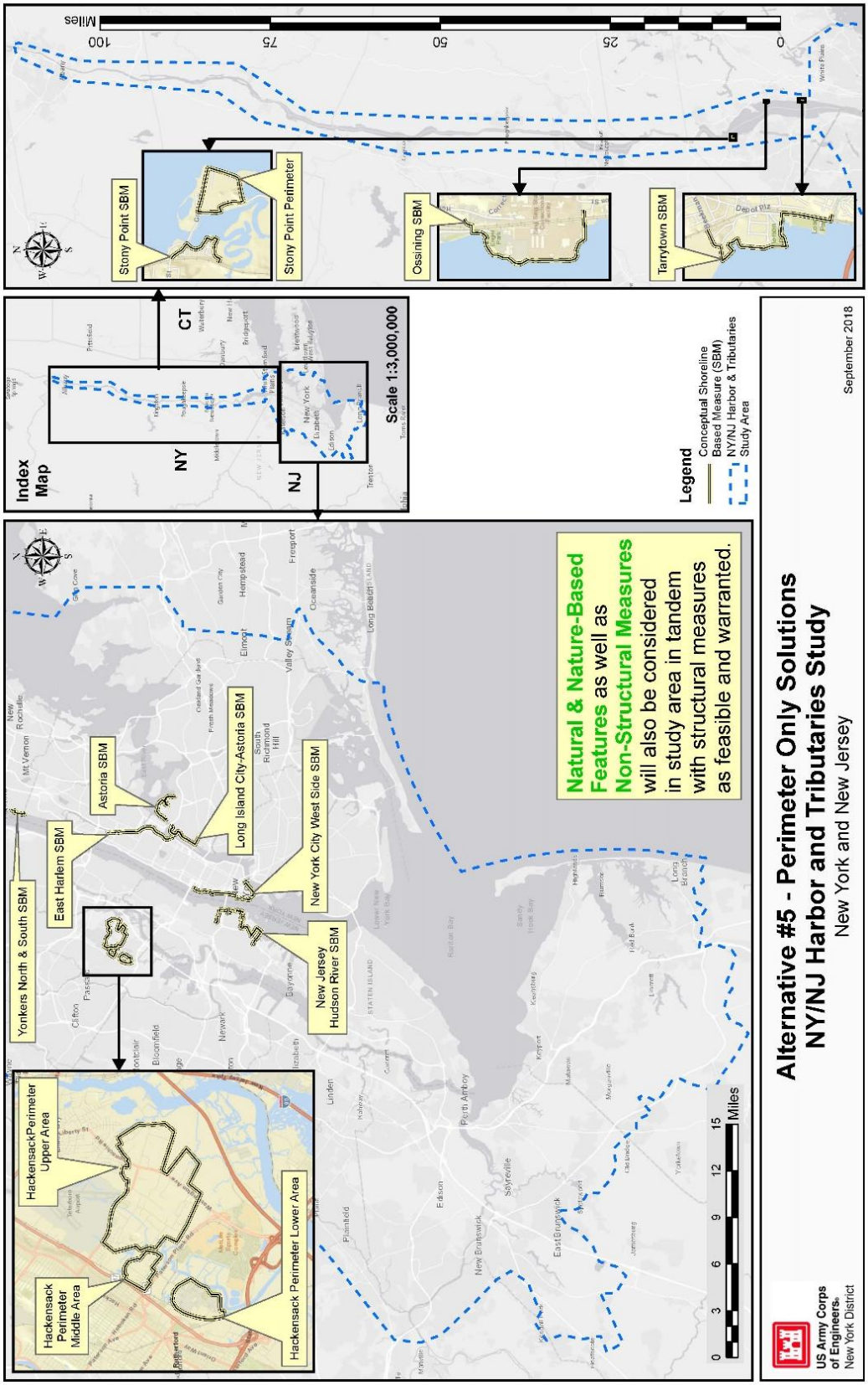
75
100
Miles


Alternative #3B - Multiple Bay/Basin Gate/Floodwall/Levee
NY/NJ Harbor and Tributaries Study
New York and New Jersey
September 2018
US Army Corps of Engineers.
New York District
Legend
Conceptual Surge Gate
Conceptual Shoreline Based Measure
NY/NJ Harbor & Tributaries Study Area
Index Map
NY
NJ
CT
Scale 1:3,000,000
Natural & Nature-Based Features as well as Non-Structural Measures will also be considered in study area in tandem with structural measures as feasible and warranted.
Bronx River/ Westchester Creek Gates
Pelham Gate
Flushing Creek Gate
Astoria SBM
New York City West Side SBM
Jamaica Bay Gate
Yonkers North & South SBM
East Harlem SBM
Long Island City-Astoria SBM
New Jersey Hudson River SBM
Arthur Kill Gate
Newtown Creek Gate
Kill Van Kull Gate
Gowanus Gate
Stony Point SBM
Stony Point Perimeter
Ossining SBM
Tarrytown SBM
Miles
0 3 6 9 12 15
0 25 50 75 100


Alternative #4 - Single Waterbody Gate/Floodwall/Levee
NY/NJ Harbor and Tributaries Study
New York and New Jersey
September 2018
US Army Corps of Engineers.
New York District
Natural & Nature-Based Features as well as Non-Structural Measures will also be considered in study area in tandem with structural measures as feasible and warranted.
Legend
Conceptual Surge Gate
Conceptual Shoreline Based Measure (SBM)
NY/NJ Harbor & Tributaries Study Area
Index Map
NY
NJ
CT
Scale 1:3,000,000
Pelham Gate
Bronx River/ Westchester Creek Gates
Flushing Creek Gate
Jamaica Bay Gate
Newtown Creek Gate
Gowanus Canal Gate
Yonkers North & South SBM
Astoria SBM
Long Island City-Astoria SBM
East Harlem SBM
Hackensack River Gate
New Jersey Hudson River SBM
New York City West Side SBM
Stony Point SBM
Stony Point Perimeter
Ossining SBM
Tarrytown SBM
Miles
0
3
6
9
12
15
25
50
75
100


Alternative #5 - Perimeter Only Solutions
NY/NJ Harbor and Tributaries Study
New York and New Jersey
September 2018
US Army Corps of Engineers
New York District
Natural & Nature-Based Features as well as Non-Structural Measures will also be considered in study area in tandem with structural measures as feasible and warranted.
Legend
Conceptual Shoreline Based Measure (SBM)
NY/NJ Harbor & Tributaries Study Area
Index Map
NY
NJ
CT
Scale 1:3,000,000
Miles
0 3 6 9 12 15
Yonkers North & South SBM
East Harlem SBM
Astoria SBM
Long Island City-Astoria SBM
New York City West Side SBM
New Jersey Hudson River SBM
Hackensack Perimeter Upper Area
Hackensack Perimeter Middle Area
Hackensack Perimeter Lower Area
Stony Point SBM
Stony Point Perimeter
Ossining SBM
Tarrytown SBM
0 25 50 75 100
Miles

B. APPENDIX B – LOCATION MAPS

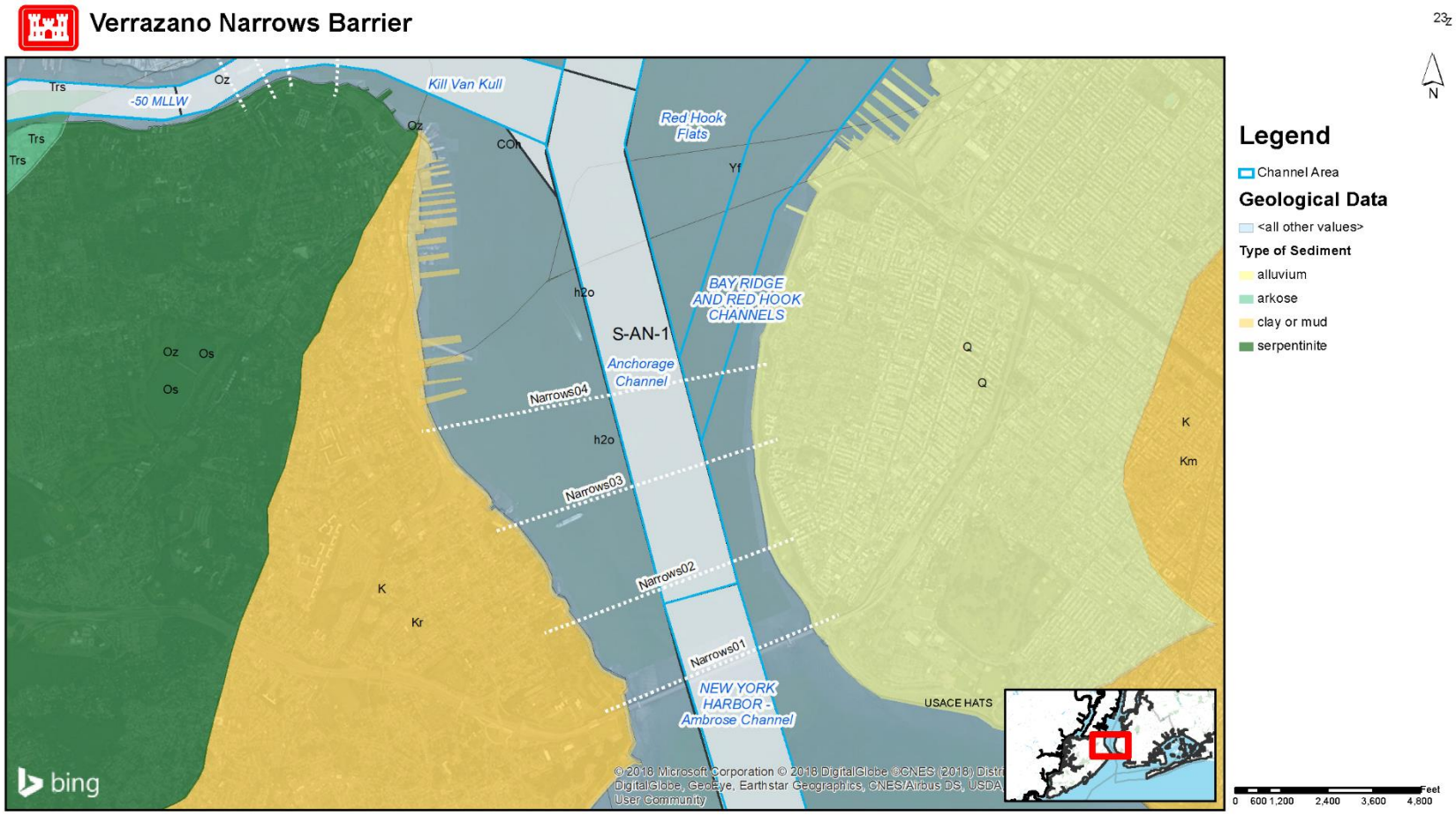
Verrazano Narrows

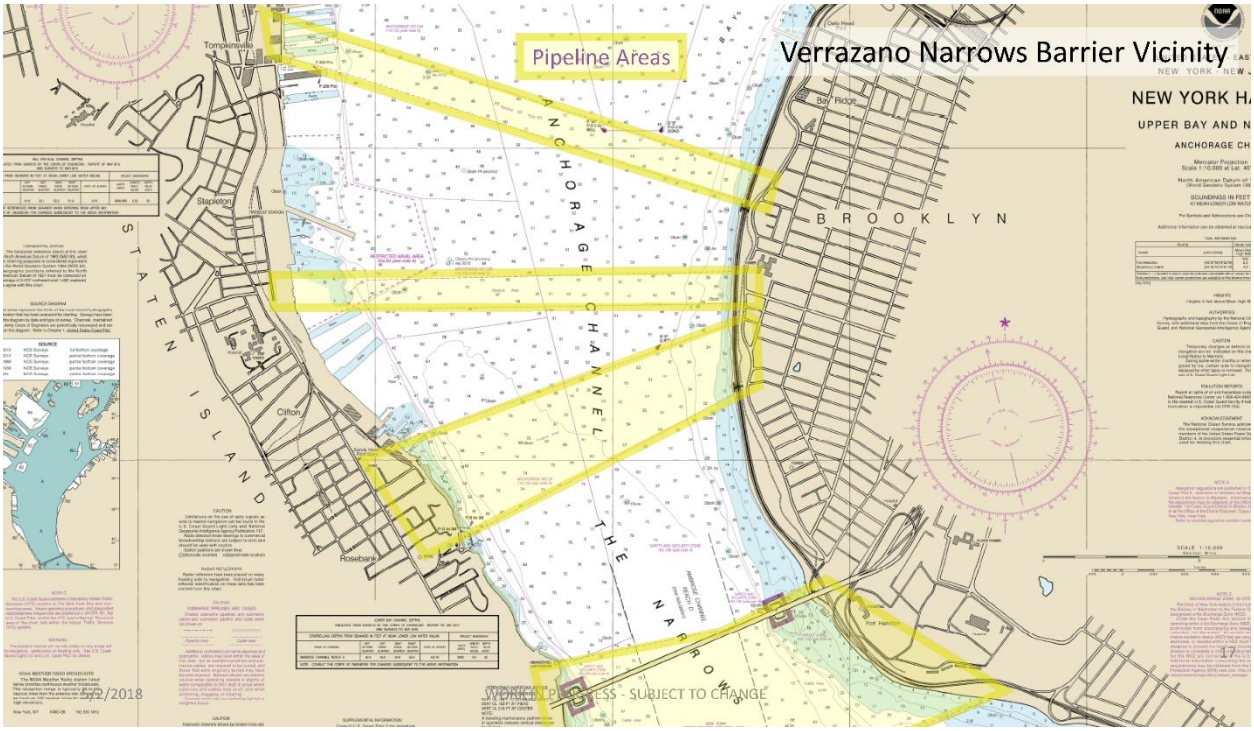
The following pages display two maps for the area of interest:

1. Location Map
2. Bathymetry Map
3. Subaqueous Utilities Map
4. Surface Geological Overview Map
5. Subaqueous Utilities Map
6. Proposed Siphon map (from https://www1.nyc.gov/html/dep/html/dep_projects/cp_si_bk_siphon_project.shtml)

Verrazano Narrows Barrier
Legend
Channel Area
Depth Countours
Depth ft
0
2.952756
5.905512
11.81102
17.71654
29.85564
35.76115
59.71128
Harbour Cable Area
Harbour Submarine Cable
Harbour Pipeline
Underwater Awash Rock
Ship Wreck
Feet
0 600 1,200 2,400 3,600 4,800
N
Red Hook Flats
Anchorage Channel
BAY RIDGE AND RED HOOK CHANNELS
NEW YORK HARBOR - Ambrose Channel
Narrows01
Narrows02
Narrows03
Narrows04
Narrows05
Kill Van Kull
KillVanKull01
-50 MLLW
USACE HAT9
© 2018 Maxar Technologies © 2018 DigitalGlobe
bing

Verrazano Narrows Barrier
Legend
Channel Area
Bathymetry
Feet, NAVD 88
-9.9 - 0
-19.9 - -10
-29.9 - -20
-39.9 - -30
-49.9 - -40
-59.9 - -50
-69.9 - -60
-79.9 - -70
-153.8 - -80
Elevations
Feet, NAVD88
0.142 - 1
1.01 - 2
2.01 - 4
4.01 - 6
6.01 - 8
8.01 - 10
10.1 - 12
12.1 - 14
14.1 - 16
16.1 - 18
Feet
0 450 900 1,800 2,700 3,600
N
Red Hook Flats
BAY RIDGE AND RED HOOK CHANNELS
Anchorage Channel
Narrows05
Narrows04
Narrows03
Narrows02
Narrows01
NEW YORK HARBOR Anchorage Channel
USACE NATS
bing


Verrazano Narrows Barrier
23Z
N
Legend
Channel Area
Geological Data
<all other values>
Type of Sediment
alluvium
arkose
clay or mud
serpentinite
Trs
-50 MLLW
Oz
Kill Van Kull
Red Hook Flats
COh
Yf
h2o
BAY RIDGE AND RED HOOK CHANNELS
S-AN-1
Anchorage Channel
Os
Q
K
Km
Narrows04
Narrows03
Narrows02
Narrows01
Kr
NEW YORK HARBOR - Ambrose Channel
USACE HATS
bing
0 600 1,200 2,400 3,600 4,800 Feet

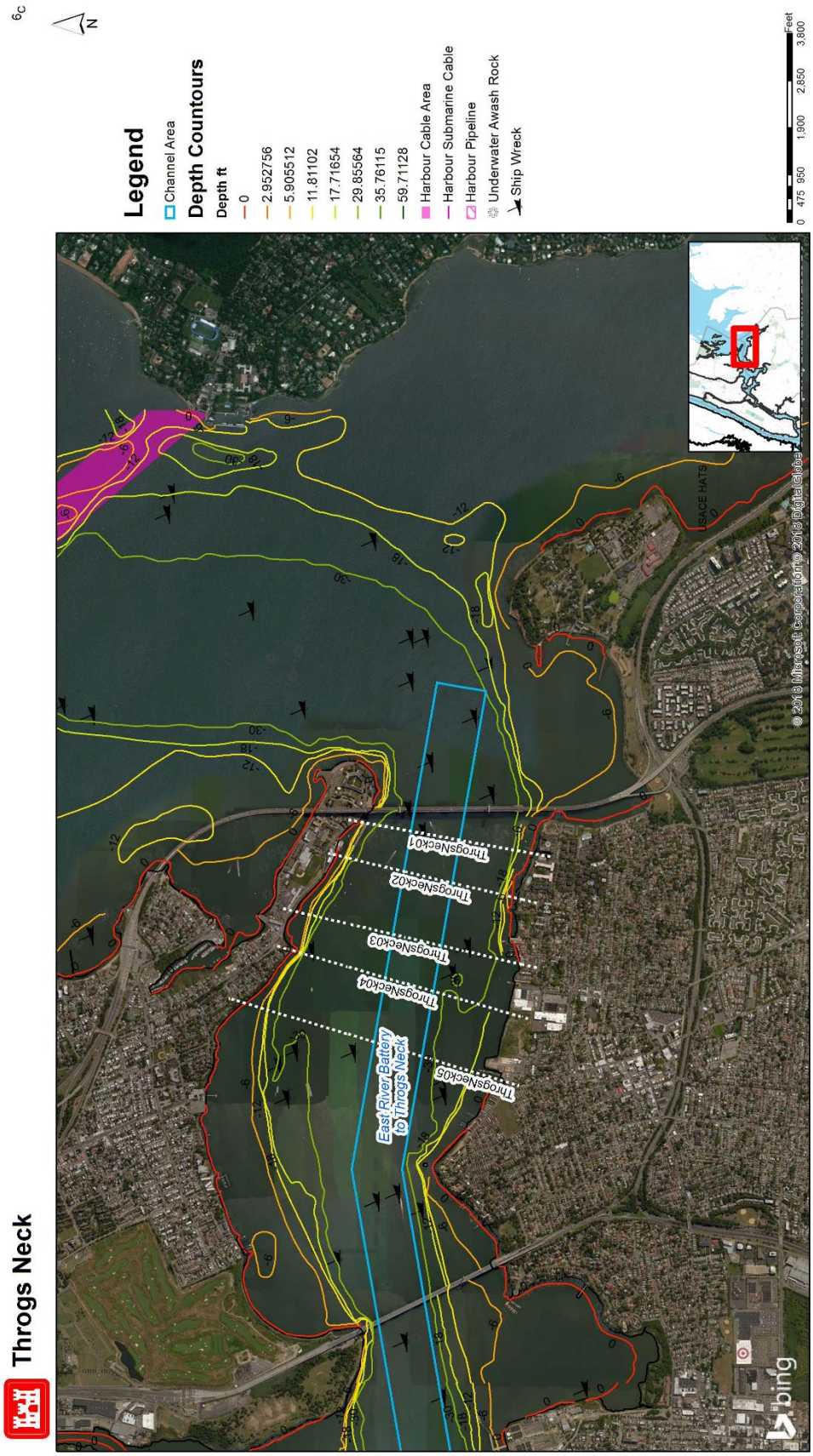

Pipeline Areas
Verrazano Narrows Barrier Vicinity
NEW YORK HA
UPPER BAY AND N
ANCHORAGE CH
BROOKLYN
STATEN ISLAND
ANCHORAGE CHANNEL
THE NARROWS
Tompkinsville
Stapleton
Clifton
Rosebank
Bay Ridge
Fort Hamilton
WORK IN PROGRESS - SUBJECT TO CHANGE


Existing Siphon
No. 1
Siphon #1
Staten
Island
Proposed Siphon
Brooklyn
Siphon #2
Existing Siphon
No. 2

Throgs Neck

The following pages display two maps for the area of interest:

1. Location Map
2. Bathymetry Map
3. Subaqueous Utilities Map
4. Surface Geological Overview Map


Throgs Neck
Legend
Channel Area
Depth Countours
Depth ft
0
2.952756
5.905512
11.81102
17.71654
29.85564
35.76115
59.71128
Harbour Cable Area
Harbour Submarine Cable
Harbour Pipeline
Underwater Awash Rock
Ship Wreck
Feet
0 475 950 1,900 2,850 3,800
N
East River Battery to Throgs Neck
ThrogsNeck01
ThrogsNeck02
ThrogsNeck03
ThrogsNeck04
ThrogsNeck05
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SACE HATS
bing

Throgs Neck
Legend
Channel Area
Bathymetry
Feet, NAVD 88
-9.9 - 0
-19.9 - -10
-29.9 - -20
-39.9 - -30
-49.9 - -40
-59.9 - -50
-69.9 - -60
-79.9 - -70
-153.8 - -80
Elevations
Feet, NAVD88
0.142 - 1
1.01 - 2
2.01 - 4
4.01 - 6
6.01 - 8
8.01 - 10
10.1 - 12
12.1 - 14
14.1 - 16
16.1 - 18
0 350 700 1,400 2,100 2,800
Feet
East River Battery to Throgs Neck
ThrogsNeck01
ThrogsNeck02
ThrogsNeck03
ThrogsNeck04
ThrogsNeck05
USACE HATC
bing

Throgs Neck
Legend
Channel Area
Geological Data
<all other values>
Type of Sediment
alluvium
amphibolite
clay or mud
East River Battery to Throgs Neck
ThrogsNeck01
ThrogsNeck02
ThrogsNeck03
ThrogsNeck04
ThrogsNeck05
ThrogsNeck06
USACE HATS
bing

Arthur Kill

The following pages display two maps for the area of interest:

1. Location Map
2. Bathymetry Map
3. Surface Geological Overview Map
4. Subaqueous Utilities Map

Arthur Kill Barrier
Legend
Channel Area
Depth Countours
Depth ft
0
2.952756
5.905512
11.81102
17.71654
29.85564
35.76115
59.71128
Harbour Cable Area
Harbour Submarine Cable
Harbour Pipeline
Underwater Awash Rock
Ship Wreck
Feet
0 420 840 1,680 2,520 3,360
N
ArthurKill04
ArthurKill03
ArthurKill02
ArthurKill01
-35 MLLW
-20 MLLW
-37 MLLW
-25 MLLW
Seguin Pt-Ward Pt-Outerbridge
Perth Amboy Anch & 2nd Chnl
Raritan River to AK Cutoff (Whole)
Raritan River w/Spur Channel
USACE HATS
© 2018 DigitalGlobe
bing

Arthur Kill Barrier
Legend
Channel Area
Bathymetry
Feet, NAVD 88
-9.9 - 0
-19.9 - -10
-29.9 - -20
-39.9 - -30
-49.9 - -40
-59.9 - -50
-69.9 - -60
-79.9 - -70
-153.8 - -80
Elevations
Feet, NAVD88
0.142 - 1
1.01 - 2
2.01 - 4
4.01 - 6
6.01 - 8
8.01 - 10
10.1 - 12
12.1 - 14
14.1 - 16
16.1 - 18
0 310 620 1,240 1,860 2,480 Feet
N
-35 MLLW
ArthurKill04
ArthurKill03
ArthurKill02
Seguin Pt - Ward Pt - Outerbridge
ArthurKill01
-35 MLLW
-35 MLLW
Perth Amboy Anch & 2nd Chnl
-37 MLLW
Raritan River to AK Cutoff (whole)
-25 MLLW
Raritan River w/Spur Channel
-20 MLLW
-35 MLLW
USACE HATS
© 2018 Microsoft Corporation © 2018 DigitalGlobe

Arthur Kill Barrier
Legend
Channel Area
Geological Data
<all other values>
Type of Sediment
clay or mud
sand
0 420 840 1,680 2,520 3,360
Feet
N
-35 MLLW
-25 MLLW
-20 MLLW
-37 MLLW
-30 MLLW
ArthurKill03
ArthurKill02
ArthurKill01
Raritan River w/Spur Channel
Raritan River to AK Cutoff (whole)
Perth Amboy Anch & 2nd Chnl
Seguin Pt - Ward Pt Outer Ridge
Arch & 2nd Chnl
USACE HATS
h2o
Kr
K
bing
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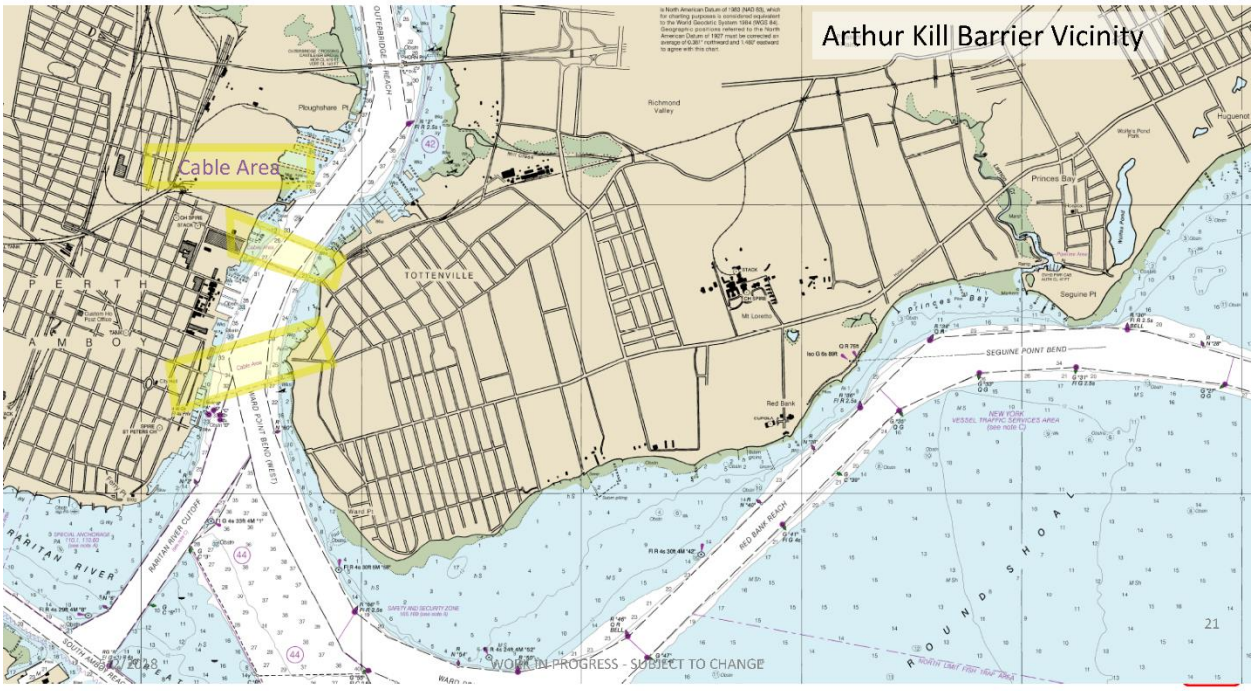
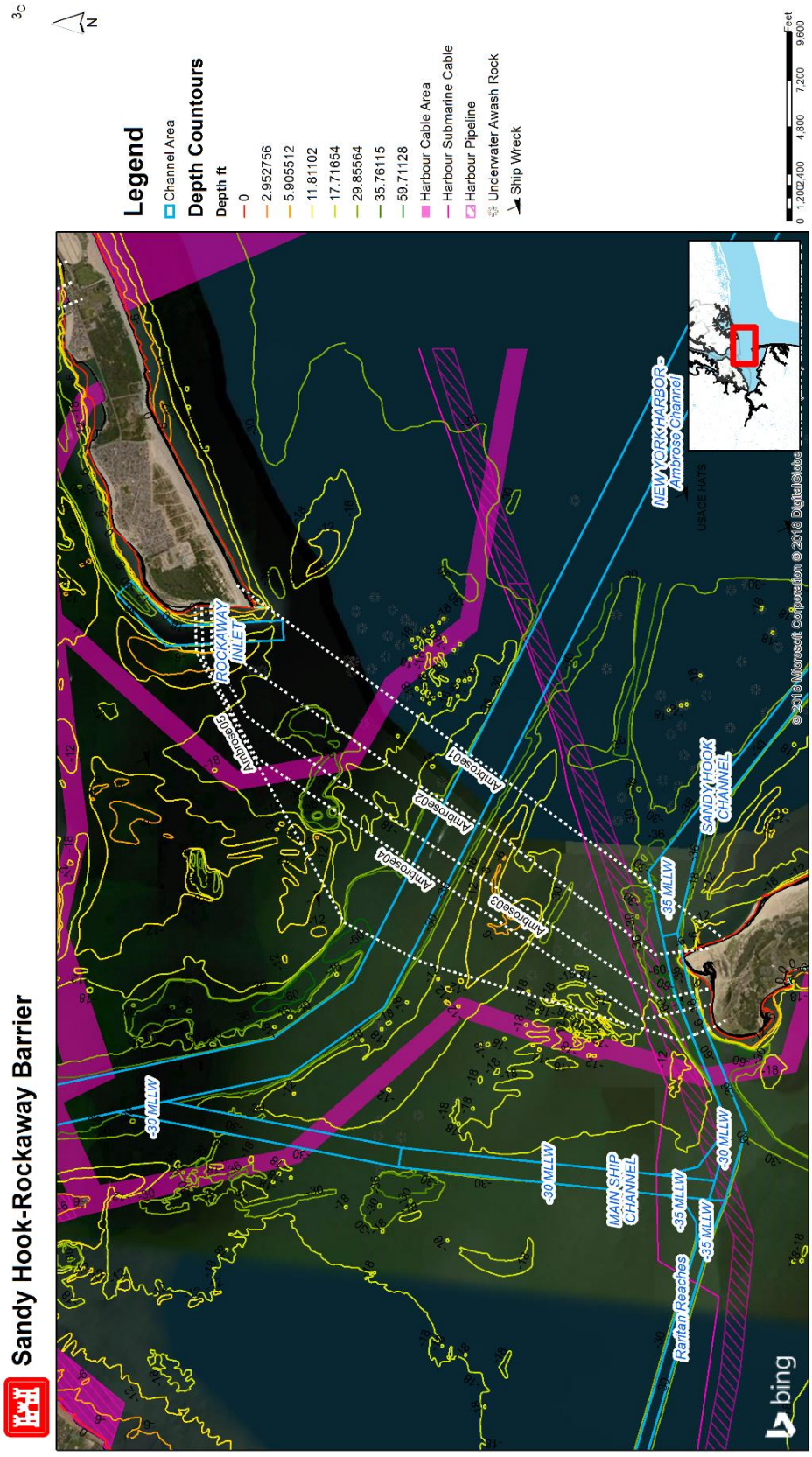


Outer Harbor

1. Location Map
2. Bathymetry Map
3. Surface Geological Overview Map
4. Subaqueous Utilities Map


3c
N
Legend
Channel Area
Depth Countours
Depth ft
0
2.952756
5.905512
11.81102
17.71654
29.85564
35.76115
59.71128
Harbour Cable Area
Harbour Submarine Cable
Harbour Pipeline
Underwater Awash Rock
Ship Wreck
0 1,200 2,400 4,800 7,200 9,600
Feet
Sandy Hook-Rockaway Barrier
ROCKAWAY INLET
Ambrose05
Ambrose01
Ambrose02
Ambrose04
Ambrose03
-30 MLLW
-30 MLLW
MAIN SHIP CHANNEL
-35 MLLW
-35 MLLW
Raritan Reaches
-30 MLLW
-35 MLLW
SANDY HOOK CHANNEL
NEW YORK HARBOR - Ambrose Channel
USACE HATS
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Sandy Hook-Rockaway Barrier
Legend
Channel Area
Bathymetry
Feet, NAVD 88
-9.9 - 0
-19.9 - -10
-29.9 - -20
-39.9 - -30
-49.9 - -40
-59.9 - -50
-69.9 - -60
-79.9 - -70
-153.8 - -80
Elevations
Feet, NAVD88
0.142 - 1
1.01 - 2
2.01 - 4
4.01 - 6
6.01 - 8
8.01 - 10
10.1 - 12
12.1 - 14
14.1 - 16
16.1 - 18
0 875 1,750 3,500 5,250 7,000
Feet
N
ROCKAWAY INLET
Ambrose01
Ambrose02
Ambrose03
Ambrose04
Ambrose05
NEW YORK HARBOR - Ambrose Channel
-30 MLLW
MAIN SHIP CHANNEL
-30 MLLW
-35 MLLW
SANDY HOOK CHANNEL
-35 MLLW
-35 MLLW
SANDY HOOK CHANNEL
USACE HATS
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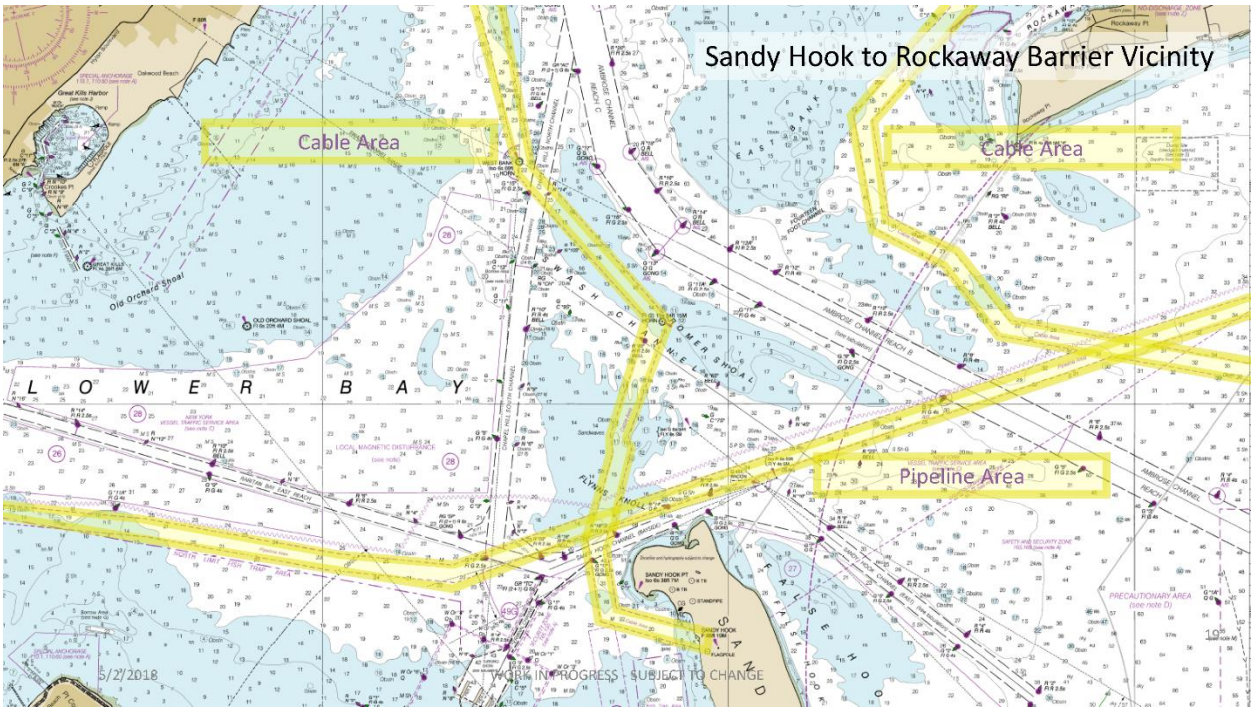
Reaches

Sandy Hook-Rockaway Barrier
Legend
Channel Area
Geological Data
<all other values>
Type of Sediment
clay or mud
0 1,200 2,400 4,800 7,200 9,600 Feet
N
ROCKAWAY INLET
NEW YORK HARBOR - Ambrose Channel
MAIN SHIP CHANNEL
SANDY HOOK CHANNEL
Raritan Reaches
-30 MLLW
-35 MLLW
Ambrose01
Ambrose02
Ambrose03
Ambrose04
Ambrose05
h2o
water
COI
Km
USACE HATS
bing



Kill van Kull

1. Location Map
2. Bathymetry Map
3. Surface Geological Overview Map
4. Subaqueous Utilities Map

Kill Van Kull Barrier
Legend
Channel Area
Depth Countours
Depth ft
0
2.952756
5.905512
11.81102
17.71654
29.85564
35.76115
59.71128
Harbour Cable Area
Harbour Submarine Cable
Harbour Pipeline
Underwater Awash Rock
Ship Wreck
Feet
0 460 920 1,840 2,760 3,680
N
Anchorage Channel
Red Hook Flats
Port Jersey
New Jersey Pierhead
Kill Van Kull
NEWARK BAY-Port Elizabeth
NEWARK BAY-Main Channel
KillVanKull04
KillVanKull03
KillVanKull02
KillVanKull01
-20 MLLW
-50 MLLW
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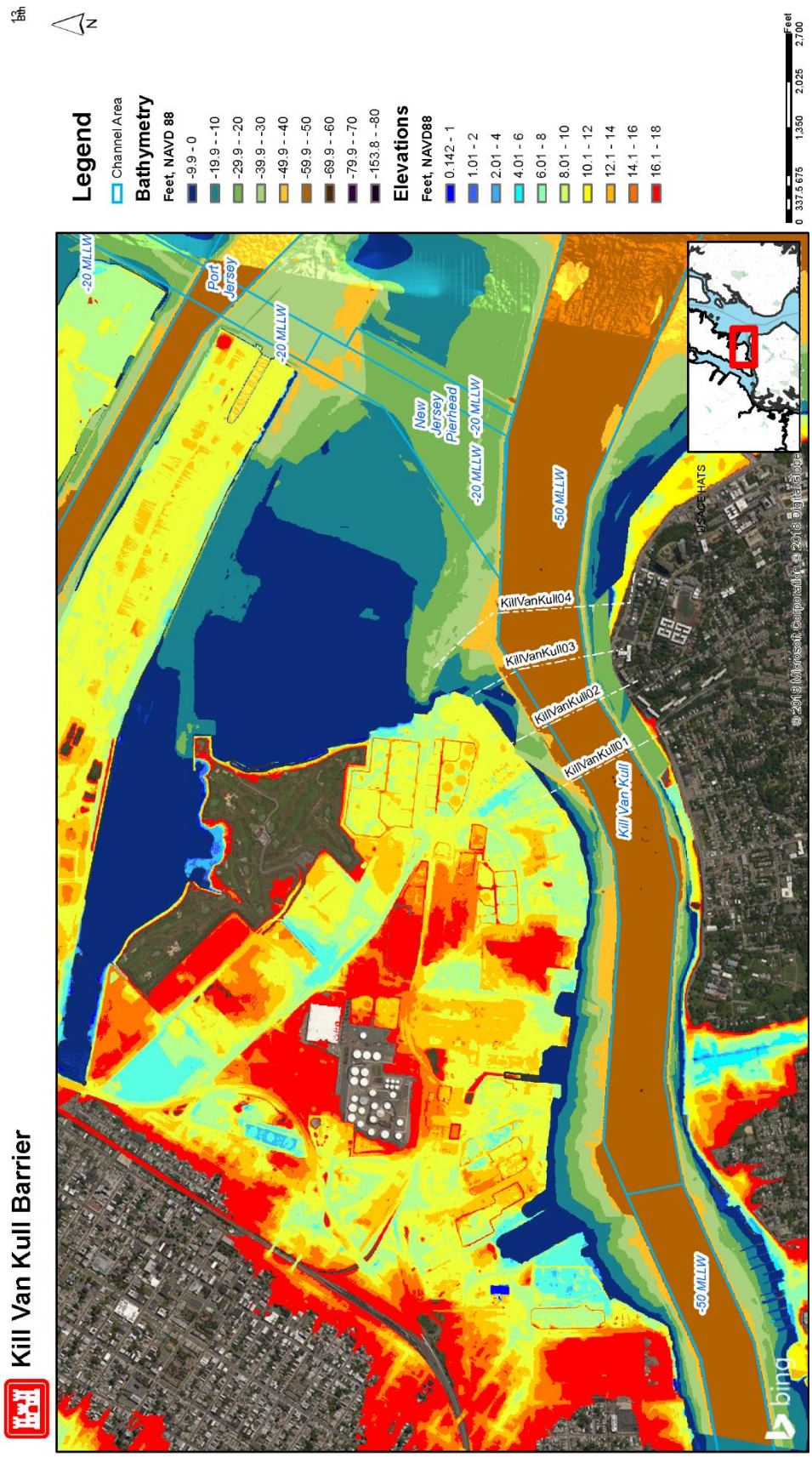
Kill Van Kull Barrier
Legend
Channel Area
Bathymetry
Feet, NAVD 88
-9.9 - 0
-19.9 - -10
-29.9 - -20
-39.9 - -30
-49.9 - -40
-59.9 - -50
-69.9 - -60
-79.9 - -70
-153.8 - -80
Elevations
Feet, NAVD88
0.142 - 1
1.01 - 2
2.01 - 4
4.01 - 6
6.01 - 8
8.01 - 10
10.1 - 12
12.1 - 14
14.1 - 16
16.1 - 18
0 337.5 675 1,350 2,025 2,700 Feet
-20 MLLW
Port Jersey
-20 MLLW
New Jersey Pierhead
-20 MLLW
-20 MLLW
-50 MLLW
KillVanKull04
KillVanKull03
KillVanKull02
KillVanKull01
Kill Van Kull
-50 MLLW
USACE HATS
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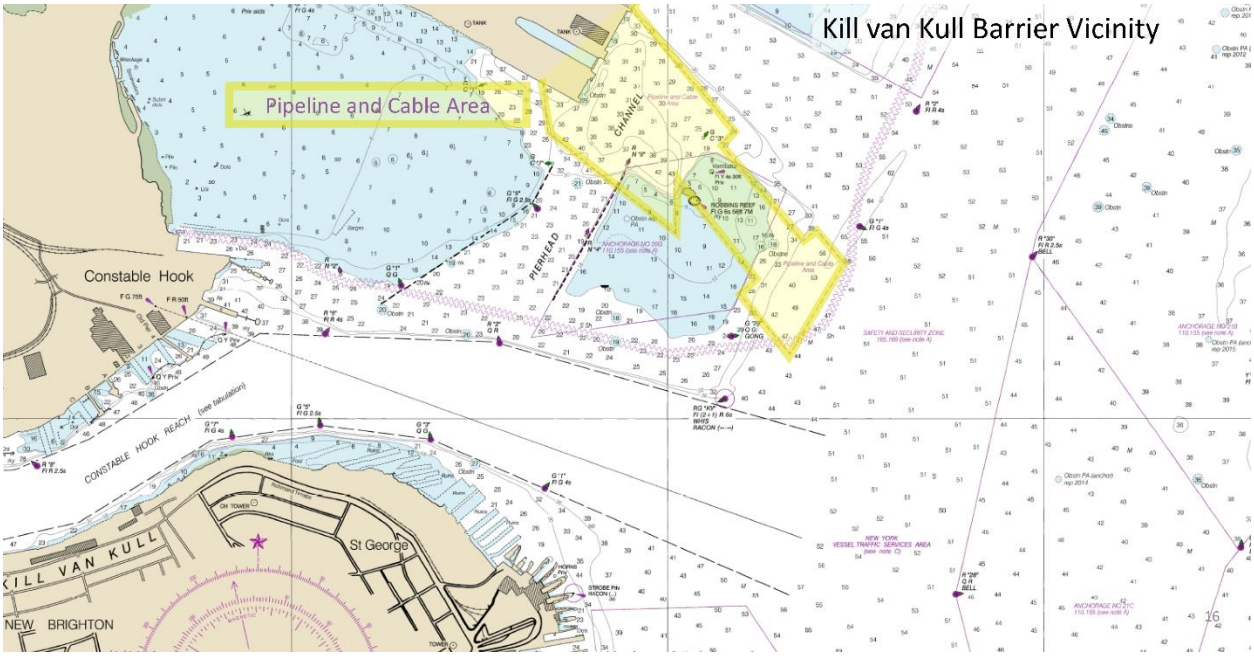
Kill Van Kull Barrier
Legend
Channel
Geological Data
<all other values>
Type of Sediment
argillite
arkose
clay or mud
diabase
serpentinite
siltstone
0 455 910 1,820 2,730 3,640
Feet
N
S-KVK-2
S-KVK-1
S-AN-
Kill Van Kull
Port Jersey
Anchorage Channel
Red Hook Flats
New Jersey Pierhead
NEWARK BAY-3 Main Channel
-20 MLLW
-50 MLLW
KillVanKull01
KillVanKull02
KillVanKull03
KillVanKull04
USACE HATS
Ow
Oz
Os
Trs
Trl
Trla
Trpa
Trp
Jd
COh
h2o
K
Kr
bing



Jamaica Bay

1. Location Map
2. Bathymetry Map
3. Surface Geological Overview Map
4. Subaqueous Utilities Map

Jamaica Bay Barrier
Legend
Channel Area
Depth Countours
Depth ft
0
2.952756
5.905512
11.81102
17.71654
29.85564
35.76115
59.71128
Harbour Cable Area
Harbour Submarine Cable
Harbour Pipeline
Underwater Awash Rock
Ship Wreck
Feet
0 400 800 1,600 2,400 3,200
N
JamBay03
JamBay02
JamBay01
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bing

Jamaica Bay Barrier
Legend
Channel Area
Bathymetry
Feet, NAVD 88
-9.9 - 0
-19.9 - -10
-29.9 - -20
-39.9 - -30
-49.9 - -40
-59.9 - -50
-69.9 - -60
-79.9 - -70
-153.8 - -80
Elevations
Feet, NAVD88
0.142 - 1
1.01 - 2
2.01 - 4
4.01 - 6
6.01 - 8
8.01 - 10
10.1 - 12
12.1 - 14
14.1 - 16
16.1 - 18
Feet
0
295
590
1,180
1,770
2,360
N
JamBay03
JamBay02
JamBay01
USACE HATS
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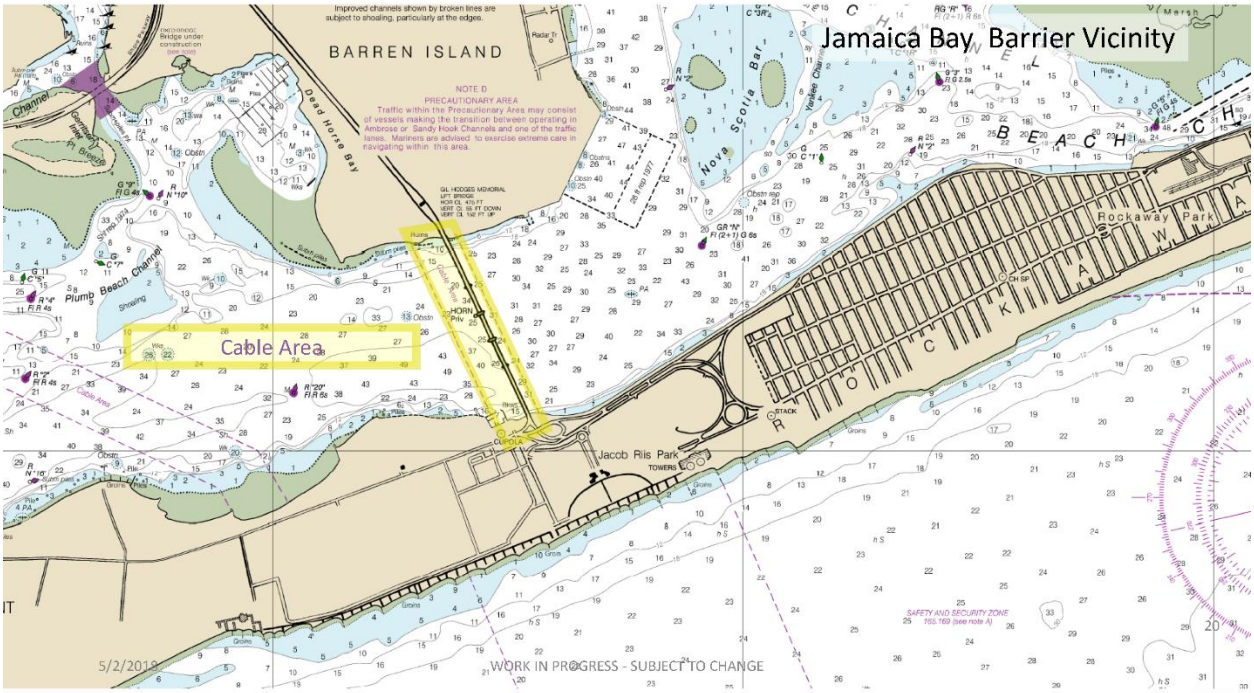
Bing

Jamaica Bay Barrier
Legend
Geological Data
<all other values>
Type of Sediment
clay or mud
0 400 800 1,600 2,400 3,200
Feet
N
JamBay03
JamBay02
JamBay01
K
Km
COi
h2o
USACE HATS
bing



Hackensack River

1. Location Map
2. Bathymetry Map
3. Surface Geological Overview Map
4. Subaqueous Utilities Map

Hackensack River Barrier
Legend
Channel Area
Depth Countours
Depth ft
0
2.952756
5.905512
11.81102
17.71654
29.85564
35.76115
59.71128
Harbour Cable Area
Harbour Submarine Cable
Harbour Pipeline
Underwater Awash Rock
Ship Wreck
Feet
0 550 1,100 2,200 3,300 4,400
N
Hackensack03
Hackensack02
Hackensack01
USACE-HATS
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Hackensack River Barrier
Legend
Channel Area
Bathymetry
Feet, NAVD 88
-9.9 - 0
-19.9 - -10
-29.9 - -20
-39.9 - -30
-49.9 - -40
-59.9 - -50
-69.9 - -60
-79.9 - -70
-153.8 - -80
Elevations
Feet, NAVD88
0.142 - 1
1.01 - 2
2.01 - 4
4.01 - 6
6.01 - 8
8.01 - 10
10.1 - 12
12.1 - 14
14.1 - 16
16.1 - 18
0 430 860 1,720 2,580 3,440 Feet
Hackensack03
Hackensack02
Hackensack01
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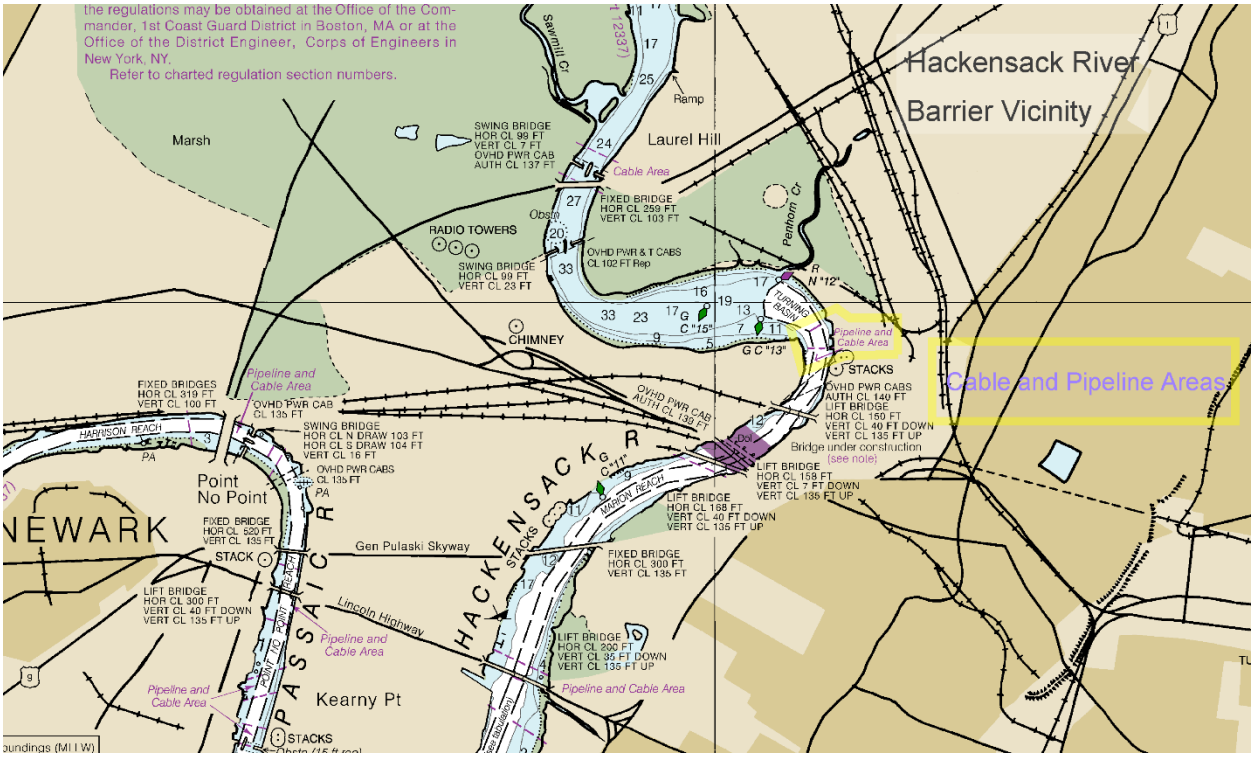
bing

Hackensack River Barrier
Legend
Geological Data
<all other values>
Type of Sediment
argillite
arkose
diabase
sandstone
siltstone
Feet
0 550 1,100 2,200 3,300 4,400
N
Trla
Trl
Jd
Trpg
JTrpms
JTrps
JTrpa
USACE HATS
bing


Hackensack River
Barrier Vicinity
Cable and Pipeline Areas
the regulations may be obtained at the Office of the Commander, 1st Coast Guard District in Boston, MA or at the Office of the District Engineer, Corps of Engineers in New York, NY.
Refer to charted regulation section numbers.
Marsh
Laurel Hill
Ramp
Sawmill Cr
Penhorn Cr
SWING BRIDGE
HOR CL 99 FT
VERT CL 7 FT
OVHD PWR CAB
AUTH CL 137 FT
Cable Area
FIXED BRIDGE
HOR CL 259 FT
VERT CL 103 FT
RADIO TOWERS
Obstn
OVHD PWR & T CABS
CL 102 FT Rep
SWING BRIDGE
HOR CL 99 FT
VERT CL 23 FT
TURNING BASIN
CHIMNEY
Pipeline and Cable Area
STACKS
OVHD PWR CABS
AUTH CL 140 FT
LIFT BRIDGE
HOR CL 150 FT
VERT CL 40 FT DOWN
VERT CL 135 FT UP
Bridge under construction
(see note)
FIXED BRIDGES
HOR CL 319 FT
VERT CL 100 FT
OVHD PWR CAB
CL 135 FT
HARRISON REACH
SWING BRIDGE
HOR CL N DRAW 103 FT
HOR CL S DRAW 104 FT
VERT CL 16 FT
OVHD PWR CABS
CL 135 FT
Point
No Point
NEWARK
FIXED BRIDGE
HOR CL 520 FT
VERT CL 135 FT
STACK
Gen Pulaski Skyway
LIFT BRIDGE
HOR CL 300 FT DOWN
VERT CL 135 FT UP
PASSAIC R
Lincoln Highway
Pipeline and Cable Area
Kearny Pt
STACKS
HACKENSACK R
MARION REACH
LIFT BRIDGE
HOR CL 168 FT
VERT CL 40 FT DOWN
VERT CL 135 FT UP
FIXED BRIDGE
HOR CL 300 FT
VERT CL 135 FT
LIFT BRIDGE
HOR CL 200 FT
VERT CL 35 FT DOWN
VERT CL 135 FT UP
Pipeline and Cable Area
LIFT BRIDGE
HOR CL 158 FT
VERT CL 7 FT DOWN
VERT CL 135 FT UP

C. APPENDIX C – TIDAL DATA

Table C-1: Tidal datums for Station 8518750, The Battery, NY.

Tidal Datum	Abbreviation	NAVD88 ft	MLLW ft
Highest Observed 10/30/2012	Max Tide	11.27 (failed)	14.04 (failed)
Highest Astronomical Tide 10/16/1993	HAT	3.58	6.35
Mean Higher-High Water	MHHW	2.28	5.05
Mean High Water	MHW	1.96	4.73
Mean Sea Level	MSL	-0.2	2.57
Mean Low Water	MLW	-2.57	0.2
Mean Lower-Low Water	MLLW	-2.77	0
Lowest Astronomical Tide 1/21/1996	LAT	-4.13	-1.39
Lowest Observed 2/2/1976	Min Tide	-7.06	-4.29
Mean Tidal Range	MN	4.53	4.53

Table C-2: Tidal datums for Station 8531680, Sandy Hook, NJ

Tidal Datum	Abbreviation	NAVD88 ft	MLLW ft
Highest Observed 10/29/2012	Max Tide	9.21 (failed)	12.03 (failed)
Highest Astronomical Tide	HAT	3.78	6.60
Mean Higher-High Water	MHHW	2.41	5.23
Mean High Water	MHW	2.08	4.90
Mean Sea Level	MSL	-0.24	2.58
Mean Low Water	MLW	-2.62	0.20
Mean Lower-Low Water	MLLW	-2.82	0
Lowest Astronomical Tide	LAT	-4.19	-1.37
Lowest Observed 2/2/1976	Min Tide	-7.53	-4.71
Mean Tidal Range	MN	4.70	4.70

Table C-3: Tidal datums for Station 8516945, Kings Point, NY

Tidal Datum	Abbreviation	NAVD88 ft	MLLW ft
Highest Observed 10/30/2012	Max Tide	10.09 (failed)	14.25 (failed)
Highest Astronomical Tide 10/16/1993	HAT	5.5	9.66
Mean Higher-High Water	MHHW	3.64	7.80
Mean High Water	MHW	3.28	7.44
Mean Sea Level	MSL	-0.27	3.89
Mean Low Water	MLW	-3.88	0.28
Mean Lower-Low Water	MLLW	-4.16	0
Lowest Astronomical Tide 1/21/1996	LAT	-5.77	-1.61
Lowest Observed 2/2/1976	Min Tide	-8.17	-4.01
Mean Tidal Range	MN	7.16	7.16

Table C-4: Tidal datums for Station 8519483, Bergen Reach West, NY

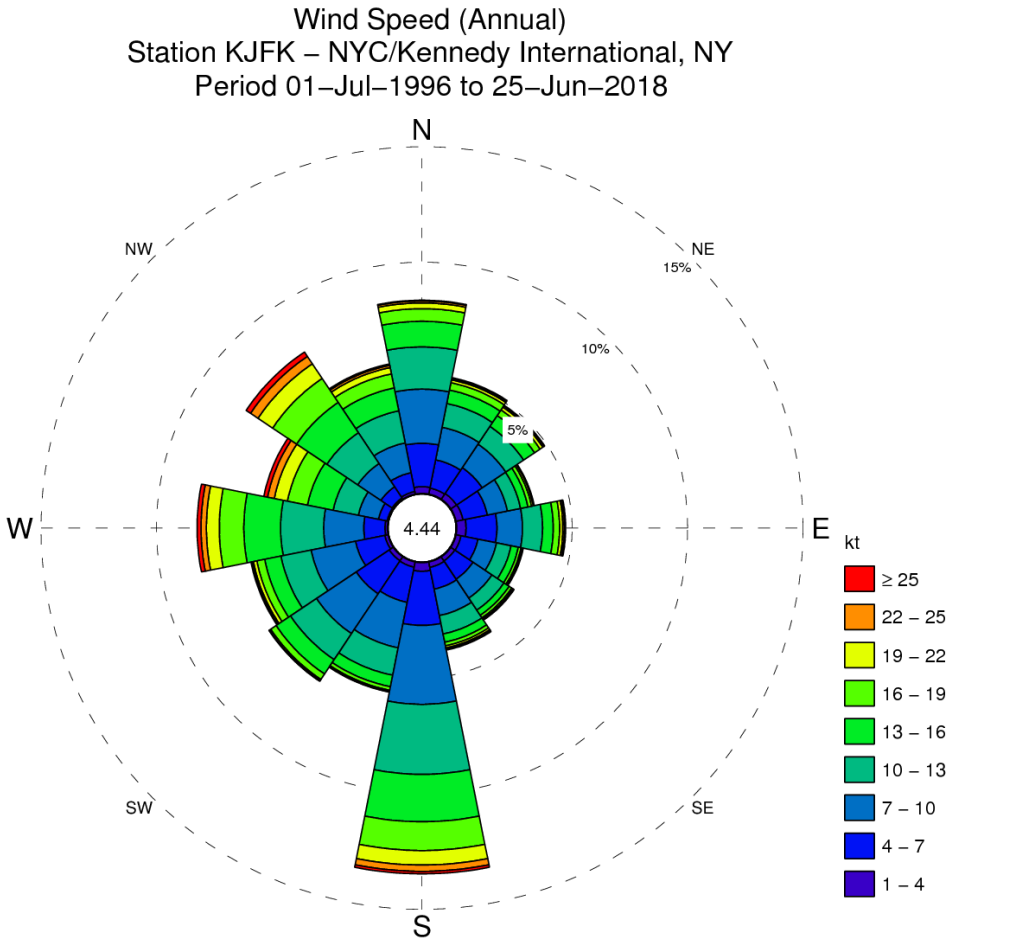
Tidal Datum	Abbreviation	NAVD88 ft	MLLW ft
Highest Observed 10/30/2012	Max Tide	11.69	14.57
Highest Astronomical Tide 10/14/1995	HAT	4.07	6.95
Mean Higher-High Water	MHHW	2.63	5.51
Mean High Water	MHW	2.31	5.19
Mean Sea Level	MSL	-0.18	2.7
Mean Low Water	MLW	-2.67	0.21
Mean Lower-Low Water	MLLW	-2.88	0
Lowest Astronomical Tide 2/08/1997	LAT	-4.51	-1.63
Lowest Observed 2/9/1985	Min Tide	-6.36	-3.48
Mean Tidal Range	MN	4.98	4.98

*used NACCS save points to convert MSL to NAVD

Table C-5: Tidal datums for Station 853069, Amtrak RR Swing bridge, Hackensack River, NJ

Tidal Datum	Abbreviation	NAVD88 ft	MLLW ft
Highest Observed	Max Tide	N/A	N/A
Highest Astronomical Tide	HAT	N/A	N/A
Mean Higher-High Water	MHHW	2.97	5.79
Mean High Water	MHW	2.68	5.5
Mean Sea Level	MSL	0.22	3.04
Mean Low Water	MLW	-2.59	0.23
Mean Lower-Low Water	MLLW	-2.82	0
Lowest Astronomical Tide	LAT	N/A	N/A
Lowest Observed	Min Tide	N/A	N/A
Mean Tidal Range	MN	5.79	5.79

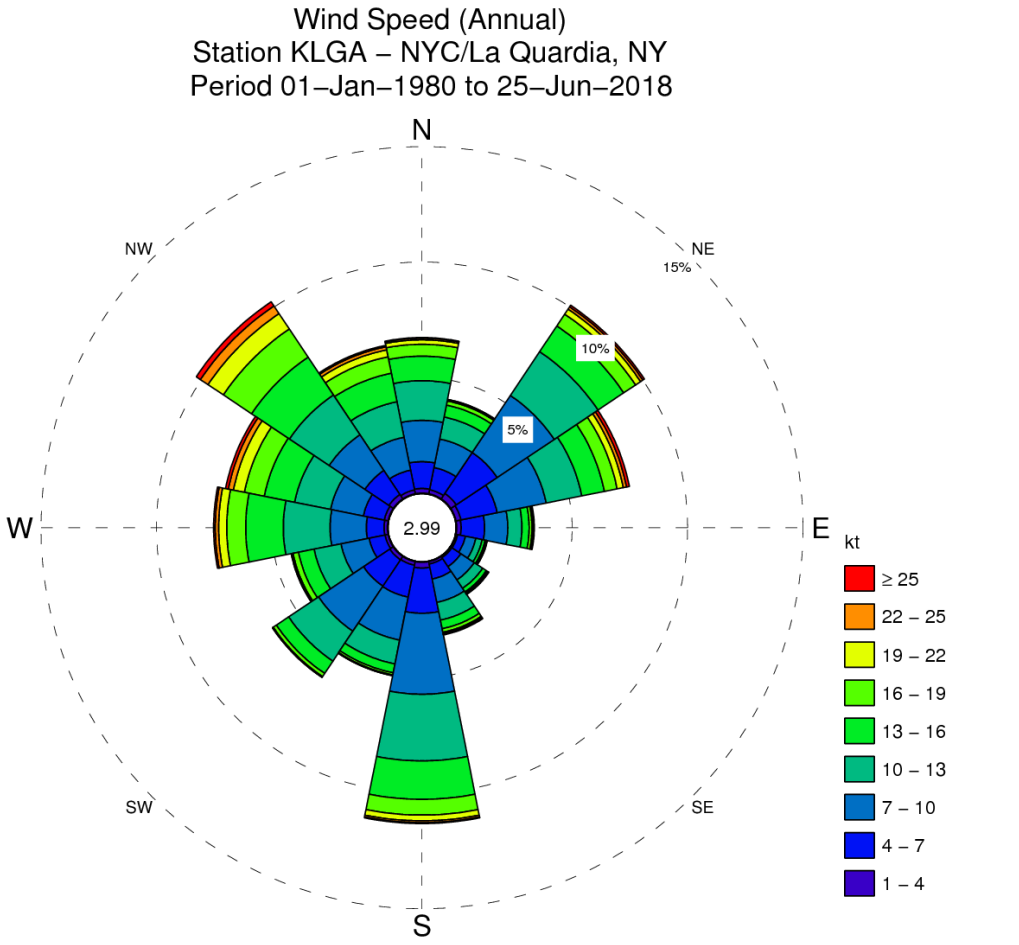
D. APPENDIX D – WIND DATA



Percentage of Occurrence

Wind Speed, kt	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	8.36	5.13	4.89	3.45	4.72	2.97	3.29	3.88	13.46	5.74	6.47	6.05	8.24	5.49	7.66	5.74	95.56
25									0.11				0.17	0.17	0.23		1.06
22									0.27				0.23	0.30	0.39	0.10	1.80
19	0.25	0.13	0.15		0.14				0.63			0.15	0.56	0.58	0.76	0.28	4.01
16	0.54	0.32	0.29	0.16	0.28	0.12	0.13	0.16	1.24	0.16	0.28	0.34	1.06	0.88	1.25	0.62	7.85
13	1.11	0.59	0.57	0.33	0.46	0.32	0.27	0.36	2.05	0.51	0.75	0.72	1.59	1.14	1.58	1.01	13.35
10	1.83	1.05	0.93	0.58	0.87	0.64	0.63	0.81	3.03	1.21	1.36	1.33	1.87	1.11	1.61	1.41	20.27
7	2.34	1.44	1.23	0.88	1.10	0.82	0.99	1.17	3.42	2.01	2.07	1.96	1.74	0.89	1.21	1.31	24.57
4	1.88	1.25	1.34	1.08	1.34	0.75	0.95	0.96	2.34	1.55	1.71	1.30	0.92	0.38	0.58	0.83	19.15
1	0.29	0.26	0.30	0.31	0.41	0.22	0.21	0.24	0.37	0.23	0.21	0.14	0.10			0.11	3.50
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total

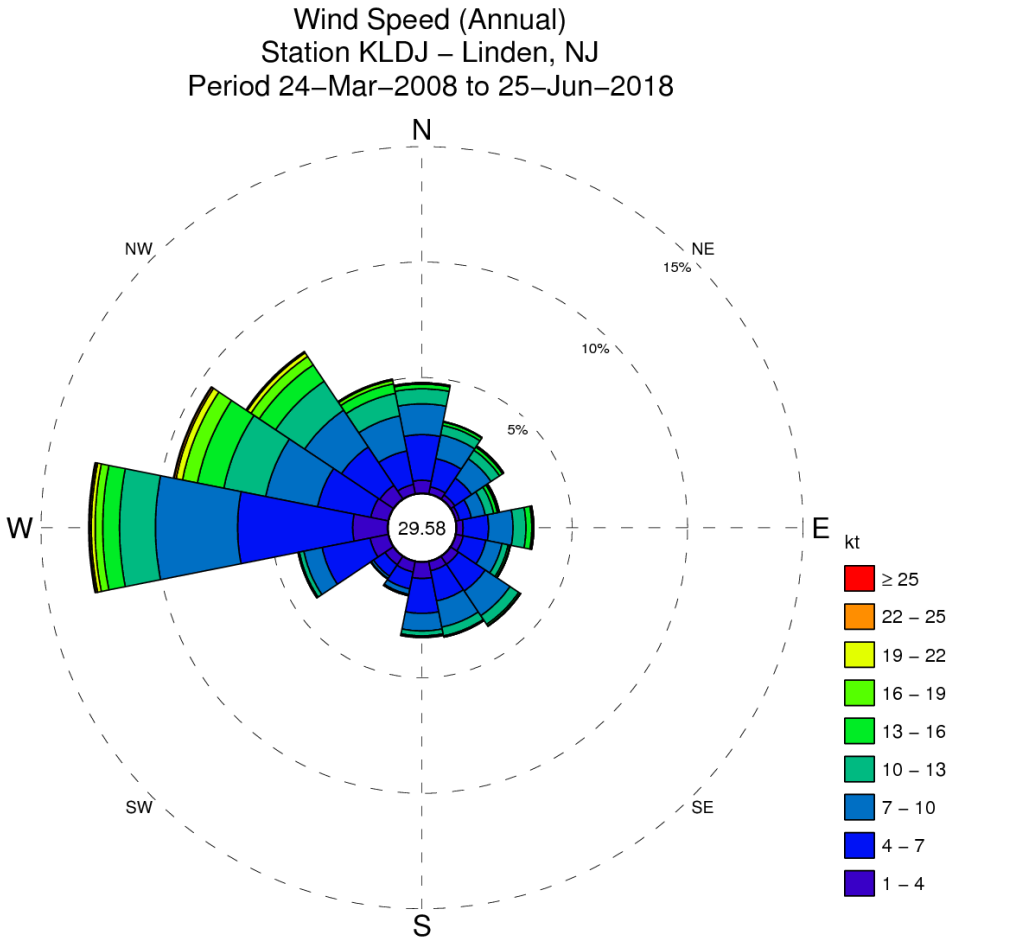
Figure D-1: Percentage of Occurrence for Wind Speed in knots for John F. Kennedy International Airport (Station KJFK) for a 22-year period.



Percentage of Occurrence

Wind Speed, kt	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	6.73	4.16	10.08	7.66	3.36	1.38	1.98	3.23	11.31	5.03	6.29	4.28	7.53	7.17	10.26	6.55	97.01
25				0.13										0.12	0.22		0.83
22			0.12	0.14									0.15	0.22	0.40	0.14	1.44
19	0.19		0.26	0.27					0.25				0.35	0.49	0.80	0.31	3.25
16	0.52	0.18	0.62	0.56	0.12			0.16	0.68		0.16	0.24	0.82	0.88	1.47	0.72	7.35
13	1.06	0.52	1.34	1.00	0.31	0.13	0.19	0.38	1.66	0.39	0.64	0.69	1.63	1.34	1.97	1.26	14.52
10	1.71	0.97	2.28	1.57	0.61	0.28	0.44	0.78	2.89	1.05	1.56	1.18	2.03	1.59	2.07	1.56	22.58
7	1.79	1.26	2.97	2.16	1.00	0.46	0.69	1.03	3.51	1.90	2.34	1.24	1.57	1.44	1.86	1.45	26.69
4	1.16	0.98	2.13	1.60	1.02	0.34	0.47	0.66	1.96	1.41	1.37	0.70	0.77	0.92	1.25	0.87	17.63
1	0.20	0.17	0.28	0.22	0.19			0.10	0.25	0.16	0.18	0.13	0.14	0.17	0.23	0.17	2.72
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total

Figure D-2: Percentage of Occurrence for Wind Speed in knots for La Guardia Airport (Station KLGA) for a 38-year period.



Percentage of Occurrence

Wind Speed, kt	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	4.77	3.19	2.76	1.91	3.34	2.41	3.63	3.33	3.26	1.51	1.18	3.98	12.94	9.48	7.68	5.06	70.42
25																	0.10
22																	0.24
19													0.16	0.28	0.19		0.75
16													0.33	0.66	0.43	0.15	1.90
13	0.20	0.14	0.16	0.16	0.25								0.73	1.17	0.85	0.42	4.40
10	0.65	0.42	0.44	0.37	0.56	0.27	0.45	0.39	0.22			0.20	1.56	1.85	1.51	1.00	9.96
7	1.34	1.05	1.01	0.62	1.06	0.71	1.29	1.16	0.76	0.24	0.12	0.81	3.56	2.29	1.92	1.55	19.46
4	1.98	1.24	0.88	0.53	1.10	1.00	1.39	1.33	1.51	0.75	0.60	2.12	4.98	2.35	2.00	1.48	25.24
1	0.55	0.30	0.21	0.15	0.28	0.32	0.40	0.40	0.70	0.46	0.43	0.79	1.50	0.77	0.70	0.41	8.36
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total

Figure D-3: Percentage of Occurrence for Wind Speed in knots for Linden Airport, NJ (Station KLDJ) for a 10-year period.

E. APPENDIX E – GEOTECHNICAL CONDITIONS

This appendix provides a brief overview of the geological setting for each storm surge barrier location and the assumed soil and geotechnical conditions.

A more detailed site specific analysis is recommended for the next stage for those storm surge barriers that are selected for further study. Such an analysis should investigate the existing geotechnical conditions and any available existing borings at the site. This would help further inform the conceptual design of the storm surge barriers.

Verrazano Narrows

The following characterization is based on publicly available subsurface information for the study area. Appendix A contains a map with the surficial geological conditions.

Geologic Setting:

- Recent Alluvium
- Glacial Moraine (very variably sorted and typically permeable)

The following is assumed for the soil and geotechnical conditions:

- Recent Alluvium – Loose Fine Sands with minor amounts of silt
    - 2 - 5 feet thick over the majority of the alignment
    - Alluvium layer thickens and gradation becomes finer moving Northward
- Glacial Moraine – medium dense to very dense sands, gravels, boulders and lesser amounts of silts and clays

Throgs Neck

The following characterization is based on publicly available subsurface information for the study area. Appendix A contains a map with the surficial geological conditions.

Geologic Setting:

- Recent Alluvium
- Glacial Till
- Lloyd Sand or Raritan Clay

The following is assumed for the soil and geotechnical conditions:

- Recent Alluvium – organic soft silts with varying amounts of fine sand
    - 25+ feet thick over the majority of the alignment
- Glacial till - mainly medium dense to very dense sands and gravels with lesser amounts of silts and clays

- Lloyd Sand – mainly medium dense to very dense sands and gravels with minor amounts of silts and clays
- Raritan Clay

Arthur Kill

The following characterization is based on publicly available subsurface information for the study area. Appendix A contains a map with the surficial geological conditions.

Geologic Setting:

- Recent Alluvium
- Glacial Moraine (very variably sorted and typically permeable)
- Raritan Clay

The following is assumed for the soil and geotechnical conditions:

- Recent Alluvium – organic soft silts with varying amounts of fine sand
    - 3 – 25 feet thick over the majority of the alignment
- Glacial Moraine – medium dense to very dense sands, gravels, boulders and lesser amounts of silts and clays
- Raritan Clay

Outer Harbor

The following characterization is based on publicly available subsurface information for the study area. Appendix A contains a map with the surficial geological conditions.

Geologic Setting:

- Recent Alluvium
- Glacial Outwash (proglacial fluvial deposition)

The following is assumed for the soil and geotechnical conditions:

- Recent Alluvium – loose fine sands with minor amounts of silt
    - 2 - 5 feet thick over the majority of the alignment
    - Alluvium layer thickens and gradation becomes finer as one moves out to sea
- Glacial Outwash - medium dense to dense sands and gravels
    - Gradation becomes finer moving out to sea

Kill van Kull

The following characterization is based on publicly available subsurface information for the study area. Appendix A contains a map with the surficial geological conditions.

Geologic Setting:

- Recent Alluvium
- Bedrock near the mudline

The following is assumed for the soil and geotechnical conditions:

- Recent Alluvium – loose fine sands with minor amounts of silt
    - 2 - 5 feet thick over the majority of the alignment
    - Alluvium layer thickens and gradation becomes finer moving Northward
- Serpentine bedrock

Jamaica Bay

The following characterization is based on publicly available subsurface information for the study area. Appendix A contains a map with the surficial geological conditions.

Geologic setting:

- Beach deposits and alluvium
- Glacial outwash (proglacial fluvial deposition)

The following is assumed for the soil and geotechnical conditions:

- Recent alluvium – loose fine sands with minor amounts of silt
    - 5 - 40 feet thick
    - Alluvium layer thickens as one moves closer to shore and thins out in the deeper portions of the bay
- Glacial outwash - medium dense to dense sands with minor amounts of fine gravel
    - Gradation becomes coarser with depth
- No bedrock encountered up to 150 feet below Mean Low Water (MLW)

Hackensack River

The following characterization is based on publicly available subsurface information for the study area. Appendix A contains a map with the surficial geological conditions.

Geologic Setting:

- Salt-Marsh Deposits
- Alluvium

- Glacial Lake (Hackensack Lake) Deposits
- Sedimentary Rock

The following is assumed for the soil and geotechnical conditions:

- Salt-Marsh Deposits – organic silt and clay with varying amounts of sand and shells
    - 5 - 10 feet thick along the edge of the river bank
    - Salt-marsh deposits gradually thin with an increase in water depth, towards the middle of the river.
- Alluvium – fine to medium sand and clay
    - 5 - 10 feet thick primarily on the southern end of the alignment
    - Alluvium layer pinches out moving out into the river
- Glacial Lake Deposits – soft to stiff varved silts, clays and sand
    - Glacial lake deposits increase in stiffness with depth and/or overburden thickness
- Sedimentary Rock – Shale and/or Arkosic Sandstone
    - Top of rock at approximately 60 to 80 feet below MLW
    - Shale on south end of alignment
    - Arkosic Sandstone on north end of alignment

F. APPENDIX F – GATE TYPE SELECTION FOR THE NAVIGABLE PASSAGE

The storm surge barriers will include navigable passages that will be gated openings with the minimum dimensions as listed in Table 4-1. The table is repeated below for completeness.

Location	Federal Channel	Existing Depth (ft)	Minimum Practical Width of Opening [ft]	Minimum Depth of Opening (ft, MLLW)	Minimum Depth of Opening [ft NAVD88]	Air Clearance [ft NAVD88]
Verrazano	Ambrose Channel	70-75	1400	-55	-58	Unrestricted
Verrazano	Secondary Navigation Channel	20-25	200	-45	-35	50 ft
Throgs Neck	Throgs Neck	40-55	450	-37	-40	Unrestricted
Arthur Kill	Arthur Kill Channel	35	600	-37	-40	Unrestricted
Outer Harbor	Ambrose Channel	75-80	1500	-55	-58	Unrestricted
Outer Harbor	Sandy Hook	60-70	800	-37	-40	Unrestricted
Outer Harbor	Rockaway Inlet	20-30	200	-22	-25	50 ft
Kill Van Kull	Kill Van Kull	40-55	800	-52	-55	Unrestricted
Jamaica Bay	Rockaway Inlet	20-30	200	-22	-25	Unrestricted
Hackensack River	Hackensack River	20-25	100	-20	-23	Unrestricted

Following the requirements above and the information provided in Enclosure 2 a cursory evaluation[8] of gate types and their suitability for the navigable passage has been completed for the Verrazano, Throgs Neck and Outer Harbor storm surge barriers in Table F-1 and for the Arthur Kill, Kill van Kull, Jamiaca Bay and Hackensack storm surge barriers in Table F-2. Pictograms for the various gate types are modified after (Dijk 2010).

[8] It should be noted that without a complete gate type study and evaluation of gate alternatives for the storm surge barriers (inclusive of preliminary cost estimates) this can only be regarded as a preliminary assumption which will have to be verified at a later stage. The evaluation of gate types is to establish a reasonable preliminary selection of gate types such that a preliminary geometry of the openings of the barrier can be established. The geometry and its performance can then be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the barrier designs.

Table F-1: Cursory Review of Hydraulic Gate Types for Application Within the Navigable Passages of the Verrazano, Throgs Neck and Outer Harbor Storm Surge Barriers

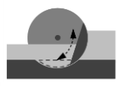
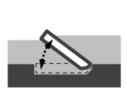
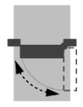
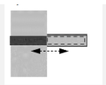
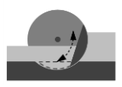
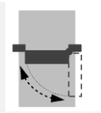
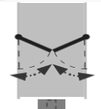
	GATE TYPE	Verrazano Suitability for Main Navigation Gate	Verrazano Suitability for Secondary Navigation Gate	Throgs Neck - Suitability for Main Navigation Gate	Outer Harbor - Suitability for Ambrose Channel Gate	Outer Harbor - Suitability for Sandy Hook Gate	Outer Harbor - Suitability for Rockaway Inlet Gate
	Horizontal Rolling Gate	Impractical due to area needed to dock the gate	Impractical due to area needed to dock the gate	Impractical due to area needed to dock the gate	Impractical due to area need to dock the gate	Impractical due to area need to dock the gate	Impractical due to area need to dock the gate
	Sector Gate (Vertical Axis)	Not suitable for such a large span (no proven constructed concept)	**Suitable**, proven concept for similar span and sill elevation.	Not suitable for such a large span (no proven constructed concept)	Not suitable for such a large span (no proven constructed concept)	Not suitable for such a large span (no proven constructed concept)	**Suitable**, proven concept for similar span and sill elevation.
	Floating Sector Gate	**Suitable**, proven concept for large navigable opening. Note that the 1400ft wide opening would be approx. 200 ft wider than existing largest built gate (Maeslant Barrier)	Not suitable, deemed relatively expensive and too complex system	**Suitable**, proven concept for large navigable opening.	**Suitable**, proven concept for large navigable opening. 300 ft wider than existing largest built gate (Maeslant Barrier)	**Suitable**, proven concept for large navigable opening	Not suitable, deemed relatively expensive and too complex system
	Rotating Segment Gate	Not suitable for such a large span (no proven constructed concept)	**Possible**, but deemed relatively expensive and complex system	Not suitable for such a large span (no proven constructed concept)	Not suitable for such a large span (no proven constructed concept)	Not suitable for such a large span (no proven constructed concept)	**Possible**, but deemed relatively expensive and too complex system
	Inflatable Gate or Dam	Not suitable for such a large deep span (no proven constructed concept)	**Possible**, but no proven concept for the depths at this location	Not suitable for such a large deep span (no proven constructed concept)	Not suitable for such a large deep span (no proven constructed concept)	Not suitable for such a large deep span (no proven constructed concept)	**Possible**, but no proven concept for the depths at this location
	Flap Gate	Not suitable for reverse head conditions and deemed too challenging from a maintenance perspective. In addition, no proven concept for water depths exceeding 48ft.	Not suitable for reverse head conditions and deemed too challenging from a maintenance perspective	**Possible**, as a series of flap gates, but not suitable for reverse head conditions and deemed too challenging from a maintenance perspective	Not suitable for reverse head conditions and deemed to challenging from a maintenance perspective. In addition, no proven concept for water depths exceeding 48ft.	Not suitable for reverse head conditions and deemed to challenging from a maintenance perspective	Not suitable for reverse head conditions and deemed to challenging from a maintenance perspective
	Barge Gate	Not suitable for such a large span (no proven constructed concept)	Not suitable, deemed too challenging to operate under wave and current conditions	Not suitable for such a large span (no proven constructed concept)	Not suitable for such a large span (no proven constructed concept)	Not suitable for such a large span (no proven constructed concept)	Not suitable, deemed too complex and challenging to operate under wave and current conditions
	Miter Gate	Not suitable for such a large span or reverse head conditions	Not suitable due to reverse head conditions	Not suitable for such a large span or reverse head conditions	Not suitable for such a large span or reverse head conditions	Not suitable for such a large span or reverse head conditions	Not suitable due to reverse head conditions
	Vertical Lift Gate	Not suitable since unrestricted air clearance is required	Not suitable since the required air clearance would prohibitively increase gate costs	Not suitable since unrestricted air clearance is required	Not suitable since unrestricted air clearance is required	Not suitable since unrestricted air clearance is required	Not suitable since the required air clearance would prohibitively increase gate costs
	Vertical Rising Gate	Deemed too challenging from a maintenance perspective and no proven concept for such a large gate span	**Possible**, but deemed to challenging from a maintenance perspective	Deemed too challenging from a maintenance perspective and no proven concept for such a large gate span	Deemed to challenging from a maintenance perspective and no proven concept for such a large gate span	Deemed to challenging from a maintenance perspective and no proven concept for such a large gate span	**Possible**, but deemed to challenging from a maintenance perspective
	Tainter Gate	Not suitable since unrestricted air clearance is required	Not suitable since the required air clearance would prohibitively increase gate costs	Not suitable since unrestricted air clearance is required	Not suitable since unrestricted air clearance is required	Not suitable since unrestricted air clearance is required	Not suitable since the required air clearance would prohibitively increase gate costs

Table F-2: Cursory Review of Hydraulic Gate Types for Application Within the Navigable Passages of the Arthur Kill. Kill van Kull, Jamaica Bay and Hackensack Storm Surge Barriers

	GATE TYPE	Arthur Kill - Suitability for Main Navigation Gate	Kill van Kull - Suitability for Main Navigation Gate	Jamaica Bay - Suitability for Main Navigation Gates	Hackensack River - Suitability for Main Navigation Gate
	Horizontal Rolling Gate	Impractical due to area needed to dock the gate	**Possible**, the land area to the south could function as dock area. However difficult to operate under wave and current conditions.	Impractical due to area needed to dock the gate	Impractical due to area needed to dock the gate
	Sector Gate (Vertical Axis)	Not suitable for such a large span (no proven constructed concept)	Not suitable for such a large span (no proven constructed concept)	**Suitable**, proven concept for similar span and sill elevation.	**Suitable**, proven concept for similar span and sill elevation.
	Floating Sector Gate	**Suitable**, proven concept for large navigable opening.	**Suitable**, proven concept for large navigable opening.	Not suitable, deemed relatively expensive and too complex system	Not suitable, deemed relatively expensive and too complex system
	Rotating Segment Gate	Not suitable for such a large span (no proven constructed concept)	Not suitable for such a large span (no proven constructed concept) and too challenging to construct in rock substrate	**Possible**, but deemed relatively expensive and complex system. Potential for sediment built-up as a result of the sediment transport for the Jamaica Bay Inlet	**Possible**, but deemed relatively expensive and complex system.
	Inflatable Gate or Dam	Not suitable for such a large deep span (no proven constructed concept)	Not suitable for such a large deep span (no proven constructed concept)	**Possible**, but no proven concept for tidal inlet applications. In addition, considered a relatively complex system.	**Suitable**, proven concept at other similar location (Ramspol, NL).
	Flap Gate	**Possible**, as a series of flap gates, but not suitable for reverse head conditions and deemed too challenging from a maintenance perspective	Not suitable for reverse head conditions and deemed too challenging to construct in rock substrate.	Not suitable for reverse head conditions and deemed too challenging from a maintenance perspective	**Possible**, but deemed to challenging from a maintenance perspective as it relates to sediment built-up on the sill
	Barge Gate	**Possible**, however for such a large span there is no proven constructed concept	Not suitable for such a large span (no proven constructed concept)	Not suitable, deemed too challenging to operate under wave and current conditions	**Suitable**, proven concept.
	Miter Gate	Not suitable for such a large span or reverse head conditions	Not suitable for such a large span or reverse head conditions	Not suitable due to reverse head conditions	Not suitable for large reverse head conditions
	Vertical Lift Gate	Not suitable since unrestricted air clearance is required	Not suitable since unrestricted air clearance is required	Not suitable since the required air clearance would prohibitively increase gate costs	Not suitable since the required air clearance would prohibitively increase gate costs
	Vertical Rising Gate	Not suitable, deemed too challenging from a maintenance perspective and no proven concept for such a large gate span	Not suitable, deemed too challenging from a maintenance perspective and no proven concept for such a large gate span and too challenging to construct in rock substrate.	**Possible**, however deemed too challenging from a maintenance perspective Potential for sediment built-up as a result of the sediment transport for the Jamaica Bay Inlet	**Possible**, but deemed to challenging from a maintenance perspective as it relates to sediment built-up on the sill
	Tainter Gate	Not suitable since unrestricted air clearance is required	Not suitable since unrestricted air clearance is required	Not suitable since the required air clearance would prohibitively increase gate costs	Not suitable since the required air clearance would prohibitively increase gate costs

G. APPENDIX G – GATE TYPE SELECTION FOR AUXILIARY FLOW GATES

The storm surge barriers will include auxiliary flow gates to minimize the impact on the water exchange through the opening in order to minimize impacts on the inner basin environmental conditions. Requirements for the dimensions for the auxiliary flow gates were established in section 4 and summarized in Table 4-4. The table is repeated here for completeness.

Location	Existing Depth (ft)	Width of Flow Gate Opening (ft)	Depth of Flow Gate Opening (ft, NAVD88)	Required Bottom of the gate in raised position[9]
Verrazano Narrows Auxiliary Flow Gates	Varying between 70ft to 30ft	150	-60, -25, and -20	+7ft NAVD88
Throgs Neck Auxiliary Flow Gates	Varying between 55ft to 40ft	150	-45	+8ft NAVD88
Arthur Kill Auxiliary Flow Gates	Varying between 20ft to 10ft	70*	-10	+7ft NAVD88
Outer Harbor Auxiliary Flow Gates	Varying between 50ft to 20ft	150	varying	+7ft NAVD88
Kill Van Kull Auxiliary Flow Gates	Varying between 40ft to 60ft	150	-28 to -30	+7ft NAVD88
Jamaica Bay Auxiliary Flow Gates	Varying between 300ft to 20ft	150	varying	+7ft NAVD88
Hackensack River Auxiliary Flow Gates	Varying between 20 to 25	50**	varying	+8ft NAVD88

* A gate span of 70ft is preliminarily selected based on the limited width and depth available between the navigable gate and gate housing and the shoreline
** A gate span of 50ft is preliminarily selected based the previous USACE-NAN study (USACE-NAN, 1989).

Following the requirements above and the information provided in Enclosure 2 a cursory evaluation of gate types and their suitability for both the navigable passage as well as the auxiliary flow gates has been completed for all storm surge barriers discussed herein. From the hydraulic

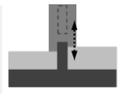
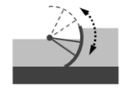
[9] Headwall Elevation is the sum of MHHW for the reference tidal station (see section 3.6.2)

gates as listed in Enclosure 2, a number are omitted from the table below as they are deemed unsuitable;

- The horizontal rolling gate and the sector gate (both floating and vertical axis) are impractical to function as auxiliary flow gate due to area needed to dock the gate,
- The barge gate is tot suitable and too complex of a system to operate and to be used for auxiliary flow,
- The miter gate is not suitable for reverse head conditions or operation during flow conditions,
- The flap gate, although potentially possible in some location, it is not suitable for reverse head conditions and in general is considered to be too challenging from a maintenance perspective and as such not selected for the conceptual design
- The vertical rising gate, similar to the flap gate, could potentially be utilized in some locations, but is in general considered to be relatively expensive and too complex a system to be used as auxiliary flow gate.

The evaluation of gate types for auxiliary flow is summarized in Table G-1.

Table G-1: Cursory Review of Hydraulic Gate Types for Use as Auxilary Flow Gates for all Storm Surge Barriers

	GATE TYPE	Verrazano - Suitability for Auxiliary Flow Gates	Throgs Neck - Suitability for Auxiliary Flow Gates	Arthur Kill - Suitability for Auxiliary Flow Gates	Outer Harbor - Suitability for Auxiliary Flow Gates	Kill van Kull - Suitability for Auxiliary Flow Gates	Jamaica Bay - Suitability for Auxiliary Flow Gates	Hackensack - Suitability for Auxiliary Flow Gates
	Rotating Segment Gate	Not suitable, no need for unrestricted air clearance, furthermore deemed relatively expensive and too complex system. No proven concept for water depths exceeding of 30ft.	Not suitable, no need for unrestricted air clearance, furthermore deemed relatively expensive and too complex system. No proven concept for water depths exceeding of 30ft.	**Possible**, but deemed relatively expensive and complex system for relatively small auxiliary flow gate	Not suitable, deemed relatively expensive and too complex a system to operate in open ocean wave conditions	Not suitable, no need for unrestricted air clearance, furthermore deemed relatively expensive and too complex system	**Possible**, but deemed relatively expensive and complex system	**Possible**, but no need for unrestricted air clearance, furthermore deemed relatively expensive and too complex system
	Inflatable Gate or Dam	**Possible**, but no proven concept for the depths at this location	**Possible**, but no proven concept for the depths at this location	**Possible**, but deemed relative complex system for such a relatively small gate.	**Possible**, but no proven concept for the depths at this location	**Possible**, but no proven concept for the depths at this location	**Possible**, but no proven concept for tidal inlet applications. Furthermore, deemed relatively complex system.	**Suitable**, proven concept for tidally influenced river application (albeit one constructed project).
	Vertical Lift Gate	**Suitable**, proven concept	**Suitable**, proven concept	**Suitable**, proven concept	**Suitable**, proven concept	**Suitable**, proven concept	**Suitable**, proven concept	**Suitable**, proven concept
	Tainter Gate	**Suitable**, proven concept, however reverse head conditions will need to be investigated	**Suitable**, proven concept, however reverse head conditions will need to be investigated	**Suitable**, proven concept, however reverse head conditions will need to be investigated	**Suitable**, proven concept, however reverse head conditions will need to be investigated	**Suitable**, proven concept, however reverse head conditions will need to be investigated	**Suitable**, proven concept, however reverse head conditions will need to be investigated	**Suitable**, proven concept, however reverse head conditions will need to be investigated

US Army Corps of Engineers®
New York District

INTERIM REPORT

New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study

Enclosure 1

Maritime Traffic Analysis

February 2019

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New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study

Maritime Traffic Analysis

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1 EXECUTIVE SUMMARY

This report documents maritime traffic analyses and recommendations for design vessels for storm surge barriers with navigable passages under the New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study (HAT Study). From this vessel evaluation, a preliminary assessment is made for minimum feasible widths of the navigable passages through the storm surge barriers. The dimensions identified in this study are very preliminary. There are numerous factors that would affect the final selection of gate widths for the navigable barriers that go well beyond the scope of this brief study.

The preliminary assessment of navigable passage width shall not be construed as recommendations or requirements for actual width necessary for implementation. Significant additional study is required to substantiate the width, location, and configuration of the passages, including a full evaluation of issues including, but not limited to, navigation, environmental, ecological, and cost considerations.

A preliminary listing of potential future studies that may be required include the following:

- One-way versus two-way vessel passage, including meeting, passing, and overtaking
- Number, frequency, and intensity of vessel passage
- Vessel wait areas, queuing, and wait times
- Cross-traffic conditions
- Barrier positioning and fairway lengths for maneuvering
- Currents, cross currents, wind, tides, surge, weather, night, visibility, and other environmental considerations for vessel passage
- Trends for future vessel traffic, including vessel size, frequency
- Requirements for vessel traffic service, including advisory / control / restrictions on navigation
- Requirements for Aids-to-Navigation, guide structures, and protective structures
- Navigation evaluations, including pilot and navigation industry input, and real-time simulations to assess, amongst others:
    - Flow and cross-current considerations
    - Gate approach and departure
    - Passing vessel assessments
- Environmental considerations, including water quality impacts, ecological issues
- Project cost evaluations

This report is part of a series of reports that document the preliminary analysis completed in support of the feasibility of the New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study.

2 INTRODUCTION

2.1 Introduction

This report documents maritime traffic analyses and recommendations for design vessels for the following storm surge barriers with navigable passages under the HAT Study.

Table 2-1: Storm Surge Barriers under HAT Study discussed herein

Storm Surge Barrier Name under HAT Study	Abbreviation used throughout this document and other HAT Study documents
Outer Harbor (Also referred to as the Sandy Hook to Breezy Point storm surge barrier)	SHBP Storm Surge Barrier
Throgs Neck	TN Storm Surge Barrier
Arthur Kill	AK Storm Surge Barrier
Verrazano Narrows	VN Storm Surge Barrier
Jamaica Bay	Jamaica Bay Storm Surge Barrier
Kill van Kull	KVK Storm Surge Barrier
Newtown Creek	Newtown Creek Storm Surge Barrier
Gowanus Canal	Gowanus Canal Storm Surge Barrier

It is assumed that the reader is familiar with the geographic extent of the HAT Study area and the location of the barriers. For location references, authorized channel dimensions and maps the reader is referred to other reports which are being completed for this study.

The purpose of this report is to document a preliminary assessment of the minimum required dimensions for the navigable passages of the proposed storm surge barriers under the HAT Study. The dimensions identified in this study are very preliminary. There are numerous factors that would affect the final selection of gate widths for the navigable barriers that go well beyond the scope of this brief study. This study is performed under the following list of broad assumptions:

- No future channel widening projects are considered. M&N is not aware of any future plans to widen the federally authorized navigation channels under consideration. Recommended gate widths do not make any allowance for future, wider channels.
- No future channel deepening projects are considered. M&N is not aware of any future plans to deepen the federally authorized navigation channels under consideration. Recommended sill elevations do not make any allowance for future, deeper channels.
- A very limited forecast of future vessel sizes was performed, providing a design vessel of a 400m long container ship for the main shipping channel for this study. For other channels (e.g., Arthur Kill), no evaluation of future vessels was performed. A future phase of study for this project should include much more detailed evaluation of potential future vessels.

- Both one-way and two-way traffic widths are computed for barriers in locations where the existing channel allows for two-way traffic. It is beyond the scope of this study to provide a conclusion regarding whether it would be acceptable to restrict passage to one-way traffic, for certain vessel sizes, in an existing two-way channel (e.g., Ambrose Channel). A large majority of the deep draft harbor traffic passes through Kill van Kull, which currently already is a one-way channel for larger vessels. The feasibility of adding an additional one-way traffic area is only discussed at a very high level in this study and must be confirmed by further study, in coordination with the port and the harbor pilots.
- For navigable gate width calculations, guidance on channel widths is taken from both EM 1110-2-1613 *Hydraulic Design of Deep-Draft Navigation Projects* and PIANC Working Group 121 report *Harbour Approach Channels Design Guidelines*. Neither of these reports provide methodology specifically for sizing navigable storm surge barrier gates. With regard to this topic, EM states that "the width and depth of the navigation gap should be designed to allow adequate clearance by normal size ships with due regard for safety of ship transits inside the barrier." The PIANC guidance includes a factor in the channel width calculation for "hard structures" along the banks, but does not provide any discussion for storm surge barriers specifically. A more detailed study is required, in coordination with the harbor pilots, to firmly establish navigable opening widths that would be acceptable for these locations. The estimates provided here are very preliminary and are subject to change upon a more detailed analysis.
- A detailed analysis of current velocity magnitudes and increases in currents through the navigable passages of the storm surge barriers has been omitted at this point and will need to be completed at a later date.
- In addition, an analysis of the vessel-flow interactions through the navigable passes has been omitted at this point. I.e., during transit the vessel partially blocks the navigable passage and the reduction in cross-sectional area can result in increases of current flow velocities thereby. A more detailed study of vessel-flow interactions is recommended after a feasible alternative has been selected.
- It is recognized that there are additional storm surge barriers that are part of the HAT Study Alternatives beyond the ones listed in **Table 2-1**. Those storm surge barriers are; the Hackensack River, East Chester Creek, Westchester Creek, Sheepshead Bay, Flushing Creek, Bronx River and Gerritsen Creek storm surge barrier, are not discussed in detail in this report. For this set of storm surge barriers there is insufficient reliable AIS data. This provides the basis for the assumption that to no ore very infrequent deep draft maritime traffic is expected to traverse those smaller barriers. Navigable passage dimensions for those barriers will instead be based on existing federal channel dimensions or on other known channel restrictions.

2.2 AIS DATA

Both the International Maritime Organization (IMO, SOLAS, Chapter V, Regulation 19, Paragraph 2.4) and the United States Government (33 CFR 164.46) require that most commercial vessels[1] maintain an operational Automatic Identification System (AIS). The shipborne AIS provides regular (every 10 seconds, or more frequent) very high frequency (VHF) radio signals containing basic information regarding the ship's identity, location, speed, etc. This system was originally outlined by the IMO in Resolution A.917(22) in 2002 and was implemented progressively over the following years.

AIS data is broadcast over two specific VHF channels and is not encrypted, making the data easily accessible to any other shipborne or shore-based receiver. AIS data is collected, stored and shared by a variety of organizations for different purposes. Moffatt & Nichol maintains a large in-house database of AIS data collected from in-house receivers, as well as global data from participation in real-time data-sharing co-ops.

Taken as spatial data, AIS signals are individual points. However, by grouping points from individual vessels, the track of each vessel can be drawn (Figure 2-1, for Verrazano Narrows). Vessel track data provides a straightforward way of generating statistics for how many vessels pass certain locations. While vessel tracks are helpful for statistics, they are difficult to visualize when numerous tracks are present. Traffic intensity maps (Figure 2-2, for Verrazano Narrows) are often more helpful for visualizing what areas see higher traffic than others. As Figure 2-2 shows, traffic maps can be prepared for an entire dataset or for only a portion (e.g., a specific vessel type).

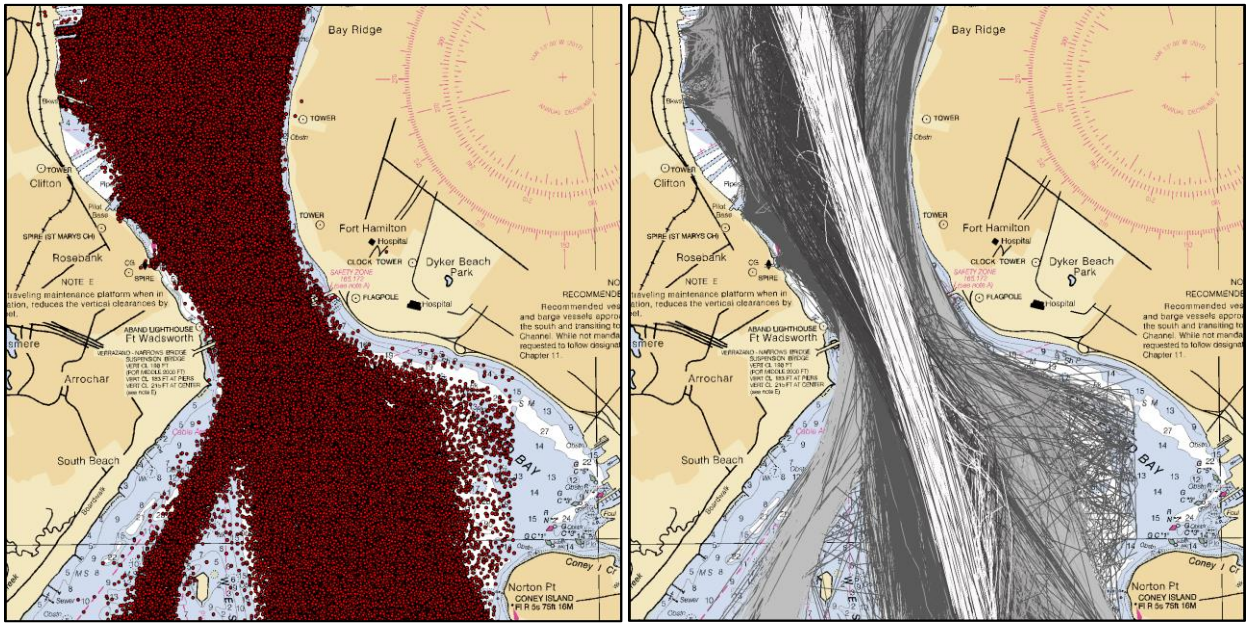

Figure 2-1: AIS Data Signals (left) and Tracks (right)

[1] All vessels over 500 gross tonnage, and smaller vessels under certain conditions.

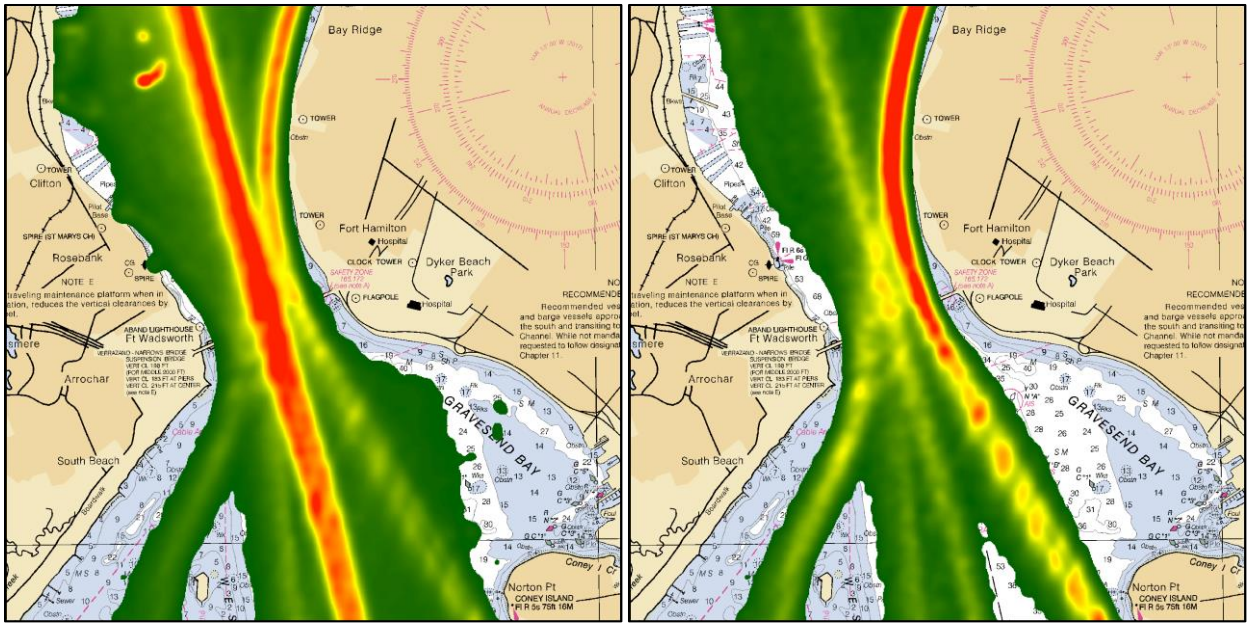

Figure 2-2: Vessel Traffic Intensity Maps – All Traffic (left) – Passenger Vessels (right)

For this study, AIS data was queried for five of the storm surge barrier locations under consideration. AIS data was not used to develop traffic statistics for the other barrier locations, because the vessels transiting those locations are smaller and do not reliably report AIS data. Other sources are more appropriate for those locations. AIS data is presented for the following five barrier locations:

- Verrazano Narrows – Section 3.
- Sandy Hook to Breezy Point – Section 4.
- Throgs Neck – Section 5.
- Arthur Kill – Section 6.
- Kill van Kull – Section 7.

3 VERRAZANO NARROWS

3.1 Traffic Summary

Traffic statistics were prepared for Verrazano Narrows based on part-year data from 2017 and 2018, representing a total of 310 days of data. The purpose of this study is not to develop absolute/precise numbers for vessel passes, so the gaps present in the dataset were not of concern for this analysis. Table 3-1, Table 3-2, Figure 3-1, and Figure 3-2, present summaries of the vessel traffic passing through the Verrazano Narrows. Table 3-1 and Figure 3-1 present the data categorized by vessel length. Table 3-2 and Figure 3-2 present the data categorized by vessel beam (width).

Table 3-1: Vessel Length Statistics - Verrazano Narrows

LOA [ft]	Bulker	Container	General Cargo	Other	Passenger	Ro-Ro	Tanker	Tug	Total
0-100				5771	10330		15	2580	18696
100-200				3495	3597		2	3346	10440
200-300				102	7		37	25	171
300-400		87	53	16	13		14	1	184
400-500	4	8	38	9	3		119	42	223
500-600	70	199	34	1	16	52	340		712
600-700	218	244	61	2	14	582	842	3	1966
700-800	53	82	4	1	22	108	220		490
800-900		493	1	1	45	4	185	1	730
900-1000		938	83	7	143		5		1176
1000-1100		739			91				830
1100-1200		344			93				437
1200-1300		85							85
2000-2100				1					1
2200-2300				2					2
2400-2500				2					2
Unspecified				148				1	149
Total	**345**	**3219**	**274**	**9558**	**14374**	**746**	**1779**	**5999**	**36294**

Table 3-2: Vessel Beam Statistics - Verrazano Narrows

Beam [ft]	Bulker	Container	General Cargo	Other	Passenger	Ro-Ro	Tanker	Tug	Total
0-20				3251	57			197	3505
20-40				6010	13869		15	5649	25543
40-60		1	49	98	8		14	75	245
60-80	4	97	54	32	15		182	71	455
80-100	73	434	39	12	41	2	233	2	836

100-120	268	766	53	16	166	725	1054	3	3051
120-140		770	79	1	111	19	116		1096
140-160		1129			34		165	1	1329
160-180		22			73				95
Unspecified				138				1	139
Total	**345**	**3219**	**274**	**9558**	**14374**	**746**	**1779**	**5999**	**36294**

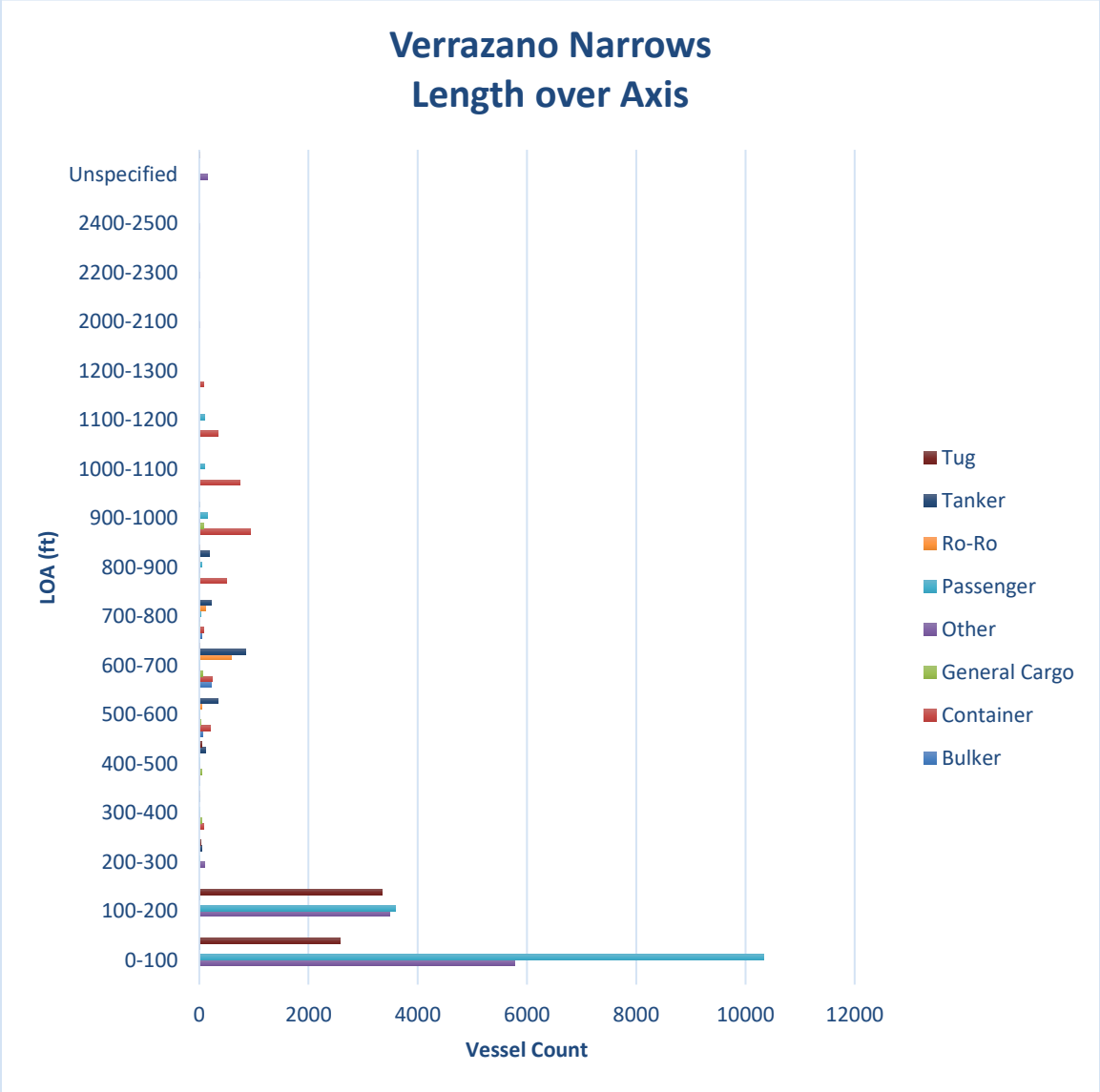

Figure 3-1: Vessel Length Statistics - Verrazano Narrows

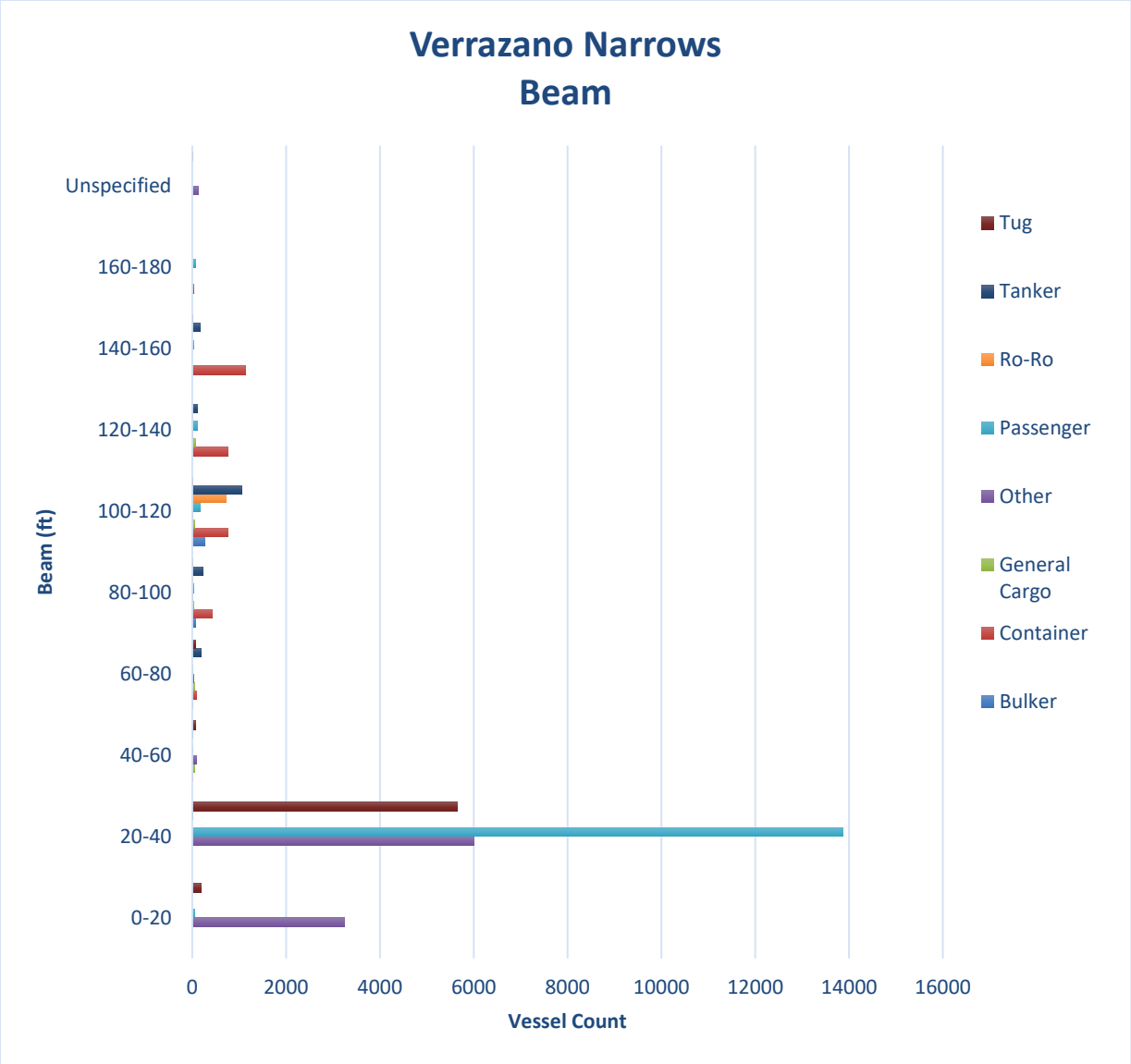

Figure 3-2: Vessel Beam Statistics - Verrazano Narrows

3.2 Minimum Practical Channel Width (Main Navigable Passage) – Verrazano Narrows

The vessel data presented above provides a perspective of the current traffic. However, future traffic may involve larger vessels, and either greater or lesser vessel count. For this study a larger vessel was used for sizing the gate opening and based on a cursory review of vessel size trends (included in Appendix A). It is beyond the scope of this study to address the question of whether or not larger vessels could safely navigate the NY harbor channels, particularly the Kill van Kull.

The design vessel for the main gate opening was taken to be the OOCL Hong Kong, an Ultra Large Container Vessel (ULCV), with summary specifications as follows:

- Container Capacity: 21,413 TEU,
- Length over Axis (LOA): 400m (approx. 1310ft),
- Beam: 58.8m (193ft)

Using this design vessel, the USACE Engineering manual EM 1110-2-1613 *Hydraulic Design of Deep-Draft Navigation Projects* was consulted first for insight into appropriate one and two-way channel widths. The channel width beam multipliers, for three channel cross-section categories, provided in the EM are summarized in Table 3-3 and Table 3-4 for one-way and two-way channels, respectively. Unfortunately, none of the three channel cross-section categories presented in the EM apply very well to the Verrazano Narrows channel, which is naturally deep in the areas currently under consideration for the barrier.

Table 3-3: One-Way Ship Traffic Channel Width Design Criteria (EM 1110-2-1613, Table 8-2)

Channel Cross Section	Design Ship Beam Multipliers for Maximum Current, Knots		
	0.0 to 0.5	0.5 to 1.5	1.5 to 3.0
	Constant Cross Section, Best Aids to Navigation		
Shallow	3.0	4.0	5.0
Canal	2.5	3.0	3.5
Trench	2.75	3.25	4.0
	Variable Cross Section, Average Aids to Navigation		
Shallow	3.5	4.5	5.5
Canal	3.0	3.5	4.0
Trench	3.5	4.0	5.0

Table 3-4: Two-Way Ship Traffic Channel Width Design Criteria (EM 1110-2-1613, Table 8-3)

Channel Cross Section	Design Ship Beam Multipliers for Maximum Current, Knots		
	0.0 to 0.5	0.5 to 1.5	1.5 to 3.0
	Constant Cross Section, Best Aids to Navigation		
Shallow	5.0	6.0	8.0
Canal	4.0	4.5	5.5
Trench	4.5	5.5	6.5

As far as geometry is concerned, the channel cross section, upstream and downstream of the navigable passage, would be most similar to the "Shallow" type section (EM 1110-2-1613, page 8-2), which would lead to a channel width of 5.0 to 5.5 times the vessel beam, inclusive of the assumption that one would experience high currents through the navigable passage (depending on aids to navigation, etc., however for a world-class port like the port of New York and New Jersey, *best aids* to navigation may be assumed). However, the reason "Shallow" type channels are specified as wider is that they are often subject to cross currents, which are expected to be minor to merit consideration in the Verrazano Narrows. Thus, it seems reasonable to state that the channel width requirements would be less than required for the "Shallow" designation. The geometry of Verrazano Narrows does not fit the Canal or Trench designations well. It does however, seem reasonable to use the width multiplier specified for the Trench category as a lower bound because the naturally deep waters in the Verrazano narrows would be less restricting (e.g., no bank effects) than a Trench channel.

Given the discussion above, a first estimate of a one-way channel width for Verrazano Narrows is expected to be between 4.0 and 5.0 times the design vessel beam (772 to 965 ft). This width range assumes "best" aids to navigation, but allows for relatively high currents (1.5 to 3.0 knots), which can reasonably be expected in Verrazano Narrows. By similar logic, we would expect a first estimate for a two-way channel width to be between 6.5 and 8.0 times the design vessel beam (1,254 ft to 1,543 ft, assuming passing of two 21,000 TEU vessels, which is conservative). Further study would be required to identify the appropriate design vessels for the design of a two-way passing width if the navigable passage is to be designed to accommodate two-way traffic.

These widths do not explicitly account for the effects of the barrier itself. Even in the absences of a hard barrier, setting the width of navigable waterways is a complex question. Further study is necessary to determine the actual required barrier opening width, with consideration for numerous factors, including, but not limited to:

- Design ship beam, length, and draft.
- Local piloted ship control.
- Channel cross section and alignment.
- River and tidal currents.
- Navigation traffic pattern (one- or two-way).
- Vessel traffic intensity and congestion.
- Wind and wave effects.
- Visibility.
- Quality and spacing of navigation aids.
- Composition of channel bed and banks.
- Variability of channel and currents.
- Speed of design ship.

Because the USACE manual does not provide any discussion of how hard structures on each bank would affect the required channel width, a separate calculation was performed following the approach outlined by the World Association for Waterborne Transport Infrastructure (PIANC) in the Working Group 121 report *Harbour Approach Channels Design Guidelines*. This calculation is summarized in Table 3-5 for the Verrazano Narrows main navigation gate. Based on this method, a minimum channel width for one-way traffic in Verrazano Narrows would be 968 ft. According to PIANC, the two-way channel width can be estimated by doubling all of the factors in Table 3-5 except the Bank Allowance factors. This results in a two-way width of 7.4 times the vessel beam (1,428 ft).

Based on both USACE and PIANC channel width analyses, it seems appropriate to recommend 1,000 ft as a preliminary gate width for one-way and 1,400 ft as a preliminary gate width for two-way traffic. While it is not possible at this point to determine definitively whether two-way traffic is required (or what size vessels would be used for the design of a two-way gate width), a gate width of 1,400ft seems conservatively appropriate for use as preliminary estimate in the overall storm surge barrier feasibility study.

Table 3-5: PIANC Channel Width Calculation for Verrazano Narrows Main Channel (Navigable Passage)

Parameter	Specification	Factor [x Beam] One-way traffic	Factor [x Beam] Two-way traffic
Basic Maneuvering Lane	Good	1.3	2.6
Location	Inner Channel (Protected Water)	0	0
Vessel Speed	Fast (>= 12 kts)	0.1	0.2
Prevailing Cross-wind	Moderate (<33 kts)	0.3	0.6
Additional Factor for High Wind Area	(based on vessel type)	0	0
Prevailing Cross-current	Negligible (<0.2 kts)	0	0
Prevailing Longitudinal Current	Strong (>=3 kts)	0.1	0.2
Wave Effects	Mild (Hs<=1m)	0	0
AtoN	Excellent	0	0
Bottom Surface	Rough and Hard	0.2	0.4
Depth of Waterway, h	h < 1.25T	0.4	0.8
Bank Allowance, Red Side (left)	Steep and Hard Embankments/ Structures	1.3	1.3
Bank Allowance, Green Side (right)	Steep and Hard Embankments/ Structures	1.3	1.3
Total Channel Width [x Beam]		5.0	7.4
Total Channel Width [ft]		968	1428

3.3 Minimum Practical Channel Width (Additional Passage for Smaller Vessels)

In addition to the main navigation channel, the AIS data (Section 3.1) suggests that there is a high volume of smaller traffic. It would be advisable to include at least one smaller, navigable gate in the barrier to pass smaller vessels, keeping the smaller vessels out of the main navigation channel. For this purpose, a ferry was identified as the design vessel. The particulars for this design vessel are as follows:

- Design Vessel Name: Seastreak
- Length: 42.9m (LOA),
- Beam: 10.45m.

Using this design vessel, a design channel width of 178ft was computed using the PIANC method (Table 3-6) for a one-way channel. Given the space available in the main navigable passage, it was deemed appropriate to size the additional passage for one-way traffic only. Further study would be needed to confirm this assumption. Based on this estimate, it seems reasonable to recommend a 200 ft wide opening for the supplementary / smaller vessel navigable passage for this feasibility study. From the AIS data and traffic patterns it is recommended to place the secondary navigable passage on the east side of the main passage.

Table 3-6: PIANC Channel Width Calculation for Verrazano Narrows (Additional Navigable Passage)

Parameter	Specification	Factor [x Beam] One-way traffic
Basic Maneuvering Lane	Good	1.3
Location	Inner Channel (Protected Water)	0
Vessel Speed	Fast (>= 12 kts)	0.1
Prevailing Cross-wind	Moderate (<33 kts)	0.3
Additional Factor for High Wind Area	(based on vessel type)	0.2
Prevailing Cross-current	Negligible (<0.2 kts)	0
Prevailing Longitudinal Current	Strong (>=3 kts)	0.1
Wave Effects	Mild (Hs<=1m)	0
Aids to Navigation	Excellent	0
Bottom Surface	Rough and Hard	0.2
Depth of Waterway, h	h < 1.25T	0.4
Bank Allowance, Red Side (left)	Steep and Hard Embankments/ Structures	1.3
Bank Allowance, Green Side (right)	Steep and Hard Embankments/ Structures	1.3
Total Channel Width [x Beam]		5.2
Total Channel Width [ft]		178

4 SANDY HOOK TO BREEZY POINT

Traffic and gate widths are presented for three different locations along the Sandy Hook to Breezy Point Barrier, corresponding to the three federally-maintained navigation channels that cross the barrier alignment:

- Sandy Hook Channel – Section 4.1.
- Ambrose Channel – Section 4.2.
- Rockaway Inlet – Section 4.3.

In each case, the same approach is applied as for Verrazano Narrows (Section 3). Traffic statistics were prepared based on part-year data from 2017 and 2018, representing a total of 329 days of data.

4.1 Sandy Hook Channel

4.1.1 Traffic Summary

Table 4-1, Table 4-2, Figure 4-1 and Figure 4-2 present summaries of the vessel traffic passing through Sandy Hook. Table 4-1 and Figure 4-1 present the data categorized by vessel length and Table 4-2 and Figure 4-2 present the data categorized by vessel length.

Table 4-1: Vessel Length Statistics - Sandy Hook Channel

LOA [ft]	Other	Passenger	Tanker	Tug	Total
0-100	174	33	1	329	537
100-200	55	2	1	273	331
200-300	39	1			40
300-400	1				1
400-500			16	4	20
500-600			92		92
600-700	8		305	3	316
700-800	8		65		73
800-900			78		78
900-1000			1		1
Unspecified	102			1	103
Total	**387**	**36**	**559**	**610**	**1592**

Table 4-2: Vessel Beam Statistics - Sandy Hook Channel

Beam [ft]	Other	Passenger	Tanker	Tug	Total
0-20	145			1	146
20-40	86	4	1	589	680
40-60	36	31	1	11	79
60-80	1		29	5	35
80-100	1	1	53		55
100-120	16		366	3	385
120-140			37		37
140-160			72		72
Unspecified	102			1	103
Total	**387**	**36**	**559**	**610**	**1592**

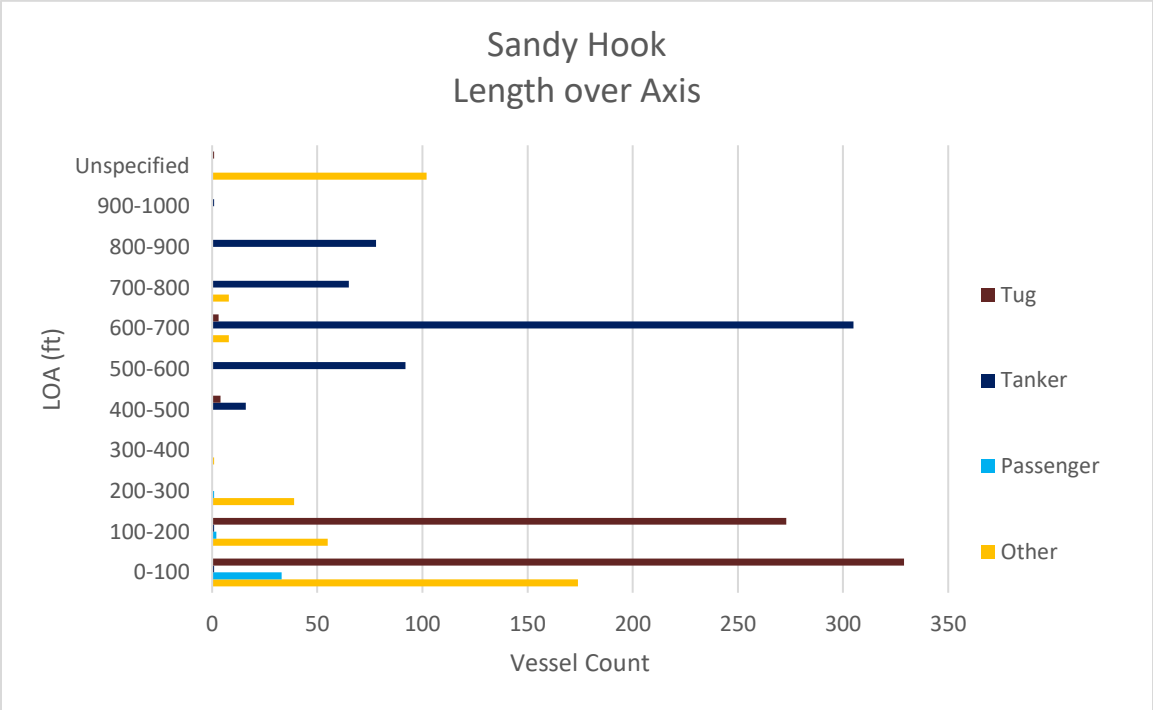

Figure 4-1: Vessel Length Statistics - Sandy Hook Channel

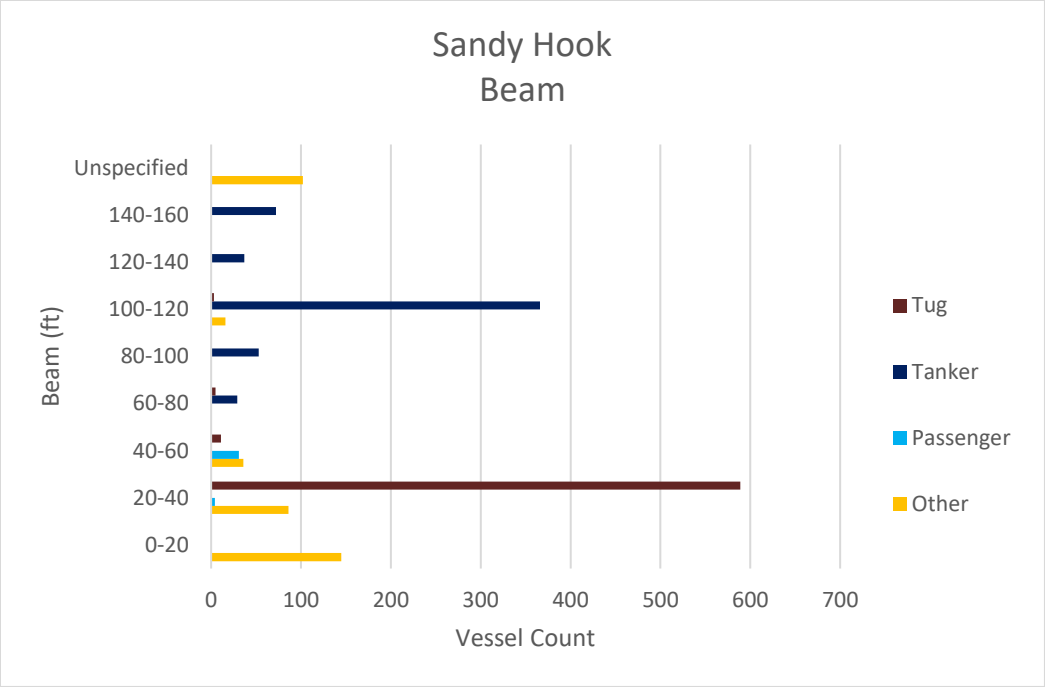

Figure 4-2: Vessel Beam Statistics - Sandy Hook Channel

4.1.2 Minimum Practical Channel Width – Sandy Hook

Based on the vessel data presented above, the largest vessel expected to transit the Sandy Hook Channel is a tanker. The design vessel for the Sandy Hook Channel gate is taken to be the following:

- Design Vessel Name: Cape Bonny
- Length: 274.5m (LOA)
- Beam of 48m.

Using this design vessel, the channel width estimates were obtained as 630-866 feet based on the USACE manual and 835ft based on PIANC guidance (Table 4-3) for one-way traffic. These estimates confirm that the current authorized channel width of 800 ft is appropriate to serve as the width of the corresponding storm surge barrier gate.

Table 4-3: PIANC Channel Width Calculation for Sandy Hook Channel (Navigable Passage)

Parameter	Specification	Factor [x Beam] One-way Traffic
Basic Maneuvering Lane	Good	1.3

Location	Outer Channel (Open Water)	0
Vessel Speed	Fast (>= 12 kts)	0.1
Prevailing Cross-wind	Moderate (<33 kts)	0.3
Additional Factor for High Wind Area	(based on vessel type)	0
Prevailing Cross-current	Negligible (<0.2 kts)	0
Prevailing Longitudinal Current	Strong (>=3 kts)	0.1
Wave Effects	Moderate (Hs>1m)	0.5
AtoN	Excellent	0
Bottom Surface	Rough and Hard	0.2
Depth of Waterway, h	h < 1.25T	0.2
Bank Allowance, Red Side (left)	Steep and Hard Embankments/ Structures	1.3
Bank Allowance, Green Side (right)	Steep and Hard Embankments/ Structures	1.3
Total Channel Width [x Beam]		5.3
Total Channel Width [ft]		835

4.2 Main Ambrose Channel

4.2.1 Traffic Summary

Similar to Verrazano Narrows, the Ambrose Channel is on the primary shipping route for deep draft vessels entering the harbor. As expected, the deep draft traffic statistics for larger vessels appear very similar to the statistics for Verrazano Narrows (Section 3.1). Table 4-4, Table 4-5, Figure 4-3, and Figure 4-4 summarize the vessel traffic passing through Ambrose Channel. Table 4-4 and Figure 4-3 present data categorized by vessel length. Table 4-5 and Figure 4-4 present the data categorized by vessel beam.

Table 4-4: Vessel Length Statistics - Ambrose Channel

LOA [ft]	Bulker	Container	General Cargo	Other	Passenger	Ro-Ro	Tanker	Tug	Total
0-100				162	11			284	457
100-200				237	2		2	1338	1579
200-300				48	3		1	17	69
300-400		90	54	8	9		4	2	167
400-500	4	8	41	8	3		116	12	192
500-600	76	190	33	1	17	52	310		679
600-700	222	249	56	2	14	610	800	1	1954
700-800	53	88	4		22	117	235		519
800-900		491	1		45	4	200		741
900-1000		956	88	6	150		4		1204
1000-1100		769			98				867
1100-1200		360			98				458
1200-1300		96							96
2200-2300				1					1
Unspecified				13					13
Total	355	3297	277	486	472	783	1672	1654	8996

Table 4-5: Vessel Beam Statistics - Ambrose Channel

Beam [ft]	Bulker	Container	General Cargo	Other	Passenger	Ro-Ro	Tanker	Tug	Total
0-20				146	1			5	152
20-40				258	10			1585	1853
40-60		1	48	45	9		4	44	151
60-80	4	97	59	7	8		141	18	334
80-100	79	434	37	10	41	2	223		826
100-120	272	753	48	6	173	759	1004	1	3016
120-140		802	85	1	111	22	126		1147

140-160		1183			40		174	1	1398
160-180		27			79				106
Unspecified				13					13
Total	355	3297	277	486	472	783	1672	1654	8996

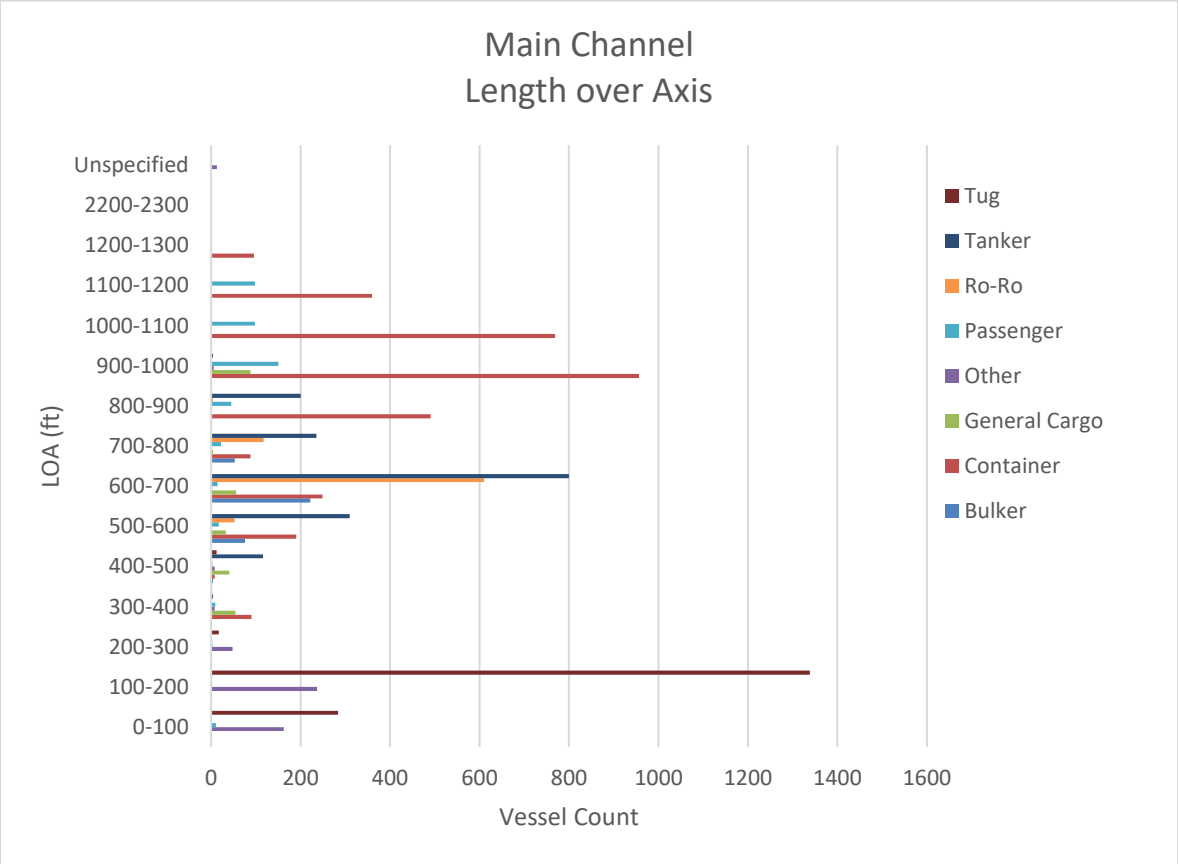

Figure 4-3: Vessel Length Statistics - Ambrose Channel (Tug statistics omitted for clarity)

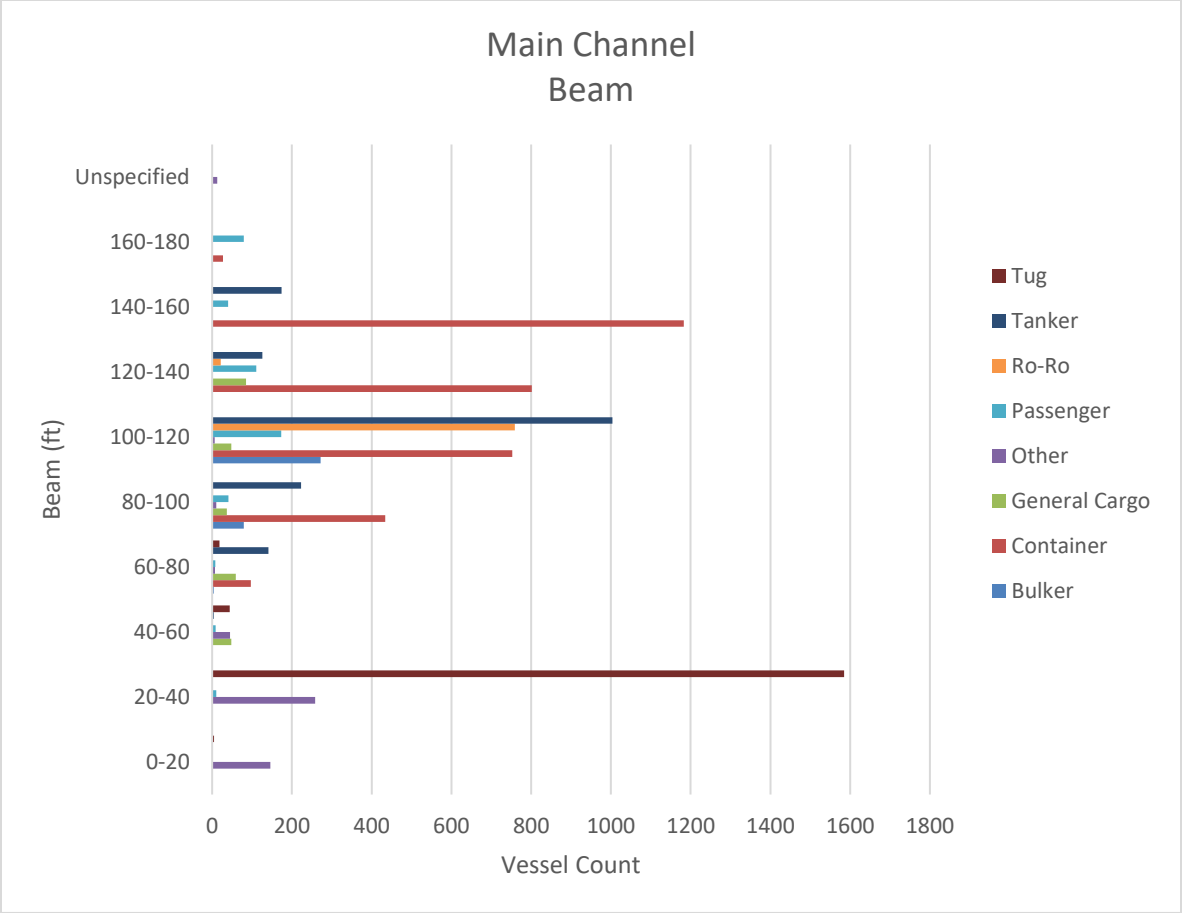

Figure 4-4: Vessel Beam Statistics - Ambrose Channel (Tug statistics omitted for clarity)

4.2.2 Minimum Practical Channel Width

As for Verrazano Narrows, the design vessel for the Ambrose Channel was identified as a large container ship, representing larger vessels that could potentially call in New York during the lifetime of the storm surge barrier project. The design vessel is summarized as follows:

- Design Vessel Name: OOCL Hong Kong,
- Container Capacity: 21,413 TEU,
- Length: 400m (LOA),
- Beam: 58.8m.

Using this design vessel, and in light of the discussion provided for the main passage for Verrazano Narrows, the channel width analysis (Table 4-6) provided shows the guidance based on PIANC. The Channel width is very similar for both Ambrose Channel and Verrazano Narrows. Based on both USACE and PIANC channel width analyses, it seems appropriate to recommend 1,000 ft as a preliminary navigable passage width for one-way and 1,500 ft as a preliminary navigable passage width for two-way traffic. While it is not possible at this point to determine definitively whether two-way traffic is required (or what size vessels would be used for the design of a two-way gate width), a gate width of 1,500ft seems conservatively appropriate for use as preliminary estimate in the overall storm surge barrier feasibility study.

Table 4-6: PIANC Channel Width Calculation for Ambrose Channel (Main Navigable Passage)

Parameter	Specification	Factor [x Beam] One-way traffic	Factor [x Beam] Two-way traffic
Basic Maneuvering Lane	Good	1.3	2.6
Location	Outer Channel (Open Water)	0	0
Vessel Speed	Fast (>= 12 kts)	0.1	0.2
Prevailing Cross-wind	Moderate (<33 kts)	0.3	0.6
Additional Factor for High Wind Area	(based on vessel type)	0	0
Prevailing Cross-current	Negligible (<0.2 kts)	0	0
Prevailing Longitudinal Current	Strong (>=3 kts)	0.1	0.2
Wave Effects	Moderate (Hs>1m)	0.5	1
AtoN	Excellent	0	0
Bottom Surface	Rough and Hard	0.2	0.4
Depth of Waterway, h	h < 1.25T	0.2	0.4
Bank Allowance, Red Side (left)	Steep and Hard Embankments/ Structures	1.3	1.3
Bank Allowance, Green Side (right)	Steep and Hard Embankments/ Structures	1.3	1.3
Total Channel Width [x Beam]		5.3	8.0

Total Channel Width [ft]	1026	1543

4.3 Rockaway Inlet

4.3.1 Traffic Summary

Table 4-7, Table 4-8, Figure 4-5, and Figure 4-6 summarize the vessel traffic passing in and out of the harbor north of the Ambrose Channel. This includes traffic in and near Rockaway Inlet channel. Table 4-7 and Figure 4-5 present the data categorized by vessel length. Table 4-8 and Figure 4-6 present the data categorized by vessel beam.

Table 4-7: Vessel Length Statistics – Rockaway Inlet

LOA [m]	Container	Other	Passenger	Tanker	Tug	Total
0-50		690	3		67	760
50-100		3		1		4
100-150		1	1			2
Unspecified		7				7
Total		**700**	**4**	**1**	**67**	**772**

Table 4-8: Vessel Beam Statistics – Rockaway Inlet

Beam [m]	Container	Other	Passenger	Tanker	Tug	Total
0-10		663	2		66	731
10-20		30	2		1	33
20-30				1		1
Unspecified		7				7
Total		**700**	**4**	**1**	**67**	**772**

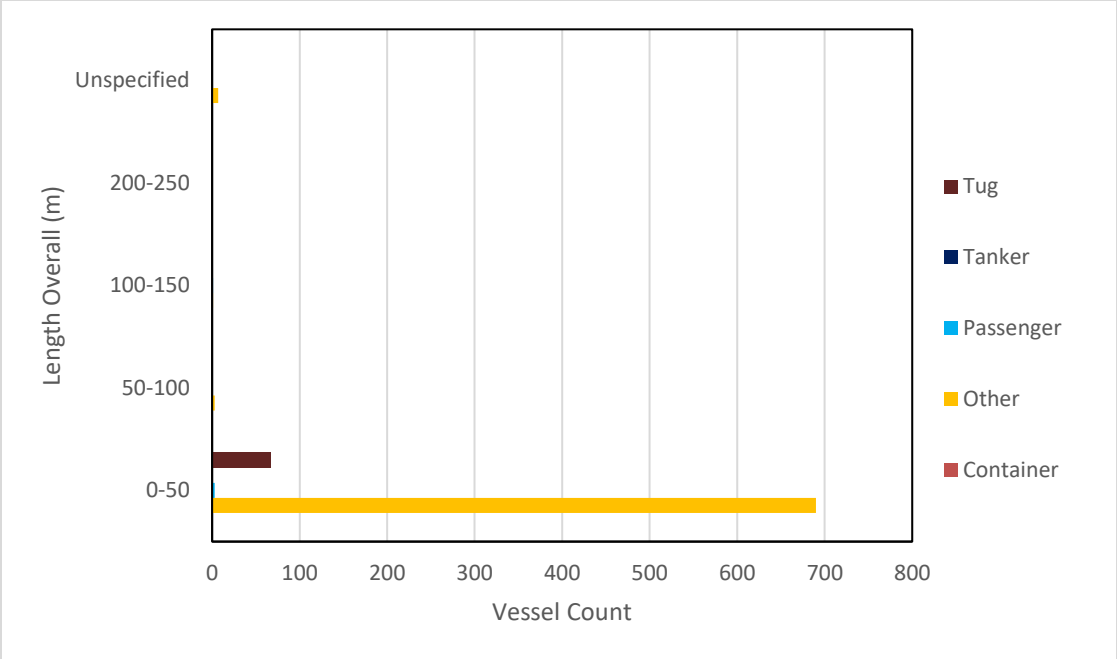

Figure 4-5: Vessel Length Statistics – Rockaway Inlet

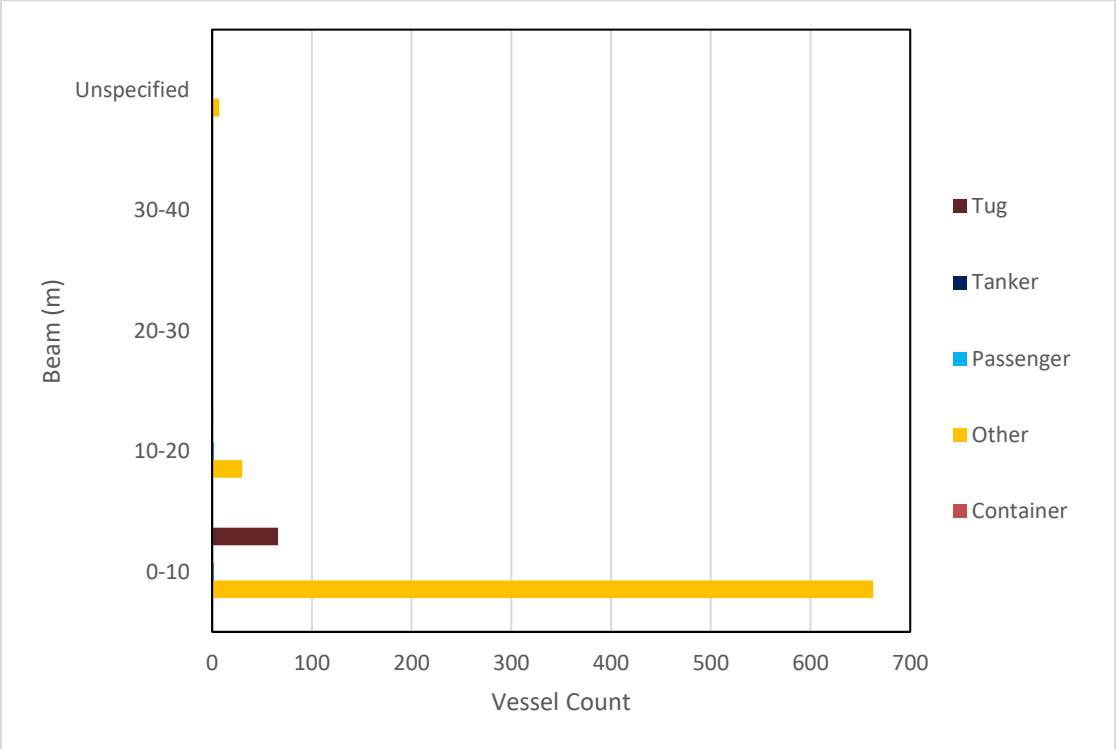

Figure 4-6: Vessel Beam Statistics – Rockaway Inlet

4.3.2 Minimum Practical Channel Width

Based on the traffic data presented above, a representative design vessel was identified for the Rockaway Channel storm surge barrier gate. The design vessel is as follows:

- Design Vessel Name: American Princess,
- Length: 45m (LOA),
- Beam: 11m.

Using this vessel, the PIANC method indicates a channel width of 198 ft (Table 4-9). Based on this, it seems reasonable to suggest a minimum practical navigable passage width of 200 ft for this location.

Table 4-9: PIANC Channel Width Calculation for Rockaway Inlet Channel (Navigable Passage)

Parameter	Specification	Factor [x Beam] One-way Traffic
Basic Maneuvering Lane	Good	1.3
Location	Outer Channel (Open Water)	0
Vessel Speed	Fast (>= 12 kts)	0.1
Prevailing Cross-wind	Moderate (<33 kts)	0.3
Additional Factor for High Wind Area	(based on vessel type)	0.2
Prevailing Cross-current	Negligible (<0.2 kts)	0
Prevailing Longitudinal Current	Strong (>=3 kts)	0.1
Wave Effects	Moderate (Hs>1m)	0.5
AtoN	Excellent	0
Bottom Surface	Rough and Hard	0.2
Depth of Waterway, h	h < 1.25T	0.2
Bank Allowance, Red Side (left)	Steep and Hard Embankments/ Structures	1.3
Bank Allowance, Green Side (right)	Steep and Hard Embankments/ Structures	1.3
Total Channel Width [x Beam]		5.5
Total Channel Width [ft]		198

5 THROGS NECK

5.1 Traffic Summary

Traffic statistics were prepared for Throgs Neck based on part-year data from 2017 and 2018, representing a total of 329 days of data. Table 5-1, Table 5-2, Figure 5-1, and Figure 5-2 summarize the vessel traffic passing Throgs Neck. Table 5-1 and Figure 5-1 present the data categorized by vessel length. Table 5-2 and Figure 5-2 present the data categorized by vessel beam.

Table 5-1: Vessel Length Statistics - Throgs Neck

LOA [ft]	Bulker	General Cargo	Other	Passenger	Tanker	Tug	Total
0-100			259	18		1449	1726
100-200			66	13	1	2380	2460
200-300			10	62		8	80
300-400			1	5		53	59
400-500						24	24
500-600	1	1			1		3
Unspecified			29	29		2	60
Total	1	1	365	127	2	3916	4412

Table 5-2: Vessel Beam Statistics - Throgs Neck

Beam [ft]	Bulker	General Cargo	Other	Passenger	Tanker	Tug	Total
0-20			99	1		145	245
20-40			173	28		3688	3889
40-60			63	6	1	3	73
60-80		1	1	3		78	83
80-100	1			60			61
100-120					1		1
Unspecified			29	29		2	60
Total	**1**	**1**	**365**	**127**	**2**	**3916**	**4412**

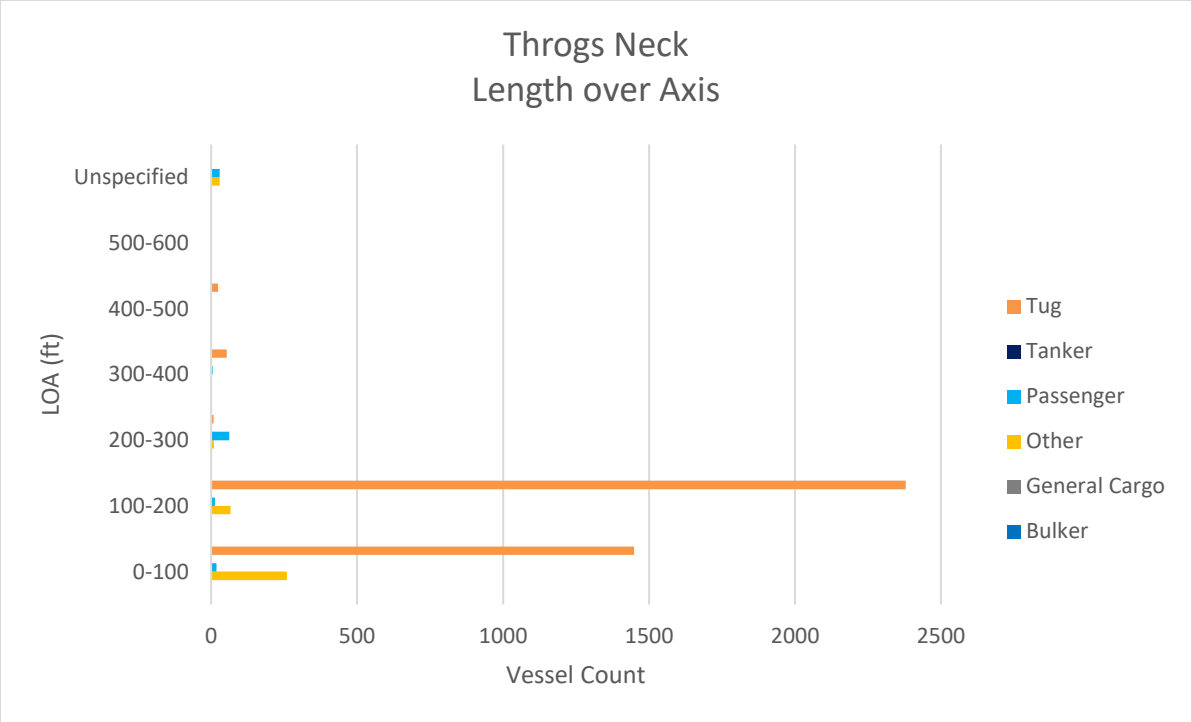

Figure 5-1: Vessel Length Statistics - Throgs Neck (Tug statistics omitted for clarity)

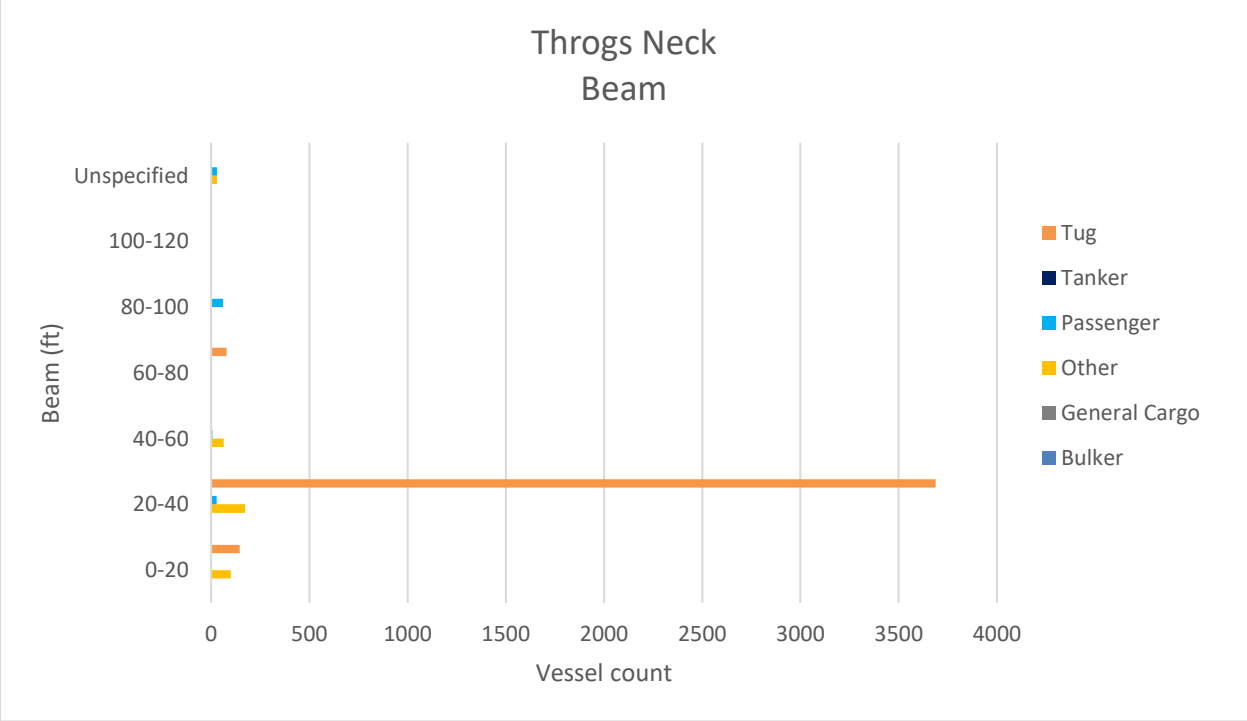

Figure 5-2: Vessel Beam Statistics - Throgs Neck (Tug statistics omitted for clarity)

5.2 Minimum Practical Channel Width

Based on the traffic data, the majority of the traffic that is reporting AIS data is tug boats. AIS data does not record information about what size barges each tug is towing, which makes it difficult to use AIS data to evaluate barge traffic. There was one larger tanker (beam of 30.6m) that passed during the period of AIS data collection. This tanker was used for the initial channel width estimate, but with the understanding that this particular vessel size infrequently navigate this waterway and most traffic would be classified by narrower beams. The design vessel is as follows:

- Design Vessel Name: Asphalt Splendor
- Length: 179.9m (LOA), (590ft)
- Beam 30.6m. (100ft)

Based on this vessel and its frequency the PIANC method for one-way traffic is used and indicates a design channel width of 478 ft. It is assumed that this represents a conservative estimate of the required navigable width, since vessels of this size do not pass frequently. If the gate width was slightly more restricting, this vessel could still pass, but perhaps would require a slower speed or greater tug assistance. In addition the majority of the vessels has a beam width of less than 16m (52ft). If passing of vessels of such dimensions is assumed to be maintained within the navigable passage of the storm surge barrier, then a width of 447ft is suggested based on PIANC calculations (see Table 5-3).

Based on the traffic data and typical PIANC channel widths, it seems reasonable to recommend a navigable passage width of 450 ft for the Throgs Neck storm surge barrier.

Table 5-3: PIANC Channel Width Calculation for Throgs Neck (Navigable Passage)

Parameter	Specification	Factor [x Beam] One-way Traffic	Factor [x Beam] Two-way Traffic
Basic Maneuvering Lane	Good	1.3	2.6
Location	Inner Channel (Protected Water)	0	0
Vessel Speed	Slow (>=5 kts)	0	0
Prevailing Cross-wind	Moderate (<33 kts)	0.6	1.2
Additional Factor for High Wind Area	(based on vessel type)	0	0
Prevailing Cross-current	Negligible (<0.2 kts)	0	0
Prevailing Longitudinal Current	Strong (>=3 kts)	0.4	0.8
Wave Effects	Mild (Hs<=1m)	0	0
AtoN	Good	0.2	0.4
Bottom Surface	Rough and Hard	0.2	0.4
Depth of Waterway, h	h < 1.25T	0.4	0.8
Bank Allowance, Red Side (left)	Steep and Hard Embankments/ Structures	0.5	0.5

Bank Allowance, Green Side (right)	Steep and Hard Embankments/ Structures	0.5	0.5
Additional Width Due to Response Time in Turning	Turning	0.4	0.8
Additional Width Due to Drift Angle	(based on vessel type & characteristics)	0.3	0.6
Total Channel Width [x Beam]		4.8	8.6
Total Channel Width [ft] (Design Beam 100ft)		480	
Total Channel Width [ft] (Design Beam 52ft)			447

6 ARTHUR KILL

6.1 Traffic Summary

Traffic statistics were prepared for Arthur Kill based on part-year data from 2017 and 2018, representing a total of 329 days of data. Table 6-1, Table 6-2, Figure 6-1, and Figure 6-2 summarize the vessel traffic through Arthur Kill. Table 6-1 and Figure 6-1 present the data categorized by vessel length. Table 6-2 and Figure 6-2 present the data categorized by vessel beam.

Table 6-1: Vessel Length Statistics - Arthur Kill

LOA [ft]	Other	Passenger	Tanker	Tug	Total
0-100	1	1		18	20
100-200		1		63	64
200-300	2				2
400-500			5		5
500-600			34		34
600-700			122		122
700-800			26		26
800-900			10		10
Total	3	2	197	81	283

Table 6-2: Vessel Beam Statistics - Arthur Kill

Beam [ft]	Other	Passenger	Tanker	Tug	Total
0-20	1			1	2
20-40		2		79	81
40-60	2			1	3
60-80			6		6
80-100			19		19
100-120			158		158
120-140			5		5
140-160			9		9
Total	3	2	197	81	283

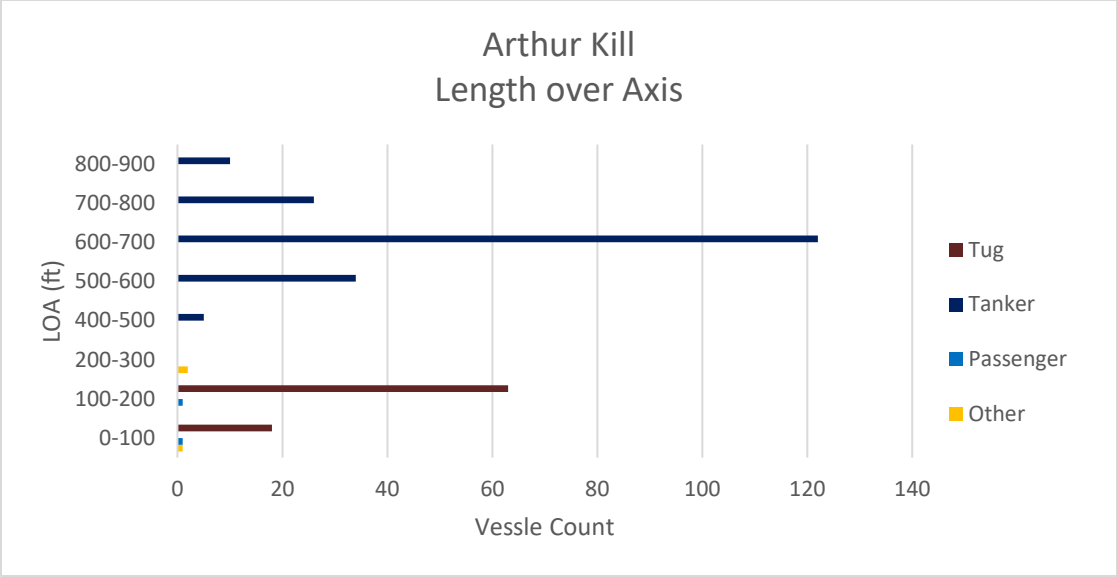

Figure 6-1: Vessel Length Statistics - Arthur Kill

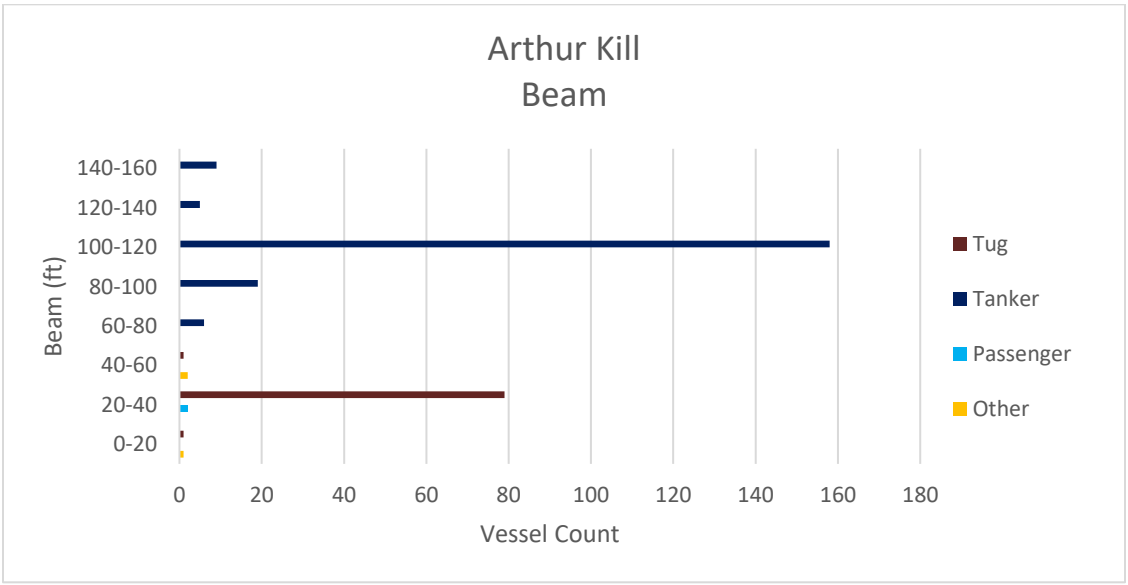

Figure 6-2: Vessel Beam Statistics - Arthur Kill

6.2 Minimum Practical Channel Width

Based on the traffic data presented above, a design vessel was identified for evaluating an appropriate width for the Arthur Kill storm surge barrier navigable passage. The design vessel identified is summarized as follows:

- Design Vessel Name: Australian Spirit (Tanker),
- Length: 256m (LOA),
- Beam: 44.8m.

Using this design vessel, the USACE channel width calculation would suggest a width of approximately 4 times the vessel beam (or 587 ft). Using the PIANC method (Table 6-3), a navigable passage width of 661 ft was computed.

Noting that the Arthur Kill channel varies in width along its length, with a typical width of 600 ft at the location of the proposed storm surge barrier, it seems appropriate to recommend the following:

- The Arthur Kill barrier gate should not be narrower than the width of the channel at the location selected for the barrier. Simulations studies at a further design phase would indicate whether there would be value in making the gate wider than the channel to provide additional maneuvering area for assisting tugs.
- The Arthur Kill barrier gate should not be less than 600 ft wide.

For the purposes of the feasibility study, a navigable passage width of 600 ft is suggested

Table 6-3: PIANC Channel Width Calculation for Arthur Kill (Navigable Passage)

Parameter	Specification	Factor [x Beam]
Basic Maneuvering Lane	Good	1.3
Location	Inner Channel (Protected Water)	0
Vessel Speed	Moderate (>=8 kts)	0
Prevailing Cross-wind	Moderate (<33 kts)	0.4
Additional Factor for High Wind Area	(based on vessel type)	0
Prevailing Cross-current	Negligible (<0.2 kts)	0
Prevailing Longitudinal Current	Strong (>=3 kts)	0.2
Wave Effects	Mild (Hs<=1m)	0
AtoN	Excellent	0
Bottom Surface	Rough and Hard	0.2
Depth of Waterway, h	h < 1.25T	0.4
Bank Allowance, Red Side (left)	Steep and Hard Embankments/ Structures	1
Bank Allowance, Green Side (right)	Steep and Hard Embankments/ Structures	1
Total Channel Width [x Beam]		4.5
Total Channel Width [ft]		661

7 KILL VAN KULL

7.1 Traffic Summary

Traffic statistics were prepared for Kill van Kull based on part-year data from 2017 and 2018, representing a total of 332 days of data. Table 7-1, Table 7-2, Figure 7-1, and Figure 7-2 summarize the vessel traffic through Kill van Kull. Table 7-1 and Figure 7-1 present the data categorized by vessel length. Table 7-2 and Figure 7-2 present the data categorized by vessel beam.

Table 7-1: Vessel Length Statistics - Kill Van Kull

LOA [ft]	Bulker	Container	General Cargo	Other	Passenger	Ro-Ro	Tanker	Tug	Total
0-100				1671	20		473	21646	23810
100-200				403	11		1	22056	22471
200-300				344			341	305	990
300-400		94	4	4	8		183	390	683
400-500			7	3			93	155	258
500-600	23	148	16			51	217		455
600-700	145	226	50			542	471	2	1436
700-800	44	69	4			123	106		346
800-900		515	2			4	122		643
900-1000		939	90				1		1030
1000-1100		687							687
1100-1200		282							282
1200-1300		92							92
Unspecified				691				359	1050
Total	212	3052	173	3116	39	720	2008	44913	54233

Table 7-2: Vessel Beam Statistics - Kill Van Kull

Beam [ft]	Bulker	Container	General Cargo	Other	Passenger	Ro-Ro	Tanker	Tug	Total
0-20				742	9			1248	1999
20-40				939	13		473	42338	43763
40-60		2	1	870	9		184	418	1484
60-80		96	12	173	8		441	546	1276
80-100	23	352	25	4		2	162	1	569
100-120	189	810	47	2		700	572	2	2322
120-140		741	88			18	64		911
140-160		1029					112	1	1142
160-180		22							22

Unspecified				386				359	745
Total	212	3052	173	3116	39	720	2008	44913	54233

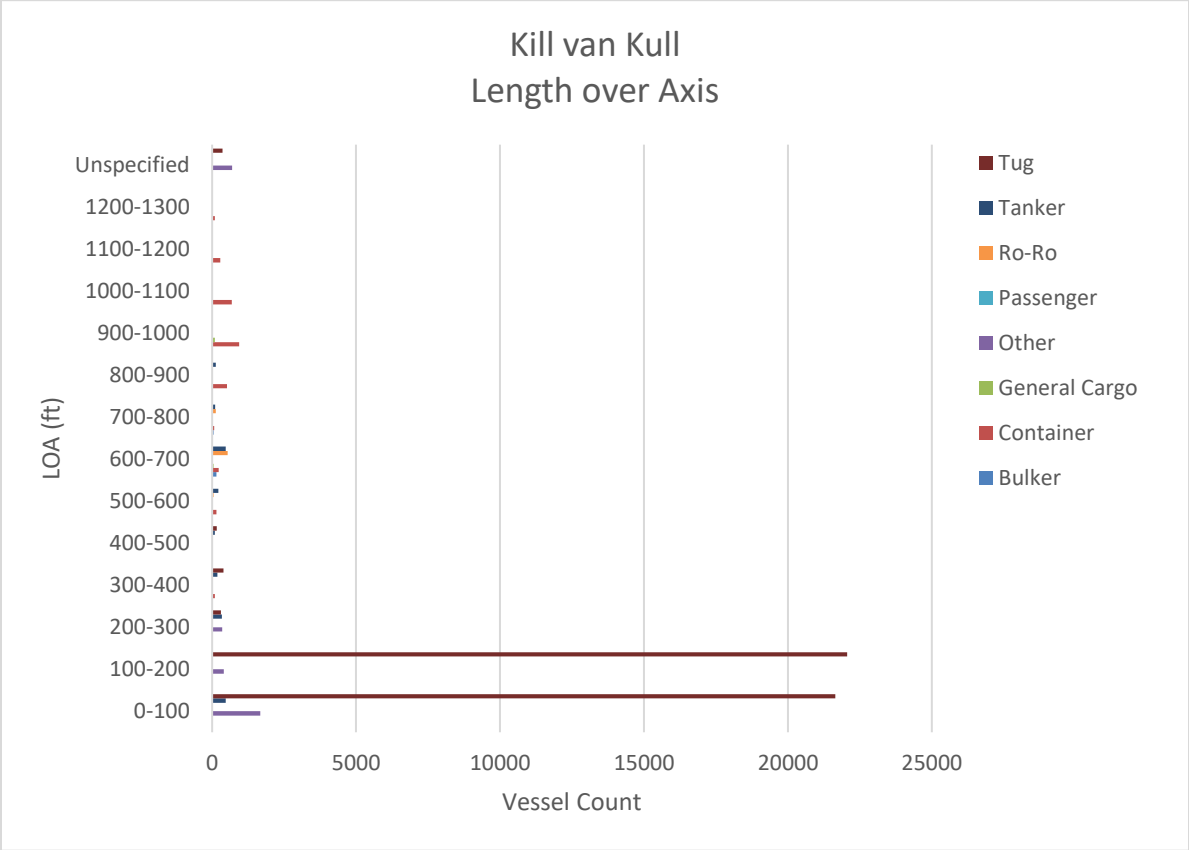

Figure 7-1: Vessel Length Statistics - Kill Van Kull (Tug statistics omitted for clarity)

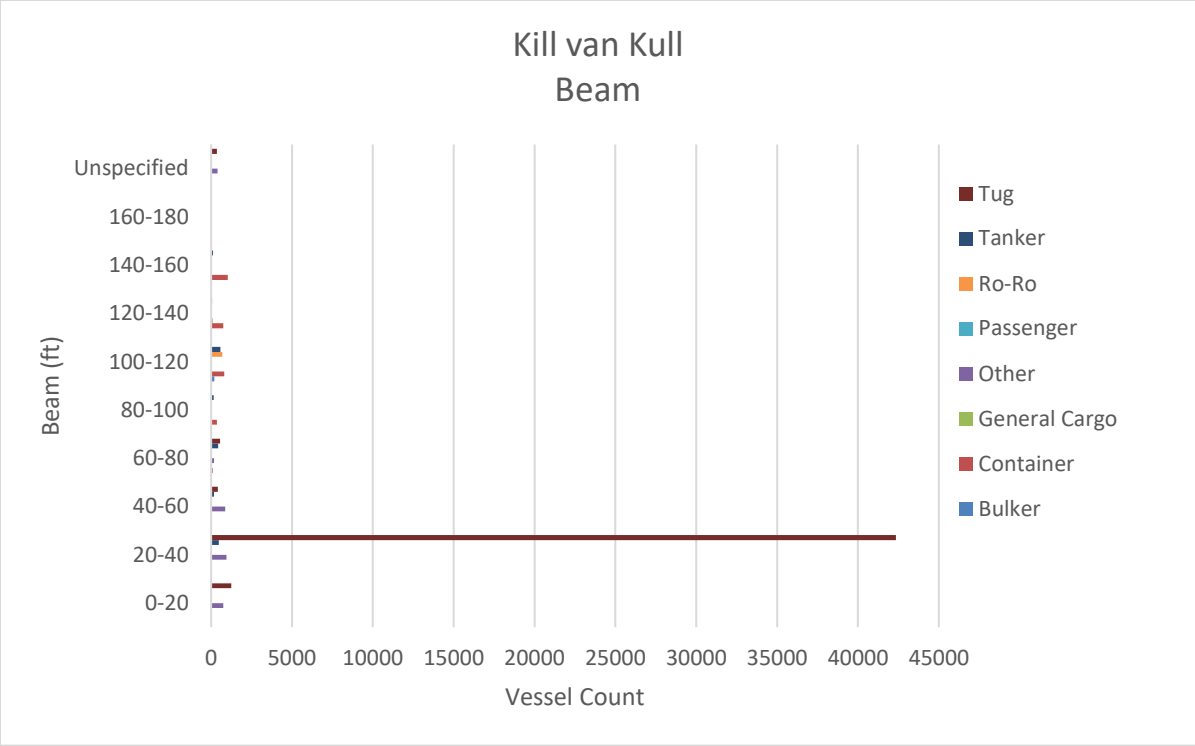

Figure 7-2: Vessel Beam Statistics - Kill Van Kull (Tug statistics omitted for clarity)

7.2 Minimum Practical Channel Width (Navigable Passage)

Similar to Verrazano Narrows and the Ambrose Channel, Kill van Kull is on the main navigation channel used by the largest container vessels entering the harbor. As such, the vessel statistics presented above present a partial perspective, but the design vessel used for the channel width calculation is representative of larger vessels that may call sometime in the future. The design vessel for Kill van Kull is as follows:

- Design Vessel Name: OOCL Hong Kong,
- Container Capacity: 21,413 TEU,
- Length: 400m (LOA), (1312ft)
- Beam: 58.8m. (193ft)

Using this vessel and the fact that this reach currently is used as a one-way traffic lane for the largest vessels, both the USACE and PIANC methods were applied to estimate an appropriate minimum channel width. The USACE method does not provide different estimate for this location as compared to the estimates for Verrazano Narrows and Ambrose Channel (771-1,061 ft). However, the PIANC method is able to account for the slower speed of the vessel in this reach, reducing the required width to 856 ft.

These channel width calculations confirm that the current Authorized channel width of 800 ft is appropriate for the minimum practical navigable passage through the storm surge barrier for Kill van Kull.

Table 7-3: PIANC Channel Width Calculation for Kill van Kull (Navigable Passage)

Parameter	Specification	Factor [x Beam] One-Way Traffic
Basic Maneuvering Lane	Good	1.3
Location	Inner Channel (Protected Water)	0
Vessel Speed	Slow (>=5 kts)	0
Prevailing Cross-wind	Moderate (<33 kts)	0.6
Additional Factor for High Wind Area	(based on vessel type)	0
Prevailing Cross-current	Negligible (<0.2 kts)	0
Prevailing Longitudinal Current	Strong (>=3 kts)	0.4
Wave Effects	Mild (Hs<=1m)	0
AtoN	Excellent	0
Bottom Surface	Rough and Hard	0.2
Depth of Waterway, h	h < 1.25T	0.4
Bank Allowance, Red Side (left)	Steep and Hard Embankments/ Structures	0.5
Bank Allowance, Green Side (right)	Steep and Hard Embankments/ Structures	0.5
Additional Width Due to Response Time in Turning	Not Turning	0.4
Additional Width Due to Drift Angle	(based on vessel type & characteristics)	0.1
Total Channel Width [x Beam]		4.4
Total Channel Width [ft]		856

8 JAMAICA BAY

8.1 Traffic Summary

For Jamaica Bay no AIS data analysis was completed, but instead it was assumed that the traffic as presented for the Rockaway Inlet (see section 4.3) would be representative and that a navigable opening within the Jamaica Bay Barrier would at a minimum need to be equal to the Rockaway Inlet opening.

8.2 Minimum Practical Channel Width

Using the data provided and based on USACE 2016 it is assumed that the majority of the commercial traffic is barge traffic with a vessel beam of 50ft. It is noted that there is a significant portion of recreational traffic within this area (there are over 50 private marinas located around the Bay providing boat slips and launch ramps – DEP Jamaica Bay Watershed protection plan). The estimate for the navigable passage width is set 200ft for this location. This estimate is based on the preliminary design provided in USACE 2016. It is further assumed that a 200ft navigable opening will be able to accommodate safe passage of vessels, and it is equal to the width of the Rockaway Inlet navigable passage and would thereby accommodate all traffic coming through that reach.

9 NEWTOWN CREEK

9.1 Traffic Summary

For Newtown Creek no extensive AIS data record was available. For the location where the navigable storm surge barrier is considered the authorized channel is authorized as a channel 23ft deep (MLLW datum) and 130ft wide. Maritime traffic was studied for a site specific study under the direction of New York City Economic and Development Corporation (EDC) and Mayor's Office of Recovery and Resiliency (ORR). A portion of the study report "Newtown Creek Storm Surge Barrier Study" is included here verbatim.

Newtown Creek is an active component of New York Harbor that has maintained a high level of commercial activity throughout the history of the port. The Creek is a narrow tidal extension of the East River and, in some locations, also serves as a boundary between the boroughs of Brooklyn and Queens. The surrounding area is high industrialized and the tributary offers reliable marine transportation access to the various commercial and industrial firms, which line its banks.

The existing federal navigation project provides for a channel 23 feet deep and 130 feet wide from East River to 150 feet north of Maspeth Avenue, with a triangular area at the north side of the entrance. A turning basin 23 feet deep at Mussel Island becomes shallower and narrower approaching Metropolitan Avenue. A survey conducted on behalf of the USACE in 2009 reveals that while the federal channel depth of 23 feet has been identified, shoaling, and sedimentation has reduced the controlling depth to approximately 17 feet on the western portion and approach channels of the Creek.

The cargo passing through the tributary is predominantly via tug and barge, however, most businesses operate barges on seasonal rather than weekly schedules as a way to supplement regular truck and pipeline shipments. In 2005, the Federal Highway Administration conducted a navigation analysis of Newtown Creek as part of the Kosciuszko Bridge Project. The report includes a review of current vessel traffic through the channel as well as an overview of vessel sizes and types. Based on a review of the report, typical vessel types and sizes are given below.

TABLE 3-5
Newtown Creek Vessel Characteristics

	Air Draft (feet)	Water Draft (feet)	Length (feet)	Beam (feet)
Barge	10 to 35	5 to 15	50 to 300	25 to 60
Tugboat	15 to 50	6 to 13	35 to 100	15 to 35

Source: FHA, 2005.

In addition to frequent commercial barge use, the NYCDEP operates a fleet of sludge vessels that call at the Newtown Creek WWTP. A fleet of three new vessels has recently come into operation. The vessels typically make one call at the WWTP per day, though in summer this can increase to twice daily as sludge production goes up. The WWTP does have 24 hours sludge holding capacity in the event that vessels are unable to call, such as in storm conditions. It is expected that longer term improvements at the plant will eventually lead to a higher percentage of solids in the sludge and, hence, a decrease in vessel frequency.

The new fleet of vessels is designed to navigate under the Pulaski Bridge, without the need for the bridge to open. The barges are self-propelled. Key characteristics are summarized in Table 3-6. Typically the vessels operate in at least 17.5 to 18.5 feet of water for safe navigation. The new vessels have an increased beam to accommodate the air draft restrictions without reducing capacity. The wider beam means that vessels are marginally less maneuverable.

TABLE 3-6
NYCDEP Vessel HUNTS POINT Vessel Characteristics (newer vessel)

Air draft (feet)	43 (mast down)
Length (feet)	290
Beam (feet)	70
Draft (feet)	14.5

TABLE 3-7
NYCDEP Vessel RED HOOK Vessel Characteristics (older vessel—no longer in service on Newtown)

Air draft (feet)	63 (mast down)
Length (feet)	360
Beam (feet)	53
Draft (feet)	21

One-way traffic operations are in effect on Newtown Creek and vessels rely on AIS data to identify and communicate with other vessels.

The minimum preferred navigation channel width is 150 feet as defined by the width at Pulaski Bridge. The navigation channel narrows between 2nd Street and Vernon Boulevard. An additional restriction to navigation—such as a barrier—would preferably be located on a straight section of channel to allow vessel traffic to approach as perpendicular as possible, particularly given the reduced maneuverability of the wider barges, so as to allow adequate forward visibility of approaching vessels.

The Kosciuszko Bridge on the Brooklyn Queens Expressway has a fixed height clearance of 125 feet.

9.2 Minimum Practical Navigable Passage Dimensions

In the absence of more detailed information regarding the vessel sizes that transit this area, it seems reasonable to base the minimum dimension of the navigable passage based on previous studies. The minimum practical channel dimensions are based on the recommendations for the recommended barrier alternative "2nd Street" in CH2MHILL 2016:

- The sill level will be set at a depth of -20ft as to lie flush with the existing bed level
- Set the gate span conservatively equal to the span of the Pulaski Bridge (150ft to 170ft), however to minimize impacts to water quality tidal exchange and flushing requirements could require a wider gate opening (or additional gates)

10 GOWANUS CANAL

10.1 Traffic Summary

For Gowanus Canal no extensive AIS data record was available. The authorized channel terminates at the Hamilton Avenue bridge and at this upper limit is authorized as a channel 18ft deep (MLLW datum) and 100ft wide. Maritime traffic was studied for a site-specific study under the direction of New York City Economic and Development Corporation (EDC) and Mayor's Office of Recovery and Resiliency (ORR). A portion of the study report "Gowanus Canal Storm Surge Barrier Study" (CH2MHill 2016) is included here verbatim.

The Gowanus Canal begins at Butler Street and extends southward approximately 8,500 feet to its mouth with depths ranging from 4 feet to approximately 15 feet at Hamilton Avenue, then rapidly increasing in depth to approximately 30 feet for the rest of the canal heading out to Gowanus Bay.

The canal is approximately 100 feet wide up to Hamilton Avenue where it widens and flows into Gowanus Bay and Upper New York Bay. Present land uses along the Canal consist primarily of manufacturing, industrial, and commercial uses. The canal shorelines are entirely altered consisting almost exclusively of bulkheads with some areas of riprap and piers. Traffic is primarily barges with tug escort. Vessel traffic through the canal is limited with only the federal navigational channel maintained south of Hamilton Avenue. There are four turning basins located north of Hamilton Avenue at 4th Street, 6th Street, 7th Street, and 11th Street. These basins experience limited marine traffic as they are not part of the main navigational channel and are primarily used as a means for vessels to reverse direction during transit.

Due to the relatively shallow water depth, narrow channel width, and numerous air draft restrictions (with moveable bridges closed), vessel traffic is restricted primarily to tugboats and barges. This is especially true for the traffic traveling further inland. Based on a review of waterfront land use, typical vessel characteristics are likely within the ranges shown below.

TABLE 3-5
Indicative Vessel Characteristics

	Air Draft (feet)	Water Draft (feet)	Length (feet)	Beam (feet)
Barge	10-35	3-12	25-150	20-45
Tugboat	20-50	8-20	50-120	20-40

10.2 Minimum Practical Channel Dimension

In the absence of more detailed information regarding the vessel sizes that transit this area, it seems reasonable to base the minimum dimension of the navigable passage based on previous studies. The minimum practical channel dimensions are based on the recommendations for the recommended barrier alternative "Hamilton Avenue" in CH2MHILL 2016:

- The sill level will be set at a depth of -22ft as to lie flush with the post-remediation bed level,
- The width of the navigation opening is to be approximately 90ft so as to provide the same channel cross-sectional area as he Hamilton Avenue bridge immediately upstream, and
- The gate would be aligned with the existing navigation channel and opening through the bridge structure.

11 RECOMMENDATIONS FOR MINIMUM DIMENSIONS OF NAVIGABLE PASSAGES

11.1 Study Synopsis

This report documents a maritime traffic analysis for a number of federal navigation channels that cross the proposed storm surge barrier alignments under the HAT Study. The storm surge barriers will include gated navigable passes that will accommodate navigation and waterborne commerce. Based on the traffic analysis design vessels are selected for each navigable passage and minimum practical dimensions are set for each opening.

It should be noted that this is a preliminary assessment and that the actual passage dimensions could be set larger in certain instances; e.g. if additional flow area is needed based on environmental considerations or if other navigation and non-navigation related design considerations require larger dimensions for the opening. The following sections provide a summary of the preliminary findings.

11.2 Recommendations for Minimum Practical Dimensions of Navigable Passages

11.2.1 Verrazano Narrows

The storm surge barrier at the Verrazano Narrows would at least require two navigable passages:

1. Main Navigable Passage Opening: 1400ft wide

- PIANC guidance to set Navigational Opening: to a minimum of 968ft and 1428ft for one-way and two-way traffic respectively. Rounded to the nearest hundred, 1400ft is deemed conservatively appropriate for use as preliminary estimate in the overall storm surge barrier feasibility study.
- Authorized Channel Depth: -53ft MLLW
- Set sill of Navigable Passage at -55ft MLLW (-58ft NAVD88)
- Air Draft is controlled by the Verrazano Bridge at 228ft. This clearance is adopted as a conservative assumption which in practicality for the design of storm surge barriers translates to unrestricted air clearance.

2. Secondary Navigable Passage on East Side of Main Channel: 200ft wide

- PIANC guidance to set Navigational Opening: minimum 178ft

- Authorized Channel Depth: -40ft MLLW
- Set sill of Navigable Passage at -42ft MLLW (-45ft NAVD88)
- Air Draft: 50ft

11.2.2 Sandy Hook to Breezy Point

The storm surge barrier from Sandy Hook to Breezy Point would at least require three navigable passages:

1. Sandy Hook Navigable Passage Opening: 800ft wide (width of existing channel).
    - PIANC guidance to set Navigational Opening for one-way traffic to a minimum of 835ft, but the Authorized Channel Width is 800ft.
    - Authorized Channel Depth: -35ft MLLW
    - Set sill at -37ft MLLW (+2ft clearance below design channel) (-40ft NAVD88)
    - Air draft: 143ft (assume up to 43.6m for this vessel size class). This clearance is adopted as a conservative assumption which in practicality for the design of storm surge barriers translates to unrestricted air clearance.
2. Ambrose Channel Navigable Passage Opening: 1500ft wide
    - PIANC guidance to set Navigational Opening: to a minimum of 1026ft and 1543ft for one-way and two-way traffic respectively. Rounded to the nearest hundred, 1500ft is deemed conservatively appropriate for use as preliminary estimate in the overall storm surge barrier feasibility study.
    - Authorized Channel Depth: -53ft MLLW
    - Set sill of Navigable Passage at -55ft MLLW (-58ft NAVD88)
    - Air Draft is controlled by the Verrazano Bridge at 228ft. This clearance is adopted as a conservative assumption which in practicality for the design of storm surge barriers translates to unrestricted air clearance.
3. Rockaway Inlet Navigable Passage Opening: 200ft wide
    - PIANC guidance to set Navigational Opening: minimum 198ft
    - Authorized Channel Depth: -20ft MLLW

- Set sill at a minimum at -22ft MLLW (-25ft NAVD88)
- Air Draft: 50ft

11.2.3 Throgs Neck

The storm surge barrier at Throgs Neck would require at least one navigable gate

1. Main Navigable Passage: 450ft wide

    - PIANC guidance to set Navigational Opening: to a minimum of 480ft and 447ft for one-way and two-way traffic respectively. Rounded to the nearest fifty feet, 450ft is deemed conservatively appropriate for use as preliminary estimate in the overall storm surge barrier feasibility study.
    - Authorized Channel Depth: -35ft MLLW
    - Set sill at -37ft MLLW (-40ft NAVD88)
    - Air Draft is controlled by the Throgs Neck Bridge at 142ft. This clearance is adopted as a conservative assumption which in practicality for the design of storm surge barriers translates to unrestricted air clearance.

11.2.4 Arthur Kill

The storm surge barrier at Arthur Kill would require at least 1 navigable gate

1. Navigable Passage Opening: 600ft wide (width of existing channel).

    - PIANC guidance to set Navigational Opening for one-way traffic to a minimum of 661ft, but the Authorized Channel Width is 600ft.
    - Authorized Channel Depth: -35ft MLLW
    - Set sill at -37ft MLLW (-40ft NAVD88)
    - Unrestricted air draft to the Outerbridge Crossing, after that it is controlled by the Outerbridge Crossing at 143ft. This clearance is adopted as a conservative assumption which in practicality for the design of storm surge barriers translates to unrestricted air clearance.

11.2.5 Kill van Kull

The storm surge barrier at the Kill van Kull would require at least 1 navigable gate

1. One navigable gate: 800ft (width of existing channel).

    - PIANC guidance to set Navigational Opening for one-way traffic to a minimum of 856ft, but the Authorized Channel Width is 800ft.
    - Authorized Channel Depth: -50ft MLLW
    - Set sill at -52ft MLLW (-55ft NAVD88)
    - Air Draft is controlled by the Verrazano Bridge at 228ft. This clearance is adopted as a conservative assumption which in practicality for the design of storm surge barriers translates to unrestricted air clearance.

11.2.6 Jamaica Bay Barrier

The storm surge barrier at the entrance of Jamaica Bay would require at least 1 navigable gate

1. Main Navigable Passage: 200ft wide

    - PIANC guidance to set Navigational Opening: minimum 198ft
    - Authorized Channel Depth: -20ft MLLW
    - Set sill at a minimum at -22ft MLLW (-25ft NAVD88)
    - Air Draft is controlled by the Gil Hodges Bridge at 152ft. This clearance is adopted as a conservative assumption which in practicality for the design of storm surge barriers translates to unrestricted air clearance

11.2.7 Newtown Creek Barrier

Newtown Creek would include one navigable gate with the following minimum dimensions for the navigable passage:

- Navigable Passage: 170ft wide
- Authorized Channel Depth: -23ft MLLW
- Set sill at a minimum at -22ft MLLW (-25ft NAVD88)
- The Kosciuszko Bridge on the Brooklyn Queens Expressway has a fixed height clearance of 125 feet. This clearance is adopted as a conservative assumption which in practicality translates to unrestricted air clearance.

11.2.8 Gowanus Canal Barrier

The Gowanus Canal Barrier at the Hamilton Avenue location would include one navigable gate with the following minimum dimensions for the navigable passage:

- Navigable Passage: 90 ft wide
- Authorized Channel Depth: -18ft MLLW
- The sill level will be set at a depth of -22ft NAVD88 as to lie flush with the post-remediation bed level
- There are no fixed height bridges downstream of the proposed barrier location and therefore unrestricted air clearance is conservatively assumed.

11.3 Recommendations for Further Study

After completion of the preliminary geometric configuration of the storm surge barriers and the evaluation of their hydrodynamic performance through numerical modeling, the modeling results should be analyzed to assess the potential for increased current velocities through the navigable passages and the potential for impacts to navigability. This report may subsequently be updated to reflect such findings and requirements for minimum navigable pass widths.

At later stages of the study a detailed navigation assessment will need to be completed. Such a study should at a minimum include:

- One-way versus two-way vessel passage, including meeting, passing, and overtaking
- Number, frequency, and intensity of vessel passage
- Vessel wait areas, queuing, and wait times
- Cross-traffic conditions
- Barrier positioning and fairway lengths for maneuvering
- Currents, cross currents, wind, tides, surge, weather, night, visibility, and other environmental considerations for vessel passage
- Trends for future vessel traffic, including vessel size, frequency
- Requirements for vessel traffic service, including advisory / control / restrictions on navigation
- Requirements for Aids-to-Navigation, guide structures, and protective structures
- Navigation evaluations, including pilot and navigation industry input, and real-time simulations to assess, amongst others:
    - Flow and cross-current considerations
    - Gate approach and departure
    - Passing vessel assessments

- Environmental considerations, including water quality impacts, ecological issues
- Project cost evaluations

The study documented herein is a preliminary assessment of maritime traffic and, except for the Ambrose Channel, did not include the identification of long term trends in traffic volume and tonnage that could affect future project impacts. In addition to the current fleet, assessing the composition of the future commercial fleet will identify and characterize the potential type and frequency of vessels using the various navigation channels.

REFERENCES:

CH2MHill 2016 a, Gowanus Canal Storm Surge Barrier Study – Final Report, New York City Economic and Development Corporation and Mayor's Office of Recovery and Resiliency. CH2MHill. January 15, 2016

CH2MHill 2016 b, Newtown Creek Storm Surge Barrier Study – Final Report, New York City Economic and Development Corporation and Mayor's Office of Recovery and Resiliency. CH2MHill. January 15, 2016

USACE Navigation Data Center http://www.navigationdatacenter.us/

USACE 2016, Atlantic Coast of New York, East Rockaway Inlet to Rockaway Inlet and Jamaica Bay, Draft Hurricane Sandy General Reevaluation Report and Environmental Impact Statement, USACE August 2016

A. APPENDIX A: PORT OF NEW YORK AND NEW JERSEY VESSEL SIZE OUTLOOK

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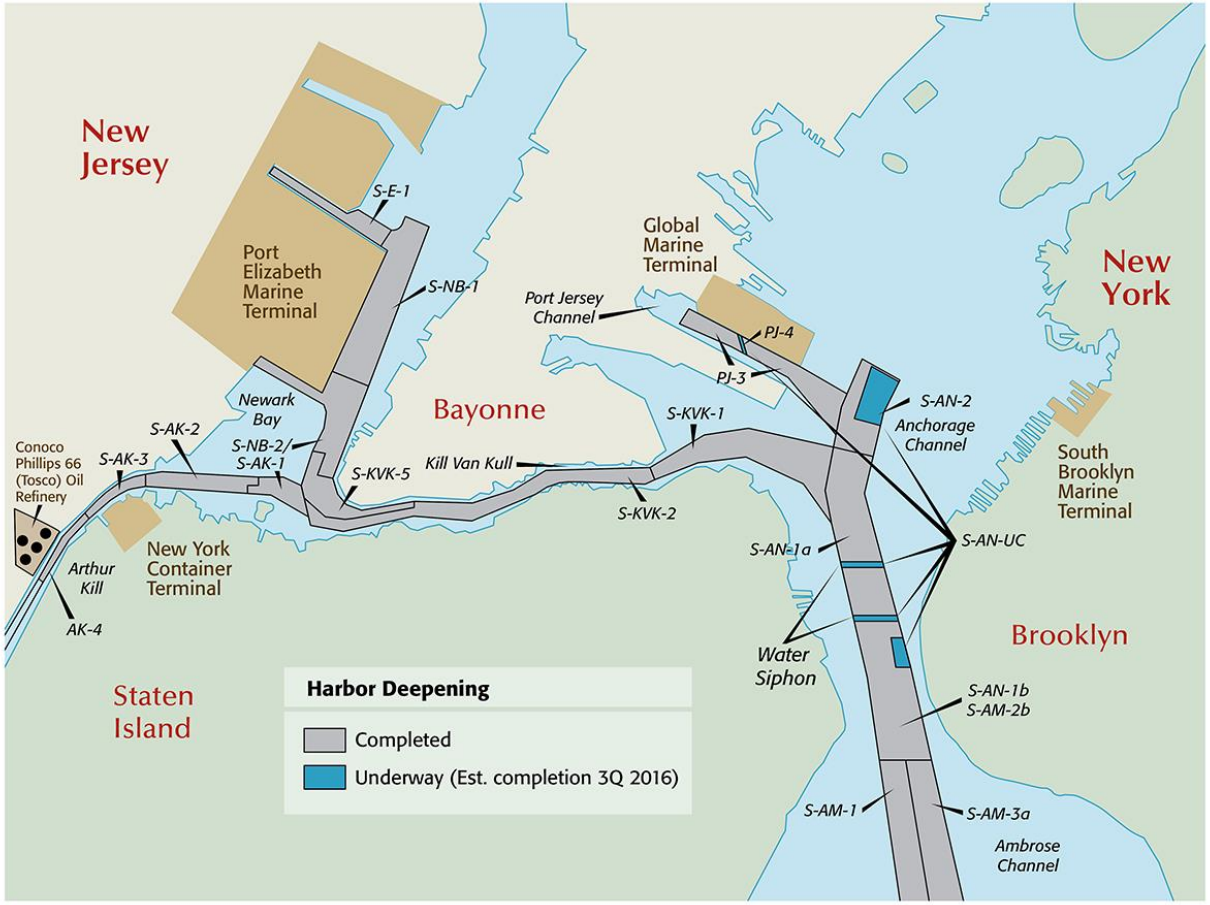
This study is qualified in its entirety by, and should be considered in light of, these limitations, conditions and considerations.

The main container terminals at the Port of New York and New Jersey can be accessed under three bridges, two of which have sufficient air draft for at least the 18,000 TEU container vessels.



Source : Google Earth / MN

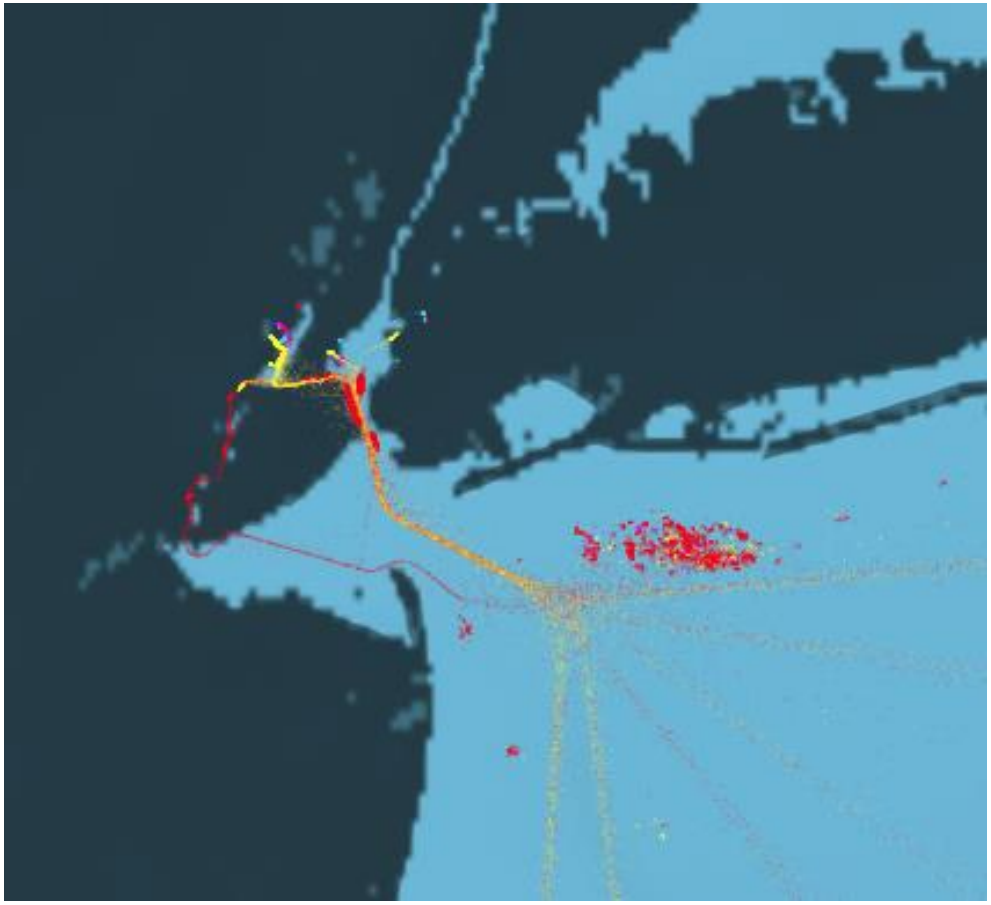
The Harbor Deepening Project to 50' is complete. Navigation depths in the southern segment of the Arthur Kill, through the Outerbridge crossing generally remains at 35' that is not sufficient for the largest container vessels.



Source : https://www.panynj.gov/photo/port/Harbor-Deepening-Map-2016-full.jpg

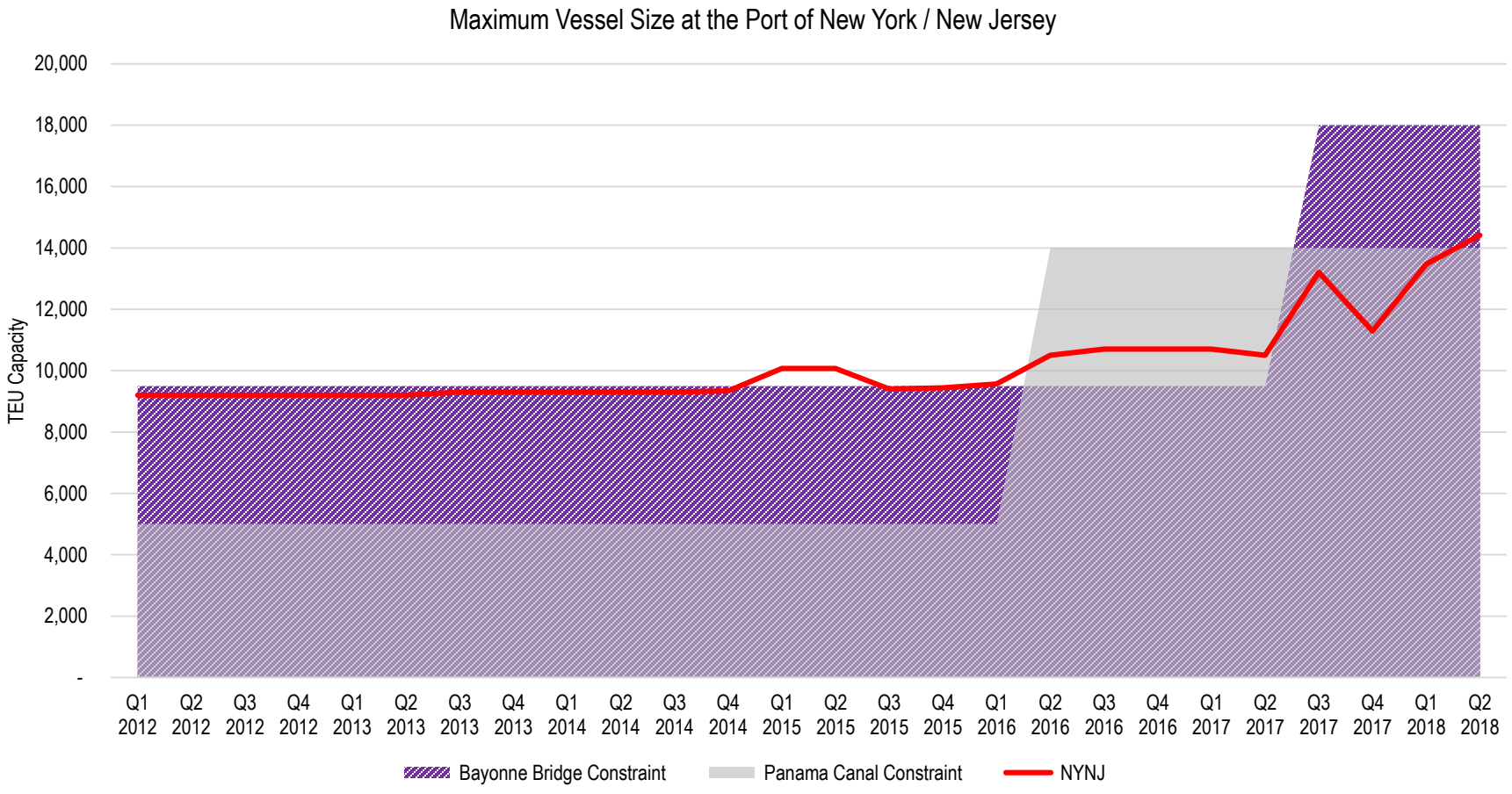
Container vessels do not move south through the Arthur Kill. The container vessel routing is through Sandy Hook / Breezy Point, Verrazano-Narrows and then to the Port Jersey Channel or under the Bayonne Bridge to the Elizabeth Channel.

- The map on the right shows the carbon emission by different types of vessels.
- This map was created based on data from the UCL Energy Institute.
- Though the data is for 2012, the navigation paths have not changed since.
- The yellow tracks represent Container vessels while the red dots/tracks show tankers.



Source : https://www.shipmap.org/

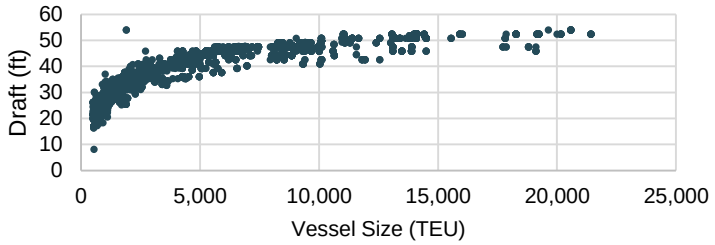
Vessel Sizes increase as constraints are mitigated; currently the largest vessel calling at the Port is 14K TEU. These constraints were the Panama Canal and the old Bayonne Bridge.

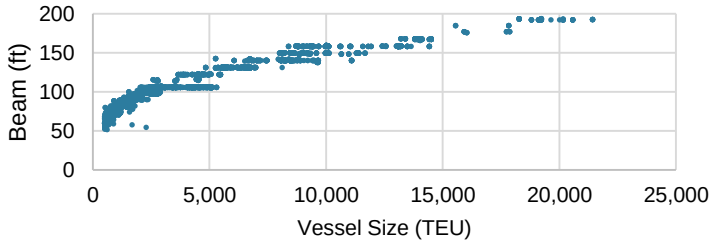


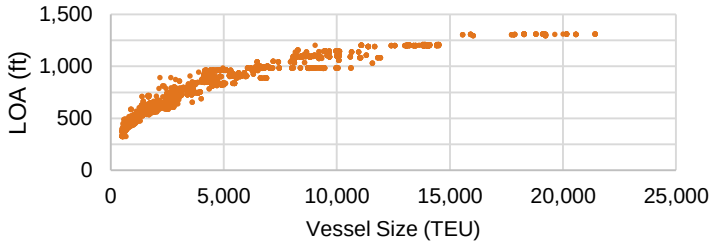
Source : Bluewater/MN

Future container vessel sizing indicates only small incremental increases to the current 21K TEU vessels. 20K TEU is the maximum size on the order book. The future enhancements will come from improving carbon footprints and operational efficiency.

- Many vessels over 10K TEU have maximum design drafts of greater than 50', meaning that they would not be able to be fully loaded (in weight) when coming to New York
- The largest vessels being built are adding incremental TEU and appear to be maintaining the maximum design beam of under 200' and a maximum design LOA of 1312 ft., corresponding to under 400 m.
- The next generation of vessels would have to break through the current Beam and LOA dimensions presenting significant infrastructural challenges.
- These largest vessels would be deployed on the Asia-Europe trade lanes and even the most productive ports and the widest and deepest canal (the Suez) cannot *efficiently* handle anything larger than a 20K TEU vessel.
- It is generally understood that vessel costs per TEU decrease with increased vessel size, whereas the handling costs per TEU increase. The addition of the two curves gives the total transport costs that indicates that the total transport cost decrease with increasing vessel size before reaching an optimum before rising again.
- All these points point to the assumption that the increase in vessel sizing is plateauing and that the 21,000 TEU class vessels, such as the OOCL Hong Kong at 21,413 TEU will remain close to the maximum size in the near to mid term.



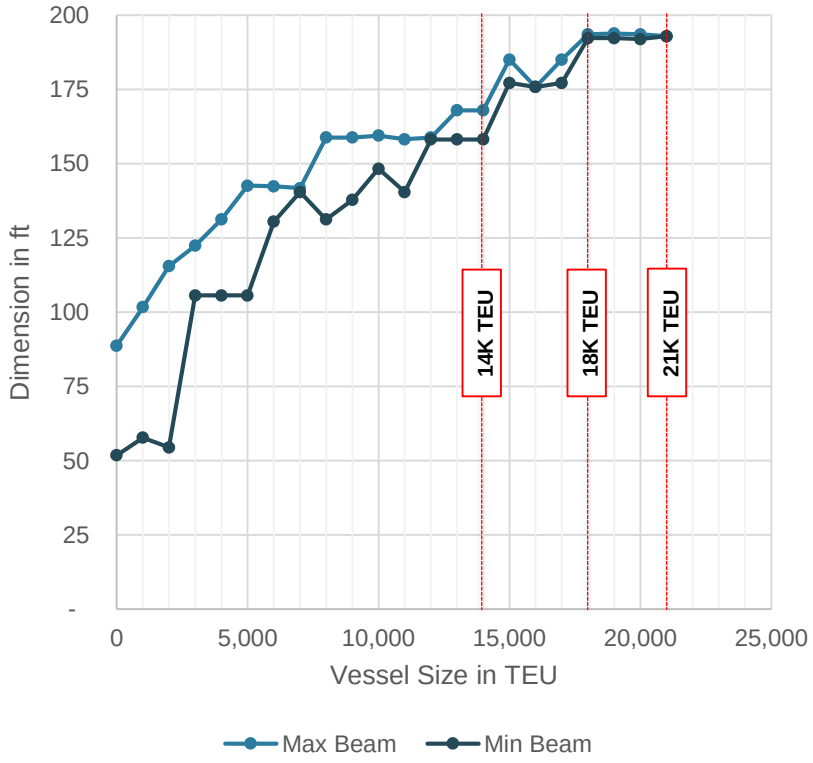




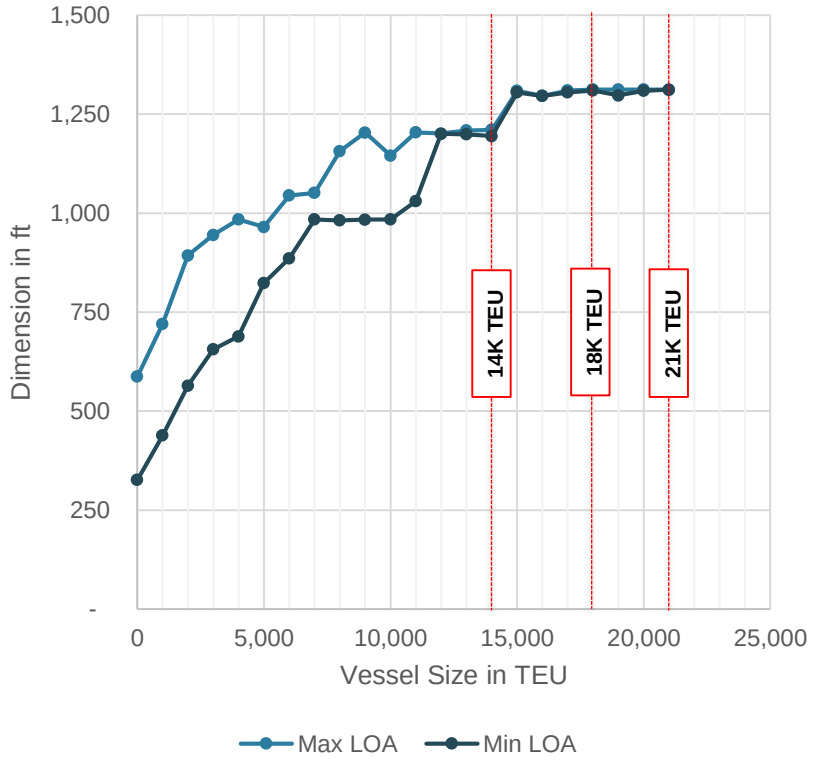
Source : Clarkson/MN

The PANYNJ 14K and 18K TEU ship simulation study reports that 18K TEU can call under some parameters. 21K TEU vessels have similar LOA and Beam indicating that these vessels should be able to call at the port under similar or modified parameters

Minimum and Maximum Beam



Minimum and Maximum LOA



Source: Clarkson/MN

The Port of New York and New Jersey is expected to service 14,000 TEU vessels with some occasional 18,000 TEU vessels in the mid to long term but should be able to handle 21,000 TEU vessels

- Market demand, profit margins and landside efficiencies will influence the deployment strategy of the largest ships
- However, irrespective of market demand, the Port of New York or the US East coast may be required to accept 18,000 TEU vessels as a result of the cascading effect, where the 20+ K TEU ships are deployed on the major east-west trade lanes and the existing tonnages on those lanes are deployed to the Asia-North America trades.
- It is expected that landside congestions would be aggravated with *regular weekly* calls of 18,000 TEU or greater vessels
- A number of requirements need to be met for the safe navigation of 14K and 18K the Port of New York and New Jersey including transit windows within one hour before or after slack at Bergen point near the Bayonne Bridge. Additionally there are vessel passing restrictions and the need for two pilots at all times. These conditions may impact the desirability of 18K vessels calling the Port of New York and New Jersey.
- With 18K TEU vessels, there are navigational constraints when docking and undocking at PNCT and Maher terminals in the Elizabeth channel as well as at GCT Bayonne in the Port Jersey Channel
- There will be an increased need for more vigilant communication between all parties at all times as well as increased crane management as these vessels are approaching or departing berths.
- For the long term, other factors need to be considered when developing demand scenario. Given the technological advances, geo-political dynamics, tariffs and other trade agreements, given a shift of manufacturing near or on-shore would lessen the need for even larger vessels than seen on the order books.
- The LOA and Beam of 18K vessel and 21K vessels are very similar with differences in stowage patterns and weight considerations allowing 3K more TEU to be handled by the 21K TEU Vessels.
- A review of potential demands, operational options, benefits and impacts point to the opinion that the maximum container vessel to call at the Port of New York and New Jersey will be 21K TEU vessels.

US Army Corps of Engineers®
New York District

DRAFT REPORT

New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study

Enclosure 2

Gate Types for Storm Surge Barriers

February 2019

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New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study

ENCLOSURE 2:

Gate Types for Storm Surge Barriers

DOCUMENT VERIFICATION

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1 INTRODUCTION

1.1 Introduction

Storm Surge Barriers and systematic overviews of constructed storm surge barriers around the world inclusive of the characterization of hydraulic gate types and functions have been discussed by various authors. Amongst others, relevant publications include: (USACE-ERDC, 2007), (PIANC, 2006), (Mooyaart L. F., Jonkman, de Vries, van der Toorn, & van Ledden, 2014), (Mooyaart & Jonkman, 2017), (Dircke, Jongeling, & Jansen, 2012), (van Ledden, Lansen, de Ridder, & Edge, 2012) and (Dijk, 2010). This document does not intend to reproduce or summarize the body of work that has been produced by others, instead, since Prof. Jonkman is part of the HATS Project Delivery Team (PDT), here we use and expand upon the content and supplemental data of (Mooyaart & Jonkman, 2017) to get a better understanding of the various storm surge barrier gate types that have been successfully applied elsewhere.

1.2 Purpose

The intent is to bracket the characteristics, application and proven concepts of the various constructed storm surge barriers and gate types such that gate types can be screened, at a level of detail commensurate to a feasibility study and be preliminarily be selected for the conceptual design of the storm surge barriers that are part of the HAT Study alternatives.

1.3 General Description of a Storm Surge Barrier with Navigable Passage

(Mooyaart & Jonkman, 2017) provide an overview of and general design considerations for storm surge barriers based on data and design documentation review of a select set of constructed barriers throughout the world. (Mooyaart & Jonkman, 2017) provide a general description of a navigable storm surge barrier. A typical layout of a storm surge barrier contains three elements; a gated section, a dam section and a navigable passage. A navigable passage can either be established with a lock or with a gated navigable opening. The difference is that the gates of a navigable passage are seldom in use, but must operate reliably during floods, while lock gates are frequently in use and must operate reliably during day-to-day lock operations and during floods. Figure 1-1 below provides a schematic plan view of a navigable storm surge barrier. The navigable passage is schematically shown as a gated navigable opening, not as a lock, since the storm surge barriers studied under the HAT Study require minimization of interruptions for maritime traffic during transit.

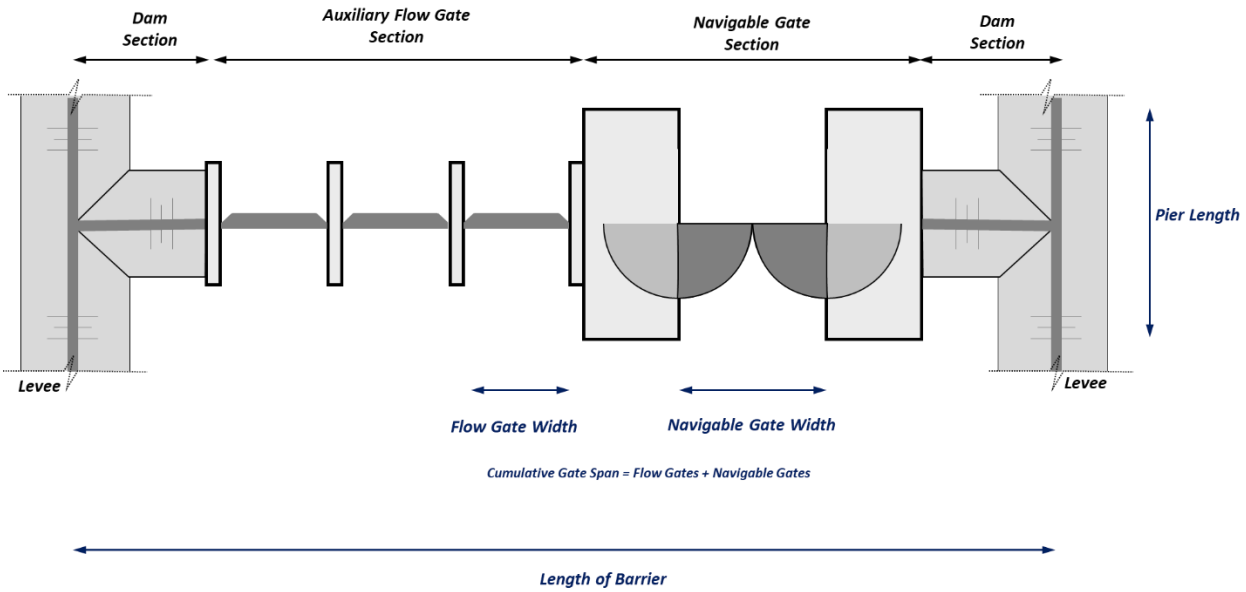


Figure 1-1: Schematic plan view of a storm surge barrier (modified after Mooyaart and Jonkman, 2017).

2 GATE TYPES FOR STORM SURGE BARRIERS

2.1 Hydraulic Steel Structures

USACE designs, constructs, and operates many types of projects and for a large number of them the primary project purpose is flood control (reservoirs), navigation, and local flood protection (or a combination thereof). Many of these projects include the use of Hydraulic Steel Structures (HSS), also referred to as hydraulic gates, to control the flow of water in various ways. ETL 1110-2-584 (USACE, 2014) prescribes guidance for designing new hydraulic steel structures and includes an overview of the most commonly gate types. It includes lock gates, tainter gates and lift gates.

For the HAT Study the gate types suitable for a storm surge barrier as reported by (Mooyaart & Jonkman, 2017) are used as a starting point, which is inclusive of the commonly used gate types above, but more comprehensive and specifically tailored to the application of hydraulic gates in constructed storm surge barriers. The reader is referred to (Mooyaart & Jonkman, 2017) for a full overview of gate types and a description of the constructed storm surge barriers, but in general the gate types can be categorized by their direction of movement (e.g. vertical axes – horizontally rotating or horizontal axes – vertical motion). The following is a list of hydraulic gate types evaluated:

- Horizontal rolling gates (horizontal lateral movement)
- Sector gate (vertical axes)
- Floating Sector Gate (ball joint)
- Rotating Segment Gate (horizontal axes)
- Flap Gates (horizontal axes)
- Barge Gate (vertical axes)
- Miter Gate (vertical axes)
- Vertical Lift Gates (vertical lateral movement)
- Vertical Rising Gates (vertical lateral movement)
- Tainter Gates (Horizontal axes)

It is recognized that other hydraulic gate types exist (e.g. drum gate, visor gate), which are applied in riverine and upland water control structures. However, those have not been utilized in a coastal setting or in constructed storm surge barriers, and as such, are omitted for preliminary design considerations for the HAT Study.

2.2 Conceptual Categorization

The application of hydraulic gates in storm surge barriers should first evaluate the specific use of the gate. As discussed, storm surge barriers contain gate structures with two distinct functions: passage of vessels and passage of flow, i.e. the gate for the navigable passage and the auxiliary flow gate. In all instances where the navigation is not locked through the storm surge barrier the gated opening of the navigable passage also provides flow exchange. Certain gate types have fewer

limitations for navigation, while others have more. The air clearance restriction associated with tainter gates and lift gates is a typical example of a limitation for navigation. Second, while all hydraulic gate types require a foundation, gate types like the vertical rising or inflatable gate require relatively more complex sill structures which effectively limit the gate height and thereby the depth of the navigable passage. Other limitations for navigation result from the feasible span of the gate types. Based on the dataset of constructed navigable storm surge barriers, floating sector gates provide the largest navigable openings while rotary segment gates for example have so far been limited to a span of 200 ft. The figure below provides a coarse categorization of the various gate types by application as navigation gate or flow gate as well as their ability to accommodate deep draft of shallow draft vessels.

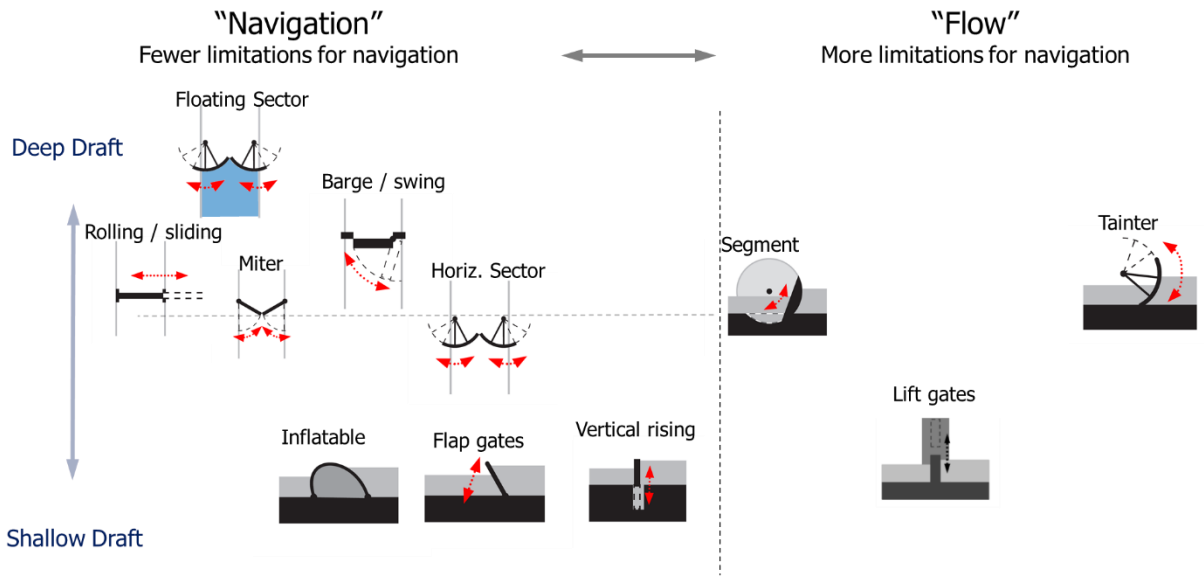


Figure 2-1: Conceptual categorization of hydraulic gate types by vertical and horizontal clearance (icons after (Dijk, 2010))

2.3 Gate Types for a Storm Surge Barrier

The following pages provide a brief summary of the gate types and lists advantages and disadvantages of each gate types when considered for storm surge barriers based on and modified after (USACE, 2014), (Dircke, Jongeling, & Jansen, 2012) and (PIANC, 2006).

Vertical Lift Gate

	Pros	Cons
Vertical lift gates are moved vertically from the sill. A tower supports the gate during its operation. Overhead cables, sheaves and bull wheels can enable lifting or hydraulic cylinders lift the gate (e.g. Seabrook Barrier, USA)	• Large gate span (up to 300 feet) possible • Little upland space required • Controlled operation under flow and wave conditions • Raised gate accessible for maintenance • Proven concept • Vertical closure (gravity assisted) • Easy discharge of excess water • Overflow and reverse flow acceptable • Relatively less complex underwater sill structures	• Limited clearance height for shipping • Raised gate subject to wind load can result in cost increase • Water depth, and required clearance determines tower and gate height • Sensitive to vibrations • Gates with wheels are a known weak spot and subject to wearing • Smooth glide and vertical motion of gate can be impeded by marine growth in gate recess

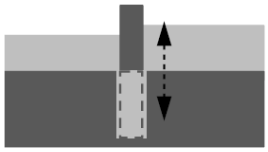


Cross Section

Examples: Seabrook (USA), IHNC (USA), Eastern Scheldt (NL), Hollandse IJssel (NL), Ems (GER)

Vertical Rising Gate

	Pros	Cons
Vertical rising gates lie beneath the sill in open position. The gates are lifted vertically to close the barrier. Both in open and in closed position the gates are positioned largely under water. In most applications, gates can be lifted above water to allow for maintenance.	• Large gate span possible • Unrestricted air-draft clearance for maritime traffic • Little upland space required • Controlled operation under flow and wave conditions • Proven concept • Easy discharge of excess water • Overflow and reverse flow acceptable	• Inspection and Maintenance is challenging • Large, deep and complex underwater sill structures • Water depth determines sill structure depth • Sensitive to vibrations • Smooth glide and vertical motion of gate can be impeded by marine growth in gate recess

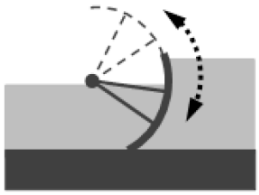


Cross Section

St. Petersburg Barrier (RU)

Tainter Gate

	Pros	Cons
The tainter gate rotates around a horizontal axis, which passes through the bearing center. In closed position, the tainter gate rests on the sill and in open position it is lifted and locked. It is also referred to as a radial or segment gate.	• Large gate span feasible • Proven Concept • Controlled operation during flow and wave conditions • Little space required • Inspection and maintenance in the dry • Limited horizontal flow contraction • Excess water through gate • Suitable for reverse head and flow	• Limited clearance height for shipping • Generally not used for vessel passage (Exception Eider Barrier, Germany) • Raised gate subject to wind load • Relatively stringent tolerance criteria for the sill structure

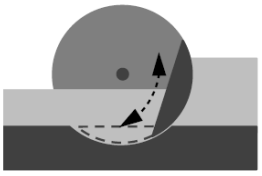


Cross Section

St. Petersburg Barrier, Eider (GER), Thames (UK), Ems (GER)

Rotating Segment Gate

	Pros	Cons
Similar to a segment gate the rotary segment gate has a horizontal axis. However, in recess, it lies in a concrete sill in the bed of the river. Thus, it is possible to sail over the gate in opened position. Operation of the gate is achieved by approximately 90deg rotation to raise the gate to the active flood protection position. An additional 90deg of rotation places the gate in the top raised position ready for inspection or maintenance.	• Large gate span feasible • Unrestricted vertical clearance for maritime traffic • Controlled operation during flow and wave conditions • Little space required • No wind loads during open position • Inspection and maintenance in the dry • Limited horizontal flow contraction • Suitable for reverse head and flow • Deployment and recovery time is relatively short	• Load transfer and concentration is complex • Complex design with relatively low tolerances • Sill structure is vulnerable to silting, objects and corrosion • Access to and maintenance of hollow gate body can be challenging • Complex mechanical operating system

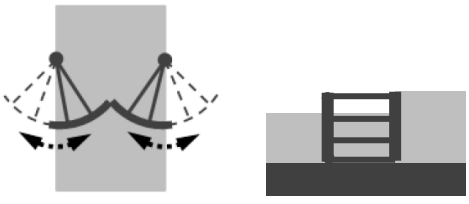


Cross Section

Thames (UK), Ems (GER)

Sector Gate (Vertical Axis)

	Pros	Cons
A sector gate consists of a double gate complex. Each gate has a circular shape, transferring forces through a steel frame to the hinges at each side of the opening. It operates by rotating around two vertical axes. During operation the gates will rest on the river bed. In non-operational condition, each gate is stored in a gate housing or recess besides the waterway.	• Large gate span feasible • Unrestricted clearance for maritime traffic • Relatively low wind loads in open and closed position • Suitable for deep waters • Immediately ready for operation • No complex sill structures • Stable structure; no load concentration • Gate housing/recess can be converted to dry docks to perform maintenance • Gates are stored in the recesses; small collision hazards • Suitable for reverse head and flow • Relatively low susceptibility to flow vibrations	• Large space and deep excavation required for gate housing • Flat and smooth slide way required • Siltation may hamper operation • Load transfer to hinges, maintenance, corrosion and marine growth are a concern • Potential for ship collision • Potential for siltation within gate housing

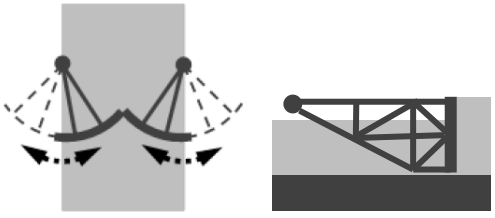


Plan view Cross-section

Seabrook (USA), New Bedford (USA), IHNC (USA), Harvey Canal (USA), West Closure (USA)

Floating Sector Gate

	Pros	Cons
A floating sector gate consists of a double gate. Each gate has a circular shape, rotates around a spherical hinge at each side of the opening transferring forces through a steel frame to the hinges and to the foundation. The gate leaves are buoyant and float into place. During operation the gates will rest on the river bed. In non-operational condition, the doors are stored in a gate housing (dry dock) besides the waterway in the river banks	• Large gate span possible • Unrestricted overhead clearance for maritime traffic • Shallow dry dock: easy inspection and maintenance and collision protection • Can be immersed if sill is covered with silt • Flatness of sill is not critical for operation • Vertical closure of flow opening (no strong horizontal flow contraction) • Separate sluice openings may be applied to reduce differential head and discharge excess water during reverse head conditions	• Large space required • Operation complicated; in-flowing water may be difficult to control • Loads are concentrated on the hinges • Reverse head conditions may cause unexpected loads on the spherical hinges • Objects on sill can cause damage • Deployment and recovery time is relatively extensive due to filling time of dry docks • Sensitivity to flow-induced oscillations • Sensitive to dynamic wave forces • Limited resistance to negative differential head



Plan view Cross-section

Maeslant Barrier (NL), St. Petersburg Barrier (RU)

Inflatable Gate or Dam

An inflatable gate is basically a sealed tube made of a flexible material, such as synthetic fiber, rubber, or laminated plastics. It is anchored to the sill and walls by means of anchor bolts and an air- and watertight clamping system. The gate is inflated with air, water, or a combination of both.

Pros

- Comparatively fewer limitations on total span
- Unrestricted overhead clearance for maritime traffic
- Marginal to wind loads
- Little space required
- Direct transfer of hydraulic load to foundation
- Invisible when not in use
- No need for hinges and driving system
- Vertical closure of the flow opening possible in adverse wave and current conditions
- Not sensitive to sediment deposition on the sill

Cons

- Relatively slow deployment/closure speeds
- Internal pressure determines stability
- Control of storage and immersion of rubber sheet can be challenging
- Not suitable for deep water (max height built today is approx. 27ft)
- Difficult inspection, maintenance and replacement of rubber sheet
- Potentially vulnerable to vandalism and debris abrasion
- Flexible structure, low frequencies, small stiffness, great mass
- Considerable response to wave loads
- No spill of excess water; overflow vibrations

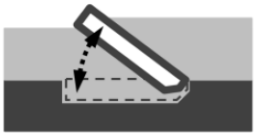


Cross Section

Ramspol Barrier (NL)

Flap Gate

	Pros	Cons
Flap gates consist of a straight or curved gate leaf surface, pivoted on a fixed horizontal axis. In Venice, the gates are operated by filling or emptying them with air. At Venice and Stamford, the gates pivot around an axis at the sill, while at the Billwerder Bucht the axis is positioned above the mean water level.	• Relative view limitations on the total span if constructed in series • Separate flaps; reduced failure risk • Unrestricted overhead clearance for maritime traffic • Little space required • Suitable for deep waters • Controlled operation under flow and wave conditions • Not subjected to wind loads • Invisible when not in use • No strong confinement of horizontal flow with alternate gates deployment • Vertical closure, single flap • Excess water through lowering the gate crest or a single flap gate	• Limitations on suitability for conditions with reverse head • Pneumatic operation is not always fully controlled • Underwater storage of flap gates makes inspection and maintenance challenging and may lead to corrosion or excessive marine growth • Abrasion of underwater hinges due to contact with granular suspended sediments • Sensitivity to vibrations • Natural frequencies low; small stiffness, large mass • Relatively more expensive structure

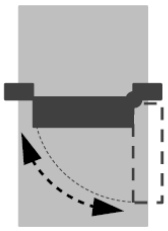


Cross Section

MOSE Venice (IT), Stamford (USA)

Barge Gate

	Pros	Cons
A barge gate is a caisson stored on one side of a waterway, pivoting around a vertical axis to close. A barge gate may be buoyant or equipped with gated openings to reduce hinge and operating forces. This type of gate is also referred to as a swing gate or caisson gate.	• Large gate span feasible • Unrestricted overhead clearance for maritime traffic • Possibility to perform maintenance offsite	• Very slow deployment/closure speeds • Not well suited for reverse head or flow • Difficult to control during opening and closing (especially in current and wave conditions) • Increase in size increases difficulties in control - not well suited for deep waters • Specialized operating equipment needed and back up tug to provide assistance • Maintenance of hollow gate body can be challenging • Not well suited to deal with sedimentation at the sill



Plan view

IHNC Barrier (USA)

Horizontal Rolling Gate

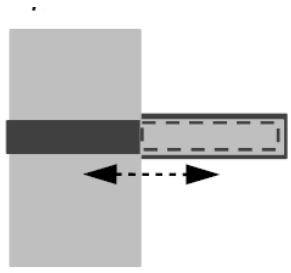
Rolling gates are sliding panels or caissons stored adjacent to the waterway. They are rolled into position in anticipation of a flood event. The new Panama locks are equipped with rolling gates. Alternative designs for the Maeslant barrier and for a barrier near Hamburg included rolling gates. For these designs the gates were equipped with gated openings to reduce the lateral loads during closure.

Pros

- Large gate span feasible
- Unrestricted overhead clearance for maritime traffic
- Gate housing/recess can be converted to dry docks to perform maintenance
- Gates are stored in the recesses; small collision hazards
- Suitable for reverse head and flow
- Relatively low susceptibility to flow vibrations
- “Invisible” when not in use

Cons

- Very slow deployment/closure speeds
- Difficult to control during opening and closing (especially in current and wave conditions)
- Specialized operating equipment needed
- Large upland area needed to “dock” gate and as such impractical as auxiliary flow gates
- Susceptible to siltation on the sill and in gate recess
- Maintenance of hollow gate body can be challenging



Plan view

Miter Gate

	Pros	Cons
Miter gates consist of two leaves mounted on the lock walls. These gates are fairly simple in construction and operation and can be opened or closed more rapidly than other types of gates. The two leaves of a miter gate form a shallow three-hinged arch angled upstream. In the closed position, under head, each leaf bears on a lock wall and on the other leaf at the center of the lock. This arch shape is very efficient for spanning larger distances between lock walls. Usually the angle of the leaf in plan, in the closed position is 1:3.	• Unrestricted overhead clearance for maritime traffic • Relatively low susceptibility to flow vibrations • Not susceptible wind loads • “Invisible” when not in use • Little upland space required • Gates are stored in the recesses; relatively small collision hazards • Suited for application in relatively deep waters • Proven concept	• It is extremely hard to close this gate type during an emergency situation with an appreciable flow through the chamber • Only relatively small gate spans are economically feasible (up to 110ft) • Not suitable for operation in reverse head conditions or waves • Sensitive to vibrations when operated in flow conditions • Susceptible to collision hazards during operations



Plan view

Eastern Scheldt (NL), Eider Barrier (GER) and applied in many locks throughout the world

3 STORM SURGE BARRIER CHARACTERISTICS THROUGH AGGREGATED DATA PLOTS

3.1 Gate Characteristics

The original supplemental data from (Mooyaart & Jonkman, 2017) has been expanded to capture a few more characteristics, such as navigable use, water body category and longitude and latitude. The following sections provide data plots and visual aids to distill characteristics of the constructed barriers and the application of gate types. These aggregated data plots are used to demonstrate the applicability of proven concepts within a finite range of parameters such as gate span or gate sill elevation. These characteristics can them be used to inform the preliminary selection of gate type and dimensions for the gated structures of the HAT Study storm surge barriers.

3.1.1 Gate Type versus Span and Sill Elevation

Figure 3-1 and Figure 3-2 presents the gate type versus the span and the gate type versus the sill elevation respectively for all gates (both auxiliary and navigable gates) that are part of the storm surge barriers. The navigable gates are marked with the letter N. From Figure 1-1 it can be clearly noted that for navigable passages where a wide maritime traffic lane needs to be maintained a floating sector gate has been selected. Floating sector gates span an opening of over 1180 ft and 650 ft at the Maeslant Barrier and St. Petersburg Barrier respectively. Other large navigable openings are spanned with a vertical rising gate or a vertical lift gate. Vertical lift gates come with air draft restrictions. With respect to the sill elevation and the water depth accommodated by the various gates, the floating sector gates are the constructed gates that accommodated the deepest sill elevations (over 50 ft of water depth measured from MSL). Flap gates are able to accommodate deep waters as well. The Venice Barrier includes a series of twenty 20 meters (65 ft) wide flap gates with a sill at 49 ft below MSL. Jointly those gates span over 1,300 ft of waterway. It can further be noted that the constructed sector gates at New Bedford has a sill elevation at 39 ft below MSL. Figure 3-3 presents the gate span versus the sill elevation respectively for both auxiliary and navigable gates. It allows one to distinguish the outliers in both span and sill elevation as well as to note that a number of gates, independent of type, span less than 200 ft and have a sill elevation less than 20 ft. The exception is the floating sector gate.

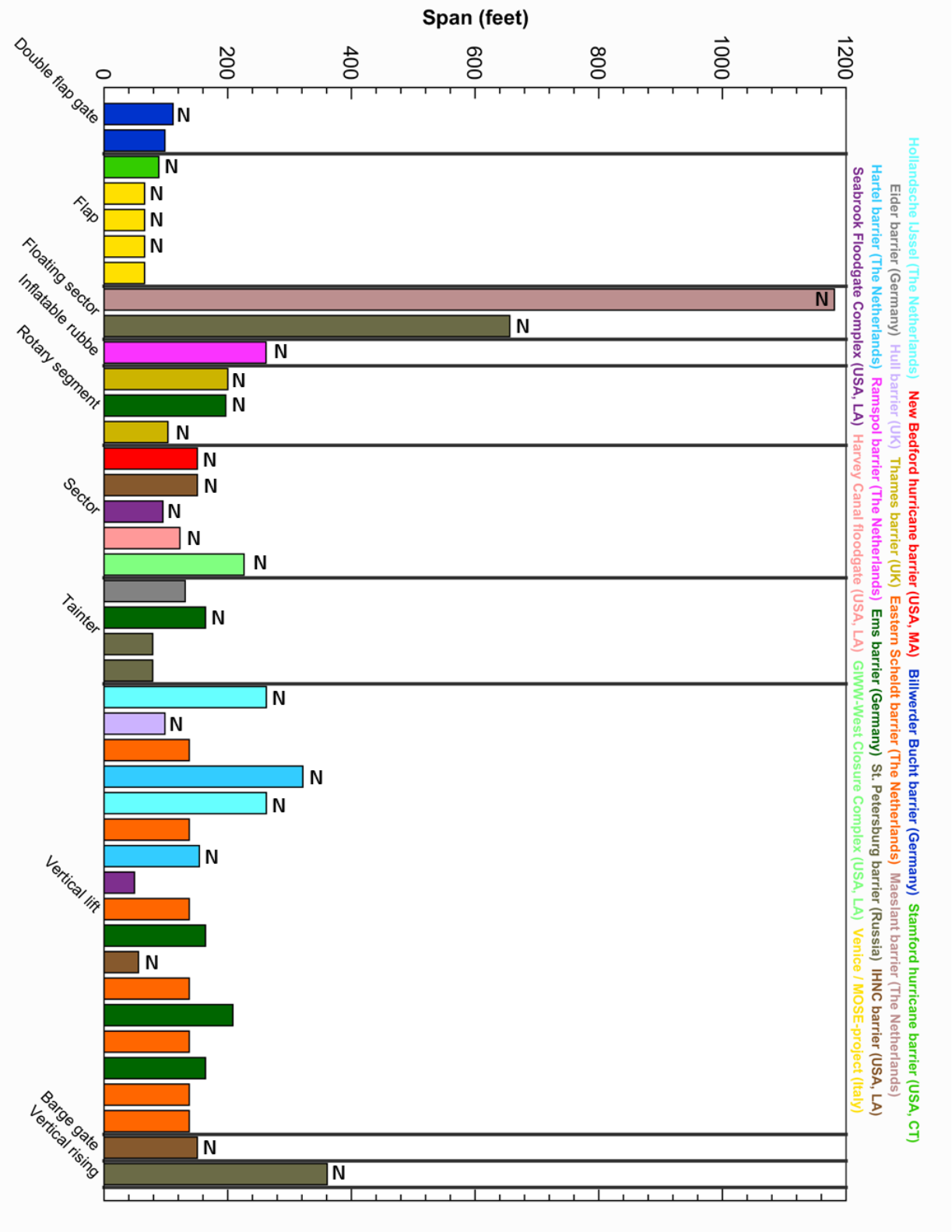


Figure 3-1: Gate Types and Gate Span for both Auxiliary Flow gates and Gates for the Navigable Passage (Navigable Gates are Marked with the Letter "N")

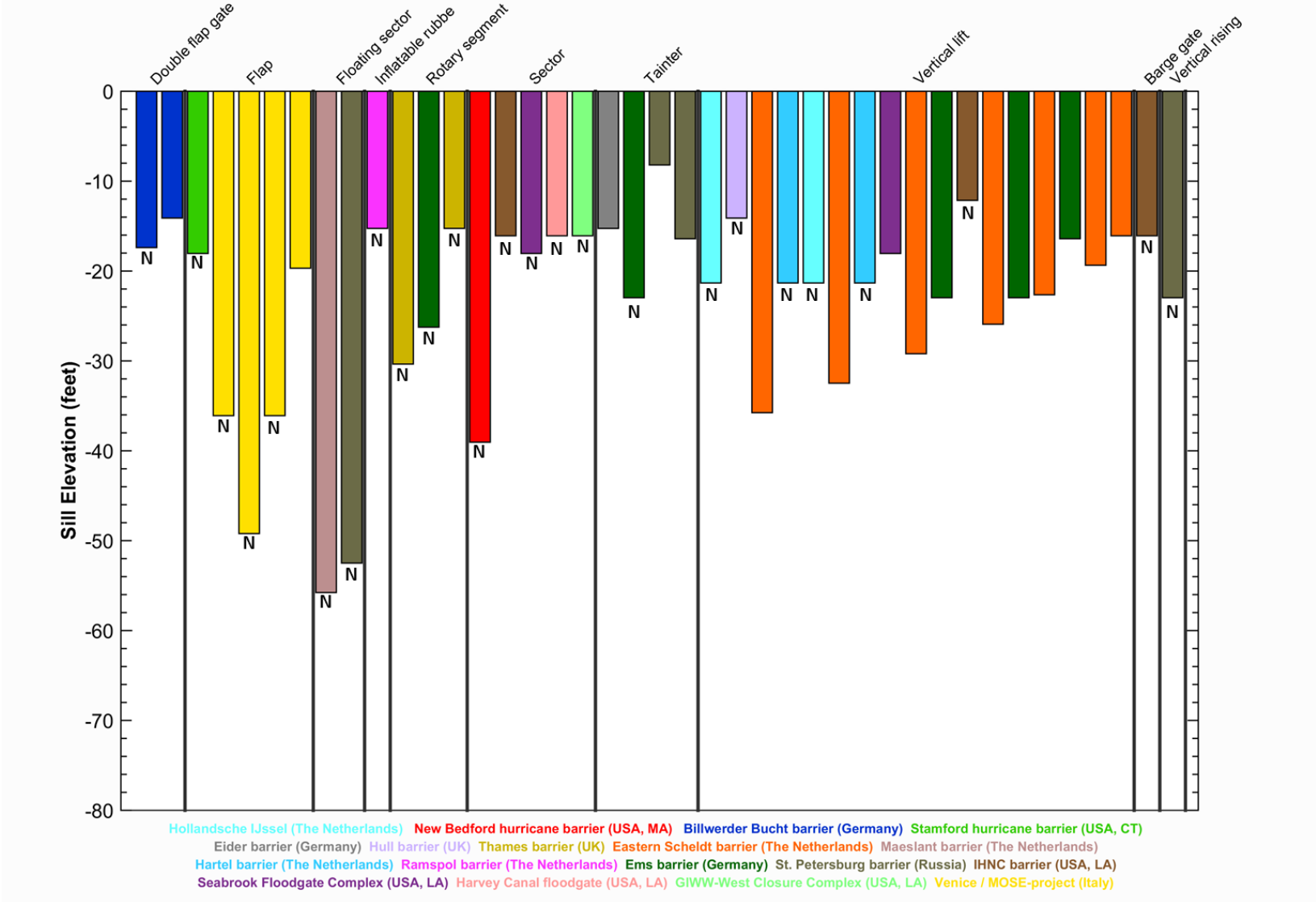


Figure 3-2: Gate Types and Gate Sill Elevation for both Auxiliary Flow gates and Gates for the Navigable Passage (Navigable Gates are Marked with the Letter "N")

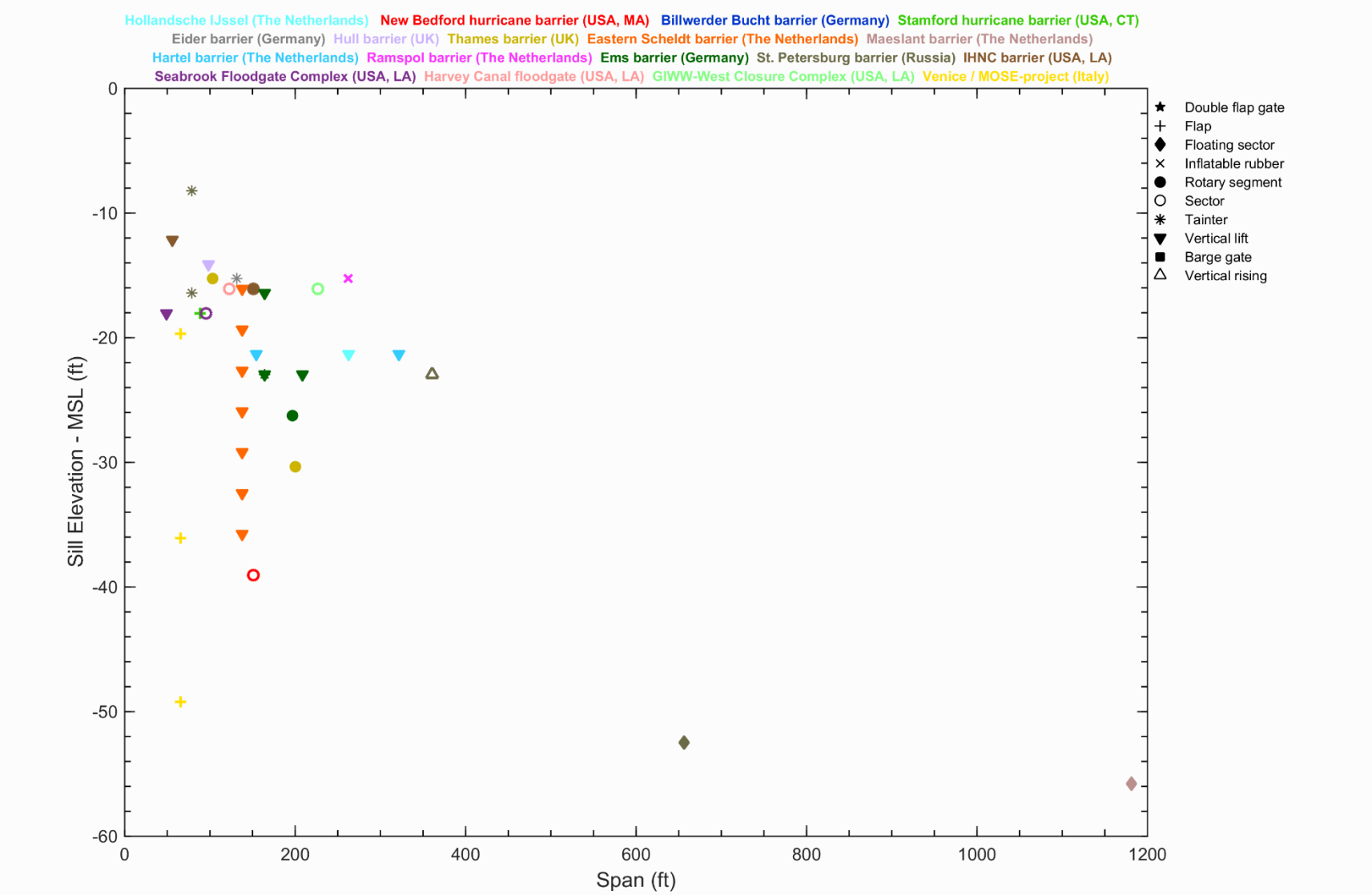


Figure 3-3: Gate Span versus Gate Sill Elevation for both Auxiliary Flow gates and Navigable Gates

3.1.2 Overall Storm Surge Barrier Characteristics

Finally a brief overview is provided of the more general characteristics of the select set of constructed storm surge barriers. Figure 3-4 provides a time line that depicts the approximate start and end date of construction for each storm surge barrier. It will have to be recognized that technological advances over time allowed for more complex constructions.

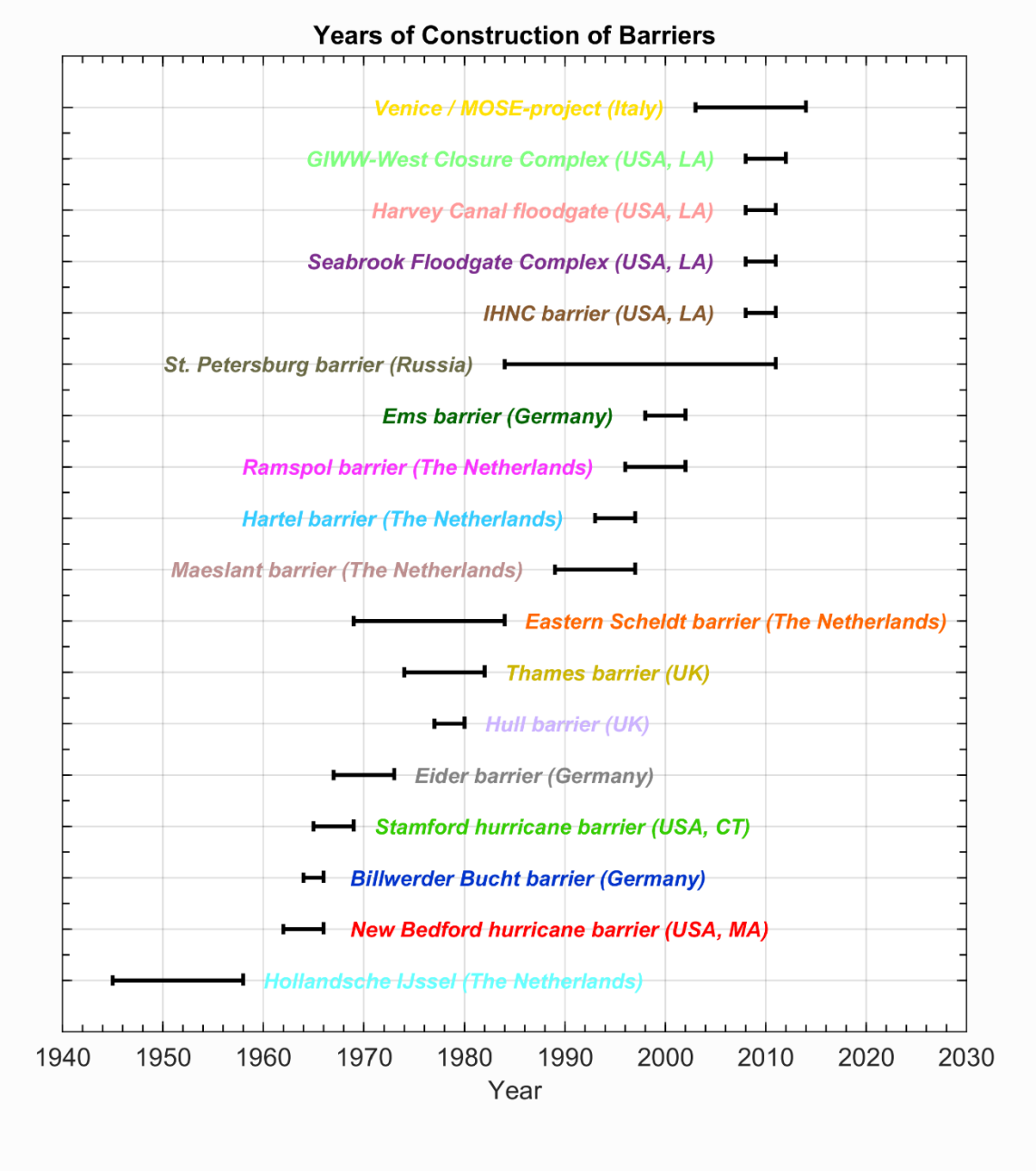


Figure 3-4: Start and End Year of Construction

Table 3-1 provides a brief overview of the data that is included within Appendix A. The gate series are numbered and distinguished by gate type, sill elevation or span. The numbering is used for ease of reference here, not a number prescribed by the owner or operator. It can be noted that some barriers have multiple gated navigable passages while the Eastern Scheldt and Eider Barrier utilize a lock. Storm surge barriers are constructed in various coastal settings. Here a general characterization is included to distinguish between barriers across open coast estuaries, tidally influenced rivers and tidally influenced canals and harbor complexes. The cumulative span of the gated section varies from comparably very short (a couple of percent) to almost equal to the barrier length. The latter indicates an open barrier where the tidal exchange between the open ocean and the protected basin is desired to be maintained to prior to construction.

Table 3-1: Summary of Gate Types within the storm surge barriers from (Mooyaart & Jonkman, 2017). Symbols used are as follows: Double Flap Gate (★), Flap Gate (✚), Floating Sector Gate (◆), Inflatable Gate (✕), Rotary Segment Gate (●), Sector Gate (○), Tainter Gate (✱), Vertical Lift Gate, (▼), Barge Gate (■), Vertical Rising Gate (△). The navigable gates are shaded in blue. Navigable passage through the Eider and Eastern Scheldt storm surge barrier is through a lock.

	Barrier Length	Gate 1 - Type	Gate 2 - Type	Gate 3 - Type	Gate 4 - Type	Gate 5 - Type	Gate 6 - Type	Gate 7 - Type	total number of gates	Cumulative Span of gated section	Cross-sectional Area below MSL	Coastal Setting
	(ft)									(ft)	(ft^2)	
Hollandsche IJssel (NL)	656	▼ (1)	▼ (1)						2	262	5,597	River
New Bedford hurricane barrier (USA, MA)	4495	○ (1)							1	151	5,920	Estuary
Billwerder Bucht barrier (Germany)	492	✚ (2)	✚ (2)						8	420	6,609	Harbor/Canal
Stamford hurricane barrier (USA, CT)	2789	✚ (1)							1	89	1,615	Harbor/Canal
Eider barrier (Germany)	16076	✱ (5)							5	656	10,010	Estuary
Hull barrier (UK)	131	▼ (1)							1	98	1,399	Estuary
Thames barrier (UK)	1739	● (4)	● (2)	✱ (4)					10	1211	26,781	Estuary
Eastern Scheldt barrier (NL)	29528	▼ (7)	▼ (11)	▼ (6)	▼ (9)	▼ (8)	▼ (6)	▼ (15)	64	8543	193,750	Estuary
Maeslant barrier (NL)	2001	◆ (1)							1	1181	73,195	River

	Barrier Length	Gate 1 - Type	Gate 2 - Type	Gate 3 - Type	Gate 4 - Type	Gate 5 - Type	Gate 6 - Type	Gate 7 - Type	total number of gates	Cumulative Span of gated section	Cross-sectional Area below MSL	Coastal Setting
Hartel barrier (NL)	820	▼ (1)	▼ (1)						2	482	10,226	River
Ramspol barrier (NL)	1476	× (1)	× (1)	× (1)					3	738	11,302	River
Ems barrier (Germany)	1562	● (1)	❋ (1)	▼ (1)	▼ (1)	▼ (3)			7	1358	26,210	River
St. Petersburg barrier (Russia)	83333	◆ (1)	△ (1)	❋ (34)	❋ (30)				66	6056	103,441	Estuary
IHNC barrier (USA, LA)	7546	○ (1)	■ (1)	▼ (1)					3	351	5,597	Harbor/Canal
Seabrook Floodgate Complex (USA, LA)	427	○ (1)	▼ (2)						3	194	3,444	Harbor/Canal
Harvey Canal floodgate (USA, LA)	394	○ (1)							1	125	3,552	Harbor/Canal
GIWW-West Closure Complex (USA, LA)	1722	○ (1)							1	226	3,552	Harbor/Canal
Venice / MOSE-project (Italy)	4921	+ (18)	+ (20)	+ (20)	+ (20)				79	4790	18,0403	Estuary

Figure 3-5 shows the cross-sectional area below MSL for the various barriers. The top three ranking are storm surge barriers build in estuarine coastal settings. From a HAT Study perspective it is worthwhile to note that the existing bathymetry and cross-sectional area of the connection between the Atlantic Ocean and New York harbor is on the order of 1,000,000 ft^2. If a storm surge barrier were to reduce the cross-sectional area to say 50% of the existing condition, the area of the gated openings would still far exceed both the Eastern Scheldt and the Venice barriers.

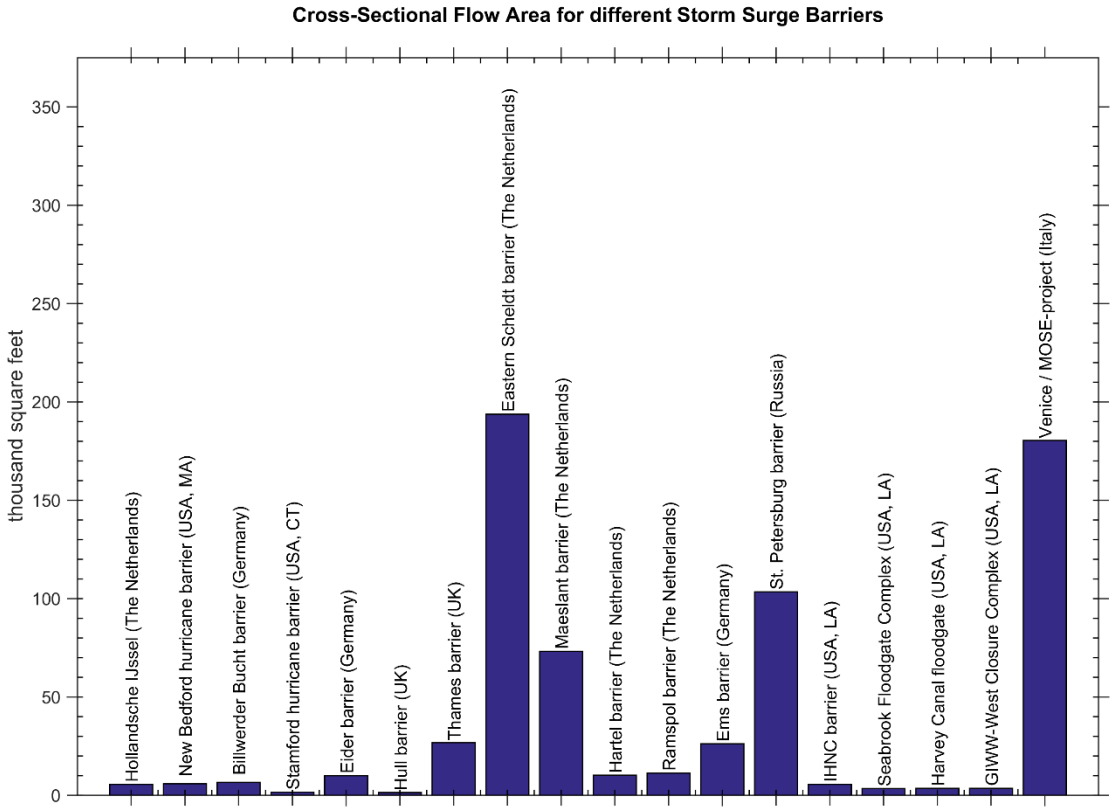


Figure 3-5: Flow Area (below MSL) for the Storm Surge Barriers listed in (Mooyaart & Jonkman, 2017)

The requirements for the conveyance of flow and flow area of the storm surge barrier is largely influenced by the tidal prism. Since the information on tidal prism is not available for all storm surge barriers Figure 3-6 presents the tidal range for the storm surge barriers as a proxy parameter.

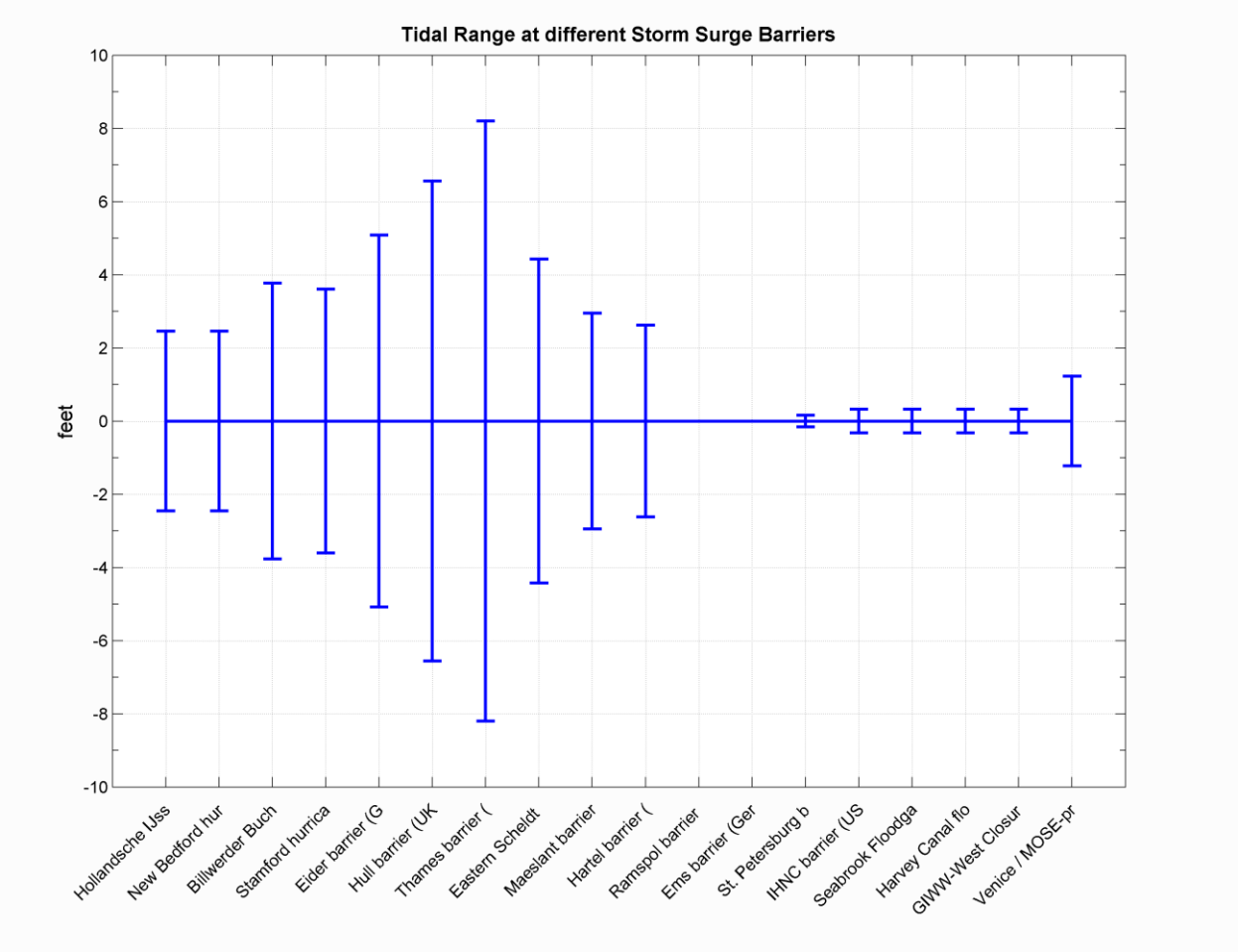


Figure 3-6: Tidal Range for the constructed Storm Surge Barriers

Lastly Mooyaart and Jonkman present a scatter plot of two key parameters: the peak flow (cfs) through the storm surge barrier openings and the cross-sectional flow area below MSL. The division of these two parameters provides for the peak velocity through the barrier and is indicated by the color of the plotted data points in Figure 3-7. A relative large tidal flow with a relative small barrier opening results in high peak flow velocities. Figure 3-7 shows the Eider Barrier and the Eastern Scheldt Barriers as the extremes with peak velocities estimated to exceed 10 ft/s. Not coincidentally these are the two barriers that do not contain gated navigable passages, but a locked passage. One of the constraints for the HAT Study is the minimization of interruptions for maritime traffic during transit. As a result the conceptual barrier designs will need to carefully consider the two parameters presented below.

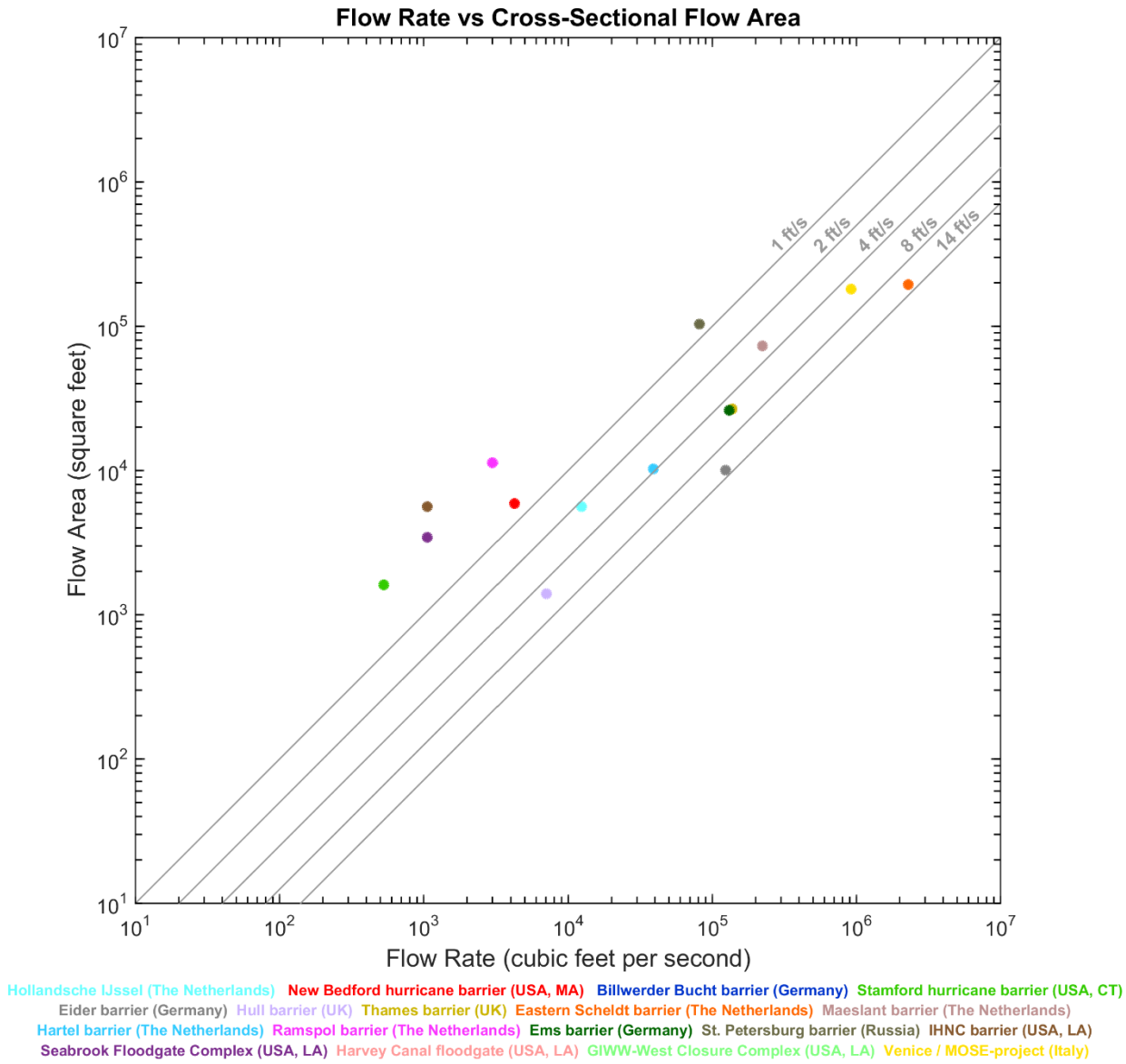


Figure 3-7: Peak Flow versus Cross-sectional Flow Area Below MSL

4 CONCLUSIONS

4.1 Synopsis

This document provides a brief overview of gate types used in storm surge barriers. This document does not intend to reproduce or summarize the body of work that has been produced by others, instead it summarizes and briefly expands upon the content and supplemental data of (Mooyaart & Jonkman, 2017) to get a better understanding of gate types that have been successfully applied elsewhere. The intent is to bracket the characteristics, application and proven concepts of the various constructed storm surge barriers and gate types such that gate types can be screened, at a level of detail commensurate to a feasibility study and be selected for the various storm surge barrier alternatives within the HAT Study.

The gate types are compared and an overview of advantages and disadvantages for each gate type were provided. It should be noted that experience thus far with developing and implementing barrier designs learns there is not one single perfect design and not one single perfect gate type. It should be emphasized that for civil work projects comparable to the barriers under consideration as part of the HAT Study, custom designs were developed that select or combine the most favorable aspects of different gate types.

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US Army Corps of Engineers®
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DRAFT REPORT

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Conceptual Design for Storm Surge Barriers

Volume 2

February 2019

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LIST OF ACRONYMS

AASHTO	American Association of State Highway Transportation Officials
ACES	Automated Coastal Engineering System
ADCIRC	Advanced Circulation Model
AIS	Automatic Identification System
AEP	Annual Exceedance Probability
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
ASTM	American Society for Testing and Materials
BOD	Basis of Design
CFS	Cubic Feet Per Second
CSRM	Coastal Storm Risk Management
CY	Cubic Yards
E&D	Engineering & Design
EL.	Elevation
EM	Engineering Manual
ETL	Engineering Technical Letter
ER	Engineering Regulation
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
FS	Factor of Safety
FWOP	Future Without Project
FWP	Future With Project
HAT	Highest Astronomical Tide
NYNJHAT	New York-New Jersey Harbor and Tributaries
HSDRRS	Hurricane Storm Damage Risk Reduction System
ICC	International Code Council
ISO	International Standards Organization
LAT	Lowest Astronomical Tide
LRFD	Load Resistance Factor Design
M&N	Moffatt & Nichol
MHHW	Mean Higher-High Water
MHW	Mean High Water
MSL	Mean Sea Level
MLW	Mean Low Water
MLLW	Mean Lower-Low Water
NAVD88	North American Vertical Datum of 1988
NJ	New Jersey
NOAA	National Oceanic and Atmospheric Administration
NNBFs	Natural and Nature-Based Features
NY	New York
NYC	New York City

PDT	Project Delivery Team
Rev	Revision
ROW	Right of Way
SBM	Shoreline Based Measure
SLR	Sea Level Rise
TM	Technical Memorandum
TOW	Top of Wall
USACE	United States Army Corps of Engineers
USACE ERDC	USACE Engineering Research and Development Center
USACE-NAN	USACE New York District
USGS	United States Geological Survey

New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study

Conceptual Design for Storm Surge Barriers – Volume 2

1 INTRODUCTION

1.1 The Study

The study purpose is to determine the feasibility of coastal storm risk management (CSRM) in the New York – New Jersey Harbor and Tributaries (NYNJHAT) study area, and to recommend a plan that will contribute to community and environmental resilience.

The study area encompasses the New York Metropolitan Area, including the most populous and densely populated city in the United States, and the six most populated cities in New Jersey. The shorelines of some of the NYNJHAT study area are characterized by low elevation areas, developed with residential and commercial infrastructure that are subject to tidal flooding during storms. The study area covers more than 2,150 square miles and comprises parts of 25 counties in New Jersey and New York. During coastal storms, storm surges are generated on the open coast and propagate through New York Harbor or through the Long Island Sound and flood the extensive low-lying areas surrounding the metropolitan area.

The NYNJHAT Study alternatives encompass multiple CSRM measures such as flood walls, levees and storm surge barriers and all include, to some extent, raising existing coastal edges. Maps of the Study alternatives are included in Appendix A.

1.2 Perimeter Flood Risk Reduction vs. Coastal Barriers

1.2.1 General

A typically employed solution for reducing flood risk is to raise the level of existing perimeter flood risk reduction systems. This solution can be challenging to implement in geometrically constrained urbanized areas where waterfront spaces have multiple uses and serve a variety of stakeholders such that social and economic impacts could be considerable. In large bays, estuaries, natural harbors and port entrance channels, coastal barriers constructed as integral part of a flood risk reduction systems can be a cost-effective alternative to reduce the risk of flooding for the area. As such, the NYNJHAT Study includes evaluation of coastal barriers, in combination with other flood risk reduction systems.

Generally, four primary types of coastal barriers are identified: closure dams, storm surge barriers, tide gates and tidal locks. Closure dams permanently close off the connection between a coastal waterbody, such as an estuary or a bay, and the ocean. Closure dams eliminate the tidal connection and result in the formation of a man-made lake, effectively minimizing the chance of coastal storm surge induced floods for the area behind it. Due to the elimination of the tidal exchange, closure dams hinder navigation and introduce a steep gradient in environmental conditions across the structure, which are generally considered a substantial negative impact.

A storm surge barrier is a fully or partial movable barrier that includes operable elements (usually gates) that can be closed temporarily to impede storm surge generated by coastal storms and limit water levels in the basin, thereby reducing flood risk for low lying coastal areas within the basin. Key characteristics of a storm surge barrier are that it allows for navigation to transit the barrier,

and it maintains tidal exchange between the ocean and the newly created inner basin during normal hydrometeorological, i.e. non-storm, conditions. Some examples of storm surge barriers are the Inner Harbor Navigation Canal storm surge barrier (New Orleans, LA), the New Bedford storm surge barrier (New Bedford, MA) and the Eastern Scheldt storm surge barrier (The Netherlands). For more examples one is referred to Enclosure 2.

Tide gates[1] or tide gate complexes are generally considered to show some similarity to storm surge barriers, with the exception that they do not provide for navigation. These structures provide a barrier between the ocean and a waterbody at a location that is considered or designed to be non-navigable. In most instances, such locations are inherently shallower in depth and smaller in span compared to storm surge barriers. As such, without any clearances needed to accommodate vessels tide gates, or tide gate complexes are in general relatively smaller structures and allow for tidal flow exchange or discharge of stream flows during normal conditions. These structures include operable gates that can be closed temporarily during storm conditions to impede storm surge and limit water levels in the waterbody behind it and provide flood risk reduction. In some instances, tide gates are accompanied by a pump station that is operated in the event of gate closure to discharge streamflows from the upstream waterbody and maintain safe water levels e.g. the 17th Street Canal Closure and Permanent Pump Station complex (New Orleans, LA).

Tidal locks or tidal lock complexes[2] are structures that have the primary function to allow for the passage of vessels between tidal and non-tidal water and where operation is affected by the state of the tides. Notwithstanding the fact that some water exchange occurs with every lock cycle, these structures are generally not designed to accommodate tidal flow exchange between the ocean and inland waterbody. When constructed in areas prone to storm surge and flood risk, coastal locks can be designed to tie in to, and be an integral part of the perimeter flood risk reduction system.

1.2.2 Structural CSRM Measures as part of the NYNJHAT Study

For the alternatives currently being analyzed as part of the NYNJHAT Study, the structural CSRM measures do not include closure dams and tidal lock complexes. Due to the elimination of the tidal exchange, closure dams and tidal locks are considered not to be aligned with the planning objectives. Therefore, structural CSRM measures considered for this study include storm surge barriers, tide gate complexes, and more typical perimeter-based structural measures such as floodwalls, levees, seawalls, deployable flood barriers, etc. Non-structural measures are considered too as part of the HAT Study alternatives but not discussed herein This report only discusses storm surge barriers as detailed in the following section.

[1] In some instances, tide gates are referred to as flood gates. However, flood gate is a more general term for a flood control structure that is not necessarily situated in the coastal plain. The term can be used to describe gated structures along rivers or structures on land that close off openings in flood risk reduction systems, hence tide gate is the preferred term here to refer to a coastal barrier as described herein.

[2] In some instances, tidal locks are included within closure dams or storm surge barriers and as such "hybrid" coastal barriers that include components or characteristics of the four primary types exist.

1.3 Organization

The engineering analysis and supporting documentation for the preliminary studies completed in support of the New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study are extensive. Therefore, the engineering work products that document the analyses of the alternative structural CSRM measures have been divided into three parts. Two reports discuss the navigable storm surge barriers that are part of the five NYNJHAT Study Alternatives and one report discusses the structural CSRM measures that are the essential "building blocks" of the perimeter-based risk reduction systems. The documentation on the storm surge barriers has been presented in two separate volumes, where the first volume describes navigable storm surge barriers that exceed a span of 1,500 ft; the second volume describes the barriers smaller than 1500 ft.

This report is R002 (Volume 2) of a series of reports, listed in Table 1-1, that document the preliminary engineering studies and analyses completed in support of the feasibility of the New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study.

Table 1-1: Reports in support of the Engineering Studies and Analyses for the NYNJHAT Study Alternatives.

ID	Title	Contents/Subject
R001	Conceptual Design for NYNJHAT Study Storm Surge Barriers – Volume 1	Conceptual Design for the following Storm Surge Barriers: Outer Harbor, Verrazano Narrows, Arthur Kill, Kill van Kull, Jamaica Bay, Hackensack River and Throgs Neck.
R002	Conceptual Design for NYNJHAT Study Storm Surge Barriers – Volume 2	Conceptual Design for the following Storm Surge Barriers: East Chester Creek, Westchester Creek, Bronx River, Flushing Creek, Newtown Creek, Gowanus Canal, Sheepshead Bay and Gerritsen Creek.
R003	Shore Based Measures	The Structural Coastal Storm Risk Management (CSRM) measures evaluated as part of the NYNJHAT Study other than the storm surge barriers.

1.3.1 Report Purpose and Reader's Guide

This report documents the general design criteria and first conceptual designs for nine (9) navigable storm surge barriers, which are part of the storm surge barrier alternatives currently considered under the New York-New Jersey Harbor and Tributaries (NYNJHAT) Study for

Coastal Storm Risk Management. The storm surge barriers are listed in Table 1-2 and presented on a map in Figure 1-1.

The purpose of this document is to provide the preliminary engineering basis to support the evaluation and development of conceptual designs for nine (9) storm surge barriers, including the minimum practical dimensions of the barrier openings and preliminarily selection of gate types. The barrier openings consist of one navigable passage, with or without auxiliary flow openings. Coincidentally the storm surge barriers discussed in this report are expected to accommodate shallow draft maritime traffic or small boats[3]. Albeit that deep-draft navigation refers to channel depths greater than 15 feet (ft) and some channels discussed herein may marginally exceed that depth, none of the channels accommodate commercial seagoing vessels. Generally, the channels do not accommodate or are not frequented by larger, more heavily laden ship traffic and as such per EM 1110-2-1613 are not considered to be deep-draft projects.

The investigations detailed within this report only document the analyses completed for the barriers, colloquially also referred to as *shallow draft storm surge barriers*, provided in Table 1-2.

Table 1-2: Shallow Draft Storm Surge Barriers - NYNJHAT Study Alternatives

Location	Study Alternative	Section within report that provides conceptual design summary
East Chester Creek (Pelham Bay) Storm Surge Barrier	2, 3A, 3B, 4	5
Westchester Creek Storm Surge Barrier	3B	6
Bronx River Storm Surge Barrier	3B	7
Flushing Creek Storm Surge Barrier	4	8
Newtown Creek Storm Surge Barrier	4	9
Gowanus Canal Storm Surge Barrier	4	10
Sheepshead Bay Storm Surge Barrier	3A, 3B, 4	11
Gerritsen Creek Storm Surge Barrier	3A, 3B, 4	12

[3] Per EM 1110-2-1206 small boats are classified as recreational craft, fishing boats, or other small commercial craft with lengths less than 100 ft (31 m).



Figure 1-1: Plan overview of all Storm Surge Barriers included within the NYNJHAT Study Area

Section 2 provides a brief overview of the Study and the locations of the storm surge barriers discussed herein. Section 3 provides a general overview of the preliminary design criteria and associated design considerations, which is subsequently used to elaborate on the conceptual design development for each storm surge barrier. Sections 5 through 12 provide a conceptual design summary of each barrier (see also Table 1-2). Recommendations for further study are documented in Section 13.

1.3.2 Scope and Limitations

The A/E has been tasked with establishing a conceptual layout and design of the storm surge barriers that are part of the Feasibility Study's alternatives. The A/E is to propose suitable gate types (e.g. flap gates, sector gates, vertical lift gates) that will accommodate navigation and other site-specific constraints. The level of detail of the proposed concepts is commensurate with that of a feasibility study. Given the size of the study area and potential number and locations of storm surge barriers, the study level is very conceptual. The A/E will define the approximate anticipated size of the gate openings, heights and general geometry of the overall storm surge barrier

structures. The number and size of the navigable gates will be based on federal navigation channel dimensions, or in accordance with USACE-NAN project-specific directions. The approximate number and size of flow gates will be based on tidal discharge requirements provided by USACE-NAN. It must be noted that the majority of the storm surge barriers presented herein span a fairly narrow waterway and may not be able to accommodate auxiliary flow gates in addition to the navigable passage. In such instances tidal flow exchange will be accommodated through the navigable passage. The objective at this stage is to provide USACE-NAN a description and early conceptual design that identifies the basis for key geometric characteristics (e.g. height, width, and depth of openings). This will enable the USACE-NAN to use a parametric cost model to estimate the approximate cost of construction of the storm surge barriers.

2 PROJECT OVERVIEW

2.1 General Description of a Storm Surge Barrier with Navigable Passage

Mooyaart and Jonkman (2017) provide general design considerations and an overview of navigable storm surge barriers based on data and design documentation review of a select set of constructed storm surge barriers throughout the world (Mooyaart & Jonkman, 2017).They also provide a general description of a storm surge barrier where a typical layout contains three elements; a gated section, a dam section and a navigable passage. A navigable passage can either be established with a lock or with a gated navigable opening. The difference is that a lock passage is usually closed during normal operational conditions and only opens for the passage of vessels; a gated navigable passage is usually open for free navigation passage and only closed during the occurrence of a storm surge event. Figure 2-1 below provides a schematic plan view of a navigable storm surge barrier. The navigable passage is schematically shown as a gated navigable opening not as a lock, since the storm surge barriers studied under the NYNJHAT Study require minimal interruptions of maritime traffic except during storm surge events. Figure 2-1 schematically shows a total of three (3) auxiliary flow gates; however the storm surge barriers discussed herein may have fewer or many more. Both navigation and tidal flow exchange can be provided through the navigable passage opening. It is further recognized that not all storm surge barriers discussed herein resemble a large civil works structure that one typically associates with the term storm surge barrier. Yet these structures are considered to be storm surge barriers, just smaller in size and of lower complexity and more akin to the Seabrook storm surge barrier complex in New Orleans, LA.

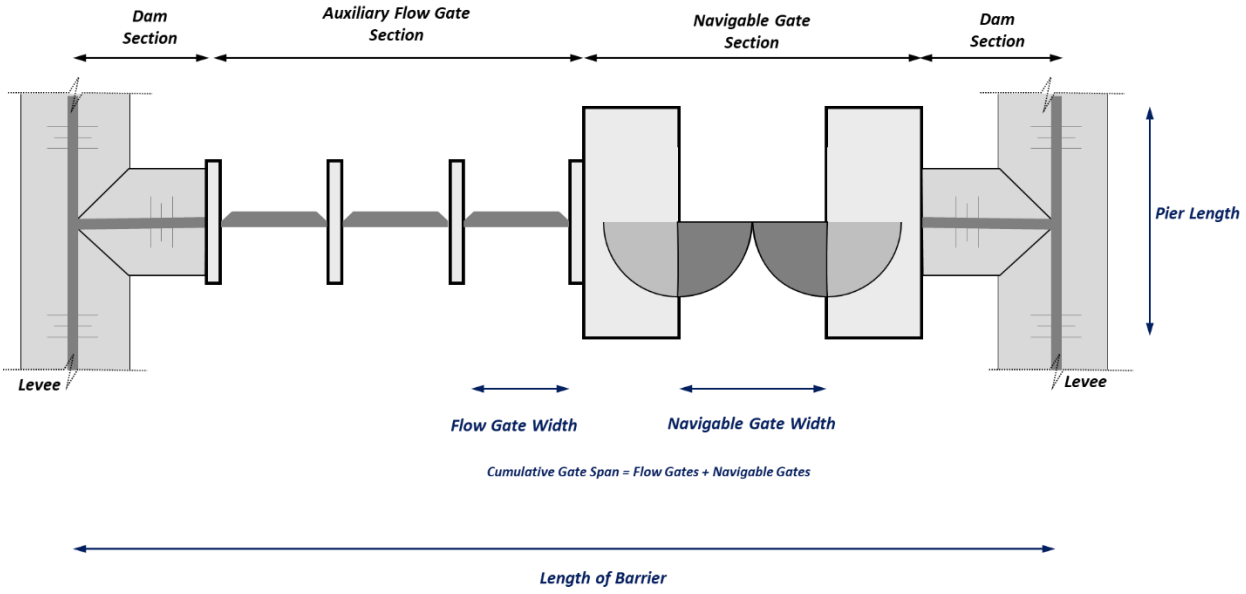


Figure 2-1: Schematic plan view of a storm surge barrier. Modified after Mooyaart and Jonkman (2017).

2.2 Locations

The following sections provide a brief overview of the locations for the shallow draft storm surge barriers discussed herein:

- East Chester Creek
- Westchester Creek
- Bronx River
- Flushing Creek
- Newtown Creek
- Gowanus Canal
- Sheepshead Bay
- Gerritsen Creek

The locations of the storm surge barriers discussed herein have been provided by USACE and are, for the most part, determined by the extent and location of the perimeter risk reduction systems that are part of the NYNJHAT Study Alternatives. Only a brief introduction is provided here. Additional maps are provided in Appendix A.

2.2.1 East Chester Creek

A navigable storm surge barrier just south of Pelham Bay Park would cross East Chester Bay at the location where East Chester Creek (otherwise known as the Hutchinson River) flows into the bay. The storm surge barrier would be located approximately 300ft south of the bridge and would span approximately 500ft across the waterbody Figure 2-2.

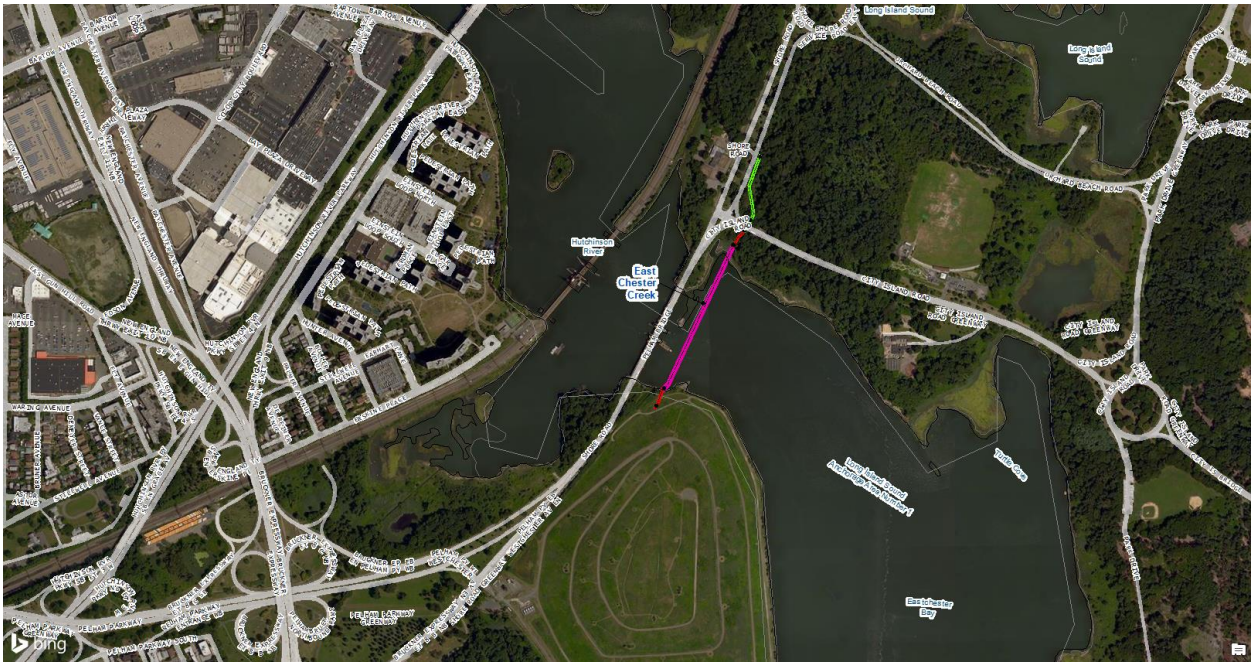


Figure 2-2: Plan view of the navigable storm surge barrier location for East Chester Creek

2.2.2 Westchester Creek

A navigable storm surge barrier in Westchester Creek would connect two reaches of land-based perimeter risk reduction systems across the creek and would be located approximately 200ft south of the Bruckner Blvd bridge. The storm surge barrier would span approximately 300ft across the water body (Figure 2-3).

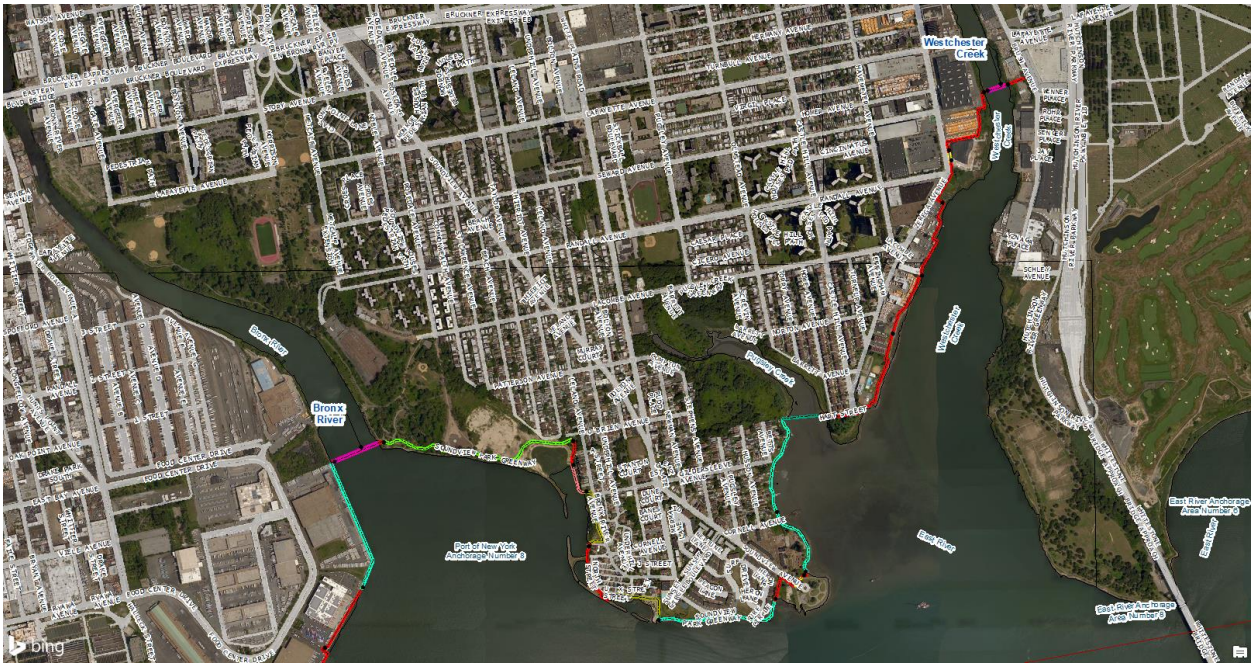


Figure 2-3: Plan view of the navigable storm surge barrier location for both the Bronx River and Westchester Creek

2.2.3 Bronx River

A navigable storm surge barrier in The Bronx River East would connect the perimeter risk reduction system across the river just to the east of the Hunts Point food market and would span approximately 200 ft (Figure 2-3).

2.2.4 Flushing Creek

A navigable storm surge barrier would cross Flushing Creek approximately 1,000 ft west of the White Stone Expressway Bridge and connect two shore-based perimeter risk reduction alignments across the creek. The storm surge barrier would span approximately 150 ft across (Figure 2-4).

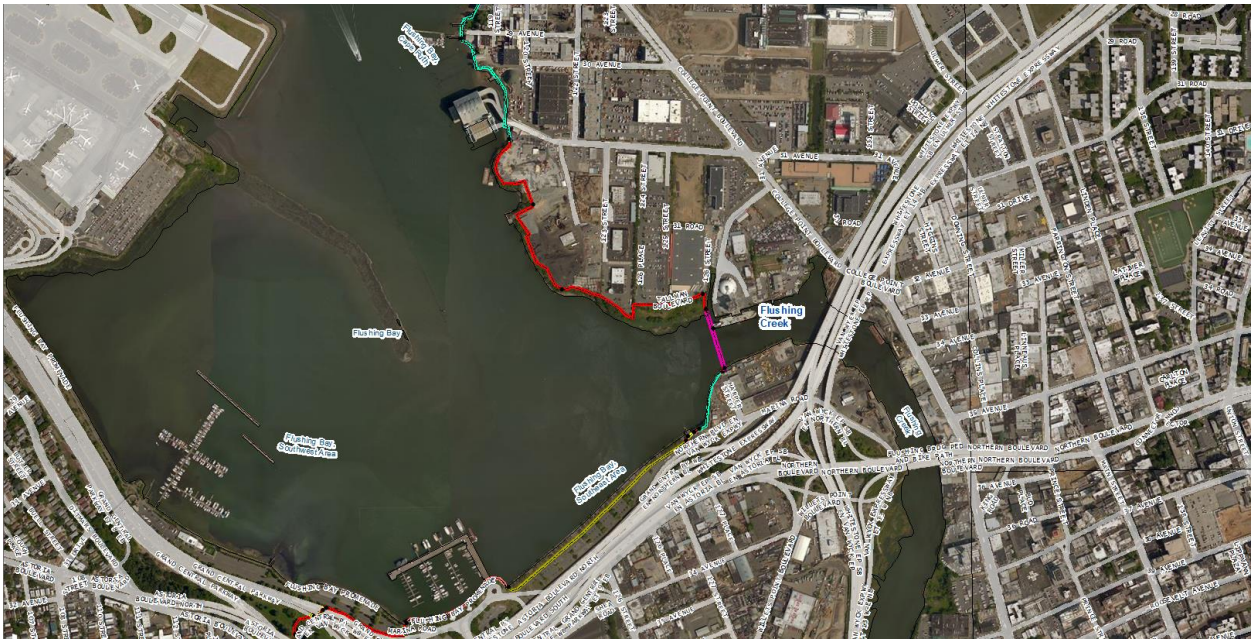


Figure 2-4: Plan view of the navigable storm surge barrier location for the Flushing Creek Storm Surge Barrier Location

2.2.5 Newtown Creek

For Newtown Creek the storm surge barrier location is informed by previous studies completed by New York City (NYC) and is proposed to have the northern tie-in located just east of the 2nd Street street-end. (CH2M Hill, 2016-B). At the proposed location the waterway is approximately 400 ft wide and 15ft deep (Figure 2-5).

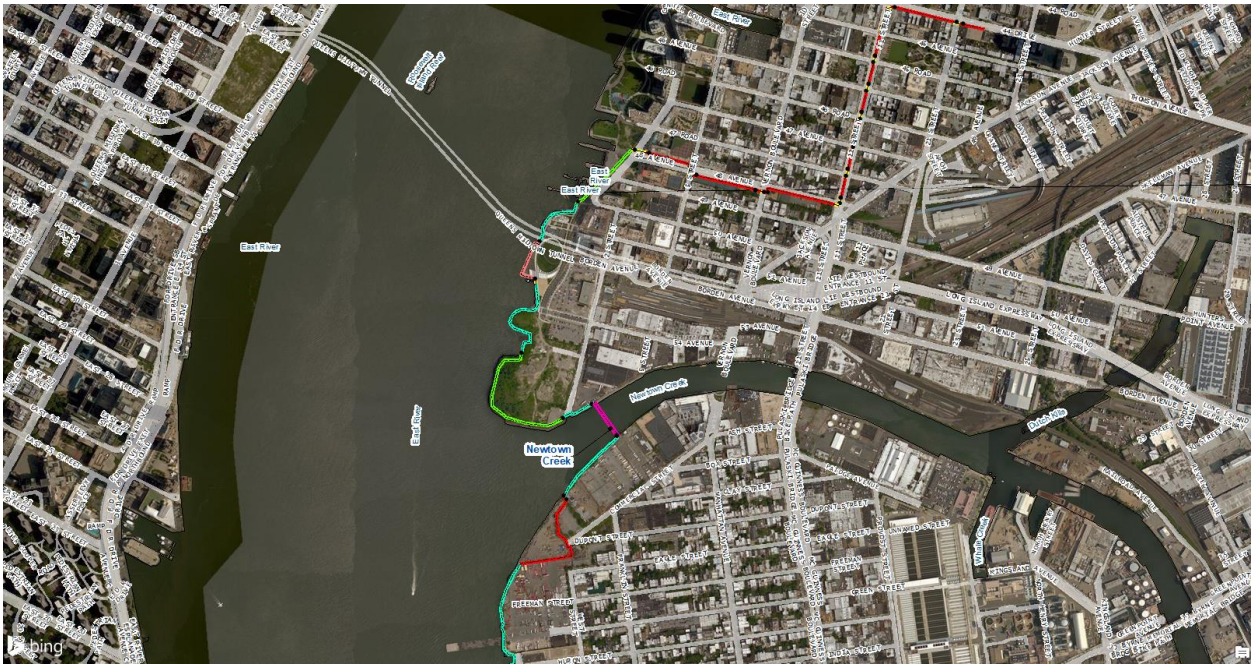


Figure 2-5: Plan view of the navigable storm surge barrier location for Newtown Creek

2.2.6 Gowanus Canal

Similar to Newtown Creek the storm surge barrier location was informed by previous studies completed by New York City (NYC) and is proposed to be located just south of the Hamilton Avenue Bridge (CH2M Hill, 2016-A). The barrier would span the canal which is approximately 100ft wide at that location, and connect the perimeter/land based flood risk reduction system on either side of the canal (Figure 2-6.

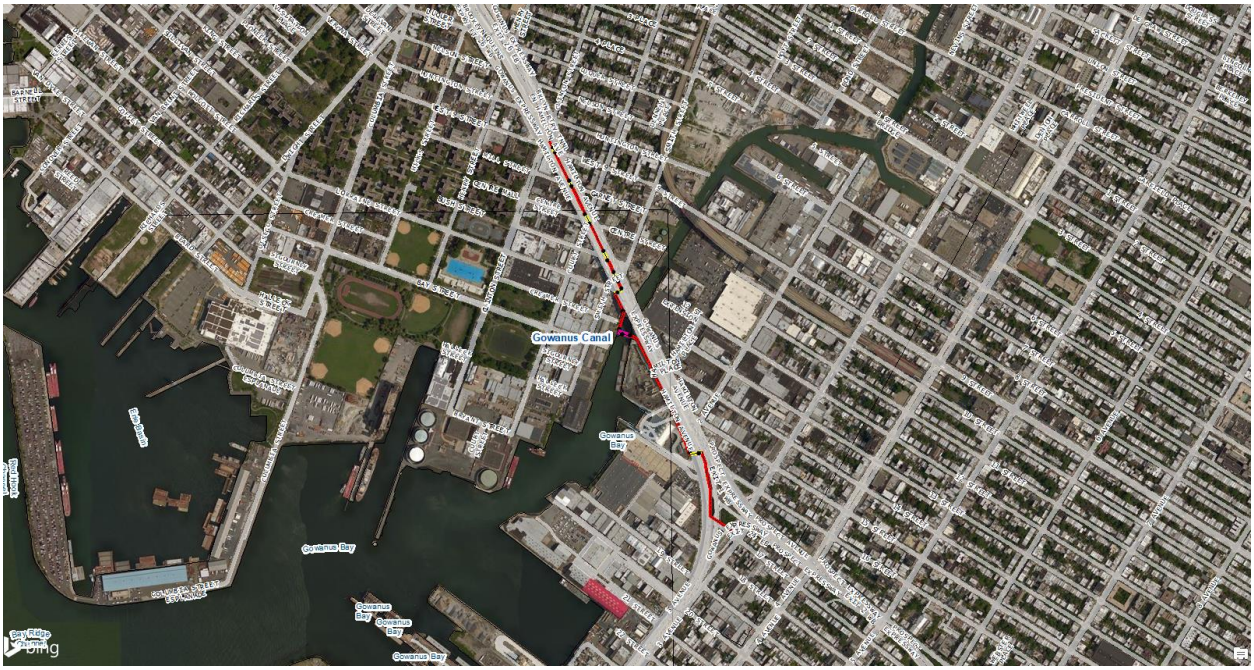


Figure 2-6: Plan view of the navigable storm surge barrier location for Gowanus Canal

2.2.7 Sheepshead Bay

For Sheepshead Bay a navigable storm surge barrier is proposed at the entrance of the bay to connect the perimeter flood risk reduction alignment on Seawall Avenue on the south side to Brigham Avenue on the North side (Figure 2-7). This navigable storm surge barrier would span approximately 700 ft. and existing depths are up to 20ft below MLLW (Figure 2-7).

2.2.8 Gerritsen Creek

Just to the east of Sheepshead Bay, flood risk reduction for Gerritsen Creek would be provided with another relatively small shallow draft navigable storm surge barrier that would be located just south of the Belt Parkway Bridge (Figure 2-7). The span from shore to shore is 250ft and existing depths are exceeding 15 ft up to 20ft below MLLW at the centerline of the creek (Figure 2-7).

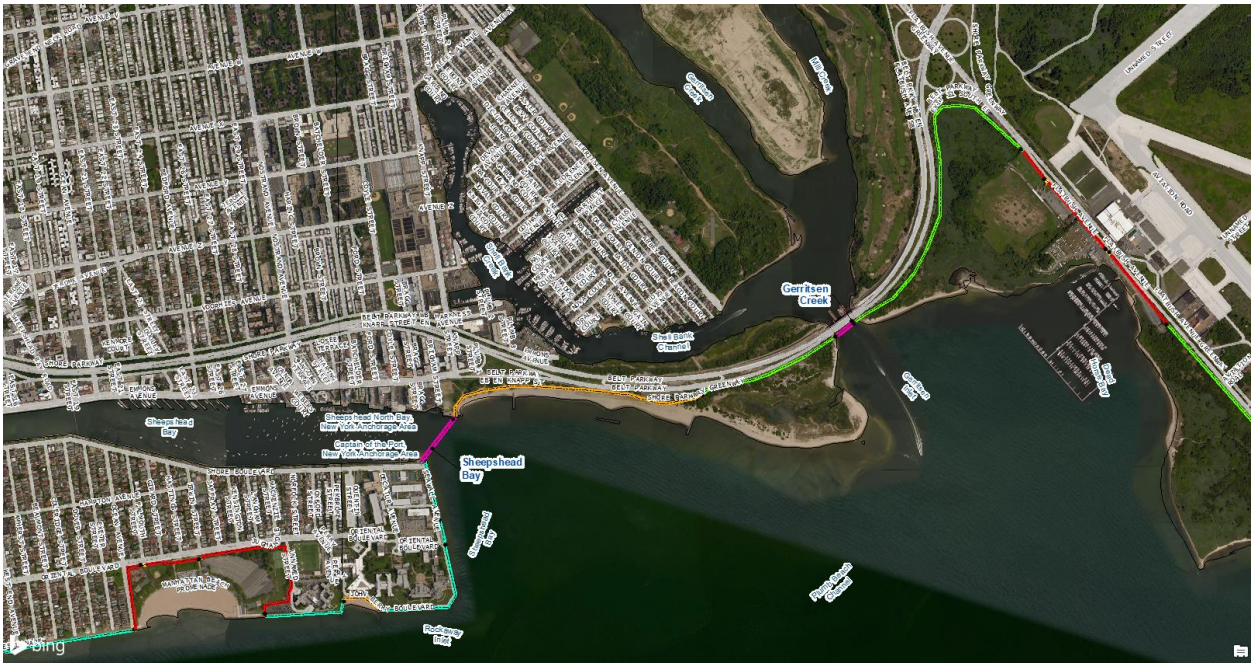


Figure 2-7: Plan view of the storm surge barrier location for both Sheepshead Bay and Gerritsen Creek

2.3 Prior Studies and Reports

There are several visioning studies, reports and presentations that have addressed the concept of storm surge barriers for the larger New York Metropolitan Area and these have been used to inform the data presented in Volume 1 and well as the data presented herein. Amongst other relevant reports and publications include the numbered items below.

1) L. Smith (2005), Closing the Doors on Storm Surge, Coastlines, vol. 34, no. 1, pp. 6-7. (Smith, 2005)
2) Bowman et al. (2004) Hydrologic Feasibility of Storm Surge Barriers to Protect the Metropolitan New York – New Jersey Region – Summary Report. (Bowman, et al., 2004)
3) Bowman, M., Hill, D., Buonaiuto, F., Colle, B., Flood, R., Wilson, R., Hunter, R. and Wang, J. (2008) 'Threats and responses associated with rapid climate change in Metropolitan New York', in M. McCracken, F. Moore, J. C. Topping (Jr.) (eds) *Sudden and Disruptive Climate Change: Exploring the Real Risks and How We Can Avoid Them*, Earthscan, London pp119–142 (Bowman, et al., 2008)

Papers and reports that provide an overview of gate types utilized in storm surge barriers and/or address the general approach to assess the feasibility of storm surge barriers in particular settings include:

4) PIANC. 2006. *Design of Movable Weirs and Storm Surge Barriers.* Brussels, Belgium: PIANC. (PIANC 2006)

5) Mooyaart, Leslie F, Sebastiaan N Jonkman, Peter A. L. de Vries, Ad van der Toorn, and Mathijs van Ledden. 2014. "Storm Surge Barrier: Overview And Design Considerations." Edited by Patrick J. Lynett. *Coastal Engineering Proceedings.* Seoul, Korea: Coastal Engineering Research Council. 808. (L. F. Mooyaart, et al. 2014)

6) Mooyaart L.F., Jonkman S.N., Overview and Design Considerations of Storm Surge Barriers. ASCE Journal of Waterway, Port, Coastal, and Ocean Engineering, Vol. 143, Issue 4. (Mooyaart & Jonkman, 2017)

7) van Ledden, Mathijs, A.J. Lansen, H.J. de Ridder, and B. Edge. 2012. "Reconnaissance level study Mississippi Storm Surge Barrier." Edited by Patrick Lynett and Jane McKee Smith. *Proceedings of 33rd Conference on Coastal Engineering.* Santander, Spain: Coastal Engineering Research Council. (van Ledden, et al. 2012)

Prior studies and reports exist which investigate storm surge barriers at Gowanus Canal, Newtown Creek, Sheepshead Bay and Gerritsen Creek. These documents have been used to inform the data presented herein.

- CH2M Hill, 2016-A, Gowanus Canal Storm Surge Barrier Study, Final Report. Submitted to the New York City Economic Development Corporation and the Mayor's Office of Recovery and Resilience, January 2016. (CH2MHill, 2016a)
- CH2M Hill, 2016-B, Newtown Creek Storm Surge Barrier Study, Final Report. Submitted to the New York City Economic Development Corporation and the Mayor's Office of Recovery and Resilience, January 2016. (CH2MHill, 2016b)
- USACE-NAN (2016). Atlantic Coast of New York, East Rockaway Inlet to Rockaway Inlet and Jamaica Bay. Draft Hurricane Sandy General Reevaluation Report and Environmental Impact Statement. Engineering Appendix A – 2, August 2016. (USACE, 2016)

3 GENERAL DESIGN CRITERIA – ABBREVIATED

As part of advancing a conceptual design for the storm surge barriers discussed herein, a number of criteria need to be established and defined, quantitatively where possible or qualitatively otherwise. Due to the preliminary nature of this feasibility study the following criteria should not be seen as comprehensive or complete. Instead the requirements and criteria form the basis for an iterative design approach of which the feasibility study and conceptual design are the first phase. Important assumptions are highlighted that influence the conceptual designs decisions and where possible, a discussion is included for issues that need to be addressed as the designs advance.

The listed design criteria are based on qualitative data and desktop analysis to a level of detail commensurate with a feasibility study. In instances where limited data was available, assumptions were made based on engineering judgement, previous experience and/or the partial data that has been collected over the course of the feasibility study phase. The implications of such assumptions along with recommendations for further data collection and refined analyses to support the design are described at the end of this report.

3.1 Available Data

A review of meteorological and oceanographic conditions at the study sites was performed to provide a basis for the geometry of the structures and the evaluation and selection of the gate types. The following were investigated:

- Authorized channel dimensions
- Storm surge elevations
- Wave climatology
- Local wind conditions
- Discharge Regime and Tidal Prism

Existing conditions for the study site were developed primarily from available data supplied by USACE and readily available public data. These data included:

- Tidal data
- Wind data
- Geological information to the extent available
- Coastal Hazard System Data (NACCS storm surge and wave modeling)

3.2 System of Units and Reference

US customary units shall be used.

The vertical datum for the project shall be NAVD88, Geoid 12B. All elevations throughout the report are referenced to NAVD88 Geoid12B unless otherwise stated. The horizontal datum shall be the North American Datum of 1983 (NAD83) State Plane.

3.3 Service Life

The storm surge barriers have various project components for which Life Cycle Design should be considered (ER 1110-2-8159). At this stage of the project (feasibility study) no such analysis has been performed. A minimum project service life of 100 years is preliminarily recommended as a result of the size and nature of the project. For certain project elements a shorter service life may be feasible.

For the storm surge barrier to meet the functional requirements regarding flood risk reduction (see Section 3.3) a period of analysis has been established that is shorter than the recommended project service life. The project will perform to meet the design criteria related to flood risk reduction in this document for a 50-year period spanning the years between 2030 and 2080. The project is to be designed for sea level rise, regional subsidence, and local settlement occurring for 50 year planning horizon (to the year 2080). After such a time, to achieve the same level of risk reduction, the structures may have to be modified or improved (i.e. adaptive management may be necessary or structural improvements may be needed if the observed sea level rise exceeds the planning criteria). Design provisions will be required to accommodate such improvements if and when needed.

3.4 Basic Functionality Requirements

The following basic functional requirements have been identified for the conceptual design of the storm surge barriers consistent with the overall objectives of the NYNJHAT Feasibility Study.

- The storm surge barriers will provide a reliable structural function as part of the NYNJHAT Study Alternatives to reduce the risk of coastal storm damage to the protected coastal region;
- The storm surge barriers will minimize impact to navigation and waterborne commerce;
- The storm surge barriers will minimize impact on the water exchange through the opening during normal operation (non-storm conditions) in order to minimize impacts on the inner basin environmental conditions; and
- The storm surge barriers will minimize the impact on upstream and downstream water levels during normal operation.

As a result of the above requirements the storm surge barrier concept will at its simplest consist of two principal parts; 1) moveable gates (both navigable gates and flow control gates) with associated and a supporting static gate support structures and 2) substructure and tie-in structures. For any system of movable gates, including navigable gates and flow control gates, there are three primary basic functional requirements:

- The moveable gates of the storm surge barrier will be able to open and close with a high degree of reliability;
- The gates will be sized and provide a range of motion suitable for purpose (e.g. navigation, flow conveyance); and

- The moveable gates and gate structures will be an integral part of the overall storm surge barrier structure and be designed such that they impede the coastal storm surge water levels and minimize the risk of coastal storm damage to the region behind it.

3.5 Site Conditions

3.5.1 Federally Authorized Channels

The majority of the shallow draft barriers proposed in this report cross federally authorized navigation channels. Channel dimensions are derived from the following resources. The barrier locations and authorized channel dimensions are provided in Table 3-1.

- Nautical Charts, National Oceanic and Atmospheric Administration, 2018.
- Controlling Depth Reports and Surveys, USACE-NAN, 2018.
- Project Maps, Rivers & Harbors, Navigation Projects, USACE-NAN, 1986.

A federal navigation channel exists at all storm surge barriers discussed herein, except for Gerritsen Creek.

Table 3-1: Federal Navigation Channels at Storm Surge Barrier Locations

	Federal Navigation Project			
Storm Surge Barrier Location	**Name**	**Authorized Width [ft]**	**Authorized Depth [ft MLLW]**	**Authorized Depth [ft NAVD88][1]**
East Chester Creek/Pelham Bay	East Chester Creek	200	-8	-12
Westchester Creek	Westchester Creek	60	-12	-16
Bronx River	Bronx River	100	-10	-14
Flushing Creek	Flushing Bay and Creek Reach B	135	-15	-19
Newtown Creek	Newtown Creek	130	-23	-26
Gowanus Canal	Gowanus Canal	100	-18	-21
Sheepshead Bay	Sheepshead Bay	100	-6	-9
Gerritsen Creek[2]		N/A	N/A	N/A

1. MLLW to NAVD88 datum conversions are provided for each tide gauge in Appendix CB.

2. Gerritsen Creek is not a federally authorized navigation channel.

3.5.2 Navigational Constraints – Bridges

The majority of the shallow draft barriers proposed in this report are adjacent to bridges. At these locations, the barriers' minimum navigable gate widths may be set by the horizontal clearance available at the bridge instead of the authorized channel width. In such instances a navigable passage of equal width to the bridge clearance will be provided to allow a vessel which can currently pass the existing bridge to pass the proposed storm surge barrier.

Vertical clearances vary for the bridge crossings. Barriers which are located adjacent to bascule bridges will provide unrestricted air clearance. This will allow the unobstructed passage of vessels with large air draft that transit the bascule bridge crossing. Storm surge barriers which are located adjacent to fixed bridges will provide vertical clearance greater than or equal to the fixed bridge clearance.

Bridge crossings and clearances are derived from the following two federal government resources. The locations and clearances are provided in Table 3-2.

- Nautical Charts, National Oceanic and Atmospheric Administration (NOAA, 2017)
- Project Maps, Rivers & Harbors, Navigation Projects, (USACE, 1986)

Table 3-2: Bridges in the Vicinity of the Proposed Storm Surge Barriers as Navigational Constraints

Location	Bridge Name	Bridge Type	Horizontal Clearance [ft]	Vertical Clearance[1] [ft NAVD88]
East Chester Creek	Pelham Parkway	Bascule Lift	59	Unrestricted
Westchester Creek	N/A	None	N/A	N/A
Bronx River	Bruckner Expressway	Bascule	90	Unrestricted
	Westchester Avenue	Fixed	65	14
Flushing Creek	Whitestone Expressway	Fixed	140	30
Newtown Creek	Pulaski	Bascule	170	Unrestricted
Gowanus Canal	Hamilton Avenue	Bascule	47	Unrestricted
Sheepshead Bay	N/A	None	N/A	N/A
Gerritsen Creek	Shore Parkway	Fixed	115	35

[1] Clearance is defined as the distance between 0 ft NAVD88 and the vertical obstruction. Clearance based on various Nautical Charts (NOAA, 2017) and Project Maps, Rivers & Harbors, Navigation Projects (USACE, 1986), converted from MHHW to NAVD88.

3.5.3 Bathymetry

Bathymetric profile plots for all storm surge barrier locations are provided in Appendix B.

3.5.4 Subaqueous Utilities

Buried utilities cross the project areas, including pipelines and cables. Appendix B provides plan views of these utilities based on the NOAA charts for the area surrounding the proposed storm surge barriers. Storm surge barriers are sited with consideration of avoiding utility conflicts to the extent practical. These utilities will have to be located precisely, identified, and potentially relocated prior to construction. Further site investigations are recommended at later stages of the study and phases of the design process.

3.6 Operational Hydrodynamic Conditions

3.6.1 Tidal Data

Tidal data for each location are derived from the NOAA gauges provided in Table 3-3. The tidal datums for each of these gauges are provided in the following tables Table 3-4, Table 3-5 and Table 3-6.

Table 3-3: Tidal Gauges

Location	Closest Tide Gauge	Station Number
East Chester Creek	Kings Point	8516945
Westchester Creek	Kings Point	8516945
Bronx River	Kings Point	8516945
Flushing Creek	Kings Point	8516945
Newtown Creek	The Battery	8518750
Gowanus Canal	The Battery	8518750
Sheepshead Bay	Sandy Hook	8531680
Gerritsen Creek	Sandy Hook	8531680

Table 3-4: Station 8561680, Sandy Hook

Tidal Datum	Abbreviation	NAVD88 ft	MLLW ft
Highest Observed 10/29/2012	Max Tide	9.21 (failed)	12.03 (failed)
Highest Astronomical Tide	HAT	3.78	6.60
Mean Higher-High Water	MHHW	2.41	5.23
Mean High Water	MHW	2.08	4.9
Mean Sea Level	MSL	-0.24	2.58
Mean Low Water	MLW	-2.62	0.2
Mean Lower-Low Water	MLLW	-2.82	0
Lowest Astronomical Tide	LAT	-1.37	-1.37
Lowest Observed 2/2/1976	Min Tide	-7.53	-4.71
Mean Tidal Range	MN	4.70	4.70

Table 3-5: Station 8518750, The Battery

Tidal Datum	Abbreviation	NAVD88 ft	MLLW ft
Highest Observed 10/30/2012	Max Tide	11.27 (failed)	14.04 (failed)
Highest Astronomical Tide 10/16/1993	HAT	3.58	6.35
Mean Higher-High Water	MHHW	2.28	5.05
Mean High Water	MHW	1.96	4.73
Mean Sea Level	MSL	-0.2	2.57
Mean Low Water	MLW	-2.57	0.2
Mean Lower-Low Water	MLLW	-2.77	0
Lowest Astronomical Tide 1/21/1996	LAT	-4.13	-1.39
Lowest Observed 2/2/1976	Min Tide	-7.06	-4.29
Mean Tidal Range	MN	4.53	4.53

Table 3-6: Station 8516945, Kings Point

Tidal Datum	Abbreviation	NAVD88 ft	MLLW ft
Highest Observed 10/30/2012	Max Tide	10.09 (failed)	14.25 (failed)
Highest Astronomical Tide 10/16/1993	HAT	5.5	9.66
Mean Higher-High Water	MHHW	3.64	7.80
Mean High Water	MHW	3.28	7.44
Mean Sea Level	MSL	-0.27	3.89
Mean Low Water	MLW	-3.88	0.28
Mean Lower-Low Water	MLLW	-4.16	0
Lowest Astronomical Tide 1/21/1996	LAT	-5.77	-1.61
Lowest Observed 2/2/1976	Min Tide	-8.17	-4.01
Mean Tidal Range	MN	7.16	7.16

3.6.2 Sea Level Change

Sea level change values are based on the USACE moderate scenario (ER 1100-2-8162) and were provided by USACE-NAN for three tidal gauge stations: Sandy Hook, The Battery, and Kings Point. These values are provided in Table 3-7.

Table 3-7: Sea Level Change per USACE's Moderate Scenario, in feet

Year	Station 8561680, Sandy Hook	Station 8518750, The Battery	Station 8516945, Kings Point
1992 (base year)	0	0	0
2030	0.62	0.47	0.42
2080	1.82	1.49	1.37

Sea level change values from the gauge nearest to the proposed storm surge barriers will be applied to obtain design water levels for both operational and extreme conditions.

3.6.3 Tidal Flows and Riverine Discharge

Information on tidal currents and discharges is obtained from a number of sources. Information on tidal currents is obtained from the NOAA's Center for Operational Oceanographic Products and

Services (CO-OPS) website[4]. Annual tidal current predictions are available at Station ACT3371 for the Bronx Rive, ACT3306 for East Chester Creek and ACT3361 for Flushing Creek (Table 3-8).

USACE-ERDC has analyzed the NYNJHAT Study Alternatives and the impacts on normal tidal conditions and circulation (ERDC, 2019). This modeling effort focused mainly on the impacts of the larger barriers (see Volume I) and no data could be extracted to inform some of the normal day to day flow or discharge regimes for the streams and waterbodies discussed herein, with the exception of Newtown Creek and Gowanus Canal. In addition for Newtown Creek and Gowanus Canal information on discharge was gleaned from the NYC reports where discharge frequency data is presented using; watershed, rainfall volumes, and waste water treatment plants (WWTP) capacity assumptions. Table 3-10 presents the discharge into the creeks using the 6 hour rainfall event.

Table 3-8: Current magnitudes for storm surge barrier locations based on CO-OPS Station ACT3556 (based on forecast for the year 2018)

Station			Flood		Ebb	
			Average maximum	Maximum	Average maximum	Maximum
ACT3371	Bronx River	Knot	Weak and Variable		Weak and Variable	
		Ft/s	-		-	
ACT3306	East Chester Creek	Knot	0.8	1.0	0.4	0.5
		Ft/s	1.35	1.7	0.7	0.85
ACT3361	Flushing Creek	Knot	Weak and Variable		Weak and Variable	
		Ft/s	-		-	

[4] http://tidesandcurrents.noaa.gov

Table 3-9: Depth averaged current magnitudes for storm surge barrier locations based AdH Modeling

Station			Flood		Ebb	
			Average Peak	Maximum Peak	Average Peak	Maximum Peak
T5	Newtown Creek (without barrier in place)	Knot	0.55	0.93	0.39	0.70
		Ft/s	0.93	1.57	0.67	1.19
T4	Gowanus Canal (without barrier in place)	Knot	0.15	0.37	0.12	0.25
		Ft/s	0.25	0.62	0.21	0.42

Table 3-10: Discharge based on rainfall volume (6-hour rainfall storm event) for Gowanus Canal (CH2MHill, 2016a) and Newtown Creek (CH2MHill, 2016b).

Location	Frequency	Average Return Period [years]	Rainfall volume MGD (6hrs)	Discharge (cfs)
Newtown Creek	1	1	315	485
	0.1	10	638	982
	0.01	100	1,022	1,572
Gowanus Canal	1	1	73	112
	0.1	10	150	231
	0.01	100	240	369

No information on current magnitudes or discharges are readily available for Sheepshead Bay, Gerritsen Creek or Westchester Creek.

3.7 Extreme Hydrodynamic Conditions

3.7.1 Extreme Water Levels

The table below provides an overview of the extreme water levels at the barrier locations.

Table 3-11: AEP Still Water Levels (50% confidence limit) from NACCS ADCIRC output, inclusive of Sea Level Rise per the site specific stations as presented in 3.6.2. Specific output nodes referenced in table

AEP	ADCIRC Output Station	Return Period	Still Water Level (1992 MSL)	Still Water Level (2030 Sea Levels)	Still Water Level (2080 Sea Levels)
		years	ft, NAVD88	ft, NAVD88	ft, NAVD88
East Chester Creek					
1%	12997	100	13.1	13.6	14.5
0.2%	12997	500	16.8	17.2	18.1
0.1%	12997	1000	18.5	18.9	19.9
Westchester Creek					
1%	13013	100	13.1	13.5	14.5
0.2%	13013	500	16.8	17.2	18.2
0.1%	13013	1000	18.5	18.9	19.9
Bronx River					
1%	13035	100	13.3	13.7	14.6
0.2%	13035	500	17.0	17.4	18.3
0.1%	13035	1000	18.7	19.1	20.1
Flushing Creek					
1%	13059	100	13.2	13.6	14.6
0.2%	13059	500	16.8	17.2	18.2
0.1%	13059	1000	18.6	19.0	19.9
Newtown Creek					
1%	13898	100	11.0	11.4	12.4
0.2%	13898	500	14.6	15.1	16.1
0.1%	13898	1000	16.2	16.7	17.7
Gowanus Canal					
1%	11862	100	11.3	11.7	12.8
0.2%	11862	500	15.0	15.5	16.5
0.1%	11862	1000	16.7	17.2	18.2
Sheepshead Bay					
1%	11967	100	9.9	10.6	11.8
0.2%	11967	500	13.1	13.7	14.9
0.1%	11967	1000	14.6	15.2	16.4

Gerritsen Creek					
1%	14085	100	9.9	10.5	11.7
0.2%	14085	500	13.1	13.7	14.9
0.1%	14085	1000	14.6	15.2	16.4

3.7.2 Extreme Wave Heights and Periods

The table below shows the wave height and period for a 1% and 0.2% AEP event for each barrier location.

Table 3-12: AEP Wave Characteristics (50% confidence limit)

AEP	ADCIRC/STWAVE output Station	Return Period	Wave Height (Hs)	Peak Period (Tp)
		years	Ft.	sec
		East Chester Creek		
1%	12997	100	3.9	3.5
0.2%	12997	500	4.7	4.5
		Westchester Creek		
1%	13013	100	3.2	2.6
0.2%	13013	500	3.6	3.1
		Bronx River		
1%	13035	100	3.0	2.3
0.2%	13035	500	3.3	2.6
		Flushing Creek		
1%	13059	100	3.3	2.6
0.2%	13059	500	3.5	2.9
		Newtown Creek		
1%	13898	100	3.7	3.2
0.2%	13898	500	4.1	3.7
		Gowanus Canal		
1%	11862	100	3.7	3.1
0.2%	11862	500	4.0	3.6
		Sheepshead Bay		
1%	11967	100	5.2	5.2
0.2%	11967	500	5.6	5.7
		Gerritsen Creek		
1%	14085	100	4.0	3.6
0.2%	14085	500	4.6	4.4

3.7.3 Wave Overtopping Criteria

For the storm surge barriers in the NY/NJ HAT Feasibility Study an overtopping criterion of 50 l/s/m is set to determine the structure height. This criterion is applied at the end of the project planning horizon (year 2080).

3.7.4 Design Crest Elevation

The storm surge barriers are conceptually designed to meet the functional criteria (impede storm surge) over the entire planning horizon (2030 – 2080). Based on the provided 1% AEP hydrodynamic characteristics for the year 2080 (extreme water levels and waves) and given the provided overtopping criterion, USACE-NAN has set the storm surge barrier design crest elevation for the storm surge barriers discussed herein, shown in Table 3-13.

Table 3-13: Crest Elevations of Storm Surge Barriers

Storm Surge Barrier	Design Crest Elevation (NAVD88)
East Chester Creek (Pelham Gate)	+19
Westchester Creek	+18
Bronx River	+19
Flushing Creek	+18
Newtown Creek	+17
Gowanus Canal	+17
Sheepshead Bay	+21
Gerritsen Creek	+18

3.7.5 Rising Water Levels After Barrier Closure

3.7.5.1 Rise in Protected Side Water Levels

During storm conditions, when the storm surge barrier is closed, the water levels on the protected side could rise as a result of wind setup, wave overtopping, river discharge, rainfall, leakage (permissible flow through the storm surge barrier when closed). This scenario has not yet been analyzed and such data will need to be quantified at a later date.

3.7.5.2 Rise in Flood Side Water Levels

Flood side water levels could increase, compared to the situation without the storm surge barrier in place, as a result of the storm surge barrier closure. This scenario has not yet been analyzed and such data will need to be quantified at a later date.

3.8 Wind Data

Wind data for each location are derived from the wind gauges in closest vicinity to the storm surge barrier location and are provided in Table 3-14. The wind roses and statistical data for each of these gauges are provided in Appendix D.

Table 3-14: Wind Gauges

Location	Closest Wind Gauge	Station Identification
East Chester Creek	LaGuardia Airport	KLGA
Westchester Creek	LaGuardia Airport	KLGA
Bronx River	LaGuardia Airport	KLGA
Flushing Creek	LaGuardia Airport	KLGA
Newtown Creek	LaGuardia Airport	KLGA
Gowanus Canal	JFK Airport	KJFK
Sheepshead Bay	JFK Airport	KJFK
Gerritsen Creek	JFK Airport	KJFK

3.9 Geotechnical Conditions

The selection of foundation types for the storm surge barriers will depend on the soil conditions and allowable capacities of geological formations at each location. In addition, the design loads from the gated structure, including overturning moment, uplift, and global stability will influence the final design of each barrier.

Limited site-specific data are available at this time and it is strongly recommended that through coordination with the NFS geotechnical data would be obtained for any (unrelated) projects that are in close vicinity to the proposed storm surge barrier sites. Previous studies of Gowanus Canal and Newtown Creek provide general subsurface conditions and publicly available geological maps provide an indication of subsurface conditions at East Chester Creek. For Sheepshead Bay and Gerritsen Creek the assumption is made that the geological subsurface conditions are similar to those used for the Jamaica Bay barrier. For the Bronx River and Westchester Creek no readily available data is available and an assumption of poor subsurface conditions and strata is assumed. Details and a synopsis of available geotechnical data for the storm surge barrier sites is presented in Appendix E.

4 CONCEPTUAL DESIGN DEVELOPMENT

4.1 Storm Surge Barriers – Principle Elements

The storm surge barriers will have a navigable passage to allow for vessels to pass. To complete the storm surge barrier a dam section will need to be constructed between the gated openings and the land-based flood risk reduction system. The purpose of the barrier is to impede storm surge yet maintain tidal flow exchange between the tidal waters and the upstream water body, e.g. river, creek or basin, during normal conditions. Per the introduction provided in section 2.1, the storm surge barrier may have, there where needed and not geometrically constrained, additional auxiliary flow gates to accommodate tidal flow exchange. Sections 4.2 and 4.3 provide more detail on the navigable passage and auxiliary flow gates, respectively.

At the tie-in locations of the storm surge barrier to the shore-based system, a dam section on the order of 100 ft would be needed. This dam section will be the transition between the operable gated structure and land-based perimeter flood risk reduction system.

As discussed in section 2.2, the locations of the storm surge barriers described herein have been provided by USACE and are, for the most part, determined by the extent and location of the perimeter risk reduction systems that are part of the NYNJHAT Study Alternatives. Once a location has been set the overall geometry is then dictated by the existing bathymetry, geometry of the navigable passage(s) and the auxiliary flow gates given the existing bathymetric profile and the design criteria as summarized in Section 3. Sections 4.2 and 0 provide additional detail on the navigable passage and auxiliary flow gates, respectively.

4.2 Navigable Passage

This section documents the preliminary assessment of the minimum required dimensions for the navigable passages and auxiliary flow gates of the storm surge barriers. It summarizes the design development of the minimum practical dimensions of the navigable passages of the storm surge barriers. The findings presented herein are a preliminary assessment and further refinement of the gate dimensions and gate configurations (including lay-out, number and width) is expected to occur during later stages in the design, e.g. if additional flow area is needed based on environmental considerations or if other navigation related design considerations require a different geometry of the openings, such evaluations and alternative comparisons would need to be evaluated. A summary overview is presented at the end of this section in Table 4-1.

4.2.1 Navigable Passage Dimensions East Chester Creek

The East Chester Creek storm surge barrier crosses a federal navigation channel (see Section 3.5.1) with an authorized width and depth of 200 ft and -12 ft NAVD88, respectively.

The Pelham Parkway Bridge to the west of the barrier, however, constricts this channel with a horizontal clearance of 59 ft. It is a bascule bridge, so the vertical clearance is unrestricted. There is also a railroad bridge combined with overhead power cables to the west of this bridge, which

provides a 68 ft horizontal clearance and 130 ft vertical clearance[5]. Given these constrictions in close proximity it is proposed to set the width of the navigable passage to 60 ft. The authorized project depth for the navigation channel is -12 ft NAVD88 (-8ft MLLW) yet at the storm surge barrier location the existing bathymetry exceeds the authorized depth. At this stage of design, a sill elevation of -17ft NAVD88 is proposed. This provides sufficient under-keel clearance to account for a hard bottom structure and accommodates tidal flow exchange.

4.2.2 Navigable Passage Dimensions Westchester Creek

The Westchester Creek storm surge barrier crosses a federal navigation channel (see Section 3.5.1) with an authorized width of 60 ft. Two bridges exist upstream of the proposed barrier, but those do not impose any constrictions of smaller dimensions than the authorized navigation channel.

Given the authorized channel dimensions, it is proposed to set the width of the navigable passage to 60 ft. The authorized project depth for the navigation channel is -16 ft NAVD88. At this stage of design, a sill elevation of -19ft NAVD88 is proposed. This includes a minimum additional 2ft of under-keel clearance to account for a hard bottom structure and is conservatively rounded to the nearest foot.

4.2.3 Navigable Passage Dimensions Bronx River

The Bronx River storm surge barrier crosses a federal navigation channel (see Section 3.5.1) with an authorized width and depth of 100 ft and -14 ft NAVD88 respectively.

The Bruckner Expressway Bridge upstream of the barrier constricts this channel with an estimated horizontal clearance of 90 ft. It is a bascule bridge, so the vertical clearance is unrestricted. The next bridge upstream is the Westchester Avenue Bridge, which is fixed with an estimated horizontal clearance of 65 feet and vertical clearance of 14 ft NAVD88. These bridges are however considered to be too far upstream to establish minimum dimensions of the navigable passage and the restrictions are not used to inform the preliminary geometric design of the navigable passage at this stage, but could potentially inform design refinements at a later date. The width of the navigable passage is set to 100 ft, equal to the authorized channel width and at this stage of design, a sill elevation of -17 ft NAVD88 is proposed. This includes a minimum additional 2 ft of under-keel clearance to account for a hard bottom structure.

4.2.4 Navigable Passage Dimensions Flushing Creek

The Flushing Creek storm surge barrier crosses a federal navigation channel (see Section 3.5.1) with an authorized width and depth of 135 ft and -19 ft NAVD88 respectively.

The Whitestone Expressway Bridge to the east of the storm surge barrier has a horizontal clearance of 140 ft. It is a fixed bridge with a vertical clearance of 30 ft NAVD88. It is proposed to set the

[5] In practicality 130 ft vertical clearance translates into unlimited vertical clearance for the design of the gated navigable passage of a storm surge barrier.

width of the navigable passage equal to the authorized channel dimensions to 135 ft and include a sill elevation of -21 ft NAVD88. This includes a minimum additional 2 ft of under-keel clearance to account for a hard bottom structure and is conservatively rounded to the nearest foot.

4.2.5 Navigable Passage Dimensions Newtown Creek

The dimensions and conceptual design for the Newtown Creek storm surge barrier have to a large extent been informed by the study completed by the New York City government agencies (CH2MHill, 2016b). A key detail that describes the gate geometry is repeated here verbatim. *The barrier width has implications for water quality due to its potential impact on the Creek's water exchange performance. The study has made a conservative assumption to size barrier gates equal to the span of the Pulaski Bridge, with a width of 150 to 170 feet, to minimize potential impact to water quality, the tolerance by regulatory agencies for absolutely no worsening of flushing performance could require a wider gate be used.* The authorized channel is 130 ft wide but for the aforementioned flow exchange reasons and due to the limit space available to fit additional auxiliary flow gates in, the width of the navigable passage is set to 170 ft.

Regarding depth the following is noted. *The future Superfund remediation works are unknown at this time. As a result, there is uncertainty around post-remediation depths at the Creek[6]. The level of contamination close to the Creek's mouth where the barrier could be sited is understood to be fairly low as compared to inland sections of the Creek, and significant dredging may not be required. However, where there are significant changes to the depth, additional infill and significant bulkheads may be required to elevate the channel bed in order to accommodate a more reasonable gate height.* (CH2MHill, 2016b). The recommendations from the 2016 report to set the sill elevation at -20 ft NAVD88 is utilized here to establish the conceptual design with the understanding that that recommendation was the result of a site-specific feasibility study at a higher level of detail than the feasibility analysis of storm surge barriers at a regional scale discussed herein. It should be emphasized however that the channel is authorized at -23 ft MLLW (-26 NAVD88) and as such a limitation to the authorized channel depth needs to be accepted at a later date.

4.2.6 Navigable Passage Dimensions Gowanus Canal

The dimensions and conceptual design for the Gowanus Canal storm surge barrier are to a large extent informed by the study completed by the New York City government agencies (CH2MHill, 2016a). Key details that describe the gate geometry are repeated here verbatim. *The sill level has been set at a depth of -22ft NAVD88 as to lie flush with the post-remediation bed-level[7]. The width of the barrier gate/opening is to be circa 90 ft so as to provide the same channel cross-sectional areas as the Hamilton Avenue Bridge immediately upstream of it.* The storm surge barrier location is at the upstream limit of the federally authorized channel, which has an authorized width of 100

[6] Newtown Creek is a superfund site (CH2MHill, 2016b)

[7] Gowanus Canal is a super fund site (CH2MHill, 2016a)

ft and depth of -21 ft NAVD88. The conceptual design will incorporate a width of 100 ft (conservatively rounded up from 90 ft) for the navigable passage to accommodate both flow and sufficient space to provide for safe vessel passage and a sill elevation at -22 ft NAVD88.

4.2.7 Navigable Passage Dimensions Sheepshead Bay

The Sheepshead Bay storm surge barrier crosses a federal navigation channel (see Section 3.5.1) with an authorized width of 100 ft. There are no constrictions in close proximity, so it is proposed to set the width of the navigable passage to 100 ft. The authorized project depth for the navigation channel is -9 ft NAVD88. A minimum sill elevation of -11ft NAVD88 is required which includes a minimum additional 2ft of under-keel clearance to account for a hard bottom structure. The existing depths at the storm surge barrier location well exceed this requirement and a lower sill elevation is considered to allow for additional cross-sectional area to maintain flow exchange between Jamaica Bay and Sheepshead Bay. As such, for the conceptual design a sill elevation of -20 ft NAVD88 is adopted.

4.2.8 Navigable Passage Dimensions Gerritsen Creek

The Gerritsen Creek storm surge barrier does not cross a federal navigation channel (see Section 3.5.1) and the dimensions of the navigable passage are solely informed by the existing Shore Parkway Bridge to the north of the storm surge barrier. The bridge piers constrict this channel with a horizontal clearance of 115 ft. Furthermore, it is a fixed bridge with a vertical clearance of 32 ft NAVD88. It is noted that the bridge is currently under construction, and satellite imagery appears to show a wider horizontal clearance between the new bridge piers at approximately 170 ft. However, for the purpose of the conceptual design it is assumed here that there is no appreciable change in boating traffic or size of vessels as a result of the bridge construction and that a horizontal clearance of 115 ft is sufficient. As such, it is proposed to set the width of the navigable passage to 115 ft and provide an air clearance of 35ft (conservatively rounded to the nearest increment of 5 ft account for changes as a result of the bridge construction). The depth of the navigable passage is determined by the existing bathymetric profile with the aim to maintain sufficient flow area. At this stage of design, a sill elevation of -19 ft NAVD88 is proposed.

4.2.9 Summary of Navigable Passage Dimensions

The navigable passage dimensions are determined by the federally authorized navigation channel dimensions, adjacent bridge clearances and flow exchange considerations as described above. For the majority of the storm surge barriers an unrestricted air clearance requirement is adopted because there are no limitations on vertical clearances for nearby bridges with the exception of Westchester Creek, Flushing Creek and Gerritsen Creek. A summary is provided here in Table 4-1.

Table 4-1: Navigable Passage Dimensions

Location	Minimum Practical Width of Opening [ft]	Authorized Channel Depth (ft, NAVD88)	Minimum Depth of Opening [ft NAVD88][1]	Air Clearance [ft NAVD88]
East Chester Creek	60	-12	-17	Unrestricted
Westchester Creek	60	-16	-19	48
Bronx River	100	-14	-17	Unknown
Flushing Creek	135	-19	-21	30
Newtown Creek	170	-26	-20	Unrestricted
Gowanus Canal	100	-21	-22	Unrestricted
Sheepshead Bay	100	-9	-20	Unrestricted
Gerritsen Creek	115	N/A	-19	35

Notes:

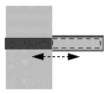
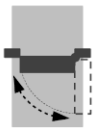
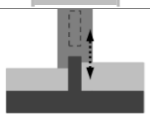
1. Sill depth is equal to the current authorized depth of channel plus two feet of under-keel clearance, conservatively rounded to the nearest foot or set at a lower depth because of flow exchange considerations (i.e. East Chester Creek, Sheepshead Bay and Gerritsen Creek), the exceptions being Gowanus Canal and Newtown Creek.

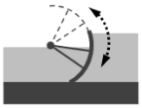
4.2.10 Navigable Gate Type Selection

Following the minimum design dimensions outlined above a suitable gate type will have to be preliminarily selected. Enclosure 2 includes an overview of the supplemental data from Mooyaart and Jonkman, 2017 which provides an overview of characteristics of constructed storm surge barriers. In addition, Enclosure 2 includes an overview of hydraulic gate types used in navigable storm surge barriers and lists general advantages and disadvantages of each gate type. Using the data set and the listed advantages and disadvantages Appendix F provides a cursory review of the suitability of each gate type for the navigable passage and a summary is presented in Table 4-2. Based on the evaluation that is provided in Appendix F a sector gate is preliminarily selected as gate type for the conceptual design of the navigable passage for the majority of the storm surge barriers discussed herein, except for Flushing Creek, Gerritsen Creek and Gowanus Canal. For the first two, a lift gate has been selected since air clearance restrictions are workable and a limited pier span is preferred such that as much flow area is maintained as possible. For Gowanus Canal

the reverse head conditions are expected to be negligible and the recommended gate type is furthermore based on the previously completed study by NYC EDC (CH2MHill, 2016a). All gate types and a generalized overview of the evaluation in Appendix F is summarized in Table 4-2.

Table 4-2: Summary of Gate Type Selection for the Navigable Passage (from Appendix F)

	GATE TYPE	Generalization of Notes and Evaluation from Appendix F	Navigable Passage where Gate Type is Selected
	Horizontal Rolling Gate	Impractical due to area needed to dock the gate	
	Sector Gate (Vertical Axis)	Deemed suitable, proven concept for similar spans and sill elevations as required for these locations.	• East Chester Creek • Westchester Creek • Bronx River • Newtown Creek • Sheepshead Bay
	Floating Sector Gate	Not suitable, too complex and likely cost prohibitive for relatively small gate spans.	
	Rotating Segment Gate	Possible, but generally deemed relatively expensive and complex system for a relatively small gate span	
	Inflatable Gate or Dam	Possible, but generally deemed relatively expensive and complex system for a relatively small gate span	
	Flap Gate	Not suitable, too complex and likely cost prohibitive for relatively small gate spans.	
	Barge Gate	Generally, possible, but challenging to operate under wave and current conditions	
	Miter Gate	Possible or suitable depending on reverse head conditions	• Gowanus Canal
	Vertical Lift Gate	Generally, deemed suitable for those locations where the air clearance is restricted. Proven concept	• Flushing Creek • Gerritsen Creek

	Vertical Rising Gate	Generally assumed to be unsuitable, related to the challenges from a maintenance perspective.
	Tainter Gate	Not suitable since either unrestricted air clearance is required or the required air clearance would prohibitively increase gate costs

The selection presented here is only preliminary and for each storm surge barrier location it is recommended to analyze, cost and compare alternate gate types at a later stage of the study. Potential alternate suitable gate types for each barrier location are documented in Appendix F.

4.3 Auxiliary Flow Gates

4.3.1 Auxiliary Flow Gate Dimensions

Following the gate type selection discussed in the previous section it is noted that only five (5) storm surge barriers have a span extensive enough to accommodate auxiliary flow gates; East Chester Creek, Bronx River, Flushing Creek, Sheepshead Bay and Gerritsen Creek storm surge barrier.

For the auxiliary flow gates, a maximum gate span of 150ft is preliminarily selected based on the review of gate characteristics as presented in (Mooyaart & Jonkman, 2017). One hundred fifty feet is considered to be a reasonable assumption, where this width falls within the gate span sizes as used in the constructed storm surge barriers. At locations where there is insufficient space to include a 150 ft wide auxiliary flow gate a smaller span can be implemented.

The sill elevation of the auxiliary flow gates is to be above the existing bed elevation such that the potential for sedimentation or siltation at the bottom of the sill is minimized. Future analysis will need to evaluate the effect that the storm surge barrier has on the hydrodynamics and morphology of the estuarine system.

When the gates are in the open position during normal day-to-day conditions, the bottom of the gates shall provide 3ft of clearance above MHHW at the end of the project service life. Positioning the bottom of the gate, when open, above this elevation will ensure that the gate will not be inundated more than needed and that flow will not be impeded by sluicing action under the gate for typical water level and operating conditions. The additional 3ft is used to account for potential wave action during normal conditions and allow for clear sight lines underneath the gate which may be needed during visual inspections. Headwall elevations are set according to the table below.

Table 4-3: Headwall Elevations

Location	MHHW (ft, NAVD88)	Sea Level Rise (ft)	Clearance (ft)	Headwall Elevation (ft, NAVD88)[1]
East Chester Creek	3.64	1.37	3	+8
Bronx River	3.64	1.37	3	+8
Flushing Creek	3.64	1.37	3	+8
Sheepshead Bay	2.41	1.82	3	+7
Gerritsen Creek	2.41	1.82	3	+7

Note:
1. Elevation = MHHW + SLR + Clearance, rounded to nearest foot.

The auxiliary flow gate only passes flow and as such the gate does not need to be raised above the elevation stated above. The design crest elevation of the storm surge barrier is provided in Section 3.7.4. To reduce the gate size, weight and overall complexity of the hoisting mechanisms, it is proposed to include a solid, non-moveable, wall between the headwall elevation and the top of the structure. This is also referred to as a headwall. While a headwall requires a fourth seal between structure and gate, since all four sides need to be sealed instead of three, the headwall reduces the overall height of the gates substantially. A gate height of 40ft would be needed to close an opening between a sill elevation of -20ft to a design elevation of +20ft, while with the use of a headwall the gate height would be 28ft (sill elevation at -20ft and top of gate at elevation +8ft). Table 4-4 summarizes the dimensions for the auxiliary flow gates.

Table 4-4: Design Dimensions for the Auxiliary Flow Gates

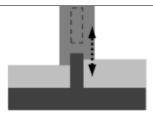
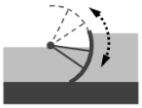
Location	Existing Depth (ft, NAVD88)	Width of Flow Gate Opening (ft)	Depth of Flow Gate Opening (ft, NAVD88)	Required Bottom of the gate in raised position (ft, NAVD88)
East Chester Creek	-15 to -20	75	-15	+8
Bronx River	-5 to -15	150	-7	+8
Flushing Creek	-5 to -20	75	-10	+8
Sheepshead Bay	-10 to -35	150	-20	+7
Gerritsen Creek	-10 to -20	50	-10	+7

4.3.2 Auxiliary Flow Gate Type Selection

Following the specifications in Table 4-2 a suitable gate type has preliminarily been selected. Appendix G provides a cursory review of the suitability of gate types for the auxiliary flow gates. Based on the evaluation that is provided in Appendix G the vertical lift gate is preliminarily selected for all of the conceptual designs of the storm surge barriers that include an auxiliary flow gate.

Gate types and a generalized overview of the evaluation in Appendix G is summarized in Table 4-5. The horizontal rolling gate, barge gate and miter gate are omitted from the table since these are the least suitable hydraulic gate options for auxiliary flow. Flap gates and vertical rising gates could potentially be utilized in some locations but are in general considered to be relatively expensive and complex systems to be used as auxiliary flow gate.

Table 4-5: Summary of Gate Type Selection for the Auxiliary Flow gates (from Appendix G)

	GATE TYPE	Generalization of Notes and Evaluation from Appendix G	Storm Surge Barrier where Conceptual Design Includes Aux. Gate Type
	Vertical Lift Gate	Suitable, proven concept	• East Chester Creek • Bronx River • Flushing Creek • Sheepshead Bay • Gerritsen Creek
	Tainter Gate	Suitable, proven concept, however reverse head conditions will need to be investigated	
	Inflatable Gate or Dam	Possible, but no proven concept for tidal inlet applications. Furthermore deemed relatively complex system.	
	Rotating Segment Gate	Possible, but no need for unrestricted air clearance, furthermore deemed relatively expensive and too complex system	

4.4 Piers and Gate Housing

The conceptual barrier design will maximize the number of auxiliary flow gates to the extent considered reasonable in order to minimize the environmental impacts and maintain as much tidal exchange as possible. The storm surge barrier locations have been set (see Section 2.2) and the dimensions of both the navigable opening and the flow gate openings have been established in the previous sections. In order to provide a conceptual geometry an assumption regarding each pier width is made. For the sector gates, each gate abutment is assumed to be, at a minimum, 65% of the span of the gate opening. For the vertical lift gate, a pier width of 30ft was conservatively assumed for lift gates that span 150 ft, and a pier width of 15 ft was assumed for spans up to 75 ft. These assumptions are based on the review of the characteristics of vertical lift gates constructed as part of storm surge barriers (Enclosure 2).

4.5 Foundations

4.5.1 Foundation Types

Three generic types of flood gate structures are being evaluated to provide flood risk reduction. These three types include sector gates, tainter gates, and vertical lift gates. Using the generic geologic and soils data presented in Appendix E, a cursory evaluation of the foundation systems has been carried out. It should be noted that this is a high level evaluation as no site specific borings are available and designs for the gates are still conceptual in nature.

The following foundation types are discussed:

- Gravity foundations with in-situ soils
- Gravity foundations with soil improvements
- Deep foundations - piles or drilled shafts

4.5.1.1 Gravity Foundation with In-Situ Soils

For study areas possessing in-situ soil conditions conducive to supporting large mat foundation structures, a gravity only foundation system may be suitable. In general, gravity type foundation systems derive their stability from the applied weight of the structure to resist sliding, bearing and overturning forces resulting from the deadweight of the structure and the applied environmental forces. In this scenario, the soils must be suitable to provide resistance to sliding and bearing failure within the soil, a function of the combined effect of both shear and compressive strength of the soil matrix. In general, the most suitable strata for gravity foundations is medium to dense sands and rock. If less desirable materials such as clay or organic silts exist at lower layers within the soil, and within the influence zone of the soil bearing pressures, then the capacity of the foundation could be significantly reduced.

A gravity foundation system is designed to address the following:

- Bearing capacity of the soils
- Sliding resistance
- Overturning stability
- Settlement

Typically, for structures such as storm surge barriers which are expected to resist significant lateral loading, such as large hydrostatic loads and dynamic wave loads, the required mass of the structure will be driven by achieving the required sliding resistance between the foundation system and the underlying soils. To this effect, different strategies may be employed to further increase the frictional resistance on the base of the structure by keying the structure into the soil to drive down the shear failure plane to within the soil matrix.

The weight of the structure must also be sufficient to ensure the structure does not overturn due to the driving lateral loads, especially significant when the driving loads are focused on the top of the

structure as is the case with storm surge barriers. Large overturning forces also contribute to increasing the soil bearing pressures under the toe of the structure, which must be checked in conjunction with the simultaneous application of the shear stresses in the soil.

Both near term and long term settlements of the foundations need to be addressed in the design, with non-uniform long term settlements due to consolidation of the weaker lower soil layers posing a significant challenge to the long term operational and maintenance requirements of the gates.

4.5.1.2 Gravity Foundation with Soil Improvements

There are instances where the desired soil conditions for a gravity based foundation exists near the surface, but are overlain by a thin layer of undesirable organic silt or clay. In these cases, the upper weak layer of soil may be removed by an appropriate dredging method and the gravity foundation either placed directly on the lower firm layer, or the removed soil is replaced with sand or gravel and the gravity based structure placed on the improved base material.

When substantially weak strata exist at lower levels in the soil profile, and within the influence zone of the soil bearing pressures, it becomes uneconomical to try to replace these soils and the foundation design reverts to a pile or drilled shaft deep foundation system, discussed hereafter.

4.5.1.3 Deep Foundations

At locations where the subsurface soils are not adequately strong to support a gravity foundation structure, or for structures which are highly dependent on controlled settlements or movements for their proper function, a deep foundation system in the form of piles or drilled shafts will be required.

Deep foundations in the form of piles or drilled shafts are designed to transfer the applied loads from the gate structure and environment to deeper soil strata, often mobilizing the distributed and additive soil strengths of multiple layers of soil into the capacity of the pile or shaft element. The difference between piles and drilled shafts are in the method of installation. Piles are structural elements driven in place, either fully displacing the surrounding soil (solid or closed ended piles) or partially displacing the soil (open ended or hollow piles). Shafts are essentially drilled piers, used commonly in bridge construction. They are deep concrete foundations installed by excavating or augering into the soil within a steel permanent or temporary casing to create a deep hole into which reinforced concrete is placed. When installed in the water, a casing is required to segregate concrete for the water; steel pipe is most often used for the casing.

Piles and shafts are designed to resist compressive, tensile and lateral forces. Depending on the subsurface soil profile, the compressive resistance is either primarily resisted by the end bearing of the pile or shaft in a deeper very hard stratum (rock or glacial till), or through the combined frictional resistance of deep layers of sand and/or clays and silts. Tensile capacity of a pile or shaft element is always a function of its frictional resistance in the underlying soils. Lateral capacity at the head of the pile is a function of the relative stiffness of the soil layers and pile structural

capacity, with larger and stiffer pile elements able to mobilize deeper layers of lateral soil resistance.

The use of deep foundation systems will reduce the reliance on the weight of the substructure to maintain stability of the structure, and as a result the substructure will not be as massive. However, the cost savings in structure mass will typically be outweighed by the added cost of the piled foundation system and the general construction method required. Gravity based structures may, in general, be able to be prefabricated offsite and transported and set into position at the gate site (following suitable seabed preparation) in the form of a float-in caisson. Structures that rely on deep foundations may be constructed in-the-dry (within a dewatered cofferdam) or may use more innovative techniques using float-in or lift-in segments placed on pre-installed deep foundation units.

Hybrid systems may also be used where the foundation base is designed to key into the soil to resist lateral loads but is supported by piles to resist bearing and overturning forces. In these cases the base may also be partially embedded below the seabed to mobilize the shear strength of the lower strata, and also the soil bearing resistance on the face of the foundation structure. In all cases, sufficient and reliable scour protection will be required to protect against significant seepage and undermining below the base of the structures which will be critical to the operation and safety of the facility.

4.5.1.4 Cellular Cofferdams and Combi-Walls

Where large sector gates are to be installed, the "gate housing structures" are more akin to the construction of small islands than to construction of a stand-alone structure, as would be the case for a vertical lift gate.

In these cases, cellular cofferdams or high-modulus sheet pile wall systems (combi-walls) may be used to construct the retaining structures for the "island" fill. Cellular cofferdams are gravity structures constructed of steel sheet piles installed around a circular template and filled with granular material to maintain their stability. Their design is based on a combination of gravity based stability, frictional resistance of the sheet piles embedded in the subsurface soils, tensile capacity generated between interlocks of adjacent sheets, and frictional resistance within interlocks. The circular sheet pile cells may be employed as the permanent perimeter walls of the "island", or as temporary structures to allow "in the dry" construction of the gate structures.

Combi-walls are large diameter steel or concrete pipe piles driven closely together with interconnecting elements to form a continuous retaining wall. The wall may be independently braced with batter piles, connected across and tied back against an opposing face, provided with a pile or sheet pile deadman anchor system, or may be free standing and cantilevered from the seabed to resist the lateral retained soil loading, depending on the water depth and loading conditions. Large capacity steel sheet piling sections may also be used for shallower water depths in lieu of combi-wall sections. As toe embedment of the retaining wall systems is critical to their stability, proper attention must be given to providing robust scour protection.

4.5.2 Preliminary Concepts for Storm Surge Barrier Foundations

These are high-level concepts based on preliminary data and require significant further study to fully vet and select an appropriate solution. Areas of further study include, but are not limited to, actual subsurface conditions, material sources, construction means and methods, construction impact on navigation and the environment, and cost. These topics are expanded upon in the final chapter of this report. Based on the limited geotechnical data, the conceptual foundation types listed below are identified as being potentially viable for the NYNJHAT Study storm surge barriers identified within this report.

- Sector gate housing, lift gate towers, sills, and foundations, miter gate recess and foundations, and dam sections:
    - deep foundation elements, either piles or drilled shafts.

At this stage this is still a high-level concept based on preliminary data and require significant further study to fully vet and select an appropriate solution. Areas of further study include, but are not limited to, actual subsurface conditions, material sources, construction means and methods, construction impact on navigation and the environment, and cost. These topics are expanded upon in the final chapter of this report.

5 EAST CHESTER CREEK CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the East Chester Creek Storm Surge Barrier.

5.1 Hydraulic Design Parameters

Table 5-1 below summarizes the key environmental criteria from Chapter 3 that determine the elevation of the conceptual storm surge barrier design.

Table 5-1: Summary of Environmental Design Criteria (East Chester Creek)

Parameter	Value
SLR for design life through 2080 (ft)	1.37
1% AEP Design Water Level (ft NAVD88) year 2080	14.5
1% AEP Significant Wave Height (ft)	3.9
1% AEP Peak Wave Period (s)	3.5
1% AEP Overtopping Criterion (l/s/m)	50

5.2 Preliminary Geometric Design

Following the considerations presented within the previous chapters, a preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 5-1. As provided in Section 4.5.2, the preliminary concept for foundations are deep foundations (piles or drilled shafts) for all sector gate housing, lift gate towers, gate sills, miter gate recesses and foundations, and dam sections. The existing cross-sectional flow area at the proposed storm surge barrier location is approximately 7,400 ft^2. The aggregated flow opening area below MSL provided by the conceptual design is 2,100 ft^2 (approximately 28% of the existing flow area at that location). For East Chester Creek it should be noted that the storm surge barrier location is just downstream of an existing constriction, i.e. at the Pelham Parkway Bridge crossing. The existing flow area at the upstream bridge location is estimated at 6,000 ft^2 and the storm surge barrier provides approximately 35% of the existing flow area.

It is proposed that this geometry will be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design.

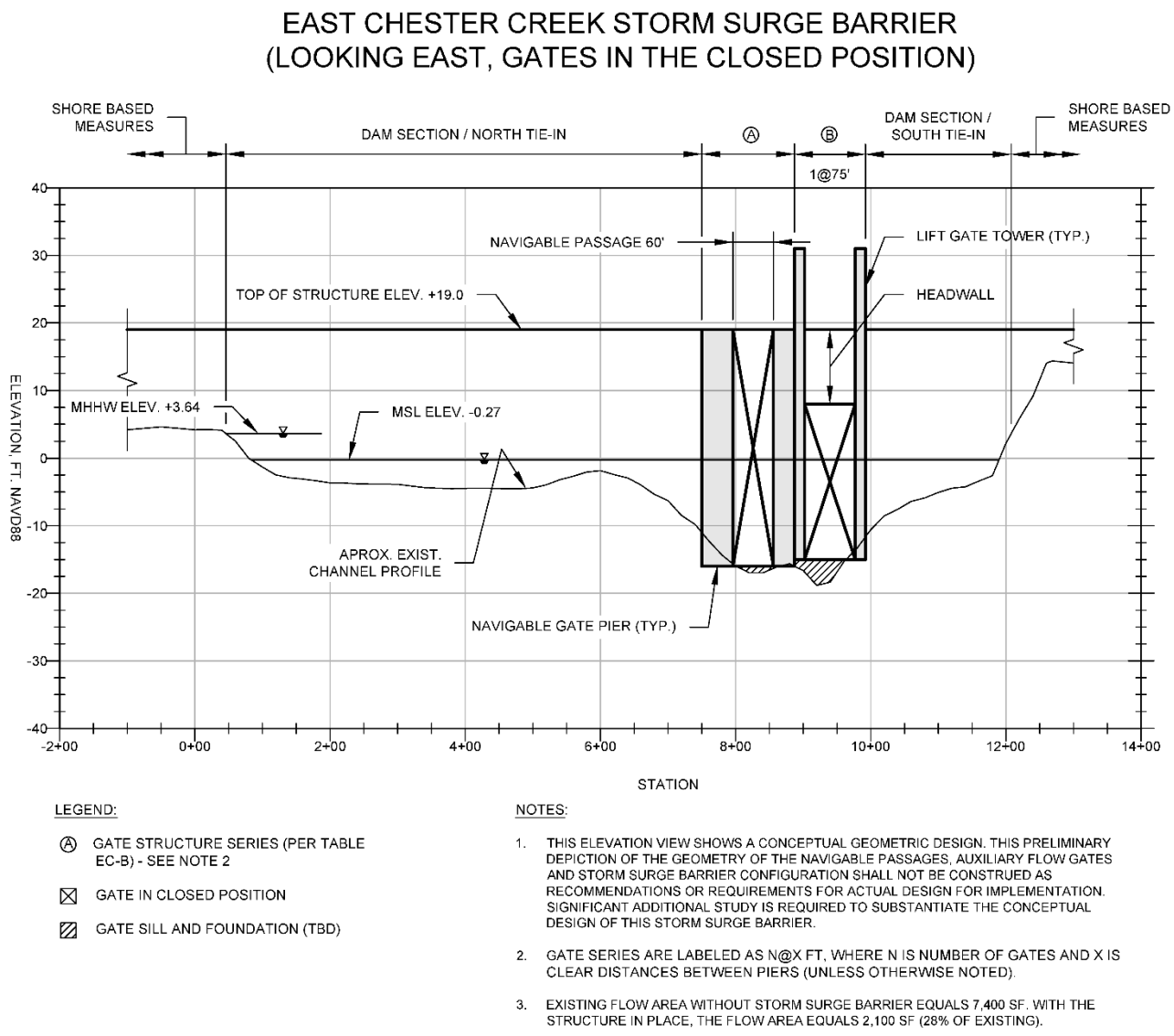


Figure 5-1: Conceptual Geometry of Gates of the East Chester Creek Storm Surge Barrier.

Table 5-2: "TABLE-EC-B": Summary of East Chester Creek Storm Surge Barrier Geometry – Navigable Passage and Auxiliary Flow Gates

Element[1] (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Notes	Top of Structure (ft, NAVD88)	Gate Height (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
A	1	-17	60	1,004	138	Navigable Passage (Sector Gate)	+19	36	N/A
B	1	-15	75	1,104	152	Auxiliary Flow Gate (Lift Gate)	+19	23	31
North Dam	0	N/A	N/A	0	705	Dam Section North Bank	+19	N/A	N/A
South Dam	0	N/A	N/A	0	215	Dam Section South Bank	+19	N/A	N/A
Total	2			2,109	1,163				

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths equal 65% of gate width and Span for auxiliary structures equals gate width plus pier widths (pier widths equal 30 ft).
3. Top of single leaf lift gate with headwall at +8ft

6 WESTCHESTER CREEK CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Westchester Creek Storm Surge Barrier.

6.1 Hydraulic Design Parameters

Table 6-1 below summarizes the key environmental criteria from Chapter 3 that determine the elevation of the conceptual storm surge barrier design.

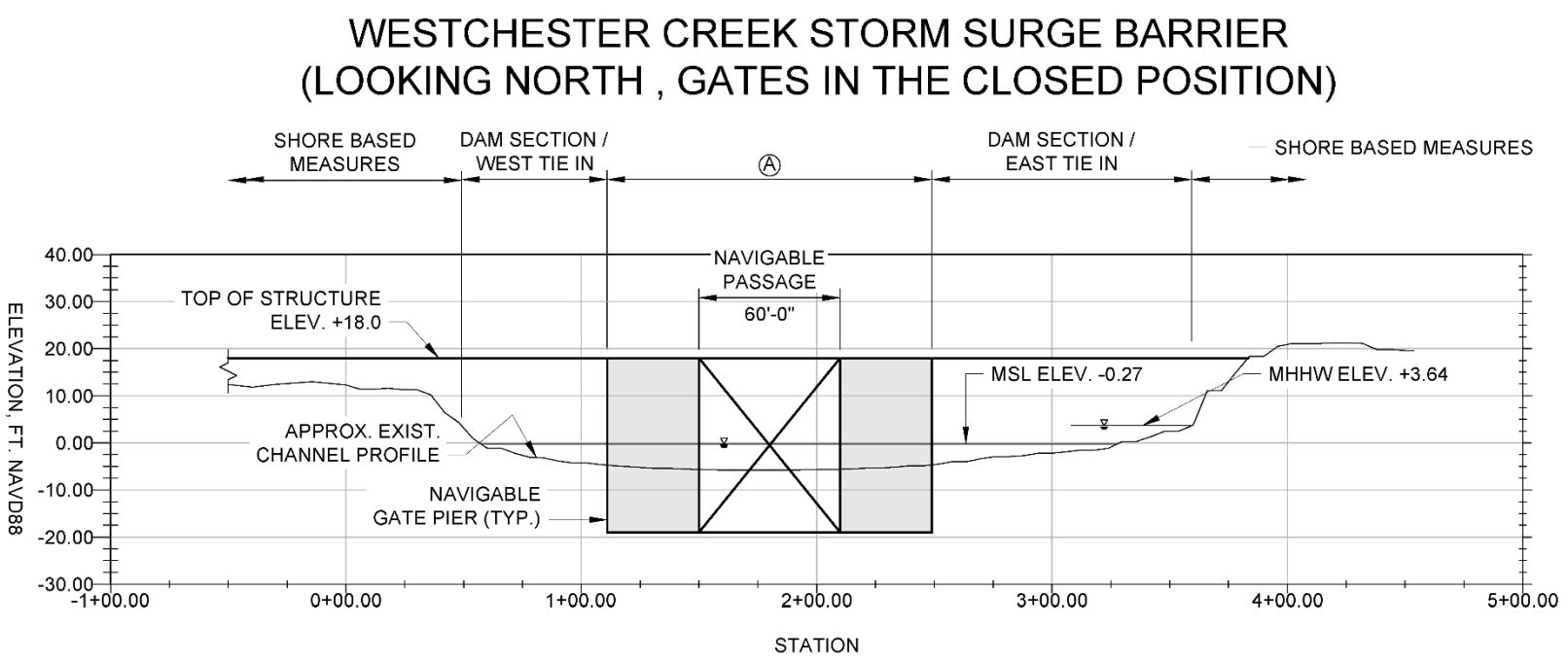
Table 6-1: Summary of Environmental Design Criteria (Westchester Creek)

Parameter	Value
SLR for design life through 2080 (ft)	1.37
1% AEP Design Water Level (ft NAVD88) year 2080	14.5
1% AEP Significant Wave Height (ft)	3.2
1% AEP Peak Wave Period (s)	2.6
1% AEP Overtopping Criterion (l/s/m)	50

6.2 Preliminary Geometric Design

Following the considerations presented within the previous chapters, a preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 6-1. As provided in Section 4.5.2, the preliminary concept for foundations are deep foundations (piles or drilled shafts) for all sector gate housing, lift gate towers, gate sills, miter gate recesses and foundations, and dam sections. The existing cross-sectional flow area below MSL at the proposed storm surge barrier location is approximately 1,100 ft^2. The aggregated flow opening area provided within the conceptual design is 1,100 ft^2 (approximately 100% of the existing flow area at that location). Due to significant shoaling in Westchester Creek, the existing channel is much shallower than the proposed sill elevation of the storm surge barrier and even if the channel would be brought back to its authorized depth prior to construction of the storm surge barrier, an analysis of long-term sedimentation impacts and need for maintenance dredging is warranted.

It is proposed that this geometry will be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design.



LEGEND:

Ⓐ GATE STRUCTURE SERIES (PER TABLE WC-B) - SEE NOTE 2.

☒ GATE IN CLOSED POSITION

NOTES:

1. THIS ELEVATION VIEW SHOWS A CONCEPTUAL GEOMETRIC DESIGN. THIS PRELIMINARY DEPICTION OF THE GEOMETRY OF THE NAVIGABLE PASSAGES, AUXILIARY FLOW GATES AND STORM SURGE BARRIER CONFIGURATION SHALL NOT BE CONSTRUED AS RECOMMENDATIONS OR REQUIREMENTS FOR ACTUAL DESIGN FOR IMPLEMENTATION. SIGNIFICANT ADDITIONAL STUDY IS REQUIRED TO SUBSTANTIATE THE CONCEPTUAL DESIGN OF THIS STORM SURGE BARRIER.
2. GATE SERIES ARE LABELED AS N@X FT, WHERE N IS NUMBER OF GATES AND X IS CLEAR DISTANCES BETWEEN PIERS (UNLESS OTHERWISE NOTED).
3. EXISTING FLOW AREA WITHOUT STORM SURGE BARRIER EQUALS 1,100 SF. WITH THE STRUCTURE IN PLACE THE FLOW AREA EQUALS 1,100 SF (100% OF EXISTING).

Figure 6-1: Conceptual Geometry of Gates of the Westchester Creek Storm Surge Barrier.

Table 6-2: "TABLE-WC-B": Summary of Westchester Creek Storm Surge Barrier Geometry – Navigable Passage

Element (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Note	Top of Structure (ft, NAVD88)	Gate Height (ft)
A	1	-19	60	1,124	138	Navigable Passage (Sector Gate)	+18	37
East Dam	0	N/A	N/A	0	110	Dam Section East Bank	+18	N/A
West Dam	0	N/A	N/A	0	65	Dam Section West Bank	+18	N/A
Total	1			1,124	313			

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths equal 65% of gate width.
3. Top of single leaf lift gate with headwall at +8ft

7 BRONX RIVER CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Bronx River Storm Surge Barrier.

7.1 Hydraulic Design Parameters

Table 7-1 below summarizes the key environmental criteria from Chapter 3 that determine the elevation of the conceptual storm surge barrier design.

Table 7-1: Summary of Environmental Design Criteria (Bronx River)

Parameter	Value
SLR for design life through 2080 (ft)	1.37
1% AEP Design Water Level (ft NAVD88) year 2080	14.6
1% AEP Significant Wave Height (ft)	3.0
1% AEP Peak Wave Period (s)	2.3
1% AEP Overtopping Criterion (l/s/m)	50

7.2 Preliminary Geometric Design

Following the considerations presented within the previous chapters, a preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 7-1. As provided in Section 4.5.2, the preliminary concept for foundations are deep foundations (piles or drilled shafts) for all sector gate housing, lift gate towers, gate sills, miter gate recesses and foundations, and dam sections. The existing cross-sectional flow area below MSL at the proposed storm surge barrier location is approximately 4,200 ft^2. The aggregated flow opening area provided within the conceptual design is 3,700 ft^2 (approximately 88% of the existing flow area at that location).

It is proposed that this geometry will be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design.

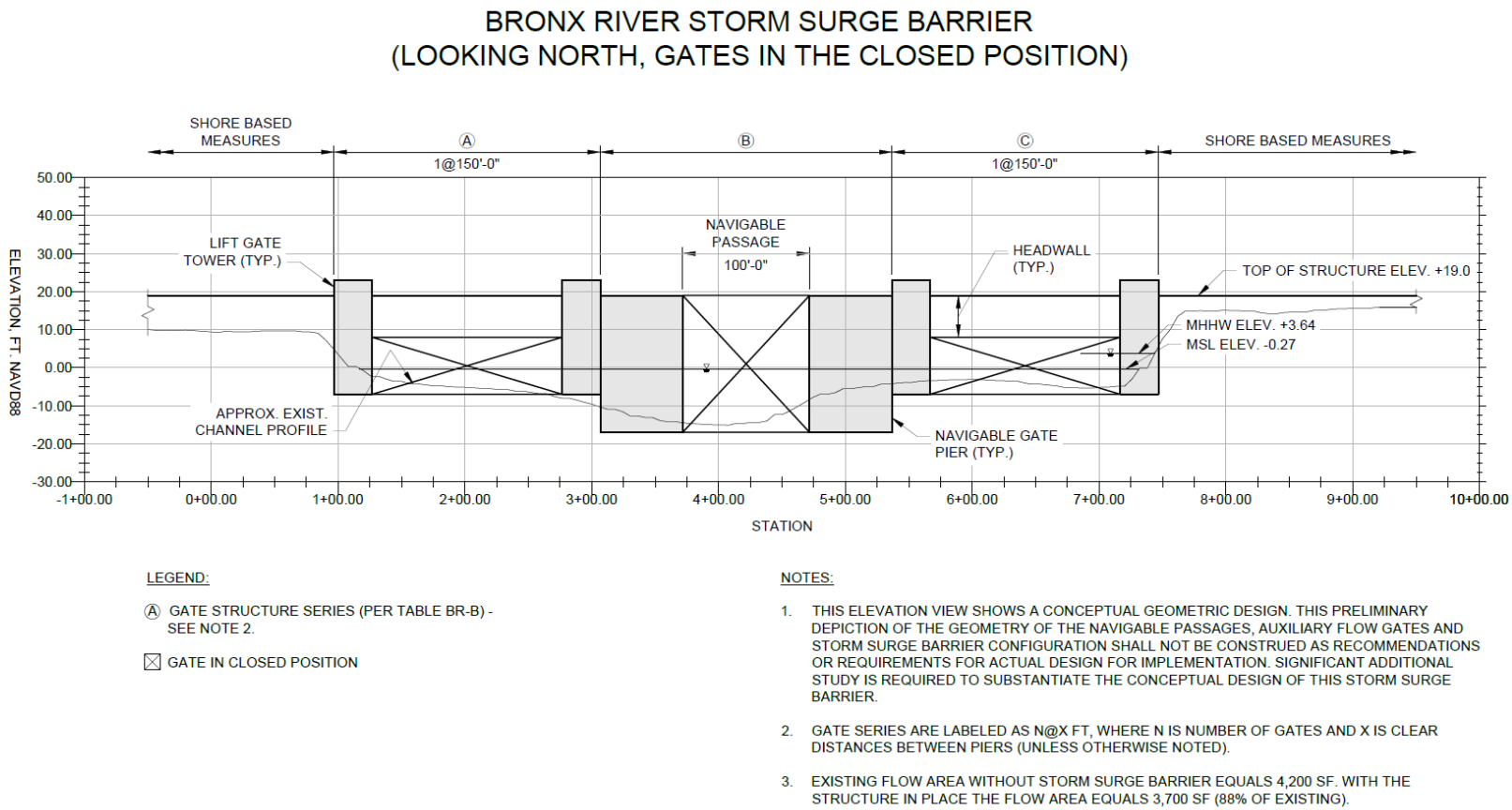


Figure 7-1: Conceptual Geometry of Gates of the Bronx River Storm Surge Barrier.

Table 7-2: "TABLE-BR-B": Summary of Bronx River Storm Surge Barrier Geometry – Navigable Passage and Auxiliary Flow Gates

Element (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Note	Top of Structure (ft, NAVD88)	Gate Height (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
A	1	-7	150	1,010	210	Auxiliary Flow Gate (Lift Gate)	+19	15	23
B	1	-17	100	1,673	230	Navigable Passage (Sector Gate)	+19	36	N/A
C	1	-7	150	1010	210	Auxiliary Flow Gate (Lift Gate)	+19	15	23
East Dam	0	N/A	N/A	0	0	Dam Section East Bank	+19	N/A	N/A
West Dam	0	N/A	N/A	0	0	Dam Section West Bank	+19	N/A	N/A
Total	3			3,693	650				

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths equal 65% of gate width and span for auxiliary structures equals gate width plus pier widths (pier widths equal 30 ft).
3. Top of single leaf lift gate with headwall at +8ft

8 FLUSHING CREEK CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Flushing Creek Storm Surge Barrier.

8.1 Hydraulic Design Parameters

Table 8-1 below summarizes the key environmental criteria from Chapter 3 that determine the elevation of the conceptual storm surge barrier design.

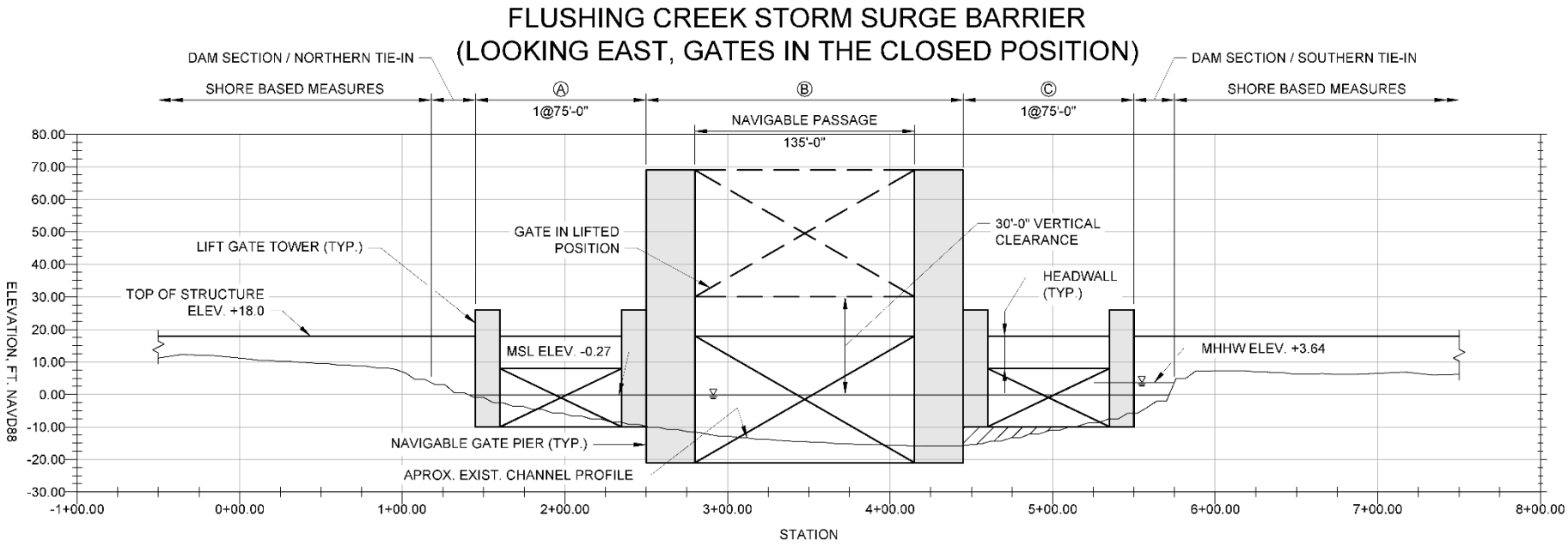
Table 8-1: Summary of Environmental Design Criteria (Flushing Creek)

Parameter	Value
SLR for design life through 2080 (ft)	1.37
1% AEP Design Water Level (ft NAVD88) year 2080	14.6
1% AEP Significant Wave Height (ft)	3.3
1% AEP Peak Wave Period (s)	2.6
1% AEP Overtopping Criterion (l/s/m)	50

8.2 Preliminary Geometric Design

Following the considerations presented within the previous chapters, a preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 8-1. As provided in Section 4.5.2, the preliminary concept for foundations are deep foundations (piles or drilled shafts) for all sector gate housing, lift gate towers, gate sills, miter gate recesses and foundations, and dam sections. The existing cross-sectional flow area below MSL at the proposed storm surge barrier location is approximately 4,500 ft^2. The aggregated flow opening area provided within the conceptual design is 4,300 ft^2 (approximately 96% of the existing flow area at that location).

It is proposed that this geometry will be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design.



LEGEND:

Ⓐ GATE STRUCTURE SERIES (PER TABLE FC-B) - SEE NOTE 2

☒ GATE IN CLOSED POSITION

▨ GATE SILL AND FOUNDATION

NOTES:

1. THIS ELEVATION VIEW SHOWS A CONCEPTUAL GEOMETRIC DESIGN. THIS PRELIMINARY DEPICTION OF THE GEOMETRY OF THE NAVIGABLE PASSAGES, AUXILIARY FLOW GATES AND STORM SURGE BARRIER CONFIGURATION SHALL NOT BE CONSTRUED AS RECOMMENDATIONS OR REQUIREMENTS FOR ACTUAL DESIGN FOR IMPLEMENTATION. SIGNIFICANT ADDITIONAL STUDY IS REQUIRED TO SUBSTANTIATE THE CONCEPTUAL DESIGN OF THIS STORM SURGE BARRIER.
2. GATE SERIES ARE LABELED AS N@X FT, WHERE N IS NUMBER OF GATES AND X IS CLEAR DISTANCES BETWEEN PIERS (UNLESS OTHERWISE NOTED).
3. EXISTING FLOW AREA WITHOUT STORM SURGE BARRIER EQUALS 4,500 SF. WITH THE STRUCTURE IN PLACE THE FLOW AREA EQUALS 4,300 SF (96% OF EXISTING).

Figure 8-1: Conceptual Geometry of Gates of the Flushing Creek Storm Surge Barrier.

Table 8-2: "TABLE-FC-B": Summary of Flushing Creek Storm Surge Barrier Geometry – Navigable Passages and Auxiliary Flow Gates

Element (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Note	Top of Structure (ft, NAVD88)	Gate Height (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
A	1	-10	75	730	105	Auxiliary Flow Gate (Lift Gate)	+18	18	26
B	1	-21	135	2799	195	Navigable Passage (Lift Gate)	+18	39	69
C	1	-10	75	730	105	Auxiliary Flow Gate (Lift Gate)	+18	18	26
North Dam	0	N/A	N/A	0	30	Dam Section East Bank	+18	N/A	N/A
South Dam	0	N/A	N/A	0	25	Dam Section West Bank	+18	N/A	N/A
Total	3			4,259	460				

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths (each) equal 30ft for navigable gate width and span for auxiliary structures equals gate width plus pier widths (pier widths equal 15 ft).
3. Top of single leaf lift gate with headwall at +8ft

9 NEWTOWN CREEK CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Newtown Creek Storm Surge Barrier.

9.1 Hydraulic Design Parameters

Table 9-1 below summarizes the key environmental criteria from Chapter 3 that determine the elevation of the conceptual storm surge barrier design.

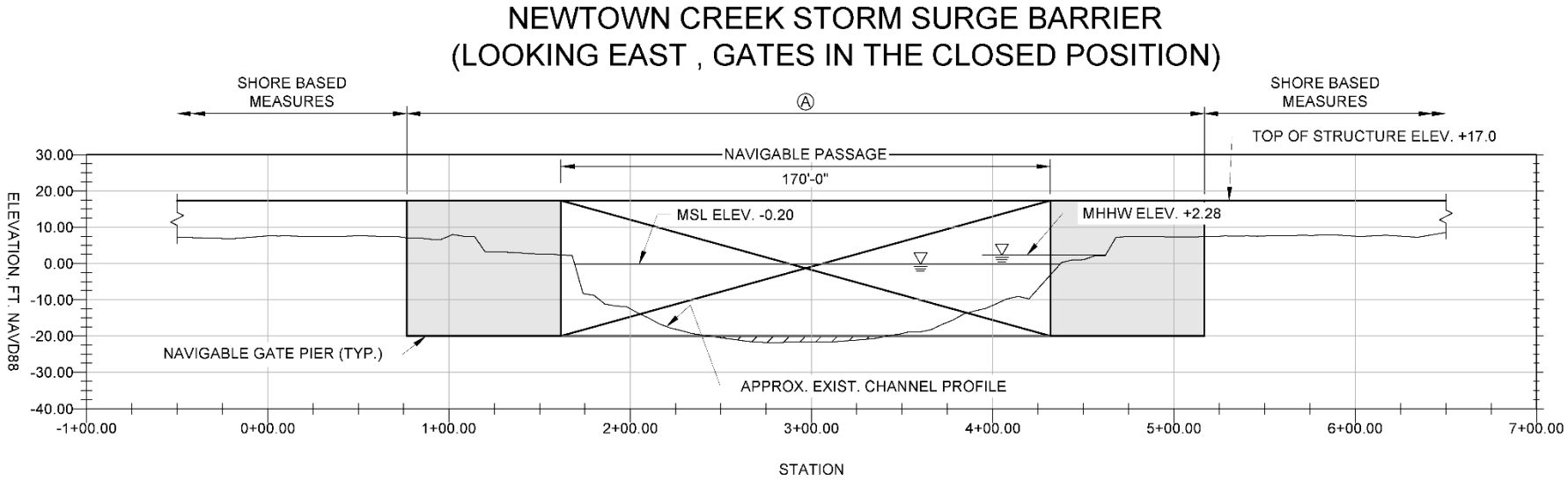
Table 9-1: Summary of Environmental Design Criteria (Newtown Creek)

Parameter	Value
SLR for design life through 2080 (ft)	1.49
1% AEP Design Water Level (ft NAVD88) year 2080	12.4
1% AEP Significant Wave Height (ft)	3.7
1% AEP Peak Wave Period (s)	3.2
1% AEP Overtopping Criterion (l/s/m)	50

9.2 Preliminary Geometric Design

Following the considerations presented within the previous chapters, a preliminary geometry for the storm surge barrier opening has been established and is shown in Figure 9-1. As provided in Section 4.5.2, the preliminary concept for foundations are deep foundations (piles or drilled shafts) for all sector gate housing, lift gate towers, gate sills, miter gate recesses and foundations, and dam sections. The existing cross-sectional flow area at the proposed storm surge barrier location is approximately 4,300 ft^2. The aggregated flow opening area provided within the conceptual design is 3,400 ft^2 (approximately 79% of the existing flow area at that location). The sector gate recesses into the existing shoreline of the creek and as such no in-water dam sections or tie-in structures are expected to be needed. The perimeter flood risk reduction features (land based) can directly tie in to the gate recess on either side of the creek.

It is proposed that this geometry will be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design.



LEGEND:

Ⓐ GATE STRUCTURE SERIES (PER TABLE NC-B) - SEE NOTE 2

☒ GATE IN CLOSED POSITION

▨ GATE SILL AND FOUNDATION

NOTES:

1. THIS ELEVATION VIEW SHOWS A CONCEPTUAL GEOMETRIC DESIGN. THIS PRELIMINARY DEPICTION OF THE GEOMETRY OF THE NAVIGABLE PASSAGES, AUXILIARY FLOW GATES AND STORM SURGE BARRIER CONFIGURATION SHALL NOT BE CONSTRUED AS RECOMMENDATIONS OR REQUIREMENTS FOR ACTUAL DESIGN FOR IMPLEMENTATION. SIGNIFICANT ADDITIONAL STUDY IS REQUIRED TO SUBSTANTIATE THE CONCEPTUAL DESIGN OF THIS STORM SURGE BARRIER.
2. GATE SERIES ARE LABELED AS N@X FT, WHERE N IS NUMBER OF GATES AND X IS CLEAR DISTANCES BETWEEN PIERS (UNLESS OTHERWISE NOTED).
3. EXISTING FLOW AREA WITHOUT STORM SURGE BARRIER EQUALS 4,300 SF. WITH THE STRUCTURE IN PLACE THE FLOW AREA EQUALS 3,400 SF (79% OF EXISTING).

Figure 9-1: Conceptual Geometry of Gates of the Newtown Creek Storm Surge Barrier

Table 9-2: "TABLE-NC-B": Summary of Newtown Creek Storm Surge Barrier Geometry – Navigable Passage

Element (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Note	Top of Structure (ft, NAVD88)	Gate Height (ft)
A	1	-20	170	3,366	340	Navigable Passage (Sector Gate)	+17	37

Notes:

1. Flow area measured from elevation MSL to sill.
4. Span for navigable structures equals gate width + pier widths. Pier widths equal 65% of gate width.

10 GOWANUS CANAL CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Gowanus Canal Storm Surge Barrier.

10.1 Hydraulic Design Parameters

Table 10-1 below summarizes the key environmental criteria from Chapter 3 that determine the elevation of the conceptual storm surge barrier design.

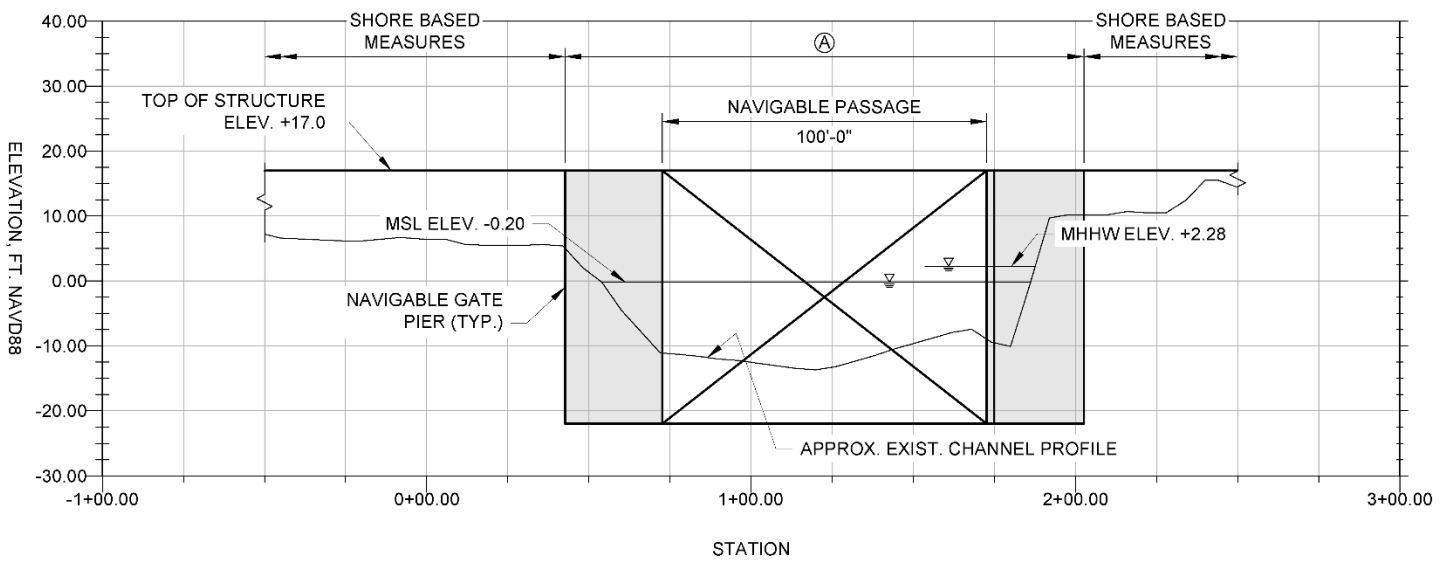
Table 10-1: Summary of Environmental Design Criteria (Gowanus Canal)

Parameter	Value
SLR for design life through 2080 (ft)	1.49
1% AEP Design Water Level (ft NAVD88) year 2080	12.8
1% AEP Significant Wave Height (ft)	3.7
1% AEP Peak Wave Period (s)	3.1
1% AEP Overtopping Criterion (l/s/m)	50

10.2 Preliminary Geometric Design

Following the considerations presented within the previous chapters, a preliminary geometry for the storm surge barrier opening has been established and is shown in Figure 10-1. As provided in Section 4.5.2, the preliminary concept for foundations are deep foundations (piles or drilled shafts) for all sector gate housing, lift gate towers, gate sills, miter gate recesses and foundations, and dam sections. The existing cross-sectional flow area below MSL at the proposed storm surge barrier location is approximately 1,300 ft^2. The aggregated flow opening area provided within the conceptual design is 2,200 ft2 (approximately 169% of the existing flow area at that location). It should be noted that because of significant shoaling in the canal, the existing canal provides less flow area than the proposed storm surge barrier would. However deepening at the barrier location as part of the Super Fund work is anticipated prior to construction of the storm surge barrier. Given the observed shoaling problems an analysis of long-term sedimentation impacts and need for maintenance dredging is warranted. The miter gate recesses, conservatively estimated at 30ft wide each, into the existing shoreline of the canal. As a result, no in-water dam sections or tie-in structures are expected to be needed. The perimeter flood risk reduction features (land based) can directly tie in to the gate recess on either side of the canal.

It is proposed that this geometry will be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design.



LEGEND:

Ⓐ GATE STRUCTURE SERIES (PER TABLE GC-B) - SEE NOTE 2

☒ GATE IN CLOSED POSITION

NOTES:

1. THIS ELEVATION VIEW SHOWS A CONCEPTUAL GEOMETRIC DESIGN. THIS PRELIMINARY DEPICTION OF THE GEOMETRY OF THE NAVIGABLE PASSAGES, AUXILIARY FLOW GATES AND STORM SURGE BARRIER CONFIGURATION SHALL NOT BE CONSTRUED AS RECOMMENDATIONS OR REQUIREMENTS FOR ACTUAL DESIGN FOR IMPLEMENTATION. SIGNIFICANT ADDITIONAL STUDY IS REQUIRED TO SUBSTANTIATE THE CONCEPTUAL DESIGN OF THIS STORM SURGE BARRIER.
2. GATE SERIES ARE LABELED AS N@X FT, WHERE N IS NUMBER OF GATES AND X IS CLEAR DISTANCES BETWEEN PIERS (UNLESS OTHERWISE NOTED).
3. EXISTING FLOW AREA WITHOUT STORM SURGE BARRIER EQUALS 1,300 SF. WITH THE STRUCTURE IN PLACE THE FLOW AREA EQUALS 2,200 SF (169% OF EXISTING).

Figure 10-1: Conceptual Geometry of Gates of the Gowanus Canal Storm Surge Barrier.

Table 10-2: "TABLE-GC-B": Summary of Gowanus Canal Storm Surge Barrier Geometry – Navigable Passage

Element (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Note	Top of Structure (ft, NAVD88)	Gate Height (ft)
A	1	-22	100	2,180	160	Navigable Passage (Miter Gate)	+17	39

Notes:

1. Flow area measured from elevation MSL to sill.
2. Total span for gate structures equals gate width plus pier widths (30ft each).

11 SHEEPSHEAD BAY CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Sheepshead Bay Storm Surge Barrier.

11.1 Hydraulic Design Parameters

Table 11-1 below summarizes the key environmental criteria from Chapter 3 that determine the elevation of the conceptual storm surge barrier design.

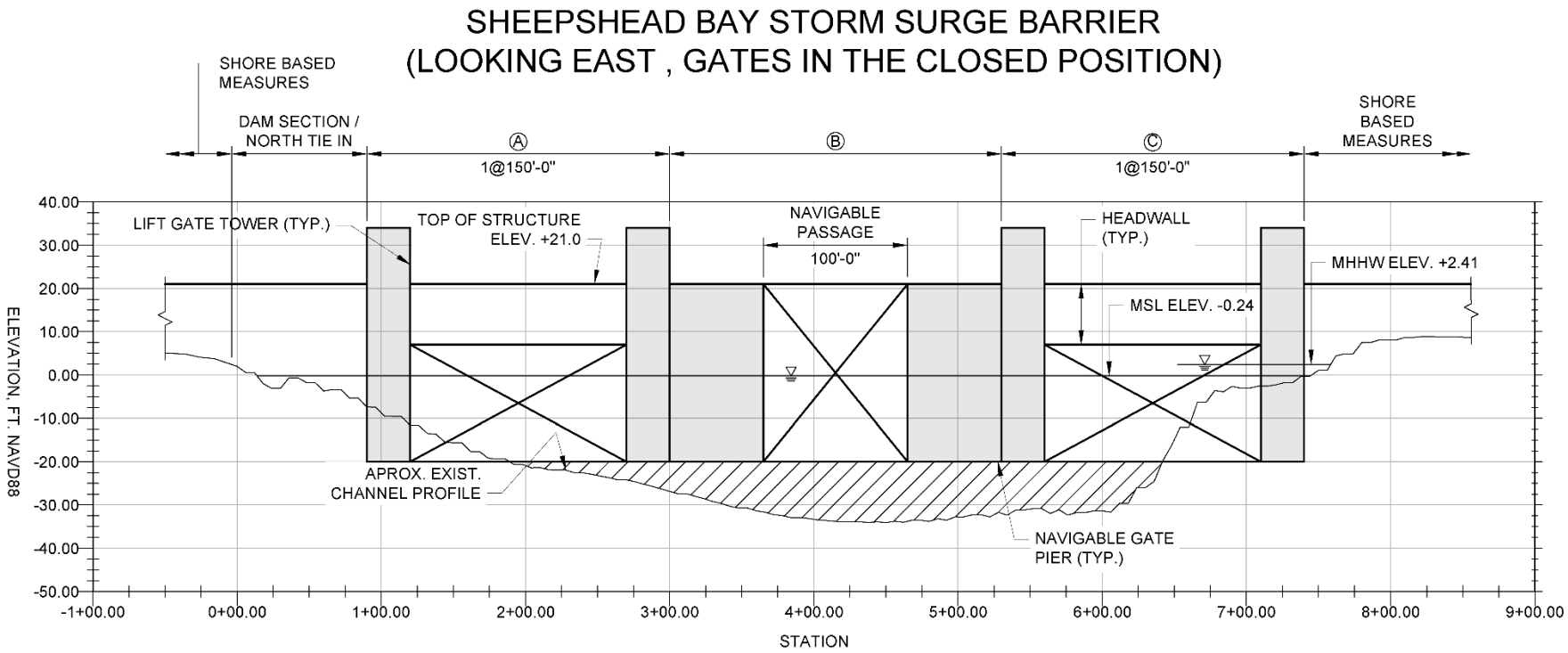
Table 11-1: Summary of Environmental Design Criteria (Sheepshead Bay)

Parameter	Value
SLR for design life through 2080 (ft)	1.82
1% AEP Design Water Level (ft NAVD88) year 2080	11.8
1% AEP Significant Wave Height (ft)	5.2
1% AEP Peak Wave Period (s)	5.2
1% AEP Overtopping Criterion (l/s/m)	50

11.2 Preliminary Geometric Design

Following the considerations presented within the previous chapters, a preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 11-1. As provided in Section 4.5.2, the preliminary concept for foundations are deep foundations (piles or drilled shafts) for all sector gate housing, lift gate towers, gate sills, miter gate recesses and foundations, and dam sections. The existing cross-sectional flow area below MSL at the proposed storm surge barrier location is approximately 15,300 ft^2. The aggregated flow opening area provided within the conceptual design is 7,900 ft^2 (approximately 52% of the existing flow area at that location).

It is proposed that this geometry will be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design.



LEGEND:

Ⓐ GATE STRUCTURE SERIES (PER TABLE SB-B) - SEE NOTE 2.

☒ GATE IN CLOSED POSITION

⧄ GATE SILL AND FOUNDATION

NOTES:

1. THIS ELEVATION VIEW SHOWS A CONCEPTUAL GEOMETRIC DESIGN. THIS PRELIMINARY DEPICTION OF THE GEOMETRY OF THE NAVIGABLE PASSAGES, AUXILIARY FLOW GATES AND STORM SURGE BARRIER CONFIGURATION SHALL NOT BE CONSTRUED AS RECOMMENDATIONS OR REQUIREMENTS FOR ACTUAL DESIGN FOR IMPLEMENTATION. SIGNIFICANT ADDITIONAL STUDY IS REQUIRED TO SUBSTANTIATE THE CONCEPTUAL DESIGN OF THIS STORM SURGE BARRIER.
2. GATE SERIES ARE LABELED AS N@X FT, WHERE N IS NUMBER OF GATES AND X IS CLEAR DISTANCES BETWEEN PIERS (UNLESS OTHERWISE NOTED).
3. EXISTING FLOW AREA WITHOUT STORM SURGE BARRIER EQUALS 15,300 SF. WITH THE STRUCTURE IN PLACE THE FLOW AREA EQUALS 7,900 SF (52% OF EXISTING).

Figure 11-1: Conceptual Geometry of Gates of the Sheepshead Bay Storm Surge Barrier.

Table 11-2: "TABLE-SB-B": Summary of Sheepshead Bay Storm Surge Barrier Geometry – Navigable Passage and Auxiliary Flow Gates

Element (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Note	Top of Structure (ft, NAVD88)	Gate Height (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
A	1	-20	150	2,964	210	Auxiliary Flow Gate (Lift Gate)	+21	27	34
B	1	-20	100	1,976	230	Navigable Passage (Sector Gate)	+21	41	N/A
C	1	-20	150	2,964	210	Auxiliary Flow Gate (Lift Gate)	+21	27	34
North Dam	0	N/A	N/A	0	120	Dam Section North Bank	+21	N/A	N/A
South Dam	0	N/A	N/A	0	0	Dam Section South Bank	+21	N/A	N/A
Total	3			7,904	770				

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths equal 65% of gate width and Span for auxiliary structures equals gate width plus pier widths (pier widths equal 30 ft).
3. Top of single leaf lift gate with headwall at +7ft

12 GERRITSEN CREEK CONCEPTUAL DESIGN SUMMARY

This chapter summarizes the key hydraulic and geometric parameters for the conceptual design of the Gerritsen Creek Storm Surge Barrier.

12.1 Hydraulic Design Parameters

Table 12-1 below summarizes the key environmental criteria from Chapter 3 that determine the elevation of the conceptual storm surge barrier design.

Table 12-1: Summary of Environmental Design Criteria (Gerritsen Creek)

Parameter	Value
SLR for design life through 2080 (ft)	1.82
1% AEP Design Water Level (ft NAVD88) year 2080	11.7
1% AEP Significant Wave Height (ft)	4.0
1% AEP Peak Wave Period (s)	3.6
1% AEP Overtopping Criterion (l/s/m)	50

12.2 Preliminary Geometric Design

Following the considerations presented within the previous chapters, a preliminary geometry for the storm surge barrier openings has been established and is shown in Figure 12-1. As provided in Section 4.5.2, the preliminary concept for foundations are deep foundations (piles or drilled shafts) for all sector gate housing, lift gate towers, gate sills, miter gate recesses and foundations, and dam sections. The existing cross-sectional flow area below MSL at the proposed storm surge barrier location is approximately 4,200 ft^2. The aggregated flow opening area provided within the conceptual design is 3,100 ft^2 (approximately 74% of the existing flow area at that location).

It is proposed that this geometry will be evaluated through numerical modeling. This step is the first step in an iterative design approach. The preliminary geometry is established such that its performance can be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the geometric design.

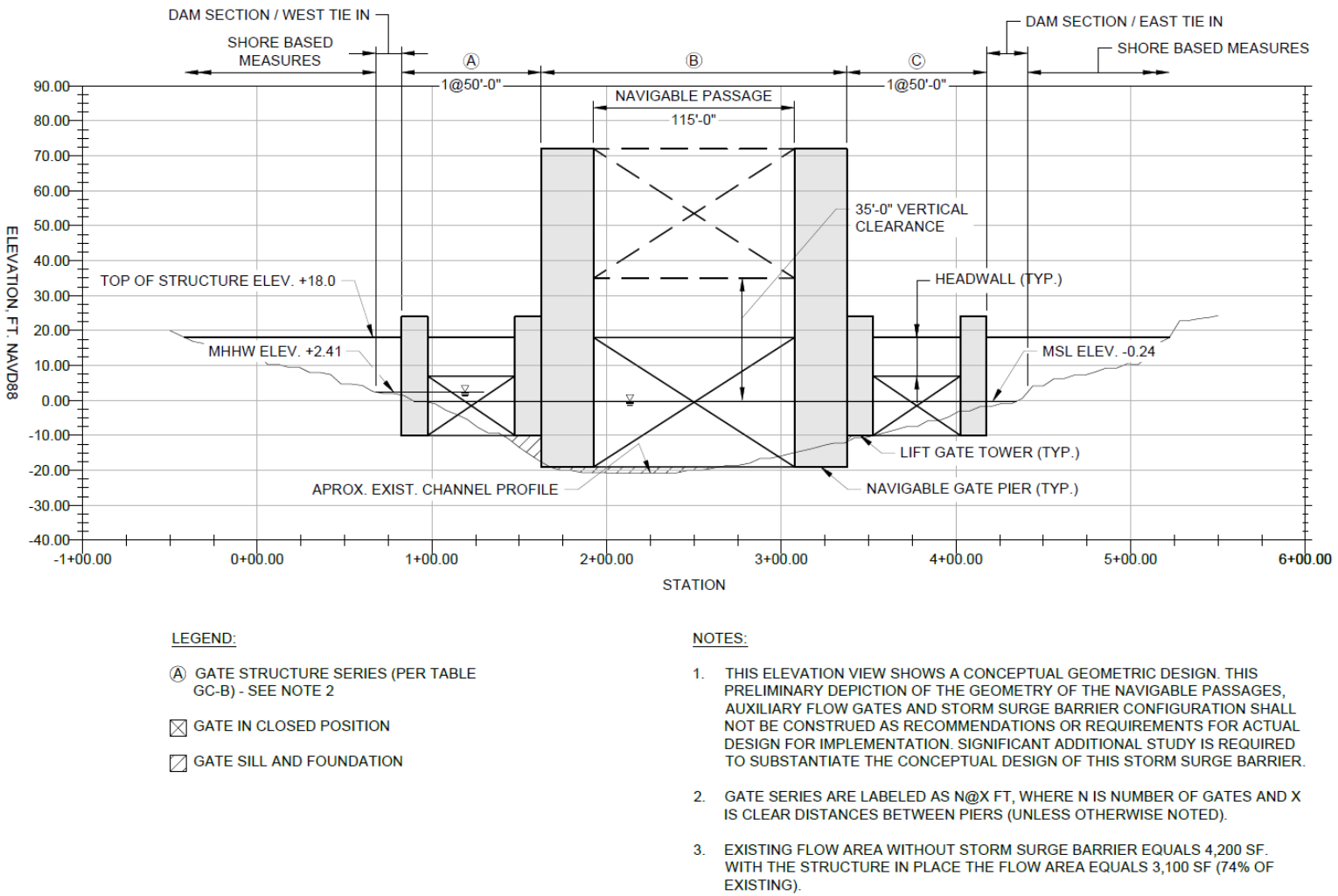


Figure 12-1: Conceptual Geometry of Gates of the Gerritsen Creek Storm Surge Barrier

Table 12-2: "TABLE-GC-B": Summary of Gerritsen Creek Storm Surge Barrier Geometry – Navigable Passage and Auxiliary Flow Gates

Element (Gate Structure with Lettered ID)	No. Gates	Sill Depth (ft, NAVD88)	Width of each Gate (ft)	Flow Area[1] (ft^2)	Span[2] (ft)	Note	Top of Structure (ft, NAVD88)	Gate Height (ft)	Top of Lift Gate in Raised Position[3] (ft, NAVD88)
A	1	-10	50	488	80	Auxiliary Flow Gate (Lift Gate)	+18	17	24
B	1	-19	115	2,157	175	Navigable Passage (Lift Gate)	+18	37	72
C	1	-10	50	488	80	Auxiliary Flow Gate (Lift Gate)	+18	17	24
West Dam	0	N/A	N/A	146	15	Dam Section West Bank	+18	N/A	N/A
East Dam	0	N/A	N/A	469	25	Dam Section East Bank	+18	N/A	N/A
Total	3			3,133	375				

Notes:

1. Flow area measured from elevation MSL to sill.
2. Span for navigable structures equals gate width + pier widths. Pier widths (each) equal 30ft for navigable gate width and span for auxiliary structures equals gate width plus pier widths (pier widths equal 15 ft).
3. Top of single leaf lift gate with headwall at +7ft

13 RECOMMENDATIONS FOR FURTHER STUDY

13.1 Introduction

This study is the first where a suite of storm surge barriers is evaluated for the New York and New Jersey Harbor. The conceptual designs for the storm surge barriers as part of the NYNJHAT Study Alternatives, are based upon a broad yet comprehensive data analysis for the entire study area with approximate equal level of detail for each storm surge barrier. The basis of design, albeit preliminary, are analogously and consistently prepared for all storm surge barrier locations and include amongst other items; the latest hydrodynamic storm surge modeling results to establish boundary conditions for design, a review of bathymetric and channel conditions, and a basis for the minimum required dimensions of the navigable passages. Most importantly the conceptual designs and geometries of the storm surge barriers for the NYNJHAT Study are evaluated using hydrodynamic models and are not solely analyzed on an individual basis but analyzed using a systems approach (USACE ERDC 2019). This document in conjunction with Volume 1 is the first step in an iterative design process using a systems approach, whilst previous completed studies in large part only provided singular concepts and did not assess impacts to the regional hydrodynamics, nor did those studies use such assessments to further the conceptual designs.

Furthermore, the gate types are selected based on a high-level but full review of hydraulic gate types and the applicability of such gates based on the review of constructed storm surge barriers. The conceptual design as presented herein is in part informed by the data and characteristics of other storm surge barriers that have been constructed throughout the world. As such the conceptual design is built upon proven concepts and principles which in turn improve the reliability of the overall concept; reliability is a key notion during the concept development of these storm surge barrier designs.

Despite the depth and breadth of preliminary evaluation, this assessment of navigable passage widths and storm surge barrier configurations shall not be construed as definitive recommendations or requirements for actual design for implementation. Significant additional study is required to substantiate the width, location, and configuration of the navigable passages and auxiliary flow gates where applicable, including a full evaluation of navigation, environmental, ecological, and cost considerations, amongst others.

The next sections provide a framework of additional studies and engineering analyses that should be considered, and what those efforts should, at a minimum, entail[8]. Certain topics can be expanded upon if there is a need to prioritize or expedite the refinement of a selected set of storm surge barrier designs.

[8] These sections are geared towards engineering analyses and studies, while it is recognized that environmental, economic, socio economic and other studies would be required similarly.

13.2 Iterative Design - Next Steps

Following the analyses described within this report there are a number of overarching topics that warrant further investigation and are considered logical next steps as part of an iterative design process. For completeness these topics are summarized here. One of the main tasks to be evaluated for each storm surge barriers is a siting study[9]. A complete siting study for the storm surge barriers would evaluate pros and cons of various alignment and conceptual design alternatives for each storm surge barrier. Such analyses should consider the following topics:

- Navigable passage dimensions:
    - The required width of the navigable passage and the requirements for one-way versus two-way traffic for the navigable passage (further detailed below under Navigation – Section 13.5)
- Existing channel conditions and proposed sill elevation - For a number of storm surge barrier locations, the reported channel conditions are less than the authorized channel dimension due to shoaling. The sill elevation of the navigable passage has been determined based on authorized channel depth, yet at certain locations this may require substantial channel maintenance (i.e. dredging) prior to construction of the storm surge barrier. A couple of examples are provided below:
    - East Chester Creek and Bronx River
        - Shoaling up and downstream of proposed storm surge barrier location
    - Westchester Creek
        - Up to 10ft of shoaling for the majority of the channel reach
    - Flushing Creek
        - Shoaling in outside quarters of the channel at the proposed storm surge barrier location
    - Newtown Creek
        - The recommendations from the 2016 report (CH2MHill, 2016b) to set the sill elevation at -20 ft NAVD88 is utilized here to establish the conceptual design with the understanding that that recommendation was the result of a site-specific feasibility study at a higher level of detail than the feasibility analysis of storm surge barriers at a regional scale discussed herein. It should be emphasized however that the channel is authorized at -23 ft MLLW (-26 NAVD88) and as such a limitation to the authorized channel depth needs to be accepted at a later date.
- Alternative gate types for the navigable passage;
    - It should be noted that this is a high-level evaluation as no site-specific borings are available and designs for the gate structures are still conceptual in nature. Further recommendations regarding geotechnical evaluations are provided below.

[9] The exceptions are Newtown Creek and Gowanus Canal for which prior studies provided a basis for site selection (CH2MHill, 2016a) (CH2MHill, 2016b)

- For those locations were air clearance is restricted, or the option exists to restrict air clearances, lift gates have been preliminarily selected.
- Detailed data for reverse head conditions for the storm surge barriers is not yet available and will need to be analyzed at a later date. A miter gate has been selected only for the Gowanus Canal storm surge barrier, because the span is relatively small and reverse head conditions are expected not to be an issue. For other storm surge barriers, i.e. Sheepshead Bay and Gerritsen Creek a miter gate may be a suitable option too if analyses show that reverse head conditions are not an issue.
- For each storm surge barrier location, it is recommended to analyze, cost and compare alternate gate types. Potential alternate gate types for each barrier location are indicated in Appendix F.

- Alternative Gate types for the auxiliary flow gates,
  - For storm surge barriers where sufficient space is available between the navigation channel and the shoreline, auxiliary gates are included. The lift gate was selected for the auxiliary flow structures. At those locations where reverse head conditions are of limited concern, a tainter gate or a miter gate for smaller spans is most likely a viable option too. Applicability of tainter gates will, in large part, depend on reverse head conditions and the potential for relatively high load concentration on the trunnion bearings. In addition, for locations that are shallower with a fairly even bathymetric profile, rotating segment gates or inflatable gates could be considered.
  - The conceptual designs presented maximized the number of auxiliary flow gates to the extent practicable, to minimize impacts on flow exchange. At some locations a different configuration (less openings or slightly less total flow area) may result in no appreciable difference in tidal flow exchange but could potentially be more economical.
  - Apart from the examples provided here, it is recommended to analyze, cost and compare alternate auxiliary flow gate types and configurations for all selected storm surge barriers. E.g. for the Bronx River storm surge barrier four 75 ft wide gates may be more economical than two 150 ft wide gates. Potential alternate gate types for each barrier location are indicated in Appendix G.
- Geotechnical site conditions,
  - Foundation concepts were based on a high-level evaluation as no site-specific borings are available and the designs for the gates are still very conceptual. It is recommended that a geotechnical data gap analyses is completed; site specific geotechnical data is gathered, and a generic geotechnical profile be compiled for each storm surge barrier location.

13.3 Constructability

The siting and eventual construction of a storm surge barrier is a complex undertaking, and practical constraints may influence the eventual design based on constructability considerations. Constructability will influence design considerations, structure type, project costs, phasing requirements, and schedule. Large civil works projects involving marine-based construction are represented by significant complexity and cost factors. These factors are generally exacerbated as water depth, flow velocities, and proximity to navigation channels are considered. Likewise, structure configurations used to overcome the spatial and loading criteria for which the structure must perform also heavily influence complexity and cost. Hence, basic constructability assessments must be performed to consider viability and provide for proof of concept for foundations and structure types under consideration.

Constructability evaluations are an inherent part of any major civil works undertaking. Among the many considerations when considering constructability, the following should be taken into account:

- Maintenance of navigation and navigational impacts during construction
- General method of construction (e.g. in-the-dry, in-the-wet)
- Temporary works (e.g. cofferdams)
- Site access (e.g. barge-based work versus land-based access via temporary trestle)
- Site staging and laydown areas
- Material deliveries to the work site (e.g. floating concrete plant)
- Contractor capabilities, and the availability of both specialized contractors and equipment needed to perform the work
- Feasibility, availability, and locations of off-site fabrication areas for modular elements (e.g. graving dock for float-in elements)
- Variability of subsurface conditions, and methods used to provide adequate foundations
- Impact from tides, current, weather, and other environmental factors on construction activities
- Extreme event scenarios, preparedness provisions, and similar risk considerations
- Potential availability of construction materials, including quality and quantity
- Waste and recycled materials considerations, including beneficial use
- Environmental considerations affecting construction activities (e.g. relocations, noise, work period restrictions)
- Construction schedule, including a variety of phasing and funding scenarios

13.4 Hydraulics, Hydrology, and the Aquatic Environment

The complexity of the regional hydraulics and hydrology warrants further study in the following topics.

- Permissible overtopping quantities and permissible leakage through the storm surge barrier to optimize structure elevation.
    - Currently an overtopping criterion of 50 l/s/m is applied, which could still be considered a conservative criterion as some coastal structures can accommodate higher overtopping discharges if properly designed for (USACE, 2002). It should be noted that, unlike the storm surge barriers discussed in volume 1, the water bodies behind the storm surge barriers discussed herein provide less storage capacity.
- Analyses of impacts to the tidal flow exchange and impacts to the tidal amplitude as a result of the proposed geometry. And analyses of impacts to current velocities, navigability and the required dimensions of openings within the storm surge barrier to minimize impacts to navigation conditions. Such analyses should further the work completed by USACE ERDC (USACE ERDC 2019) and continue the iterative design process to refine the storm surge barrier geometry, and include:
    - It should be noted that as a result of the size of the storm surge barriers discussed herein and the relative smaller dimensions of the navigable passages, not all storm surge barriers have not been evaluated with the use of hydrodynamic models.
    - USACE ERDC noted that numerical model stability issues may arise for these structures due to lower spatial resolution of the areas peripheral to the original study (USACE ERDC 2019), i.e. the model was originally set up to study the larger storm surge barriers discussed in Volume 1. It is recommended to reassess the model setup and evaluate impacts on flow velocities and tidal amplitude through numerical modeling for the storm surge barriers (further detail below under 13.4).
    - Assessment of the impact on water surface elevations, discharges and average velocities in the openings, and
    - Assessment of local hydraulic changes in the inner basin, harbor or bay such as local velocities and currents, salinities, tidal levels and circulation which are essential to pollution, fish and wildlife, and other environmental and ecological considerations
- Assessment of near-field flow patterns and alignment of the gate recesses and gate pier structures with any existing bridge piers.
- Analyses of potential changes in tidal flow exchange and the impacts on both local and far field morphology
    - Future analysis will need to evaluate the potential for erosion and sedimentation in the region of the storm surge barrier and potential impacts to scheduled maintenance dredging cycles.
- Sea level rise sensitivity and adaptability analyses
    - Perform tests with different SLC scenarios and investigate changes in hydrostatic and dynamic loading as well as changes in overtopping discharge and identify options and project features that can provide for an adaptable design

    - Adaptive management may be necessary or structural improvements may be needed if the observed sea level rise exceeds the planning criteria; such provisions would be included in the design to accommodate improvements if and when needed.

- Impacts to water levels on the protected side during gate closures (reverse head conditions);
    - Analysis of inflows and potential for a rise in water levels on the protected side of the storm surge barrier after gate closure. Inflows could be either rain-fall run-off , stream discharge or municipal treatment plant effluent discharges. This issue is relevant for all storm surge barriers discussed herein, but particularly for the Newtown Creek storm surge barrier where the earlier conceptual design included the recommendation for the inclusion of an in-line pump station (CH2MHill, 2016b).
    - Analysis of joint probability of river discharge (flood levels) and storm surge levels.
- Analysis of impacts to water quality during and post gate closure.
- Analyses of potential changes in tidal flow exchange and impacts on salinity, water quality and ecology.

13.5 Navigation

Constructing a storm surge barrier across a navigation channel will require further study. Due to the lack of comprehensive AIS data sets for the discussed navigation channels it is recommended to collect more data on vessel transits and complete a navigation assessment to address:

- Waterway traffic
    - Number, frequency, type, and intensity of vessel passage
    - One-way versus two-way vessel passage, including meeting, passing, and overtaking
    - Vessel wait areas, queuing, and wait times
    - Storm surge barrier positioning and fairway lengths for maneuvering
    - Trends for future vessel traffic, including vessel size, frequency
    - Passage of recreational vessels (this is especially relevant for the Sheepshead Bay and Gerritsen Creek storm surge barriers)
- Currents, cross currents, wind, tides, surge, weather, night, visibility, and other environmental considerations for vessel passage
- Navigation evaluations, including pilot and navigation industry input, and real-time simulations to assess, amongst others:
    - Flow and cross-current considerations
    - Gate approach and departure
    - Passing vessel assessments
- Requirements for Aids-to-Navigation, guide structures, and protective structures

- Requirements for vessel traffic service, including advisory / control / restrictions on navigation
- National security considerations.

13.6 Operations and Maintenance

Considerations for operations and maintenance affect the overall design philosophy. Operation and maintenance cost are a substantial part of the life cycle cost of storm surge barriers. Important factors that determine operation and maintenance costs are: 1) maintenance of the movable parts of the structure, 2) painting (steel) parts of the structure, 3) operations and maintenance personnel cost, 4) cost of an operational data and decision support network, 5) inspection of parts including submerged parts (after van Ledden, et al. 2012), 6) Control systems, remote operation, emergency operation, redundant systems, 7) Size, scope, equipment, and location of facilities to support Operations and Maintenance. The following topics will require further study:

- Operational criteria for gate closure and the expected frequency of gate closures.
- Time scales for deployment, reliability and operation of gate and warning systems.
- Reliable operation of the storm surge barrier (gate closures) to obtain a reduction in flood risk.
- Reliable operation of to minimize the impacts of gate closures on navigation and the aquatic environment.
- Reduce to the extent practicable the complexity of Operation and Maintenance, Repair, Replacement and Rehabilitation (OMRR&R).

13.7 Multi-functionality of a Storm Surge Barrier Complex

A number of the storm surge barriers discussed herein would be constructed in close vicinity to industrial and residential development areas. The tie-in to the shore-based perimeter flood risk reduction system and the integration of the form and function of the storm surge barrier would require further study. There may be opportunities to further blend form and function and assess shared uses and multi-functionality of this civil works complex such that it provides additional benefits to the community. Topics that require further study are:

- Inclusion of transportation infrastructure (roadways, bridges and tunnels).
- Potential for connections to existing transportation infrastructure.
- Inclusion of recreational, educational areas, and other considerations for public access, and,
- An assessment of aesthetics.

This report is part of a series of reports that document the preliminary analysis completed in support of the feasibility of the New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study.

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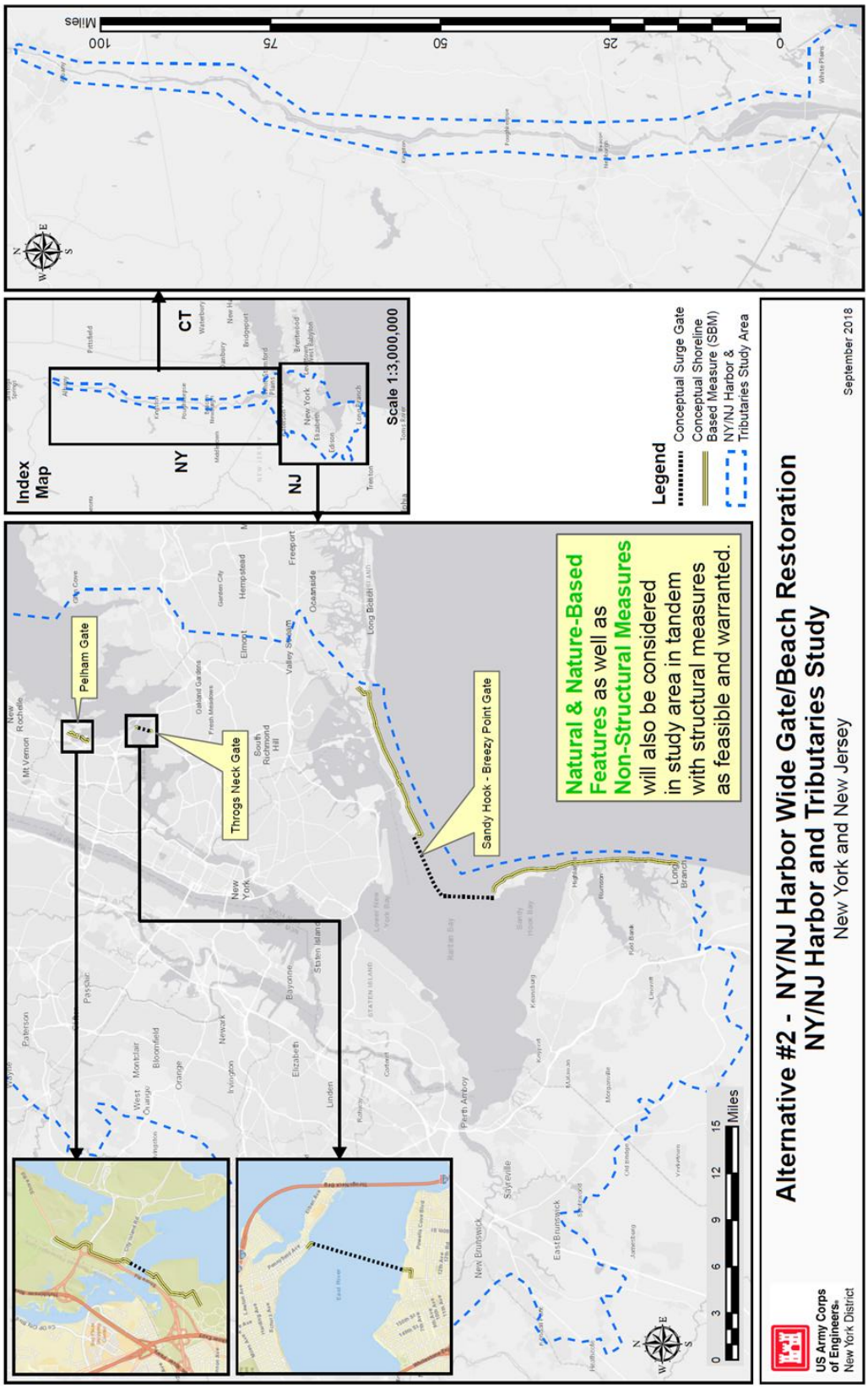
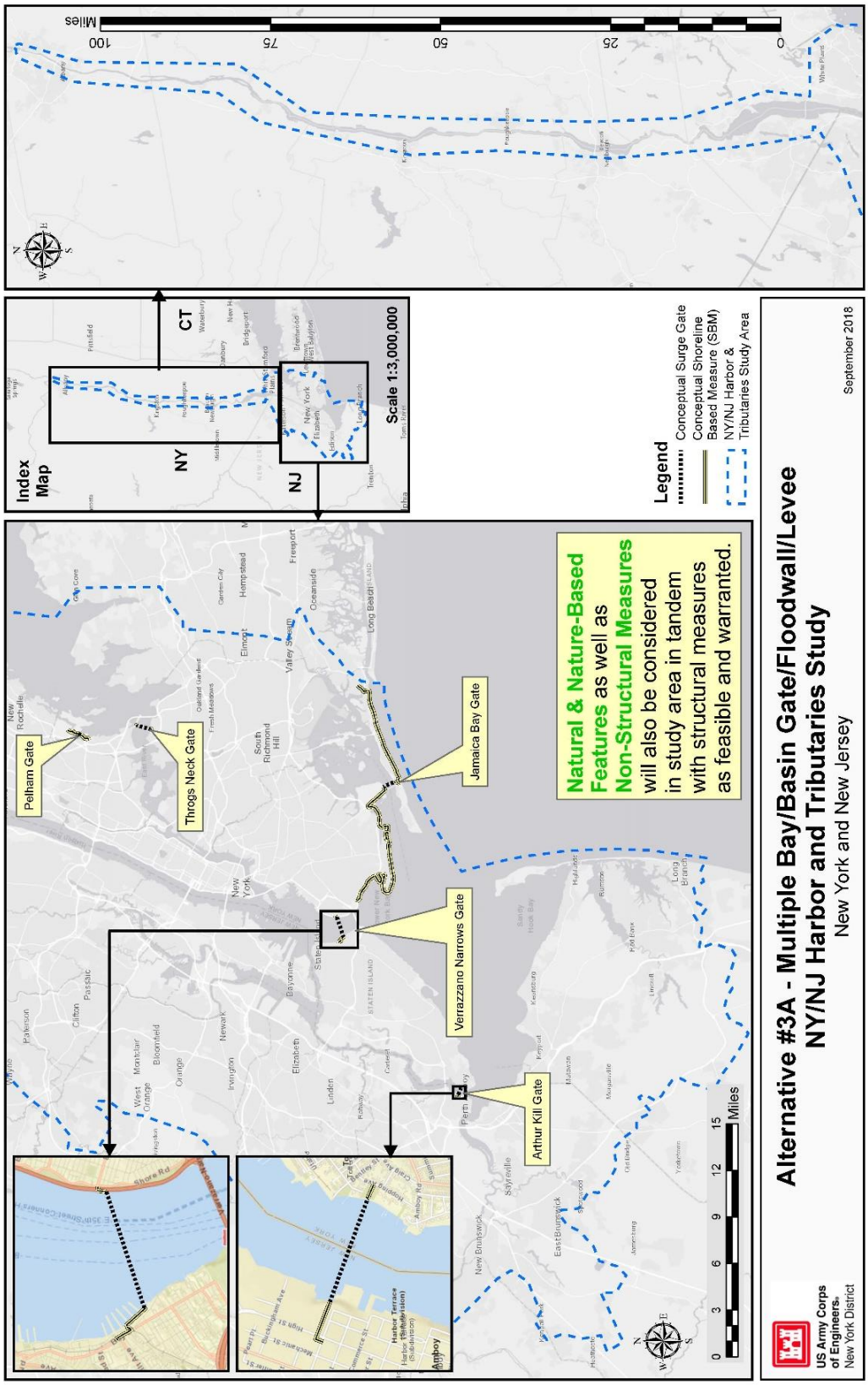
A. APPENDIX A - ALTERNATIVES MAPS

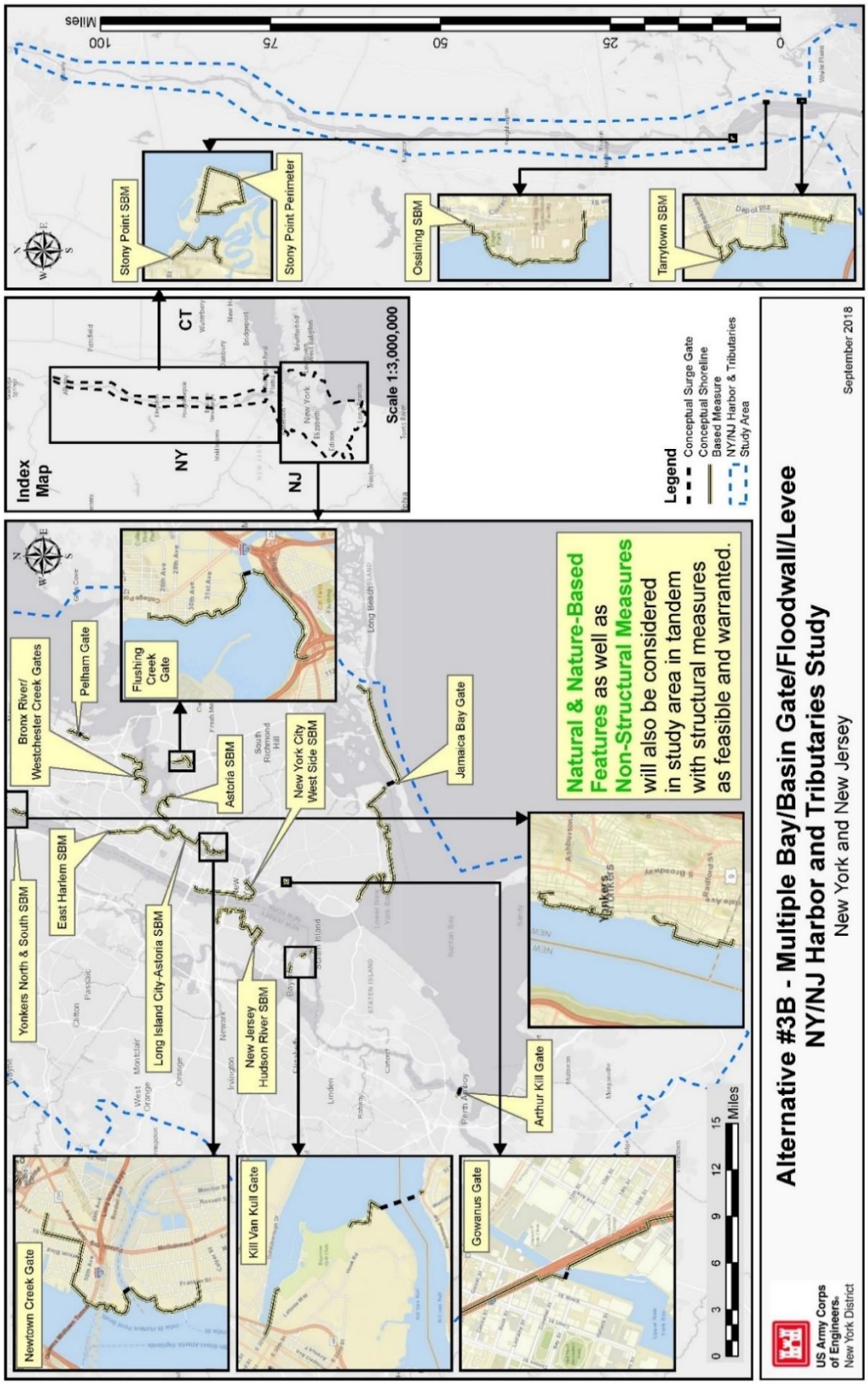
Five scales of initial alternative concepts, in addition to No Action, were developed and constitute the HAT Study Alternatives:

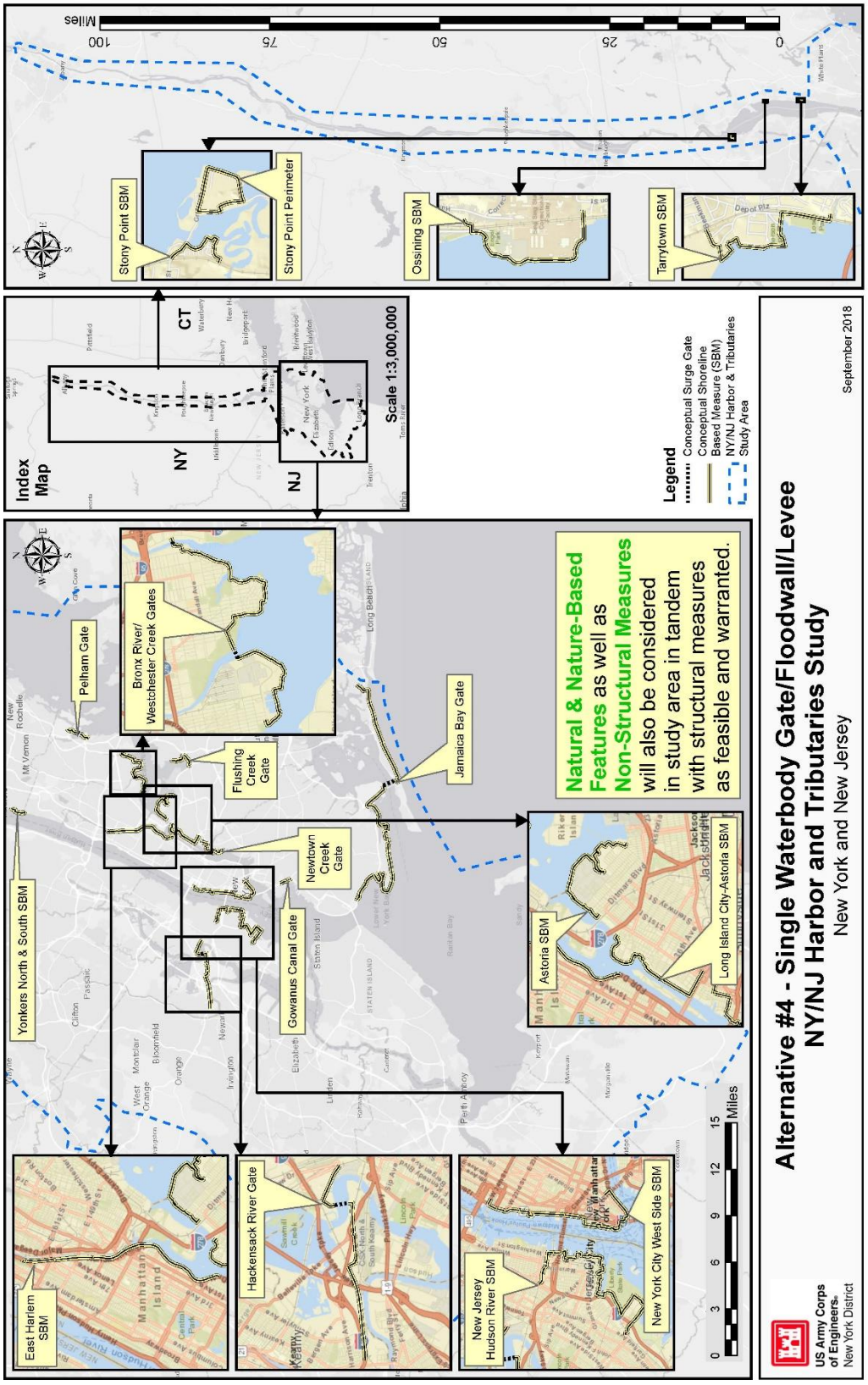
1. No Action/FWOPC (baseline)
2. Outer Harbor Barrier system (Alternative 2)
3. Multiple Barriers, Floodwalls, and Levee systems
    a. Three Barrier Plan (Alternative 3A)
    b. Two Barrier Plan (Alternative 3B)
4. Single water body barriers, Floodwalls, and Levees (Alternative 4)
5. Perimeter only (Alternative 5)

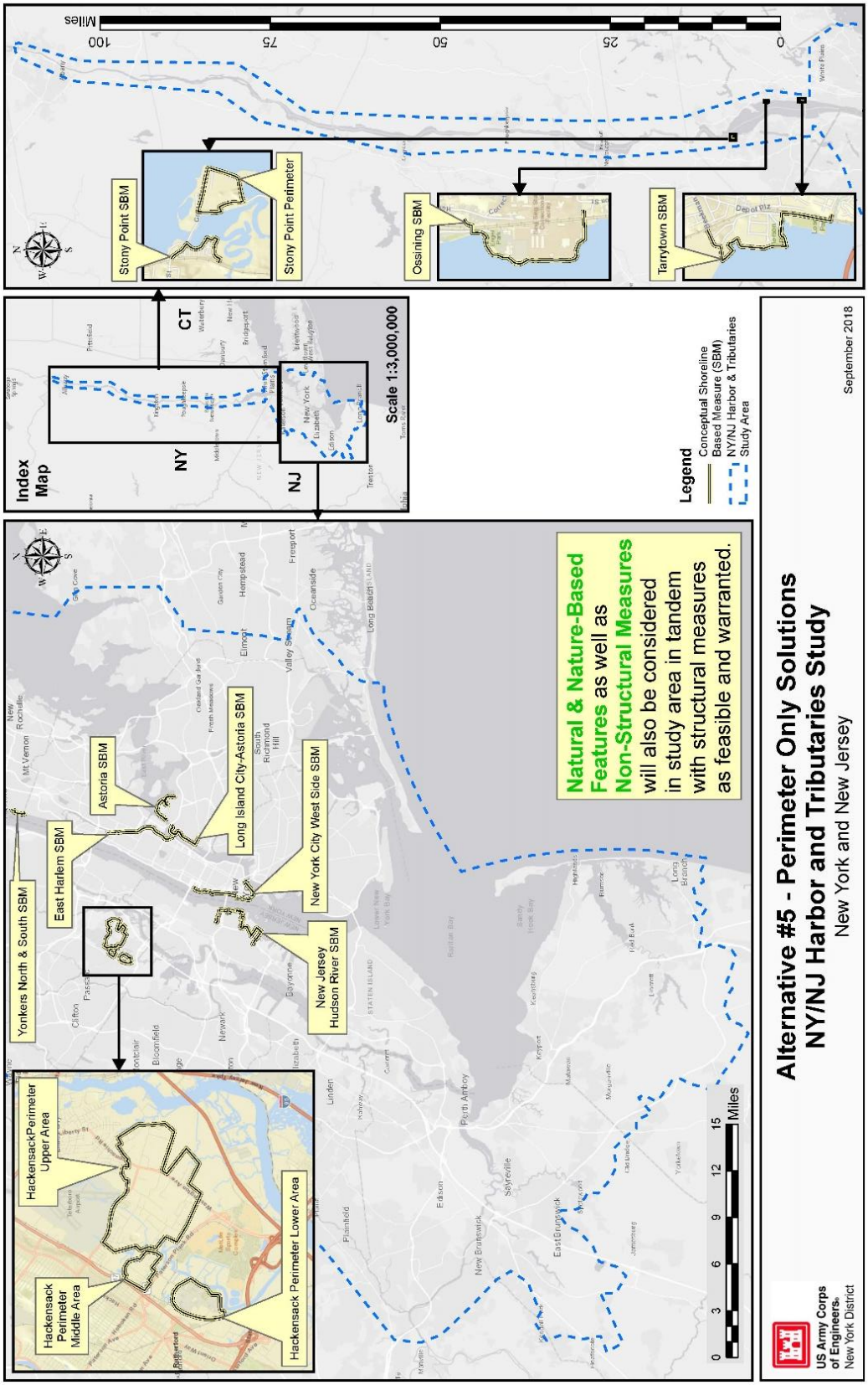
The following pages show the overall plan for the HAT Study Alternatives 2, 3A, 3B, 4 and 5:


Alternative #2 - NY/NJ Harbor Wide Gate/Beach Restoration
NY/NJ Harbor and Tributaries Study
New York and New Jersey
September 2018
US Army Corps of Engineers
New York District
Legend
Conceptual Surge Gate
Conceptual Shoreline Based Measure (SBM)
NY/NJ Harbor & Tributaries Study Area
Index Map
NY
NJ
CT
Scale 1:3,000,000
Natural & Nature-Based Features as well as Non-Structural Measures will also be considered in study area in tandem with structural measures as feasible and warranted.
Pelham Gate
Throgs Neck Gate
Sandy Hook - Breezy Point Gate
Miles
0
3
6
9
12
15
Miles
0
25
50
75
100


Alternative #3A - Multiple Bay/Basin Gate/Floodwall/Levee
NY/NJ Harbor and Tributaries Study
New York and New Jersey
September 2018
US Army Corps of Engineers.
New York District
Legend
Conceptual Surge Gate
Conceptual Shoreline Based Measure (SBM)
NY/NJ Harbor & Tributaries Study Area
Index Map
NY
NJ
CT
Scale 1:3,000,000
Natural & Nature-Based Features as well as Non-Structural Measures will also be considered in study area in tandem with structural measures as feasible and warranted.
Pelham Gate
Throgs Neck Gate
Jamaica Bay Gate
Verrazzano Narrows Gate
Arthur Kill Gate
Miles
0
3
6
9
12
15
0
25
50
75
100
Miles


Alternative #3B - Multiple Bay/Basin Gate/Floodwall/Levee
NY/NJ Harbor and Tributaries Study
New York and New Jersey
September 2018
US Army Corps of Engineers.
New York District
Legend
Conceptual Surge Gate
Conceptual Shoreline Based Measure
NY/NJ Harbor & Tributaries Study Area
Index Map
NY
NJ
CT
Scale 1:3,000,000
Natural & Nature-Based Features as well as Non-Structural Measures will also be considered in study area in tandem with structural measures as feasible and warranted.
Bronx River/ Westchester Creek Gates
Pelham Gate
Flushing Creek Gate
Astoria SBM
New York City West Side SBM
Jamaica Bay Gate
Yonkers North & South SBM
East Harlem SBM
Long Island City-Astoria SBM
New Jersey Hudson River SBM
Arthur Kill Gate
Newtown Creek Gate
Kill Van Kull Gate
Gowanus Gate
Stony Point SBM
Stony Point Perimeter
Ossining SBM
Tarrytown SBM
Miles
0
3
6
9
12
15
0
25
50
75
100


Alternative #4 - Single Waterbody Gate/Floodwall/Levee
NY/NJ Harbor and Tributaries Study
New York and New Jersey
September 2018
US Army Corps of Engineers.
New York District
Legend
Conceptual Surge Gate
Conceptual Shoreline Based Measure (SBM)
NY/NJ Harbor & Tributaries Study Area
Index Map
NY
NJ
CT
Scale 1:3,000,000
Natural & Nature-Based Features as well as Non-Structural Measures will also be considered in study area in tandem with structural measures as feasible and warranted.
Pelham Gate
Bronx River/ Westchester Creek Gates
Flushing Creek Gate
Jamaica Bay Gate
Newtown Creek Gate
Gowanus Canal Gate
Yonkers North & South SBM
Astoria SBM
Long Island City-Astoria SBM
East Harlem SBM
Hackensack River Gate
New Jersey Hudson River SBM
New York City West Side SBM
Stony Point SBM
Stony Point Perimeter
Ossining SBM
Tarrytown SBM
Miles
0
3
6
9
12
15
25
50
75
100


Alternative #5 - Perimeter Only Solutions
NY/NJ Harbor and Tributaries Study
New York and New Jersey
September 2018
US Army Corps of Engineers
New York District
Natural & Nature-Based Features as well as Non-Structural Measures will also be considered in study area in tandem with structural measures as feasible and warranted.
Legend
Conceptual Shoreline Based Measure (SBM)
NY/NJ Harbor & Tributaries Study Area
Index Map
NY
NJ
CT
Scale 1:3,000,000
Miles
0 3 6 9 12 15
Yonkers North & South SBM
East Harlem SBM
Astoria SBM
Long Island City-Astoria SBM
New York City West Side SBM
New Jersey Hudson River SBM
Hackensack Perimeter Upper Area
Hackensack Perimeter Middle Area
Hackensack Perimeter Lower Area
Stony Point SBM
Stony Point Perimeter
Ossining SBM
Tarrytown SBM
Miles
0 25 50 75 100

B. APPENDIX B – BATHYMETRIC PROFILES AND SUBAQUEOUS UTILITIES

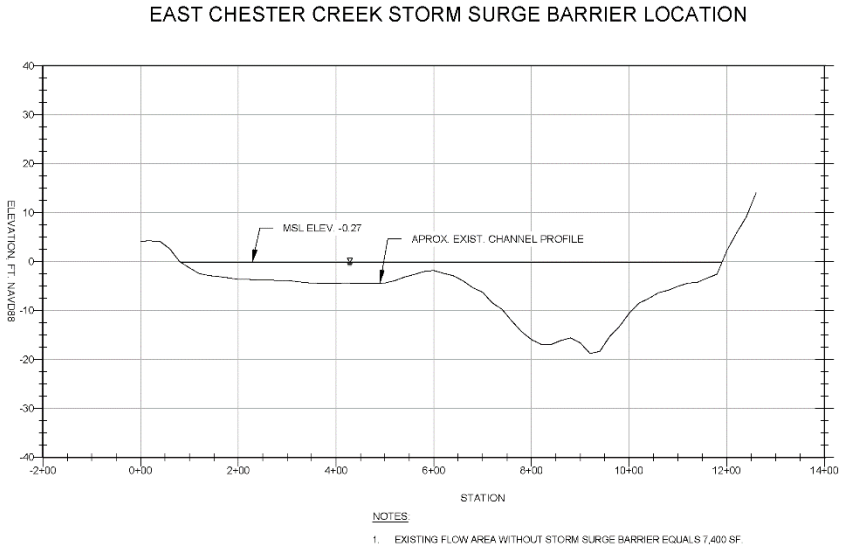


Figure B-1: Bathymetric profile for East Chester Creek

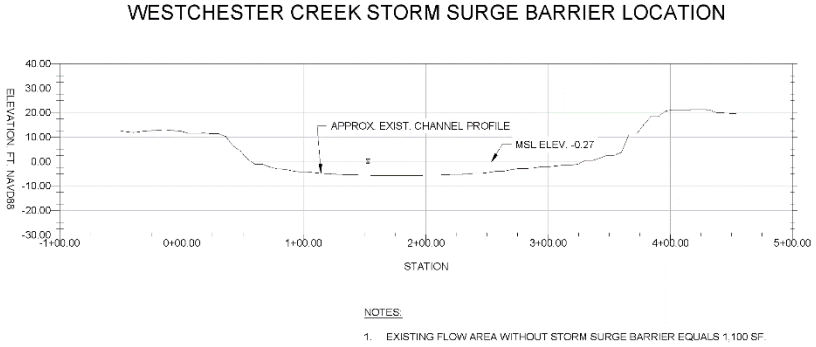


Figure B-2: Bathymetric profile for Westchester Creek

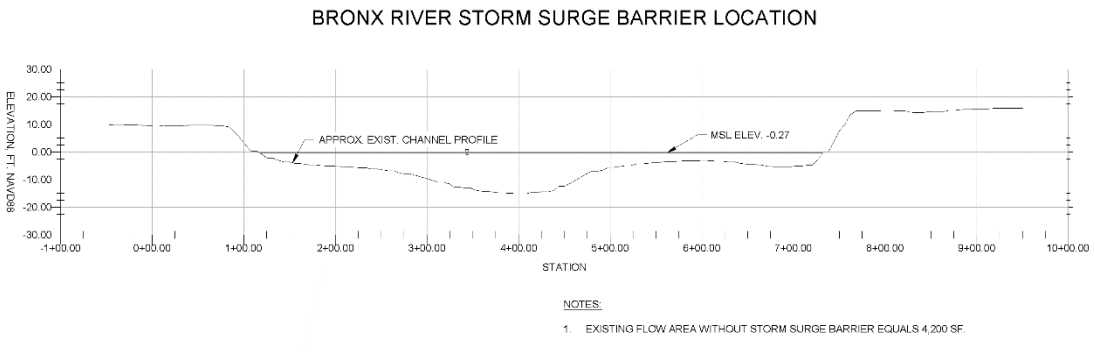


Figure B-3: Bathymetric profile for Bronx River

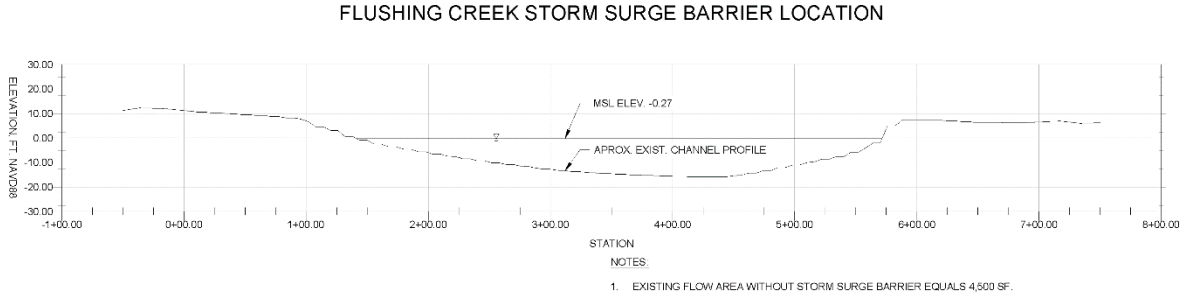


Figure B-4: Bathymetric profile for Flushing Creek

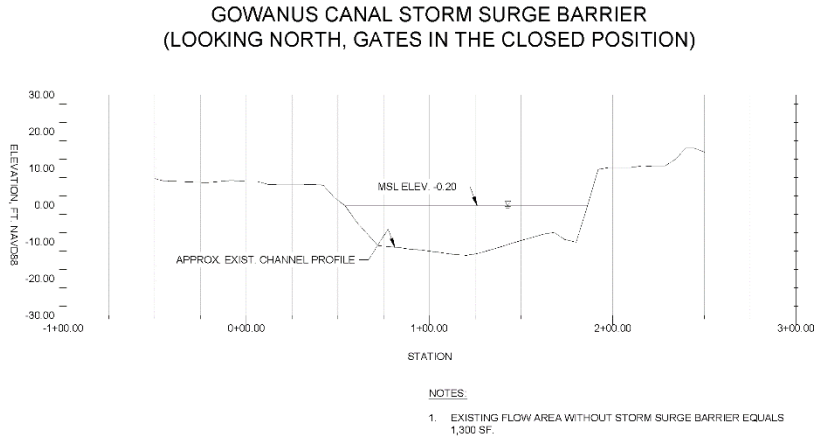


Figure B-5: Bathymetric profile for Gowanus Canal

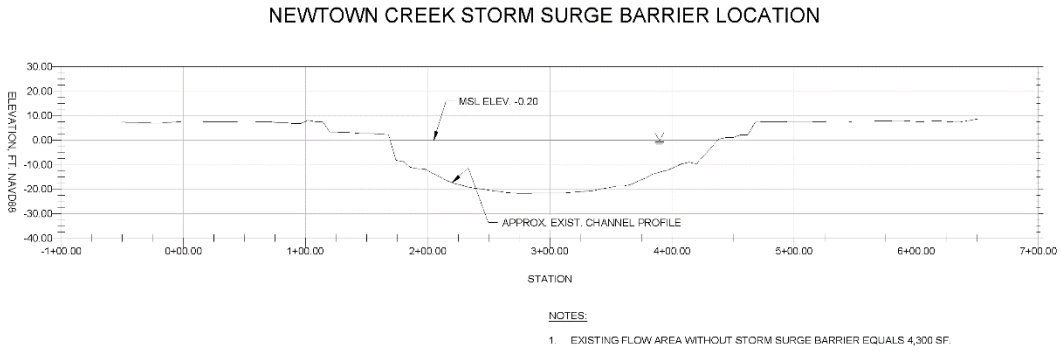


Figure B-6: Bathymetric profile for Newtown Creek

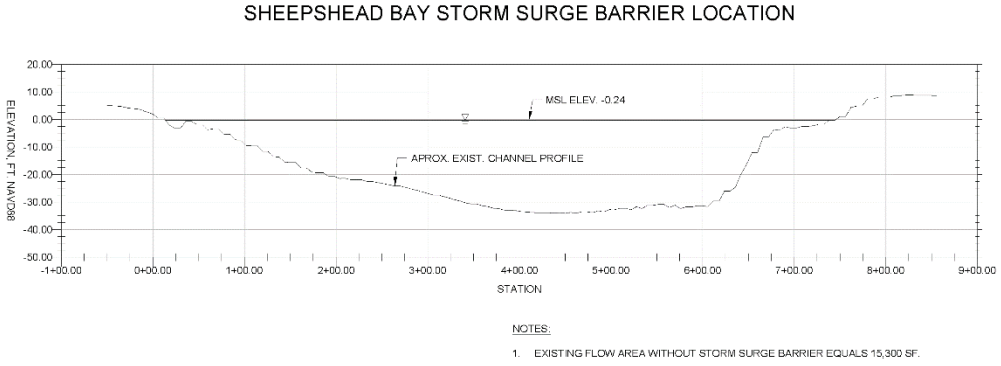


Figure B-7: Bathymetric profile for Sheepshead Bay

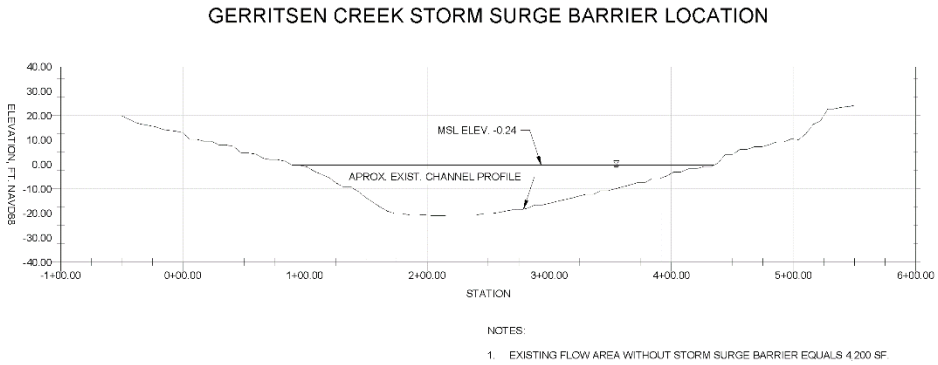


Figure B-8: Bathymetric profile for Gerritsen Creek

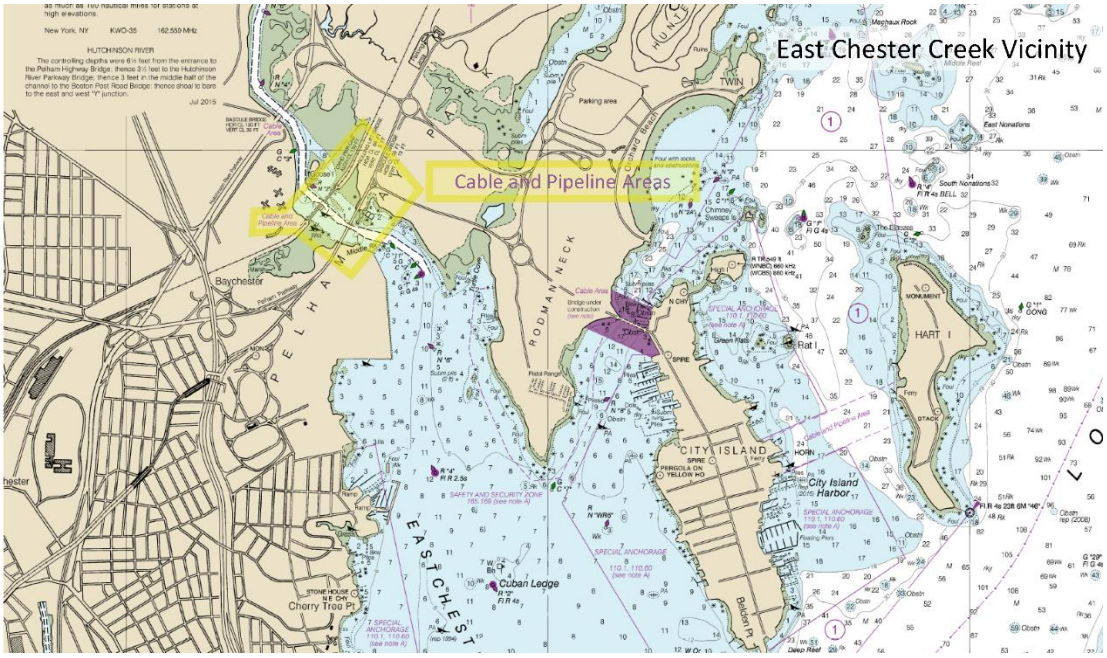


Figure B-9: Subaqueous utilities in the vicinity of the East Chester Creek barrier (adapted from NOAA Chart 12366).

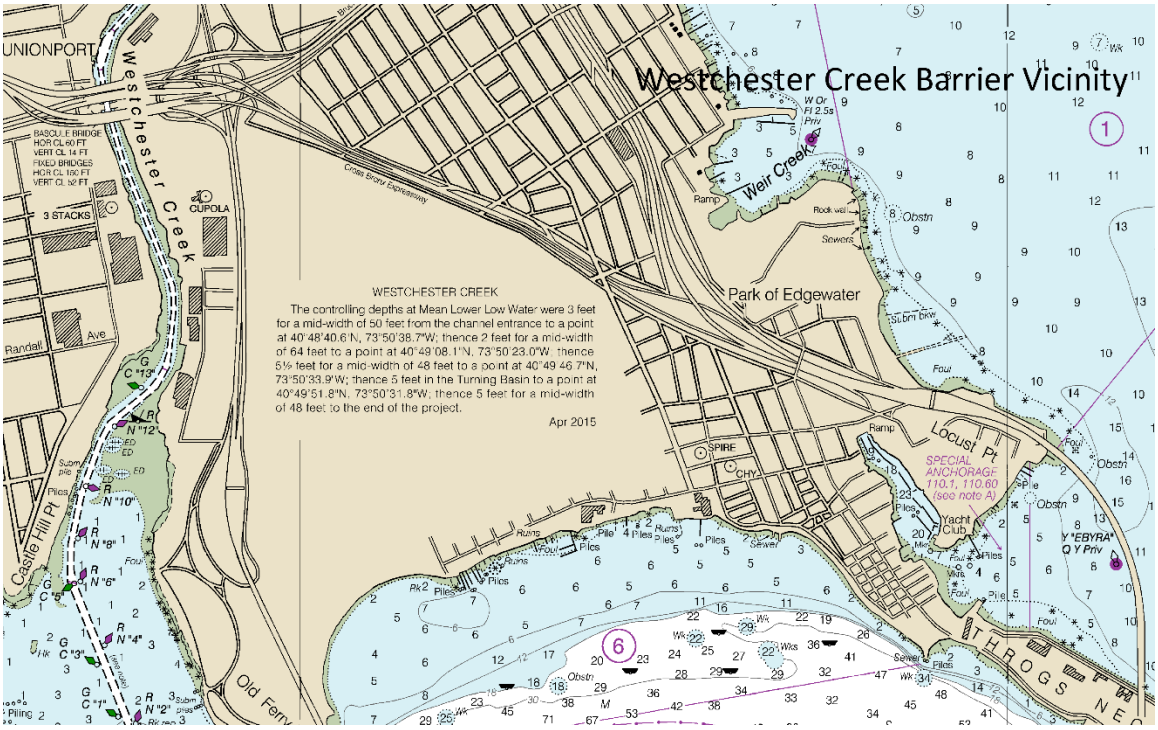


Figure B-10: Subaqueous utilities in the vicinity of the Westchester Creek barrier (adapted from NOAA Chart 12366). Note that there are no utilities shown.

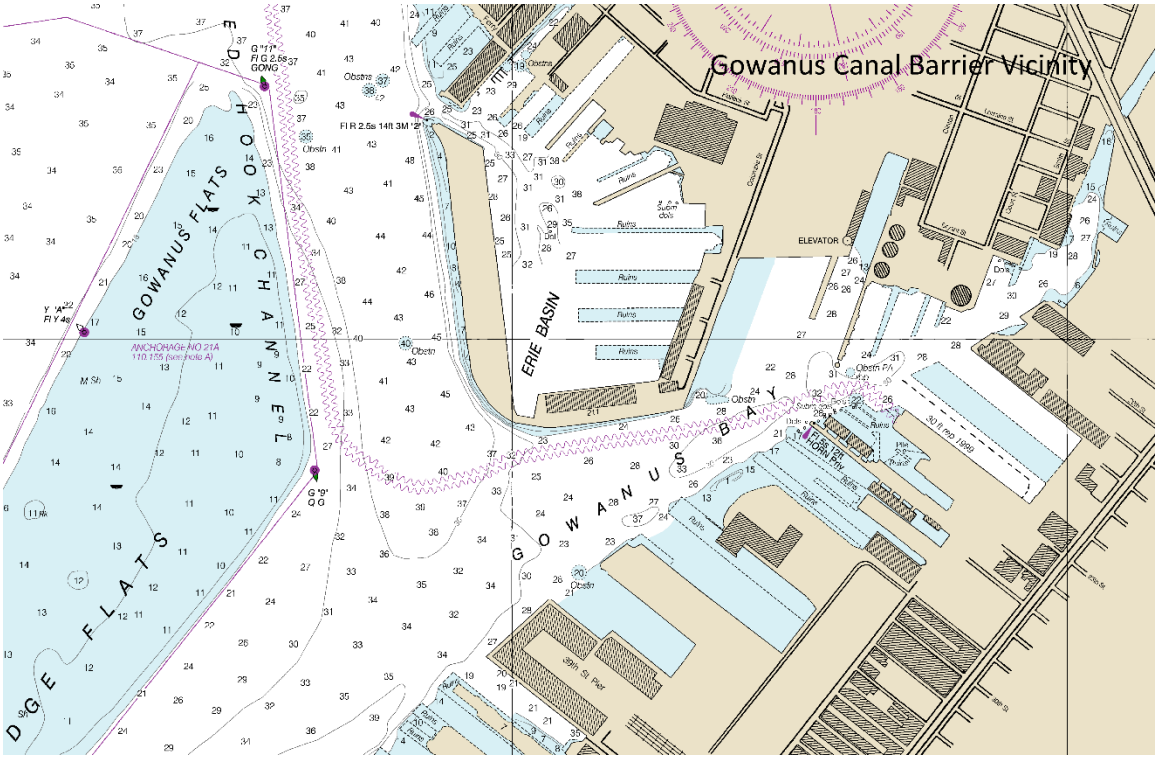


Figure B-11: Subaqueous utilities in the vicinity of the Gowanus Canal barrier (adapted from NOAA Chart 12334). Note that there are no utilities shown.

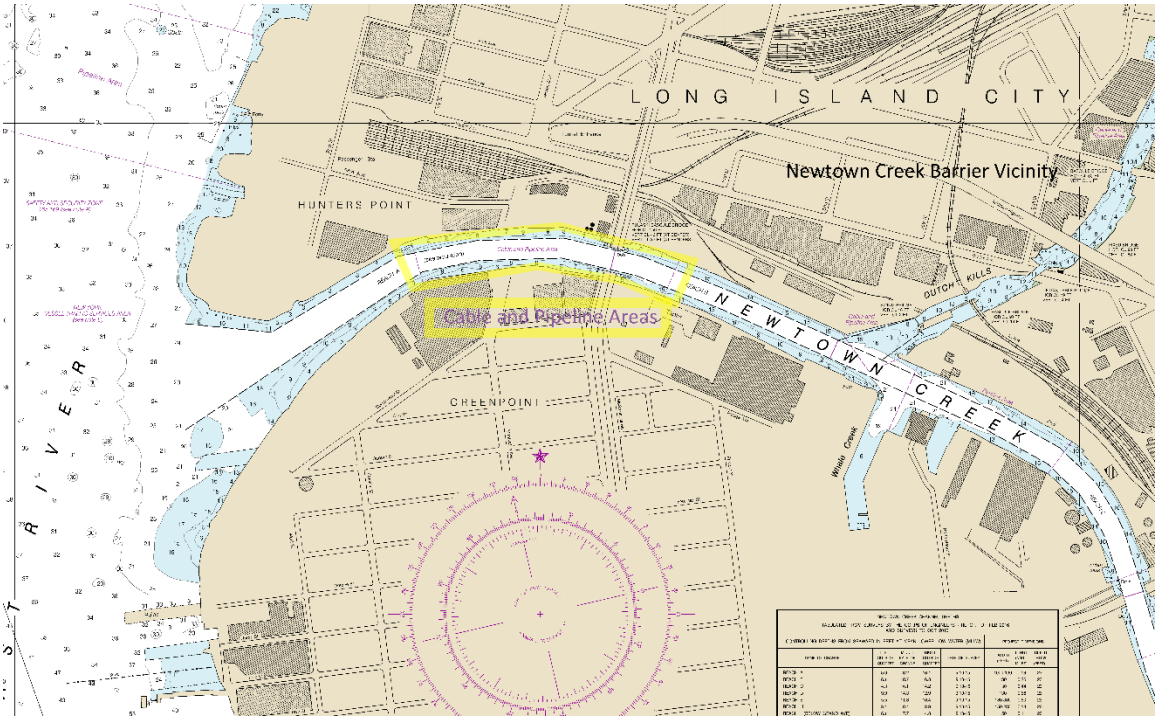


Figure B-12: Subaqueous utilities in the vicinity of the Newtown Creek barrier (adapted from NOAA Chart 12338).

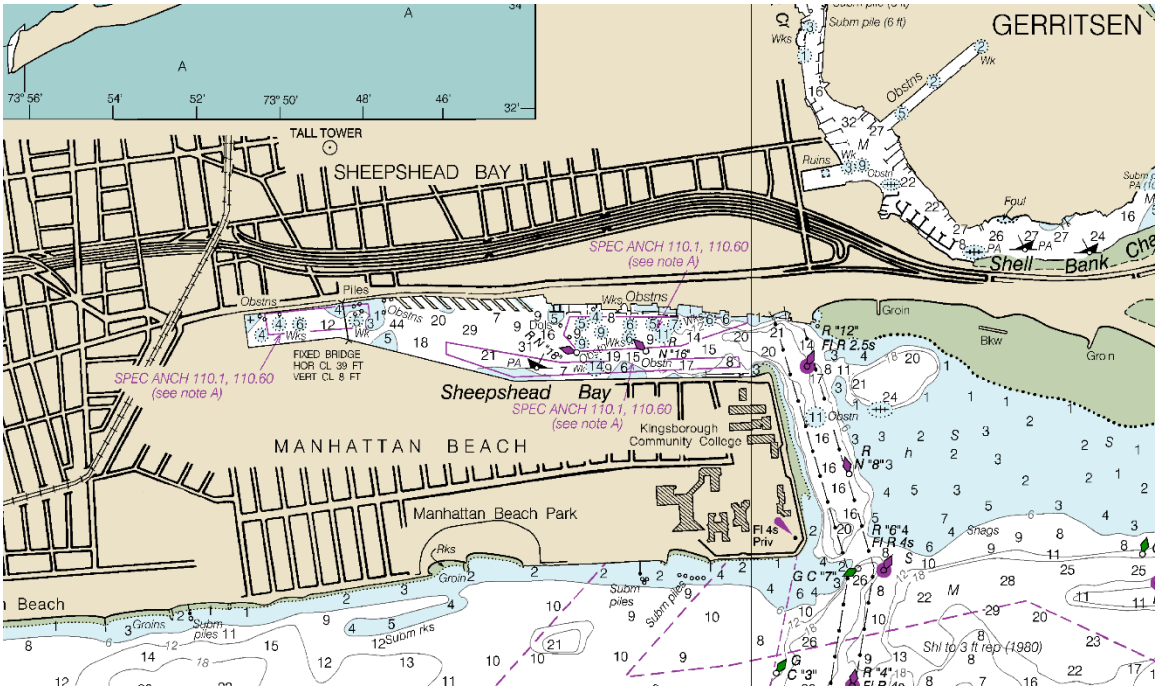


Figure B-13 Subaqueous utilities in the vicinity of the Sheepshead Bay barrier (adapted from NOAA Chart 12350). Note that there are no utilities shown at the entrance to Sheepshead Bay.

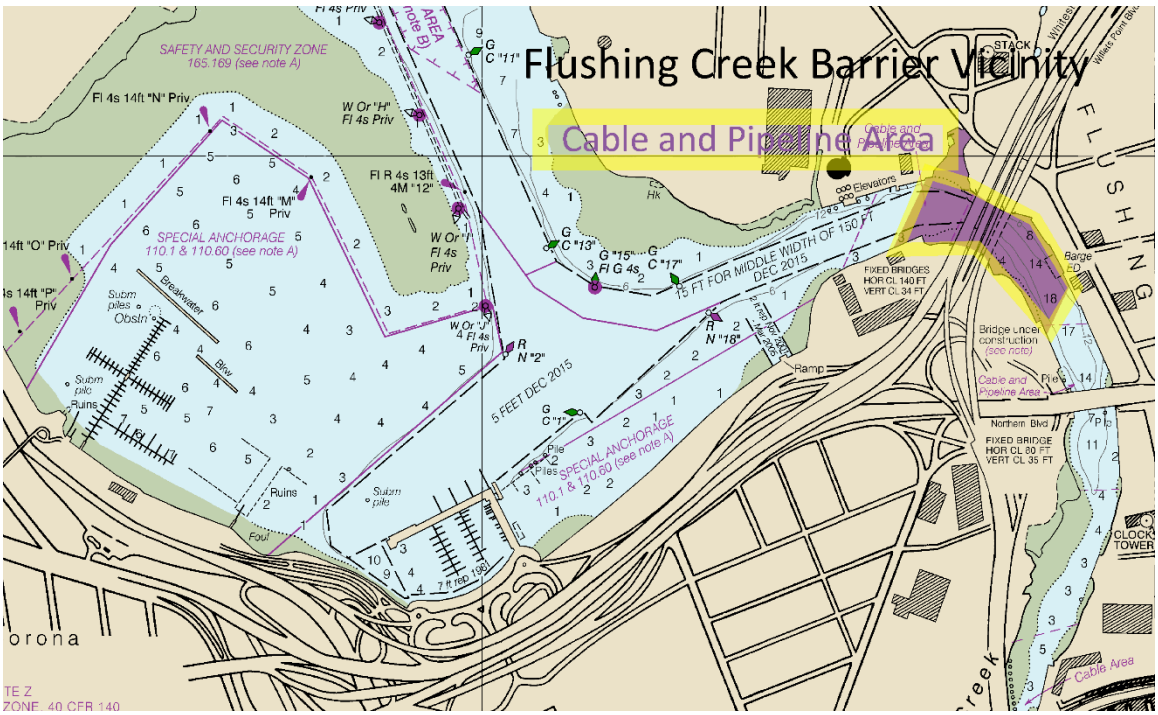


Figure B-14: Subaqueous utilities in the vicinity of the Flushing Creek barrier (adapted from NOAA Chart 12339).

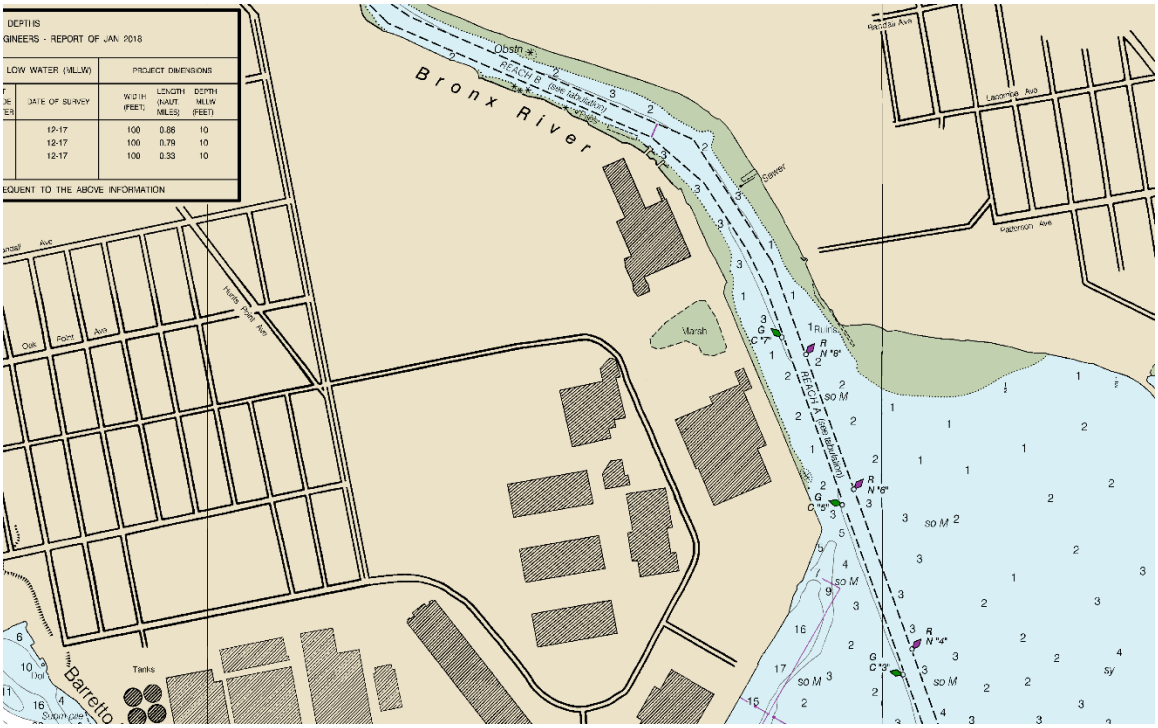


Figure B-15: Subaqueous utilities in the vicinity of the Bronx River barrier (adapted from NOAA Chart 12339). Note that the proposed barrier location is upriver and out of range of this chart. The chart indicates that there are no utilities shown at the entrance to the Bronx River.

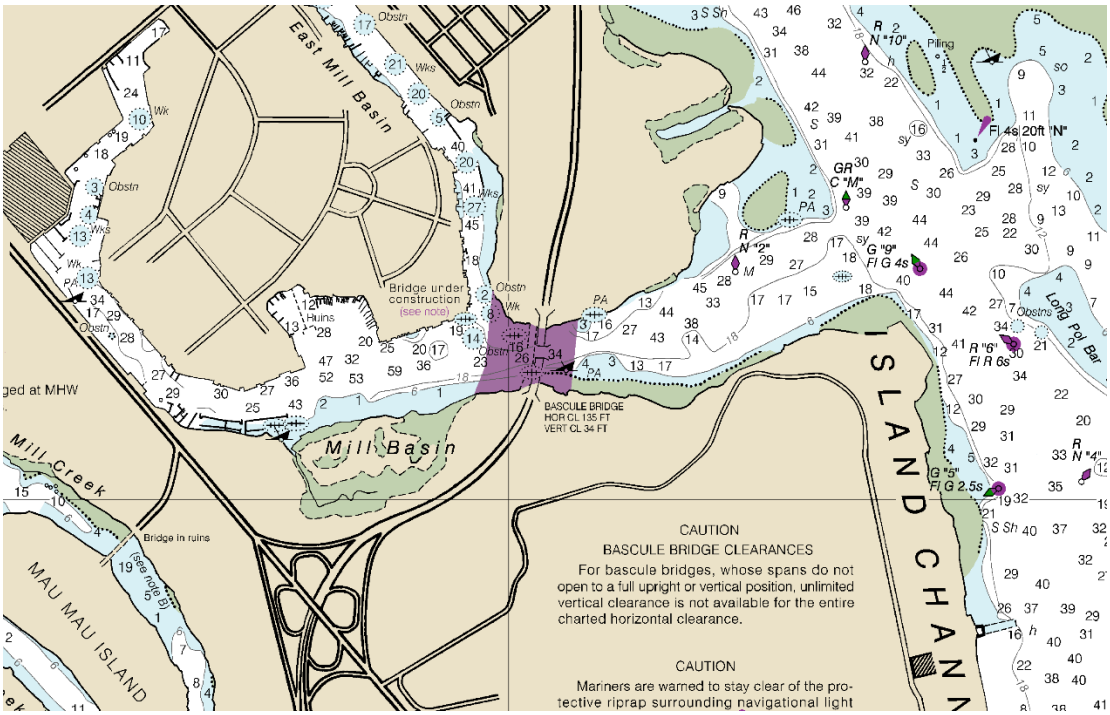


Figure B-16: Subaqueous utilities in the vicinity of the Gerritsen Creek barrier (adapted from NOAA Chart 12350). Note that there are no utilities shown in the vicinity. The purple area indicates bridge construction.

C. APPENDIX C - TIDAL DATA

Table C-1: Station 8561680, Sandy Hook

Tidal Datum	Abbreviation	NAVD88 ft	MLLW ft
Highest Observed 10/29/2012	Max Tide	9.21 (failed)	12.03 (failed)
Highest Astronomical Tide	HAT	3.78	6.60
Mean Higher-High Water	MHHW	2.41	5.23
Mean High Water	MHW	2.08	4.9
Mean Sea Level	MSL	-0.24	2.58
Mean Low Water	MLW	-2.62	0.2
Mean Lower-Low Water	MLLW	-2.82	0
Lowest Astronomical Tide	LAT	-1.37	-1.37
Lowest Observed 2/2/1976	Min Tide	-7.53	-4.71
Mean Tidal Range	MN	4.70	4.70

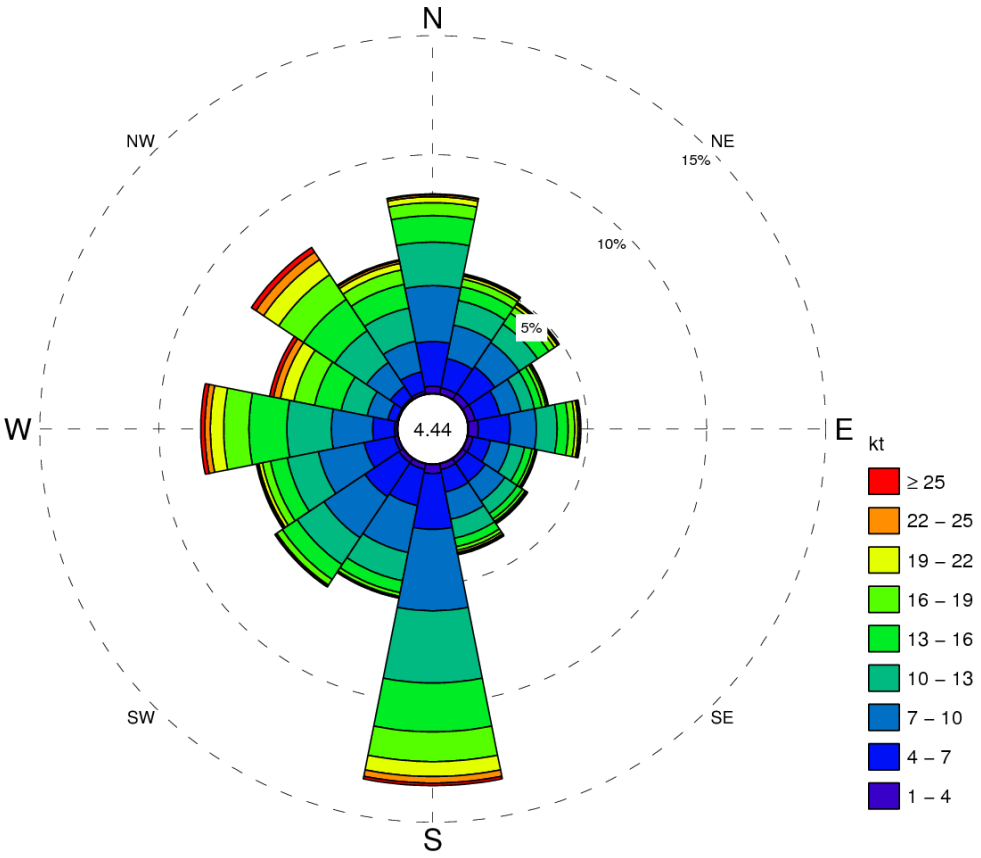
Table C-2: Station 8518750, The Battery

Tidal Datum	Abbreviation	NAVD88 ft	MLLW ft
Highest Observed 10/30/2012	Max Tide	11.27	14.04 (failed)
Highest Astronomical Tide 10/16/1993	HAT	3.58	6.35
Mean Higher-High Water	MHHW	2.28	5.05
Mean High Water	MHW	1.96	4.73
Mean Sea Level	MSL	-0.2	2.57
Mean Low Water	MLW	-2.57	0.2
Mean Lower-Low Water	MLLW	-2.77	0
Lowest Astronomical Tide 1/21/1996	LAT	-4.13	-1.39
Lowest Observed 2/2/1976	Min Tide	-7.06	-4.29
Mean Tidal Range	MN	4.53	4.53

Table C-3: Station 8516945, Kings Point

Tidal Datum	Abbreviation	NAVD88 ft	MLLW ft
Highest Observed 10/30/2012	Max Tide	10.09	14.25 (failed)
Highest Astronomical Tide 10/16/1993	HAT	5.5	9.66
Mean Higher-High Water	MHHW	3.64	7.80
Mean High Water	MHW	3.28	7.44
Mean Sea Level	MSL	-0.27	3.89
Mean Low Water	MLW	-3.88	0.28
Mean Lower-Low Water	MLLW	-4.16	0
Lowest Astronomical Tide 1/21/1996	LAT	-5.77	-1.61
Lowest Observed 2/2/1976	Min Tide	-8.17	-4.01
Mean Tidal Range	MN	7.16	7.16

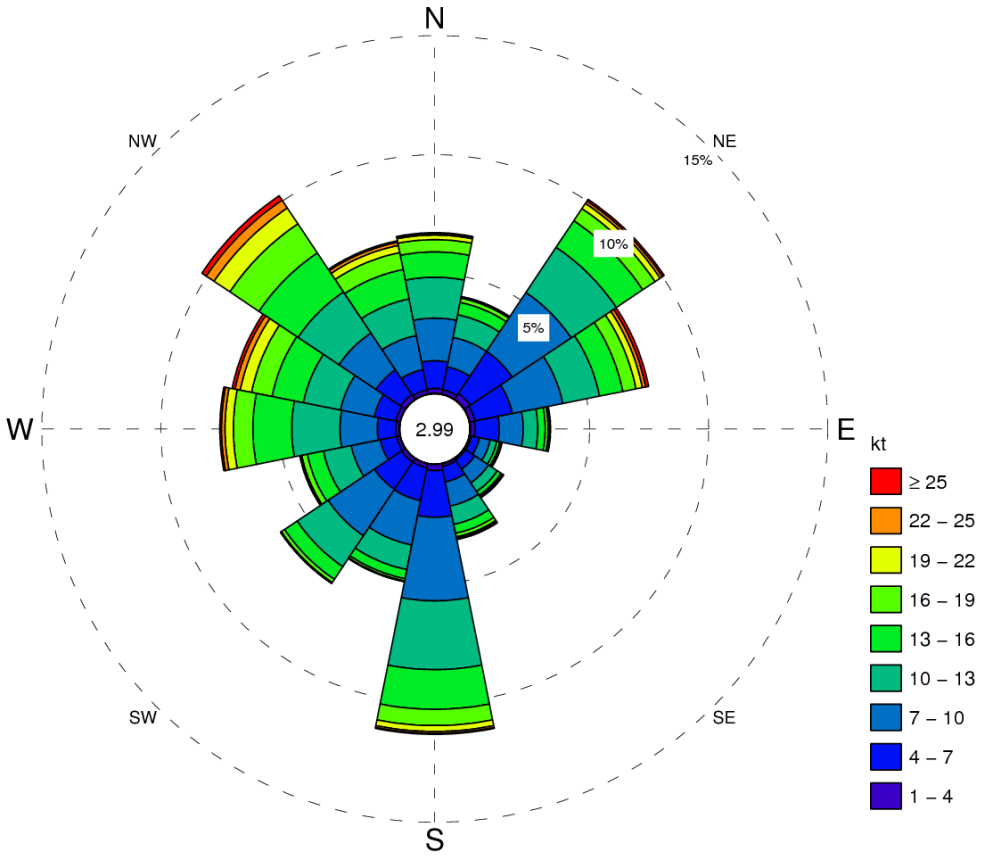
D. APPENDIX D - WIND DATA



Percentage of Occurrence

Wind Speed, kt	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	8.36	5.13	4.89	3.45	4.72	2.97	3.29	3.88	13.46	5.74	6.47	6.05	8.24	5.49	7.66	5.74	95.56
25									0.11				0.17	0.17	0.23		1.06
22									0.27				0.23	0.30	0.39	0.10	1.80
19	0.25	0.13	0.15		0.14				0.63			0.15	0.56	0.58	0.76	0.28	4.01
16	0.54	0.32	0.29	0.16	0.28	0.12	0.13	0.16	1.24	0.16	0.28	0.34	1.06	0.88	1.25	0.62	7.85
13	1.11	0.59	0.57	0.33	0.46	0.32	0.27	0.36	2.05	0.51	0.75	0.72	1.59	1.14	1.58	1.01	13.35
10	1.83	1.05	0.93	0.58	0.87	0.64	0.63	0.81	3.03	1.21	1.36	1.33	1.87	1.11	1.61	1.41	20.27
7	2.34	1.44	1.23	0.88	1.10	0.82	0.99	1.17	3.42	2.01	2.07	1.96	1.74	0.89	1.21	1.31	24.57
4	1.88	1.25	1.34	1.08	1.34	0.75	0.95	0.96	2.34	1.55	1.71	1.30	0.92	0.38	0.58	0.83	19.15
1	0.29	0.26	0.30	0.31	0.41	0.22	0.21	0.24	0.37	0.23	0.21	0.14	0.10			0.11	3.50

Figure D-1: Percentage of Occurrence for Wind Speed in knots for John F. Kennedy International Airport (Station KJFK) for a 22 year period.



Percentage of Occurrence

Wind Speed, kt	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	6.73	4.16	10.08	7.66	3.36	1.38	1.98	3.23	11.31	5.03	6.29	4.28	7.53	7.17	10.26	6.55	97.01
25				0.13										0.12	0.22		0.83
22			0.12	0.14									0.15	0.22	0.40	0.14	1.44
19	0.19		0.26	0.27					0.25				0.35	0.49	0.80	0.31	3.25
16	0.52	0.18	0.62	0.56	0.12			0.16	0.68		0.16	0.24	0.82	0.88	1.47	0.72	7.35
13	1.06	0.52	1.34	1.00	0.31	0.13	0.19	0.38	1.66	0.39	0.64	0.69	1.63	1.34	1.97	1.26	14.52
10	1.71	0.97	2.28	1.57	0.61	0.28	0.44	0.78	2.89	1.05	1.56	1.18	2.03	1.59	2.07	1.56	22.58
7	1.79	1.26	2.97	2.16	1.00	0.46	0.69	1.03	3.51	1.90	2.34	1.24	1.57	1.44	1.86	1.45	26.69
4	1.16	0.98	2.13	1.60	1.02	0.34	0.47	0.66	1.96	1.41	1.37	0.70	0.77	0.92	1.25	0.87	17.63
1	0.20	0.17	0.28	0.22	0.19			0.10	0.25	0.16	0.18	0.13	0.14	0.17	0.23	0.17	2.72
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total

Figure D-2: Percentage of Occurrence for Wind Speed in knots for La Guardia Airport (Station KLGA) for a 20 year period.

E. APPENDIX E: GEOTECHNICAL CONDITIONS

The following sections provide a brief overview of geotechnical conditions for the storm surge barrier locations discussed herein.

The selection of foundation types for the storm surge barriers will depend on the soil conditions and allowable capacities of geological formations at each location. In addition, the design loads from the gated structure, including overturning moment, uplift, and global stability will influence the final design of each barrier.

Limited site-specific data are available at this time and it is strongly recommended that through coordination with the NFS geotechnical data would be obtained for any (unrelated) projects that are in close vicinity to the proposed storm surge barrier sites. Previous studies of Gowanus Canal and Newtown Creek provide general subsurface conditions and publicly available geological maps provide an indication of subsurface conditions at East Chester Creek. For Sheepshead Bay and Gerritsen Creek the assumption is made that the geological subsurface conditions are similar to those used for the Jamaica Bay barrier.

These subsurface conditions are provided below.

East Chester Creek (geological maps, satellite imagery)

- Glacial Lake Deposits – soft to stiff varved silts, clays and sand
    - glacial lake deposits may be encountered on the south and west end of the alignment
    - glacial lake deposits increase in stiffness with depth and/or overburden thickness
- Glacial Till Deposits
    - glacial till deposits may be encountered at the surface and thinly overlying rock on the north and east end of the alignment
- Metamorphic Rock – Amphibolite
    - top of rock at or near the surface especially on the north and east end of the alignment

Westchester Creek and Bronx River

No site specific geotechnical information is available.

Flushing Creek (completed NYC EDC project at the DOT Harper Street facility)

From: Subsurface Exploration and Geotechnical Engineering Draft Report Harper Street Queens, New York - GZA GeoEnvironmental of New York, 2013.

- Organic Clayey Silt: Organic Clayey Silt with trace of fine sand, trace of roots and shells
    - The Organic Clayey Silt Stratum consists of dark gray to black, very soft organic clayey silt. This stratum was observed just below water in all water borings and

extends to depths ranging from 13 to 22 feet below MLW surface. The recorded SPT N-values in this stratum were from weight of rod ("WR") to 3 indicative of very soft soils.

- Organic Silty Clay: Organic Silty Clay with trace of fine sand, trace of roots and shells
    - The Organic Silty Clay Stratum is gray, very soft to medium stiff organic silty clay. This stratum was observed in all borings except B-8 and B-10 below the Fill Stratum in land borings or below the OS Stratum in water borings. The depths to the OC Stratum's bottom ranged from 44 to 55 feet below MLW in water borings or 37 to 72 feet below ground surface in land borings. The Organic Silty Clay Stratum contained wood, decomposed plant material, and shells. Wooden drill fragments were found in borings B-1 and B-7. The recorded SPT N-values in this stratum were generally WR to 10 blows/ft; which is indicative of very soft to stiff soils.
- Sand: Fine to coarse sand, varying amounts of silt, clay and gravel
    - The Sand stratum consists of loose to very dense sand with silt, clay, and gravel. Stratum S was encountered below Stratum OC or CS (described below) at depths of 31 to 55 feet below MLW, and 38 to 73 feet below ground surface. Borings B-1, B-3, B-7, and B-11 were terminated in this stratum, while the other borings extended through this stratum into a Hard Silty Clay Stratum (described below). The observed SPT N-values in Stratum S generally ranged from 2 blows/ft to Refusal (over 100 blows/ft) indicative of very loose to very dense soils. The thickness of this stratum ranged from 9 to 39 feet based on the borings that extended through this stratum.
- Hard silty clay: hard silty clay with trace of fine sand
    - The Hard Silty Clay Stratum consists of gray Silty Clay with little to some fine sand and trace of gravel. Stratum HC was encountered below Stratum S at depths of 59 to 77 feet below MLW, and 72 to 99 feet below ground surface. Stratum HC was observed in all water borings, but only observed in borings B-5 and B-9 on land. An obstruction was observed in boring B-2; drilling fragments suggest the obstruction was a boulder. The observed SPT N-values in this stratum generally ranged from 37 blows/ft to Refusal (over 100 blows/ft) indicative of hard soils.

Newtown Creek (CH2MHill, 2016b)

- Artificial Fill – fill materials associated with man-made development activities
- Salt-Marsh Deposits – organic silt and clay with varying amounts of sand
- Alluvium – loose, coarse to fine sands with varying amounts of gravel and silt
- Glacial Lake Deposits – varved silts, clays and sand
- Glacial Till - medium dense to dense sands and gravels with varying amounts of silt and clay

- Bedrock – gneiss

Gowanus Canal (CH2MHill, 2016a)

- Artificial Fill – Fill materials associated with man-made development activities
- Alluvium – Loose Fine Sands with minor amounts of silt
    - Alluvium gradation becomes finer in slower moving waters
- Salt-Marsh Deposits – Organic Silt and Clay with varying amounts of sand
- Glacial Till - Medium dense to dense Sands and Gravels with varying amounts of silt and clay
- Bedrock – Fordham Gneiss

Sheepshead Bay and Gerritsen Creek (see Jamaica Bay storm surge barrier description)

- Recent alluvium – loose fine sands with minor amounts of silt
    - 5 - 40 feet thick
    - Alluvium layer thickens as one moves closer to shore and thins out in the deeper portions of the bay
- Glacial outwash - medium dense to dense sands with minor amounts of fine gravel
    - Gradation becomes coarser with depth
- No bedrock encountered up to 150 feet below Mean Low Water (MLW)

A more detailed analysis regarding the subsirface conditions is recommended for the next stage, for all storm surge barriers. The analysis should investigate the existing geotechnical conditions and any available existing borings at the site. This would help further inform the conceptual design of the storm surge barrier.

F. APPENDIX F – GATE TYPE SELECTION FOR THE NAVIGABLE PASSAGE

The storm surge barriers will include navigable passages that will be gated openings with the minimum dimensions as listed in Table 4-1. The table is repeated below for completeness.

Table F-1: Navigable Passage Dimensions

Location	Federal Channel	Existing Depth (ft)	Minimum Practical Width of Opening [ft]	Minimum Depth of Opening (ft, MLLW)	Minimum Depth of Opening [ft NAVD88]	Air Clearance [ft NAVD88][1]
East Chester Creek	East Chester Creek	-5 to -20	60	-13	-17	Unrestricted
Westchester Creek	Westchester Creek	-5 to -10	60	-15	-19	48
Bronx River	Bronx River	-5 to -15	100	-13	-17	Unknown
Flushing Creek	Flushing Bay and Creek Reach B	-10 to -15	135	-18	-21	30
Newtown Creek	Newtown Creek	-15 to -20	170	-17	-20	Unrestricted
Gowanus Canal	Gowanus Canal	-10 to -15	100	-19	-22	Unrestricted
Sheepshead Bay	Sheepshead Bay	-20 to -40	100	-17	-20	Unrestricted
Gerritsen Creek[2]	-	-20 to -20	115	-16	-19	35

1. MLLW to NAVD88 datum conversions are provided for each tide gauge in Appendix B.
2. Gerritsen Creek is not a federally authorized navigation channel.

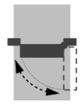
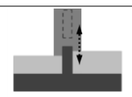
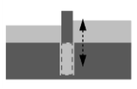
Following the requirements above and the information provided in Enclosure 2 a cursory evaluation[10] of gate types and their suitability for the navigable passage has been completed for the storm surge barriers in Table F-2. Pictograms for the various gate types are modified after

[10] It should be noted that without a complete gate type study and evaluation of gate alternatives for the storm surge barriers (inclusive of preliminary cost estimates) this can only be regarded as a preliminary assumption which will have to be verified at a later stage. The evaluation of gate types is to establish a reasonable preliminary selection of gate types such that a preliminary geometry of the openings of the barrier can be established. The geometry and its performance can then be evaluated through hydrodynamic modeling and findings can be incorporated in the next round of refinements of the barrier designs.

(Dijk, 2010). From the hydraulic gates as listed in Enclosure 2, a number are omitted from the table below as they are deemed unsuitable;

- The floating sector gate is considered too complex to be used in the smaller storm surge barriers discussed herein.
- The tainter gate is excluded as a result of the required air clearances and it would prohibitively increase gate costs

Table F-2: Cursory Review of Hydraulic Gate Types for Application Within the Navigable Passages of the Storm Surge Barriers

	GATE TYPE	East Chester Creek Navigation Gate	Westchester Creek Navigation Gate	Bronx River Navigation Gate	Flushing Creek Navigation Gate	Newtown Creek Navigation Gate	Gowanus Canal Navigation Gate	Sheepshead Bay Navigation Gate	Gerritsen Creek Navigation Gate
	Horizontal Rolling Gate	Impractical due to area needed to dock the gate	Possible, but considered challenging to operate under current conditions	Impractical due to area needed to dock the gate	Impractical due to area needed to dock the gate	Impractical due to area needed to dock the gate	Possible, but considered as impractical due to area needed to dock the gate	Impractical due to area needed to dock the gate	Impractical due to area needed to dock the gate
	Sector Gate (Vertical Axis)	Suitable, proven concept for similar span and sill elevation.	Suitable, proven concept for similar span and sill elevation.	Suitable, proven concept for similar span and sill elevation.	Suitable, proven concept for similar span and sill elevation.	Suitable, proven concept for similar span and sill elevation.	Not suitable due to geometric constraints	Suitable, proven concept for similar span and sill elevation.	Suitable, proven concept for similar span and sill elevation.
	Rotating Segment Gate	Possible, but deemed relatively expensive and complex system	Possible, but deemed relatively expensive and complex system	Possible, but deemed relatively expensive and complex system	Possible, but deemed relatively expensive and complex system	Possible, but deemed relatively expensive and complex system	Possible, but deemed relatively expensive and complex system	Possible, but deemed relatively expensive and complex system	Possible, but deemed relatively expensive and complex system
	Inflatable Gate or Dam	Possible, but deemed relatively expensive and complex system for a relatively short gate span	Possible, but deemed relatively expensive and complex system for a relatively short gate span	Possible, but deemed relatively expensive and complex system for a relatively short gate span	Possible, but deemed relatively expensive and complex system	Possible, but deemed relatively expensive and complex system	Possible, but deemed relatively expensive and complex system for a relatively short gate span	Possible, but deemed relatively expensive and complex system for a relatively short gate span	Possible, but deemed relatively expensive and complex system for a relatively short gate span
	Flap Gate	Not suitable - deemed too challenging from a maintenance perspective	Not suitable - deemed too challenging from a maintenance perspective	Not suitable - deemed too challenging from a maintenance perspective	Not suitable - deemed too challenging from a maintenance perspective	Not suitable for reverse head conditions and deemed too challenging from a maintenance perspective	Not suitable - deemed too challenging from a maintenance perspective	Not suitable - deemed too challenging from a maintenance perspective	Not suitable - deemed too challenging from a maintenance perspective
	Barge Gate	Possible, but considered relatively challenging to operate under wave and current conditions	Possible, but considered relatively challenging to operate under wave and current conditions	Possible, but considered relatively challenging to operate under wave and current conditions	Possible, but considered relatively challenging to operate under wave and current conditions	Possible, but considered relatively challenging to operate under wave and current conditions	Not suitable due to geometric constraints	Possible, but considered relatively challenging to operate under wave and current conditions	Possible, but considered relatively challenging to operate under wave and current conditions
	Miter Gate	Possible, but not selected due to potential for reverse head conditions	Possible, but not selected due to potential for reverse head conditions	Possible, but not selected due to potential for reverse head conditions	Not suitable due to the potential for reverse head conditions and a relatively large gate span	Not suitable due to the potential for reverse head conditions and a relatively large gate span	Suitable, proven concept for similar span and sill elevation.	Suitable, proven concept for similar span and sill elevation.	Suitable, proven concept for similar span and sill elevation.
	Vertical Lift Gate	Not suitable since unrestricted air clearance is required	Not suitable since the required air clearance would prohibitively increase gate costs	Not suitable since unrestricted air clearance is required	Suitable, proven concept for similar span and required air clearance	Not suitable since unrestricted air clearance is required	Not suitable since unrestricted air clearance is required	Not suitable since unrestricted air clearance is required	Suitable, proven concept for similar span and required air clearance
	Vertical Rising Gate	Possible, but deemed to challenging from a maintenance perspective	Possible, but deemed to challenging from a maintenance perspective	Possible, but deemed to challenging from a maintenance perspective	Possible, but deemed to challenging from a maintenance perspective	Possible, but deemed to challenging from a maintenance perspective	Possible, but deemed to challenging from a maintenance perspective	Possible, but deemed to challenging from a maintenance perspective	Possible, but deemed to challenging from a maintenance perspective

G. APPENDIX G – GATE TYPE SELECTION FOR AUXILARY FLOW GATES

The storm surge barriers will include, there where geometrically possible, auxiliary flow gates to minimize the impact on the water exchange through the opening in order to minimize impacts on the inner basin environmental conditions. Requirements for the dimensions for the auxiliary flow gates were established in Section 4.3 and were largely based on the remaining available space besides the gates of the main navigable passage. As a result, the Westchester Creek. Newtown Creek and Gowanus Canal storm surge barrier do not include any auxiliary flow gates. The requirements for auxiliary flow gates are summarized in Table 4-4 and is repeated here for completeness.

Table G-1: Auxiliary Gate Dimensions

Location	Existing Depth (ft)	Width of Flow Gate Opening (ft)	Depth of Flow Gate Opening (ft, NAVD88)	Required Bottom of the gate in raised position[1]
East Chester Creek	-15 to -20	75	-15	+8
Bronx River	-5 to -15	150	-7	+8
Flushing Creek	-5 to -20	75	-10	+8
Sheepshead Bay	-10 to -35	150	-20	+7
Gerritsen Creek	-10 to -20	50	-10	+7

Note:

1. Headwall Elevation is the sum of MHHW for the reference tidal station (see Section 4.3.1 Sea Level Change (+1.82 ft is used as a conservative upper bound estimate for the region) and the additional 3 ft clearance.

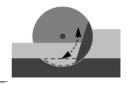
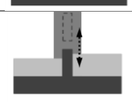
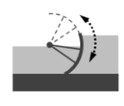
Following the requirements above and the information provided in Enclosure 2 a cursory evaluation of gate types and their suitability for both the navigable passage as well as the auxiliary flow gates has been completed for all storm surge barriers discussed herein. From the hydraulic gates as listed in Enclosure 2, a number are omitted from the table below as they are deemed unsuitable;

- The horizontal rolling gate and the sector gate (both floating and vertical axis) are impractical to function as auxiliary flow gate due to area needed to dock the gate,
- The barge gate is not suitable and too complex of a system to operate and to be used for auxiliary flow,

- The miter gate is not suitable to operate during flow conditions and as such not suitable as a flow control gate,
- The flap gate, although potentially possible in some location, is considered to be too challenging from a maintenance perspective and as such not selected for the conceptual designs
- The vertical rising gate, similar to the flap gate, could potentially be utilized in some locations, but is in general considered to be relatively expensive and too complex a system to be used as auxiliary flow gate.

The evaluation of gate types for auxiliary flow is summarized in Table G-2.

Table G-2: Cursory Review of Hydraulic Gate Types for Use as Auxilary Flow Gates for all Storm Surge Barriers

	GATE TYPE	EAST CHESTER CREEK	BRONX RIVER	FLUSHING CREEK	SHEEPSHEAD BAY	GERRITSEN CREEK
	Rotating Segment Gate	Possible, but deemed relatively expensive and complex system for relatively small auxiliary flow gate	Possible, but deemed relatively expensive and complex system for relatively small auxiliary flow gate	Possible, but deemed relatively expensive and complex system for relatively small auxiliary flow gate	Possible, but deemed relatively expensive and complex system for relatively small auxiliary flow gate	Possible, but deemed relatively expensive and complex system for relatively small auxiliary flow gate
	Inflatable Gate or Dam	Possible, but deemed relative complex system for such a relative small gate.	Possible, but deemed relative complex system for such a relative small gate.	Possible, but deemed relative complex system for such a relative small gate.	Possible, but deemed relative complex system for such a relative small gate.	Possible, but deemed relative complex system for such a relative small gate.
	Vertical Lift Gate	Suitable, proven concept	Suitable, proven concept	Suitable, proven concept	Suitable, proven concept	Suitable, proven concept
	Tainter Gate	Suitable, proven concept, however reverse head conditions will need to be investigated	Suitable, proven concept, however reverse head conditions will need to be investigated	Suitable, proven concept, however reverse head conditions will need to be investigated	Suitable, proven concept, however reverse head conditions will need to be investigated	Suitable, proven concept, however reverse head conditions will need to be investigated

US Army Corps of Engineers®
New York District

DRAFT REPORT

New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study

Shoreline Based Measures

February 2019

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LIST OF ACRONYMS

AASHTO	American Association of State Highway Transportation Officials
ACES	Automated Coastal Engineering System
ACI	American Concrete Institution
ADCIRC	Advanced Circulation
AEP	Annual Exceedance Probability
AISC	American Institute of Steel Construction
AISI	American Iron and Steel Institute
ANSI	American National Standards Institute
API	American Petroleum Institute
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
ASTM	American Society for Testing and Materials
AWS	American Welding Society
BOD	Basis of Design
CFS	Cubic Feet Per Second
CSRM	Coastal Storm Risk Management
CY	Cubic Yards
E&D	Engineering & Design
EL.	Elevation
EM	Engineering Manual
ETL	Engineering Technical Letter
ER	Engineering Regulation
FM	Factory Mutual
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
FS	Factor of Safety
FWOP	Future Without Project
FWP	Future With Project
HAT	Highest Astronomical Tide
NYNJHAT Feasibility Study	New York-New Jersey Harbor and Tributaries Feasibility Study
HSDRRS	Hurricane Storm Damage Risk Reduction System
HSS	Hollow Structural Sections
ICC	International Code Council
ISO	International Standards Organization
LAT	Lowest Astronomical Tide
LRFD	Load Resistance Factor Design
M&N	Moffatt & Nichol
MHHW	Mean Higher-High Water
MHW	Mean High Water

MSL	Mean Sea Level
MLW	Mean Low Water
MLLW	Mean Lower-Low Water
NAVD88	North American Vertical Datum of 1988
NJ	New Jersey
NOAA	National Oceanic and Atmospheric Administration
NNBFs	Natural and Nature-Based Features
NY	New York
NYC	New York City
Rev	Revision
ROW	Right of Way
SBM	Shoreline Based Measure
SLR	Sea Level Rise
TM	Technical Memorandum
TOW	Top of Wall
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
USS	United States Steel

New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study

Shoreline Based Measures

1 INTRODUCTION

1.1 Study Background

The study purpose is to determine the feasibility of coastal storm risk management measures in the New York-New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study (NYNJHAT CSRM Feasibility Study or HAT Study) study area, and to recommend a plan that will contribute to community and environmental resilience.

The study area encompasses the New York Metropolitan Area, including the most populous and densely populated city in the United States, and the six most populated cities in New Jersey. The shorelines of some of the NYNJHAT study area are characterized by low elevation areas, developed with residential and commercial infrastructure and are subject to tidal flooding during storms. The study area covers more than 2,150 square miles and comprises parts of 25 counties in New Jersey and New York.

1.2 Perimeter Flood Risk Reduction vs. Coastal Barriers

1.2.1 General

During coastal storms, storm surges are generated on the open coast and propagate through New York Harbor or through the Long Island Sound and have the potential to flood the extensive low-lying areas surrounding the metropolitan area. A typical solution for reducing flood risk is to raise the level of existing perimeter flood risk reduction systems. This solution can however, be challenging to implement in geometrically constrained urbanized areas where waterfront spaces have multiple uses and serve a variety of stakeholders such that social and economic impacts could be considerable. In large bays, estuaries, natural harbors and port entrance channels, coastal barriers constructed as integral part of a flood risk reduction systems can be a cost-effective alternative to reduce the risk of flooding for the area. As such, the HAT Study alternative includes coastal barriers (such as closure dams, storm surge barriers, tide gates and tidal locks), in combination with flood risk reduction systems.

1.2.2 Structural CSRM Measures as Part of the HAT Study

For the alternatives currently being analyzed as part of the HAT Study, the structural CSRM measures do not include any closure dams or tidal lock complexes. Structural CSRM measures under this study do include storm surge barriers, tide gate complexes and the, more typical, perimeter based structural measures such as floodwalls, levees, seawalls, deployable flood barriers, etc. Storm surge barriers are not covered within this report and more detail on the report organization is provided below.

1.3 Organization

The analysis and documentation of the preliminary engineering studies and analyses completed in support of the New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study are extensive. Therefore, the engineering work products that support the analyses of the structural CSRM measures have been divided into three parts. This report discusses the structural CSRM measures that are the essential "building blocks" of the perimeter-based risk reduction systems and are colloquially referred to as "shore based measures".

This report documents the general design criteria and conceptual design for the shoreline based flood risk reduction measures, which are components of the New York-New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study. The Coastal Storm Risk Management (CSRM) measures evaluated herein are comprised of shoreline based measures (SBMs) such as levees, floodwalls, seawalls, etc. and aim to reduce the risk of flood damages for the areas behind it. In general, SBMs are structural CSRM measures located along or inward of the shoreline where storm surge barriers span a waterbody. It should be noted that for simplicity and due to the limited number present in the study alternatives, tide gates are also included within this report. This report focuses solely on the shoreline based measures (SBMs); storm surge barriers are covered in the other two reports and hence not discussed herein. This report is part of a series of reports, provided in Table 1-1, that document the preliminary engineering studies and analyses completed in support of the feasibility of the New York – New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study.

Table 1-1: Reports in support of the Engineering Studies and Analyses for the HAT Study Alternatives.

ID	Title	Contents/Subject
R001	Conceptual Design for HAT Study Navigable Storm Surge Barriers – Volume 1	Conceptual Design for the following Storm Surge Barriers: Outer Harbor, Verrazano Narrows, Arthur Kill, Kill van Kull, Jamaica Bay, Hackensack River and Throgs Neck.
R002	Conceptual Design for HAT Study Navigable Storm Surge Barriers – Volume 2	Conceptual Design for the following Storm Surge Barriers: East Chester Creek, Westchester Creek, Bronx River, Flushing Creek, Newtown Creek, Gowanus Canal, Sheepshead Bay and Gerritsen Creek
R003	Shore Based Measures	The Structural Coastal Storm Risk Management (CSRM) measures (inclusive of Tide Gates) evaluated as part of the HAT Study other than the Storm Surge Barriers

1.4 Purpose

The purpose of this document is to provide the necessary engineering basis to support the evaluation and development of a conceptual design for the SBMs that are part of the HAT Study Alternatives. The completion of the conceptual designs will allow for an inventory of SBMs (total length and number of measures) per HAT Study Alternative and for a quantity take-off per SBM. This data set will be used by USACE-NAN as input to complete cost estimates for the study alternatives and allow for a relative comparison amongst the alternatives.

1.5 General Methodology for Development of SBM Inventory

HAT Study Alternatives were developed by the USACE New York District (USACE-NAN). Alternative 1 is a no action alternative; the other five Alternatives are composed of different combinations of SBM and storm surge barrier alignments. The discussion of each HAT Study Alternative can be found in Section 6. The purpose of the SBMs, in conjunction with the storm surge barrier, is to be an integral part of the coastal storm risk management strategy for the region and impede storm surge and reduce the risk of flooding for the area.

As its name suggested, SBMs are composed of flood risk reduction measures (e.g. levees and floodwalls) located along or inward of the shoreline. Publicly available qualitative data and aerial images were used such that the existing shoreline typology (e.g. natural shoreline, urban waterfront development, etc.) associated with each alignment could be categorized. Based on the existing shoreline typologies that prevail throughout the study area, a reasonable, but limited number, of prototypical SBMs were developed.

Again, the purpose of the study was to develop conceptual SBMs designs that are comprehensive enough to be applicable for the realm of conditions found throughout the study area yet not too detailed or site specific such that they could only be applied at one location. Due to the vast area covered by the HAT Study, generalized assumptions were made about existing conditions such as site topography, bathymetry, and soil parameters. It is emphasized that no site specific topographic survey, bathymetric survey, site condition survey and/or geotechnical analyses have been completed. Instead, publicly available qualitative data and results from desktop analyses were used to develop conceptual SBMs design, which can be used as the basis of a quantity take-off.

A quantity per linear foot for each SBM was determined; and subsequently, the SBM quantity for each Alternative was calculated. This information will be used by USACE-NAN to perform additional analyses and used as basis for cost estimates of the HAT Study Alternatives. This report focuses solely on the development of the conceptual designs, an inventory of measures per HAT Study Alternative and a quantity take-off per measure.

1.6 Limitations and Reader's guide

Section 2 provides an overview of the Project, while Section 3 provides the generalized design criteria used for the development of the SBMs. Section 4 provides the Conceptual design for the

SBMs developed for the HAT Study, and Section 5 provides the inventory of SBMs for all HAT Study Alternatives. The listed design criteria and conceptual designs are based on qualitative data and desktop level analysis analogous with the level of detail commensurate of a feasibility study and USACE's SMART planning principles. In instances where limited data was available, assumptions were made based on engineering judgement, previous experience and/or the partial data that had been collected over the course of the feasibility study phase. The implications of such assumptions along with recommendations for further data collection and refined analysis to support the design are described in Section 6.

2 PROJECT OVERVIEW

2.1 NYNJHAT CSRM Feasibility Study Alternatives

The locations of the SBMs vary amongst the five study alternatives as described in the succeeding sections. The alignment of each study alternative was developed by USACE with small modifications by the A/E where appropriate. It should be noted that all Alternatives, except Alternative 1, incorporate SBMs.

2.1.1 Alternative 1

Alternative 1 is a no action alternative. Alternative 1 does not incorporate any shoreline based measures.

2.1.2 Alternative 2

Alternative 2 incorporates SBMs in combination with the Outer Harbor storm surge barrier connecting Sandy Hook, New Jersey to Rockaway Point on the Rockaway Peninsula, as well as the storm surge barriers at Throgs Neck and Pelham Bay. The schematic concept for this Alternative is shown in Figure 2-1.

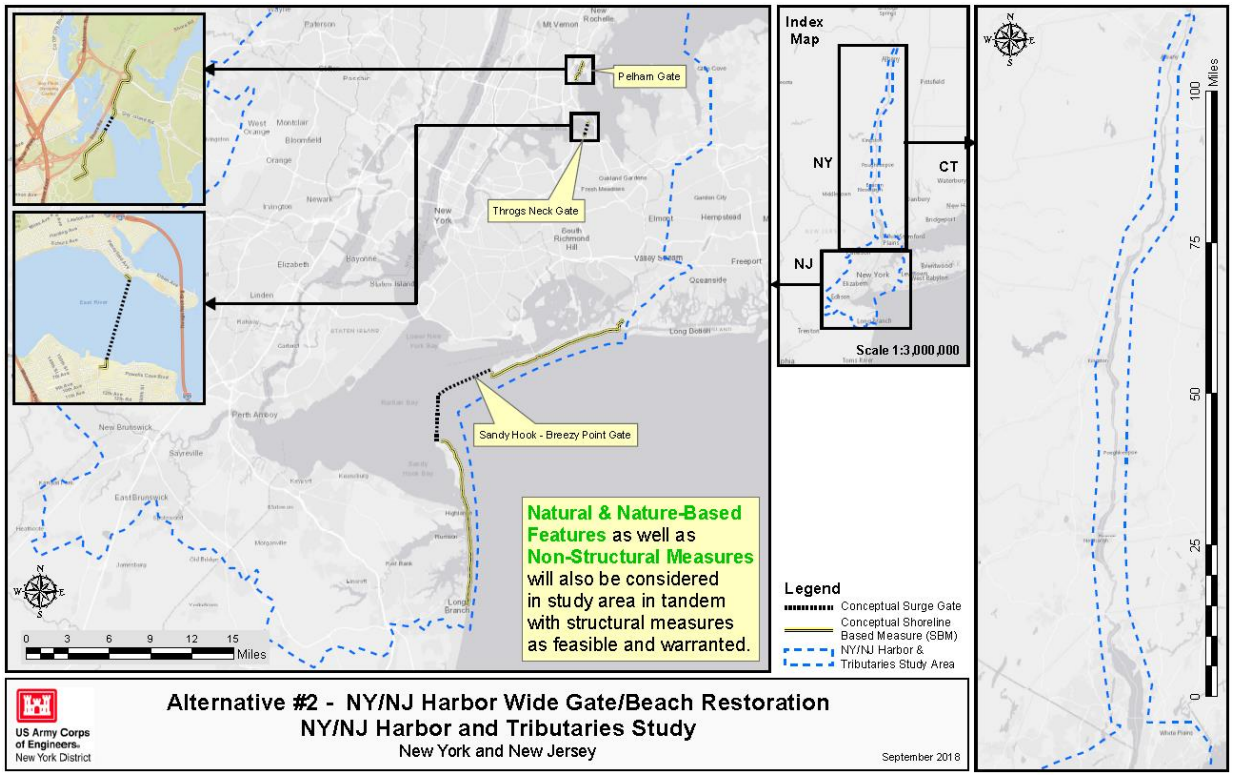

Figure 2-1: Alternative 2 Proposed Measures (USACE, Ref. 7)

2.1.3 Alternative 3A

Alternative 3A integrates SBMs with the storm surge barriers at Verrazano-Narrows, Arthur Kill, Throgs Neck, Jamaica Bay, and Pelham Bay. The schematic concept for this Alternative is shown in Figure 2-2.

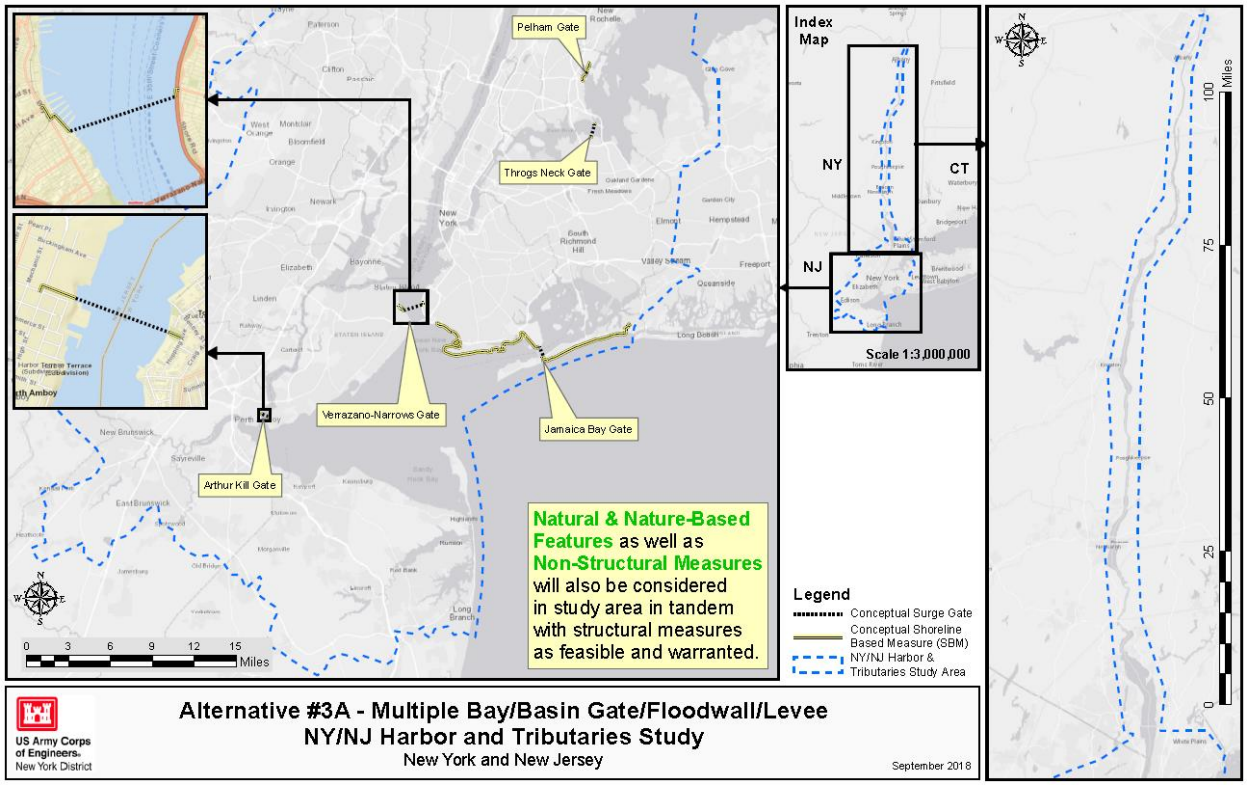

Figure 2-2: Alternative 3A Proposed Measures (USACE, Ref. 7)

2.1.4 Alternative 3B

Alternative 3B integrates SBMs along with the Arthur Kill, Kill Van Kull, Jamaica Bay, Newtown Creek, Gowanus Canal, Pelham Bay, Bronx River/Westchester Creek, and Flushing Creek storm surge barriers. The required SBMs include protection of the New Jersey Upper Bay and Hudson River shoreline from Liberty State Park to Hoboken, New York City West Side shoreline from Brooklyn Bridge to Pier 78, East Harlem shoreline from Carl Schurz Park to Washington Heights, Astoria shoreline from 80th street to Astoria Park, Long Island City-Astoria shoreline from Astoria Park to Ed Koch Queensboro Bridge, Yonkers North shoreline, Yonkers South shoreline, Haverstraw shoreline, Haverstraw perimeter, Ossining shoreline, and Tarrytown shoreline. A schematic concept for this Alternative and the referenced reaches i shown in Figure 2-3.

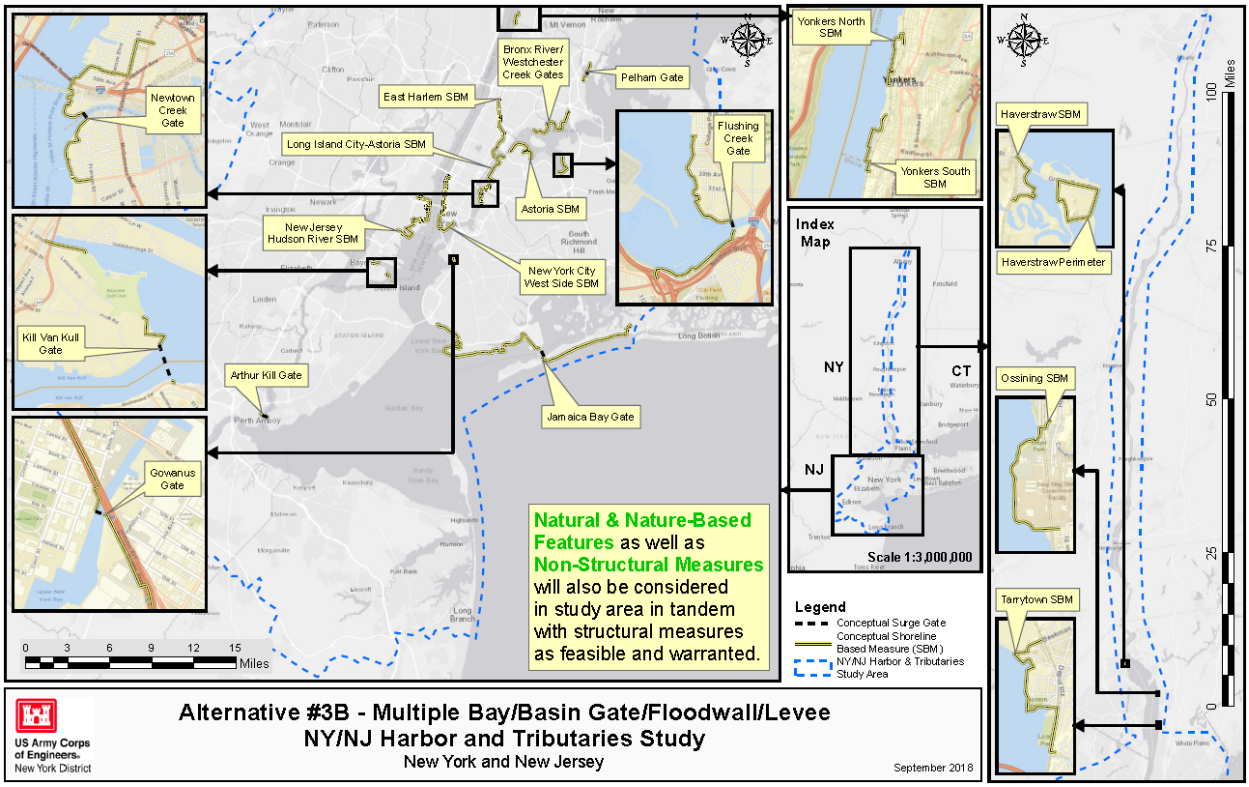

Figure 2-3: Alternative 3B Proposed Measures (USACE, Ref. 7)

2.1.5 Alternative 4

Alternative 4 incorporates SBMs along with the storm surge barriers at Jamaica Bay, Newtown Creek, Gowanus Canal, Pelham Bay, Bronx River/Westchester Creek, and Flushing Creek, and Hackensack River. These SBMs are located at the New Jersey Upper Bay and Hudson River shoreline from Liberty State Park to Hoboken, New York City West Side shoreline from Brooklyn Bridge to Pier 78, East Harlem Shoreline from Carl Schurz Park to Washington Heights, Yonkers North shoreline, Yonkers South shoreline, Haverstraw shoreline, Haverstraw perimeter, Ossining

shoreline, and Tarrytown shoreline. A schematic concept for this Alternative is shown in Figure 2-4.

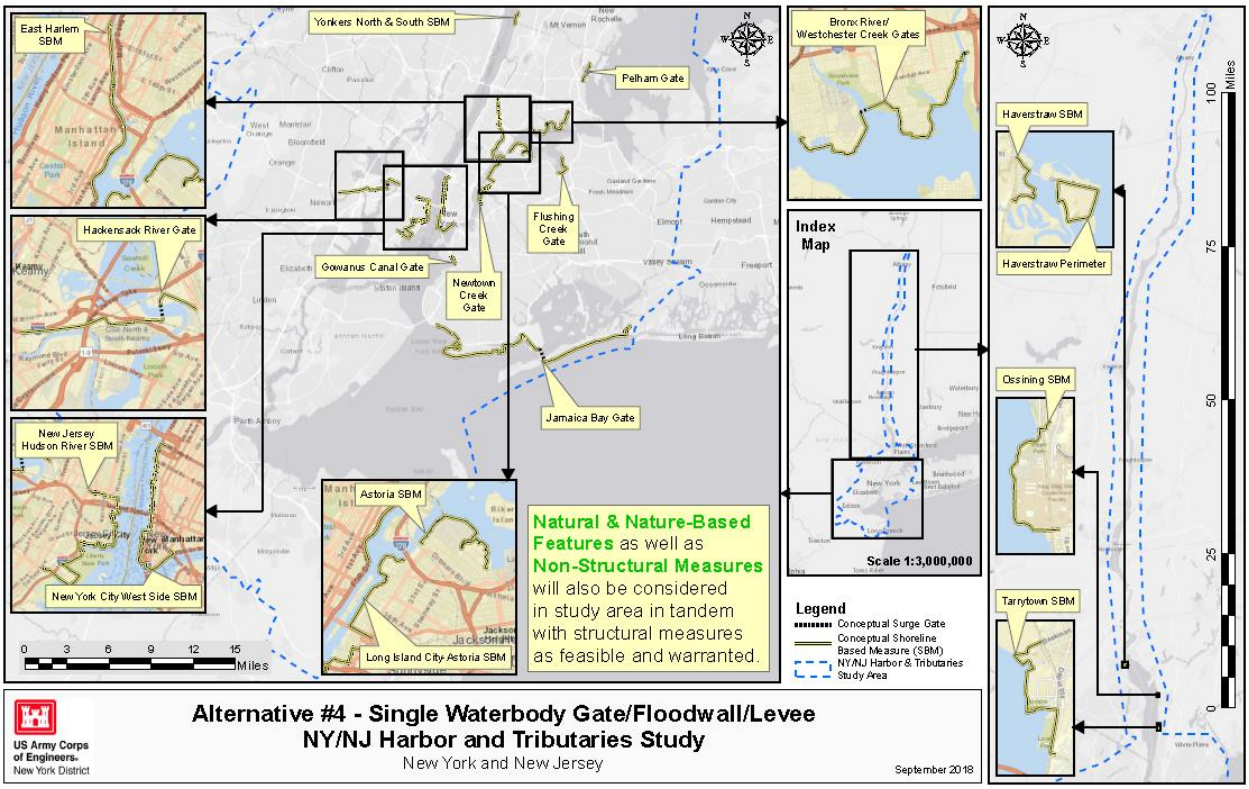

Figure 2-4: Alternative 4 Proposed Measures (USACE, Ref. 7)

2.1.6 Alternative 5

Alternative 5 presents a perimeter only risk reduction concept which excludes storm surge barriers that traverse waterways or waterbodies. SBMs would be implemented at the New Jersey Upper Bay and Hudson River shoreline, New York City West Side shoreline, Long Island City-Astoria shoreline, Astoria shoreline, East Harlem shoreline, Hackensack Perimeter Lower, Middle and Upper Areas, Yonkers North shoreline, Yonkers South shoreline, Haverstraw shoreline, Haverstraw perimeter, Ossining shoreline, and Tarrytown shoreline. A schematic concept for this Alternative is shown in Figure 2-5.

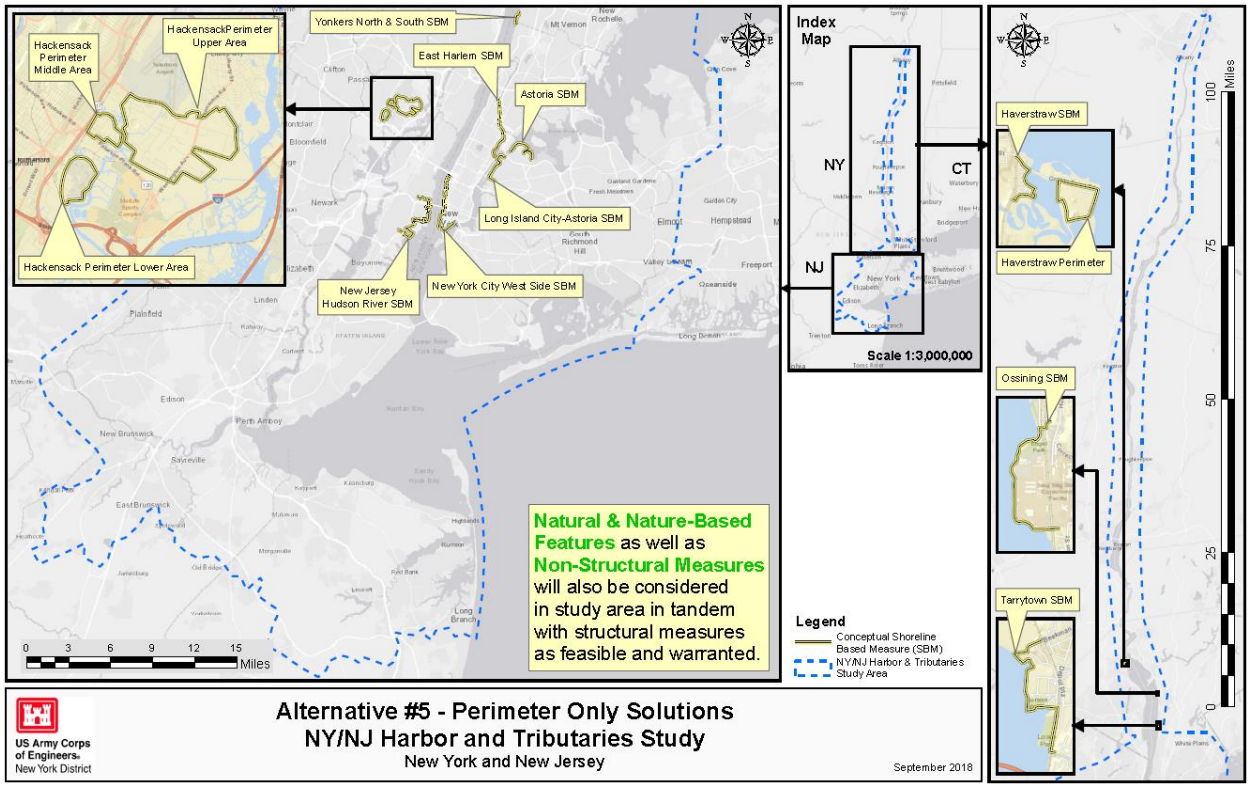

Figure 2-5: Alternative 5 Proposed Measures (USACE, Ref. 7)

2.1.7 Overview of SBM Alignments

Several Alternatives share the same SBM alignments, such as the East Harlem SBM, which are included within Alternatives 3B, 4 and 5. Figure 2-6 provides a complete overview of all SBM alignments considered. However, Northern and Southern extent of the study area are not shown due to the scale of the map. Table 2-1 provides an overview of all HAT Study Alternatives and reaches that include SBMs.

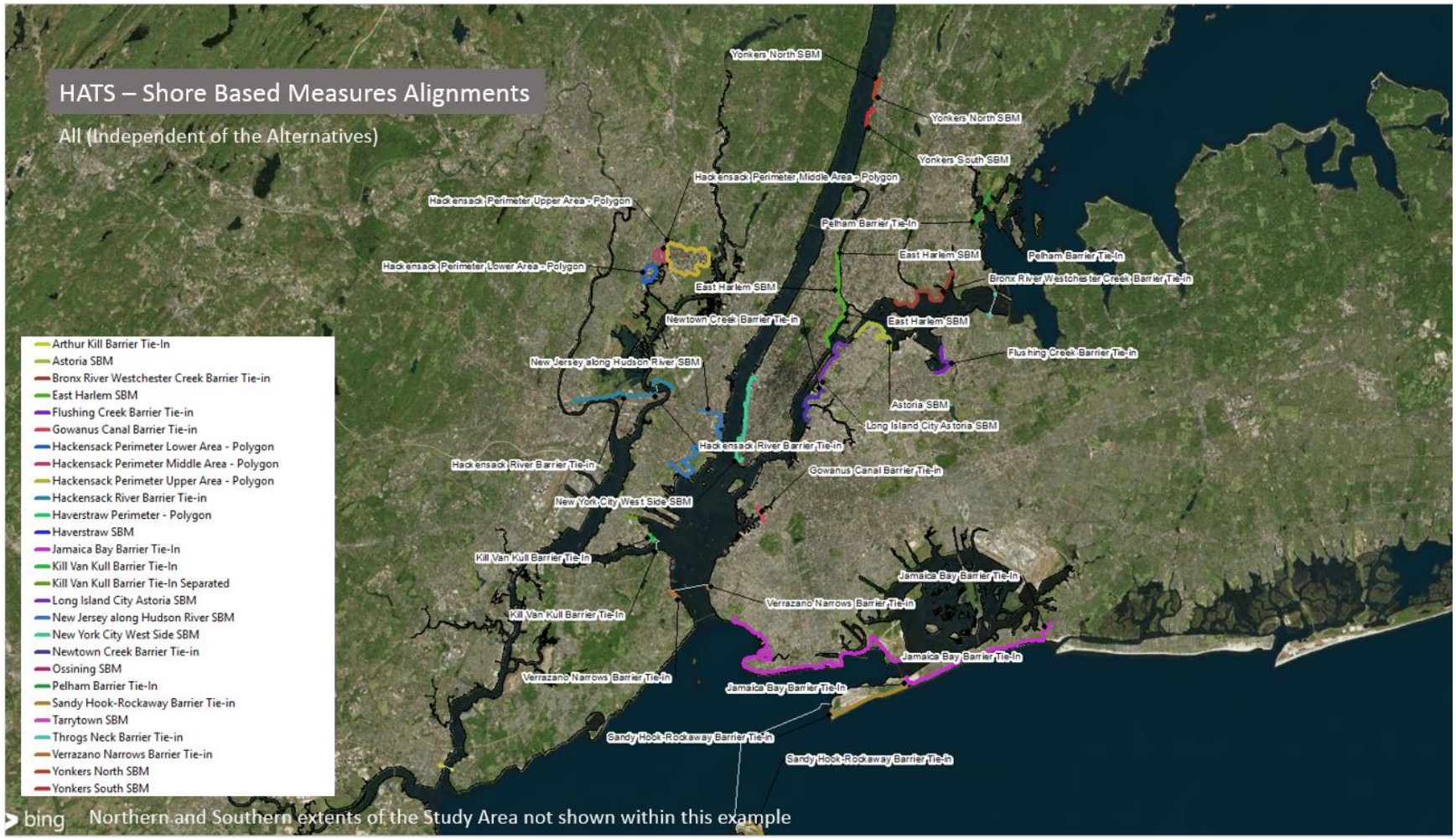

Figure 2-6: Overview of Reaches that include SBMs

Table 2-1: Overview of SBM Reaches and Inclusion per HAT Study Alternative

Name of Shore based Measure Alignment	HAT Study Alternative				
	2	3A	3B	4	5
Arthur Kill Barrier Tie-In	No	Yes	Yes	No	No
Astoria SBM	No	No	Yes	Yes	Yes
Bronx River Westchester Creek Barrier Tie-in	No	No	Yes	Yes	No
East Harlem SBM	No	No	Yes	Yes	Yes
Flushing Creek Barrier Tie-in	No	No	Yes	Yes	No
Gowanus Canal Barrier Tie-in	No	No	Yes	Yes	No
Hackensack Perimeter Lower Area - Polygon	No	No	No	No	Yes
Hackensack River Barrier Tie-in	No	No	No	Yes	No
Haverstraw Perimeter - Polygon	No	No	Yes	Yes	Yes
Haverstraw SBM	No	No	Yes	Yes	Yes
Jamaica Bay Barrier Tie-In	No	Yes	Yes	Yes	No
Kill Van Kull Barrier Tie-In	No	No	Yes	No	No
Kill Van Kull Barrier Tie-In Separated	No	No	Yes	No	No
Long Island City Astoria SBM	No	No	Yes	Yes	Yes
New Jersey along Hudson River SBM	No	No	Yes	Yes	Yes
New York City West Side SBM	No	No	Yes	Yes	Yes
Newtown Creek Barrier Tie-in	No	No	Yes	Yes	No
Ossining SBM	No	No	Yes	Yes	Yes
Pelham Barrier Tie-In	Yes	Yes	Yes	Yes	No
Sandy Hook-Rockaway Barrier Tie-in	Yes	No	No	No	No
Tarrytown SBM	No	No	Yes	Yes	No
Throgs Neck Barrier Tie-in	Yes	Yes	No	No	No
Verrazano Narrows Barrier Tie-in	No	Yes	No	No	No
Yonkers North SBM	No	No	Yes	Yes	Yes

2.2 System of Units

US customary units shall be used.

2.3 Vertical and Horizontal Reference

The vertical datum for the project shall be NAVD88, Geoid 12B. All elevations throughout the report are referenced to NAVD88 Geoid12B unless otherwise stated.

The horizontal datum shall be the North American Datum of 1983 (NAD83) State Plane.

3 GENERAL DESIGN CRITERIA – ABBREVIATED

3.1 Introduction

As part of advancing a conceptual design for the SBMs, several criteria need to be established and defined. These criteria are defined quantitatively where possible, or qualitatively otherwise. Due to the preliminary nature of this feasibility study, the size of the study area and the varying site conditions for all locations where the SBMs were proposed as part of the HAT Alternatives; the following criteria should not be seen as comprehensive, all-encompassing or complete. Instead, the requirements and criteria form the basis for an iterative design approach, of which this feasibility analysis represents the first phase.

Decisions that impact the final design are highlighted and, where possible, a discussion is included for issues that need to be addressed as the study, the HAT Alternatives, and the designs for the SBMs advance.

3.2 Design Standards, Codes and Guidelines

The following codes, references, and standards were used as a basis for the design of the SBMs:

1. American Association of State Highway and Transportation Officials (AASHTO). *Standard Specifications for Highway Bridges, 17th Edition.*
2. AASHTO. *Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Customary U.S. Units, 6th Edition.*
3. American Concrete Institute (ACI). *ACI 350-06 Code Requirements for Environmental Engineering Concrete Structures.*
4. ACI. *ACI 318-11 Building Code Requirements for Reinforced Concrete.*
5. American Institute of Steel Construction (AISC). *Manual of Steel Construction, Load and Resistance Factor Design (LRFD), 14th Edition.*
6. American Society of Civil Engineers (ASCE). *ASCE 24-14 Flood Resistant Design and Construction.*
7. ASCE. *ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures.*
8. American Welding Society (AWS). *ANSI/ AWS D1.1-2010 Structural Welding Code – Steel.*
9. Federal Emergency Management Agency (FEMA). *Guidance for Flood Risk Analysis and Mapping – Coastal Structures, November 2015.*
10. Office of the Federal Register. *Code of Federal Regulations (CFR) – 44 CFR 65.10.*
11. United States Steel (USS). *U.S. Steel Sheet Piling Design Manual, 1984.*
12. United States Army Corps of Engineers (USACE). *HSDRRSDG Hurricane and Storm Damage Risk Reduction Design Guidelines with June 2012 updates.*
13. *USACE. Engineer Circular (EC) 1110-2-6067 USACE Process for the National Flood Insurance Program (NFIP) Levee System Evaluation.*

14. USACE. *Engineer Manual (EM) 1110-2-1614 Design of Coastal Revetments, Seawalls and Bulkheads.*
15. USACE. *Engineer Manual (EM) 1110-2-2100 Stability Analysis of Concrete Structures*
16. USACE. *Engineer Manual (EM) 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures.*
17. USACE. *Engineer Manual (EM) 1110-2-2504 Design of Sheet Pile Walls.*
18. USACE. *Engineer Manual (EM) 1110-2-2906 Design of Pile Foundations.*
19. USACE. *Engineer Regulation (ER) 1100-2-8162 Incorporating Sea Level Change in Civil Works Programs.*
20. USACE. *Engineer Engineering Technical Letter (ETL) 1110-2-58 Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, And Appurtenant Structures.*
21. USACE. *Engineer Engineering Technical Letter (ETL) 1110-2-2105 Design of Hydraulic Steel Structures.*

3.3 Functional Requirements and Design Limitations

The following functional requirements have been identified for the conceptual design of the SBMs consistent with the overall objectives of the HAT Study:

1. The measures shall provide a reliable structural measure as part of the HAT Study Alternatives to reduce the risk of coastal storm damage to the coastal region behind it;
2. The measures shall minimize any adverse effect on any existing infrastructure in the study area;
3. The measures shall minimize any adverse effect on existing access and egress, such as access to existing private and public space; and
4. The measures shall seek to minimize footprint and impact to the environment.

Based on the abovementioned functional requirements, conceptual SBMs were developed for the HAT Study Alternatives. The prototypical SBMs include the following measure types: levee, floodwall, promenade, seawall, operable floodgate, flip-up barrier, and composite seawall with beach and dune.

Due to the large size and the varying site conditions throughout the study area and the preliminary nature of the HAT Study, a limited series of reasonable but conceptually generic SBMs were developed that can be used throughout the HAT Study area, and are applicable to all HAT Study Alternatives.

3.4 Service Life

CSRM measures and CSRM projects have components for which Life Cycle Design considerations would need to be completed. At this stage of the study no such analysis has been assessed and evaluated. A project service life of 50 years is preliminarily recommended as a result of the size and nature of the project. The project will perform to meet the design criteria related to

flood risk reduction in this document for a 50-year period spanning the years between 2030 and 2080. The project is to be designed for sea level rise, regional subsidence, and local settlement occurring for 50 years, assuming project construction completion in 2030. After such a time, to achieve the same level of risk reduction, the structures would likely have to be upgraded.

3.5 Site Conditions

3.5.1 Bathymetry and Topography

The site conditions for all SBMs locations were gathered using existing nautical charts in combination with flood extent maps, aerial maps, LiDAR data and land use maps that were readily available for the study area.

3.5.2 Geotechnical

No site-specific geotechnical investigation or analysis was completed as part of this study. It should be emphasized that conceptual SBMs are not site-specific but are expected to be used throughout the study area as part of any of the HAT Study Alternatives. However, For SBMs other than the composite seawall with beach and dune, geotechnical analysis and data reported in the Atlantic Coast of New York, East Rockaway Inlet to Rockaway Inlet Draft Hurricane Sandy General Revaluation Report Engineering Appendix (USACE 2016, Ref. 1) was used for indicative soil conditions. Generally, the soils are classified as fine to medium grained, poorly graded sand classified as "SP" by the Unified Soils Classification System. The soil conditions are for the Jamaica Bay area and are a conservative representation of potential soil conditions throughout the HAT study area. Since the HAT study area is expected to have deposits of alluvium, consisting of mainly granular deposits, the soil assumption is generally representative for structures located along the waterfront and adequate for this level of study.

For composite seawall with beach and dune, the assumed geotechnical conditions were taken from Draft Final Integrated Hurricane Sandy General Reevaluation Report and Environmental Impact Statement Atlantic Coast of New York, East Rockaway Inlet to Rockaway Inlet and Jamaica Bay, Appendix A1 (USACE 2018a, Ref. 5).

3.5.3 Tides and Water Levels

Tidal datums for the SBMs are based upon tidal data from NOAA station 8531680 – Sandy Hook, NJ which is summarized in Table 3-1.

Table 3-1: Tidal datums for Station 8531680 (Sandy Hook, NJ)

Tidal Datum	Abbreviation	NAVD88 ft	MLLW ft
Highest Observed 10/29/2012 (Hurricane Sandy)	Max Tide	9.21	12.03
Highest Astronomical Tide	HAT	3.78	6.60
Mean Higher-High Water	MHHW	2.41	5.23
Mean High Water	MHW	2.08	4.9
Mean Sea Level	MSL	-0.24	2.58
Mean Low Water	MLW	-2.62	0.2
Mean Lower-Low Water	MLLW	-2.82	0
Lowest Astronomical Tide	LAT	-1.37	-1.37
Lowest Observed 2/2/1976	Min Tide	-7.53	-4.71



Figure 3-1: NOAA Stations with Sandy Hook, NJ NOAA Station 8531680 marked

3.5.4 Sea Level Change

Sea level change values are based on the most conservative values gathered from the gauges located within the study area. The Sandy Hook gauge has the highest predicted sea level change

values. The USACE low scenario from 1992 (base year to 2018) and USACE intermediate scenario from 2018 to 2080 (the end of the service life) were used to develop the sea level change values as shown in Table 3-2.

Table 3-2: Sea Level Change per ER 1100-2-8162 (USACE) in feet

Sea Level Rise	1992 (base year)	2018	2030	2080
Sea Level Change under the Intermediate Scenario	0	0.26	0.56	1.80

3.5.5 Extreme Still Water Level (SWL)

The extreme still water levels vary across the HAT Study area. The results from North Atlantic Coast Comprehensive Study (NACCS), completed by the USACE in 2015, include the computed water-level statistics corresponding to different confidence intervals based on Advanced Circulation (ADCIRC) model data, and were used to inform the design water levels for this study. The design still water level (SWL) is based on a 1% Annual Exceedance Probability (AEP) with a 50% confidence limit (CL) adjusted to the start date of the project (2030) and an intermediate sea level rise assumption for the fifty-year period between 2030 and 2080 as discussed in Section 3.5.4. It is recognized that the SWL varies throughout the study area, but for the structural designs of the SBMs, this spatial variation is temporarily omitted and a representative average value for both the SWL and wave conditions are selected. This assumption allows the structural SBM designs to be based upon conservative loading assumptions, being implementable at all coastal edges throughout the study area. At the same time, this assumption allows the team to limit the number of variations of SBM designs (i.e. no separate designs are needed for each location where wave and SWL conditions vary).

Annual exceedance probability (AEP) statistics for still water levels is discussed in Appendix A. The design 1% AEP SWL for the SBMs is summarized in Table 3-3. In addition, the 0.2% (500 year RP) is noted as well. For the composite seawall with beach and dune, the design SWL, which is slightly lower than report below, was noted in *Atlantic Coast of New York, East Rockaway Inlet to Rockaway Inlet Draft Hurricane Sandy General Revaluation Report and Environmental Impact Statement Engineering Appendix* (Ref. 5).

Table 3-3: 1% AEP Still Water Levels (Expected Value) in 2080 inclusive of Sea Level Rise

AEP	Return Period	Still Water Level (2080)
	years	feet
1%	100	12.8*
0.2%	500	16.3
*For composite seawall with beach and dune concepts, see Table 4-2 in USACE 2018a (Ref.5)		

3.5.6 Extreme Wave Heights and Periods

The expected significant wave heights for the 1% AEP (i.e. 100-year return period) at the end of the project service life were based on NACCS and the Simulation of Waves Nearshore (SWAN), see also Ref 1. SBMs were designed for the 1% AEP wave loading conditions and as described in the previous section regarding SWL, a conservative but representative value was used for all SBM structural designs. The wave data are described in detail in Appendix A. The table below shows the 1% AEP design wave height and period for all SBMs except for the composite seawall with beach and dune.

For the composite seawall with beach and dune, the design SWL was noted in *Atlantic Coast of New York, East Rockaway Inlet to Rockaway Inlet Draft Hurricane Sandy General Revaluation Report and Environmental Impact Statement Engineering Appendix* (Ref. 5).

Table 3-4: 1% AEP Design Wave Characteristics

Return Period	Wave Height (Hs)	Peak Period (Tp)
years	feet	seconds
100	5*	4*
***For composite seawall with beach and dune concepts, see Table 4-4 in USACE 2018a (Ref.5)**		

3.5.7 Overtopping and Freeboard Requirements

Two overtopping requirements were used to determine the structural elevations for the SBMs.

For all SBMs, with the exception of the prototypical composite seawall with beach and dune, an overtopping criterion of 1.08×10^{-2} cubic foot per second per foot (cfs per foot) or 1 liter per second per meter (L/s/m) is set to determine the structure height. This criterion is applied at the end of the project service life. To determine the required freeboard for the given overtopping criteria, the design 1% AEP (100-year) SWL and 1% AEP (100-year) waves as discussed in Section 3.5.5 and 3.5.6 respectively, were used[1]. The analysis for the freeboard calculation is described in further detail in Appendix A.

In addition, a resiliency check was included where the design structural elevation should be set such that it is equal to or above the 0.2% AEP (500-year) SWL.

[1] The exception is the Composite Seawall with Beach and Dunes which is based on previously developed preliminary designs for the Atlantic Shorefront as documented in Ref. 5 and further detailed in section 4.5.7

3.6 Design Loads

3.6.1 Dead Loads

Structures shall be designed for material self-weight and any superimposed dead load. The unit weights of materials frequently used as part of the conceptual design of SBMs are shown in Table 3-5.

Table 3-5: Material Unit Weights

Material Unit Weight	pound/ft^3
Steel	490
Concrete (normal weight)	150
Water (salt)	64
Riprap (armor, toe and underlayer stone)	132
Fully-Compact Granular Fill, Wet	120
Fully-Compact Granular Fill, Effective	120

3.6.2 Live Loads

The structures shall be designed for a uniform live load of 100 psf per ASCE 7-10.

3.6.3 Wind Loads

The structure shall be designed for a Category IV in accordance with ASCE 7-10.

3.6.4 Seismic Loads

Seismic loads shall be determined per ASCE 7-10 and The New York City Seismic Code. This loading scenario was not assessed for this feasibility stage but will be considered during PED.

3.6.5 Vessel Impact Loads

For shoreline based measures that are directly adjacent to waterways, vessel impact loading will need to be considered. This loading scenario was not assessed for this feasibility stage but will be considered during PED.

3.6.6 Debris Impact Loads

For shoreline based measures that are directly adjacent to waterways, debris impact loading will need to be considered. This loading scenario was not assessed for this feasibility stage but will be considered during PED.

3.6.7 Flood Loads

Hydrostatic loads for the 1% AEP (100-year return period) design storm condition are based on the design 1% AEP still water level (SWL) listed in the Section 3.5.5.

Wave loads shall be considered using the 1% AEP (100-year return period) wave conditions defined in Section 3.5.6 in combination with hydrostatic loads for the 1% AEP (100-year return period) SWL.

3.6.8 Temperature and Shrinkage

The temperature and shrinkage loading considered is a uniform change, 0°F to 120°F for steel or aluminum and 10°F to 80°F for concrete, in a moderate climate.

3.7 Material Properties

All materials shall be new as described, or if not stated, to be at least in accordance with the relevant American Society of Testing Materials (ASTM) Standards. The following material specifications were used as the minimum parameters for the conceptual design of the SBMs:

- Structural steel shall conform to ASTM A992 for wide flanges, A572 Grade 50 for other structural members.
- Steel sheet pilings, combi-wall systems and HP sections (HP 14X73, HP 14X89 etc.) shall conform to ASTM A690 or ASTM A572 Grade 50; steel pipe piles shall conform to ASTM A252 Grade 3 (50 ksi).
- Steel reinforcement in concrete shall conform to ASTM A615, Grade 60.
- Reinforced concrete shall have a minimum 28-day compressive strength (f'c) of 5,000 psi, maximum water/cement ratio 0.40.
- Lean concrete shall have a minimum 28-day compressive strength (f'c) of 3,000 psi.
- Minimum cover to reinforcement for concrete exposed to marine environment shall be 3 inches.
- Concrete shall be air entrained and conform to ACI 301.
- Structural steel members exposed to marine environment shall be coated or galvanized. Steel foundations in the water, such as steel sheeting and steel piles shall be coated with coal tar epoxy and include 1/16 in. corrosion allowance in wall thickness (5/16 in. minimum).

3.8 Synopsis

Due to the large size and the varying site conditions throughout the study area and the preliminary nature of the HAT Study, a limited series of reasonable but conceptually generic SBMs are to be developed that can be used throughout the HAT Study area and are applicable to all HAT Study Alternatives. The purpose of the study is to develop generalized SBMs that are comprehensive enough to be applicable, yet not too detailed or site specific such that they could only be applied at one location. The previous sections summarize the general design criteria for the SBMs. The criteria should not be seen as comprehensive, all-encompassing or complete, as it relates to the detailed design. Instead, the requirements and criteria form the basis for an iterative design

approach of which the feasibility analysis represents the first phase. The design of the conceptual SBMs is further detailed in the next section.

4 CONCEPTUAL DESIGN OF SHORELINE BASED MEASURES

4.1 Methodology for Development SBMs as part of HATS Alternatives

A limited series of reasonable and conceptually generic structural measures to provide flood risk reduction – that can be used throughout the HAT Study area, and that are applicable to all HAT Study Alternatives – have been developed. The purpose of the study is to develop generalized SBMs that are comprehensive enough to be applicable and suitable for the entire study area, yet not too detailed or site specific such that they could only be applied at one location. The start point for the development of SBMs relied on two items: 1) The alignments of each of the HAT Study Alternatives (as developed by USACE-NAN) and 2) a sub-set of structural measures from a list of coastal storm risk management measures as identified within the NACCS report see Table 4-1.

Table 4-1: Structural Measures for Coastal Storm Risk Management

Structural Measures	Included within HAT Study as Shore Based Flood Risk Reduction Measure (SBM)	Note
Seawall + Revetment	Yes	
Groins	No	No to little flood risk reduction function
Detached Breakwater	No	No to little flood risk reduction function
Levee	Yes	
Floodwall	Yes	
Tide Gate	Yes	
Deployable Floodwall	Yes	
Road or Rail Raising	Yes	
Beach and Dune Restoration	Yes	
Storm Surge Barrier	No	Storm Surge Barrier is a flood risk reduction measure but not included within the category SBM

The selected set of structural measures from the table above (Seawall, Levee, Floodwall, Deployable Floodwall, Tide Gate, Road Raising and Beach and Dune Restoration) were then refined and expanded upon using the methodology described hereafter. The methodology is characterized as a desktop study where a high-level assessment and analysis is performed with limited data and which, in large part, relies on evaluation of aerial photography, site photos, readily available public data and on professional engineering judgement. The general process combined both the evaluation of the applicability of SBMs for each alignment as well as the evaluation of the need for minor re-alignments to accommodate the SBMs and minimize conflict. The general

methodology used for all of the SBM reaches that are part of the HATS Alternatives can be described by the following 3 steps:

1. Assess existing shorelines and coastal edges and develop a general understanding of existing conditions, shoreline type, planned projects, land uses, spatial constraints, and elevation at the coastal edge,
2. Assess the potential for application of standard SBMs (those selected from Table 4-1) and assess potential conflicts with site integration
    1. If conflict for site integration is deemed low or acceptable,
        - Select most applicable SBM and assign to reach
    2. If conflict for site integration is deemed high[2]:
        - Asses potential for re-alignment of the reach and assign the most applicable SBM (note that a full alternate alignment evaluation was not included and only small/minor changes were made where deemed feasible), OR
        - Identify a new SBM typology which would potentially result in lower degree of conflict
3. If multiple locations have been identified where a SBM different from the typical list (those selected from Table 4-1) would be preferable, then develop a conceptual design for said SBM.

Following the methodology above, and with the objective of developing generalized SBMs that are comprehensive enough to be applicable and suitable for the entire study area yet not too detailed or site specific such that they could only be applied at one location, the list of structural Shoreline Based Measures was expanded upon and for the HAT Study consists of:

- Floodwalls
    - Medium Floodwall,
    - Large Floodwall,
    - Extra Large Floodwall,
- Levees
    - Medium Levee,
    - Large Levee,
- Promenades with Flood Risk Reduction Function
    - Stepped Promenade, and
    - Elevated Promenade,
- Seawall,
- Composite Seawall with Beach and Dune,
    - Composite Seawall with Beach and Dune for Urban Application, and
    - Composite Seawall with Beach and Dune for Natural Shorelines Application (NNBF)

[2] A record of relative qualifiers are kept offline for each reach within a geographic information system database

- Deployable Flood Barriers
    - Flip-up Barrier
    - Operable Flood Gate
- Tide Gate

The following section first provides a brief explanation of the evaluation of existing shoreline conditions and the compatibility and selection of SBMs under this study (step 1 and 2 of the methodology above). Thereafter the development of the conceptual designs of the shore-based measures is presented.

4.2 Existing Shoreline Considerations

With the use of publicly available data and satellite images, existing shoreline features for all the study areas were assessed and classified into the categories shown in Table 4-2. Based on the existing features that prevail throughout the study area, several prototypical SBMs were developed and presented in Table 4-2. This table presents the general applicability of the generic SBMs by existing shoreline feature type.

It should be reiterated that no site-specific topographic survey, bathymetric survey, condition survey and geotechnical analysis have been completed. Instead, in accordance with USACE's SMART planning principles, the development of the feasibility level SBMs was based on qualitative data and desktop level analysis, which resulted in broad generalizations of existing conditions. The implications of such assumptions. along with recommendations for further data collection and refined analysis to support the design. are described in Section 6.

Table 4-2: Existing Shoreline Features and General Applicability of SBMs

SBMs / Existing Shoreline Type	Floodwall (Medium, High and Extra-High)	Levee (Medium and High)	Promenade (Stepped and Elevated)	Floodwall with Park Integration	Seawall	Composite Seawall with Beach and Dune	Flip-up Barrier	Operable Flood Gate (Roller and Swing)	Tide Gate
Natural Shoreline	√	√							
Revetment			√		√				
Parks/ Uplands		√		√					
Street End/ Crossings	√							√	
Urban Waterfront Development	√		√				√		
Industrial Waterfront development	√				√				
Promenade	√		√				√		
Beach						√			
Streams, Creeks, Waterways or Canals									√

4.3 Geometry

The SBMs were developed based on a range of generalized geometric considerations. These geometric considerations are necessary to ensure the proper function of, and safe access to and across the SBMs.

The geometric considerations for the SBMs are listed below:

Vehicle Access Ramp Slope (maximum): 1:10

Vehicle Access Ramp Width – Two-Lanes (minimum / desirable): 24 feet / 32 feet

Levee Crest Width (minimum/ desirable): 10 feet / 15 feet

Levee Front Slope (minimum/ desirable): 2h:1v / 3h:1v

Levee Back Slope (minimum/ desirable): 2h:1v / 3h:1v

Revetment Front Slope (minimum/ desirable): 2h:1v / 3h:1v

4.4 Stability Analysis for Gravity Structures

The stability analysis shall be performed to ensure the SBMs would not fail. The stability analysis investigates the followings:

1) Sliding: The structure moving horizontally
2) Bearing: The structure sinks into the ground, caused by a lack of soil bearing capacity and/or an insufficient foundation design.
3) Resultant Location: The entire base must be in compression for the usual load condition to maintain full contact between the structure and the foundation. For extreme load conditions, the resultant is permitted to be anywhere within the base.

Stability analysis shall be in accordance with United States Army Corps of Engineers (USACE) publications. The minimum factors of safety and resultant location limits are provided in the table below.

Table 4-3: Minimum Factors of Safety or Limits (EM-1110-2-2100)

	Sliding, Factor of Safety	Bearing, Factor of Safety	Resultant Location, Limit
Operational Condition	3.0	3.0	100% of Base in Compression
Design Storm Condition	2.2	3.0	Resultant Within Base

In general, the stability analysis shall also consider scour and long-term erosion effects and it shall meet criteria set forth in FEMA's Guidance for Flood Risk Analysis and Mapping – Guidance Document 42 – Coastal Structures (FEMA, 2015).

4.4.1 Pile Foundations

For axial loads in compression and tension, the ultimate capacity shall be determined based on a factor of safety of 3, in accordance with *Hurricane and Storm Damage Risk Reduction System Design Guideline* or HSDRRS (USACE, 2016). No site-specific geotechnical analysis was completed. For this study, soil characteristics as described in Chapter 3.5.2 were used. In general, the structural analysis relied on the geotechnical analysis and data completed previously and reported in USACE 2016 (Ref. 4).

4.5 Design Development of SBMs

Based on available data and desktop analyses, conceptual designs commensurate with a feasibility study level were developed for the SBMs that are proposed to be part of the HAT Study Alternatives. The description and limitation of each SBM are discussed below.

4.5.1 Medium Floodwall, Large Floodwall, and Extra-Large Floodwall

Floodwall systems are independent, single purpose structures that aim to provide flood protection. A floodwall is typically a reinforced concrete structure supported on steel H-piles, with a steel sheet pile cut-off wall as a seepage control measure. Photo 4-1 shows a section of a floodwall that has been constructed as part of the *New Orleans Hurricane Storm Damage Risk Reduction System* (HSDRRS).

Based on the range of existing site elevations and required design elevations, three types of prototypical floodwalls were designed and labeled "medium", "large", and "extra-large." All three types of floodwall are composed of an inverted T-shape reinforced concrete structure with a base of 4-foot thick, battered H-piles and a vertical steel sheet pile cut-off wall. For the medium, large and extra-large floodwalls, the existing ground elevations were assumed to be El. 11', El. 8', and El. 7.5'; the top of the wall elevations were set at El. 18', El. 20', and El. 24.5'. Typical cross-sections for medium, large and extra-large floodwall are shown in Figure 4-1, Figure 4-2 and Figure 4-3, respectively. Pile design depends on design loads and soil parameters. For this study, soil characteristics as described in Chapter 3.5.2 were used.

Due to the relatively small footprint, a floodwall is deemed suitable for flood-prone urban waterfront areas, both directly at the shoreline and farther inland, where there are no existing structures and viewshed impacts are of lesser concern. It should be noted that flood-prone waterfront areas are likely to have poor soil conditions and require excavation and backfilling prior to construction.

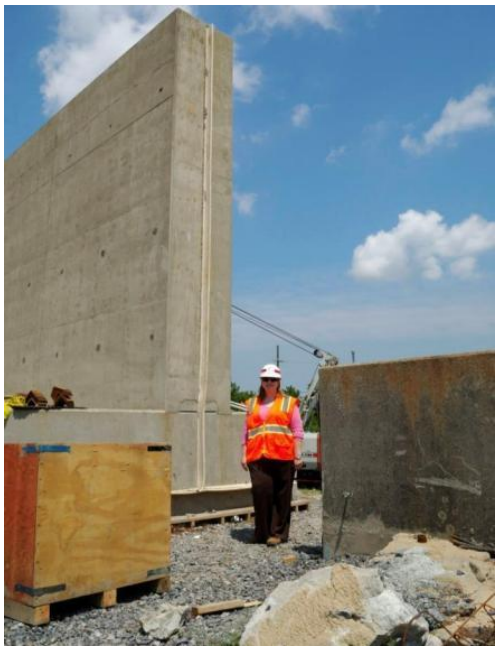

Photo 4-1: Floodwall construction in New Orleans (USACE, New Orleans District)

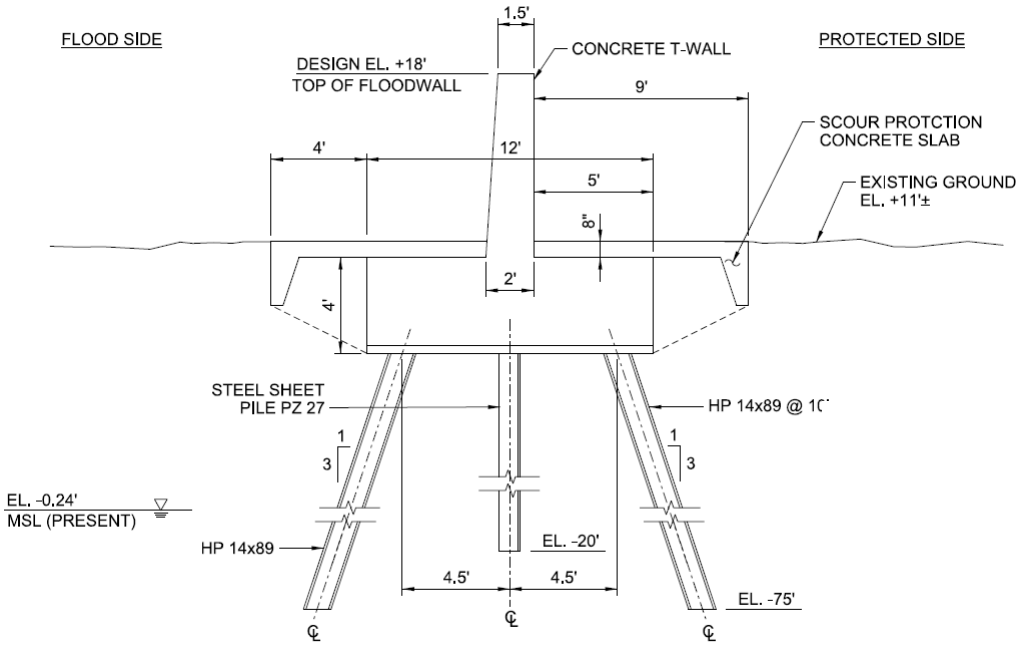

Figure 4-1: Medium Floodwall Cross-Section

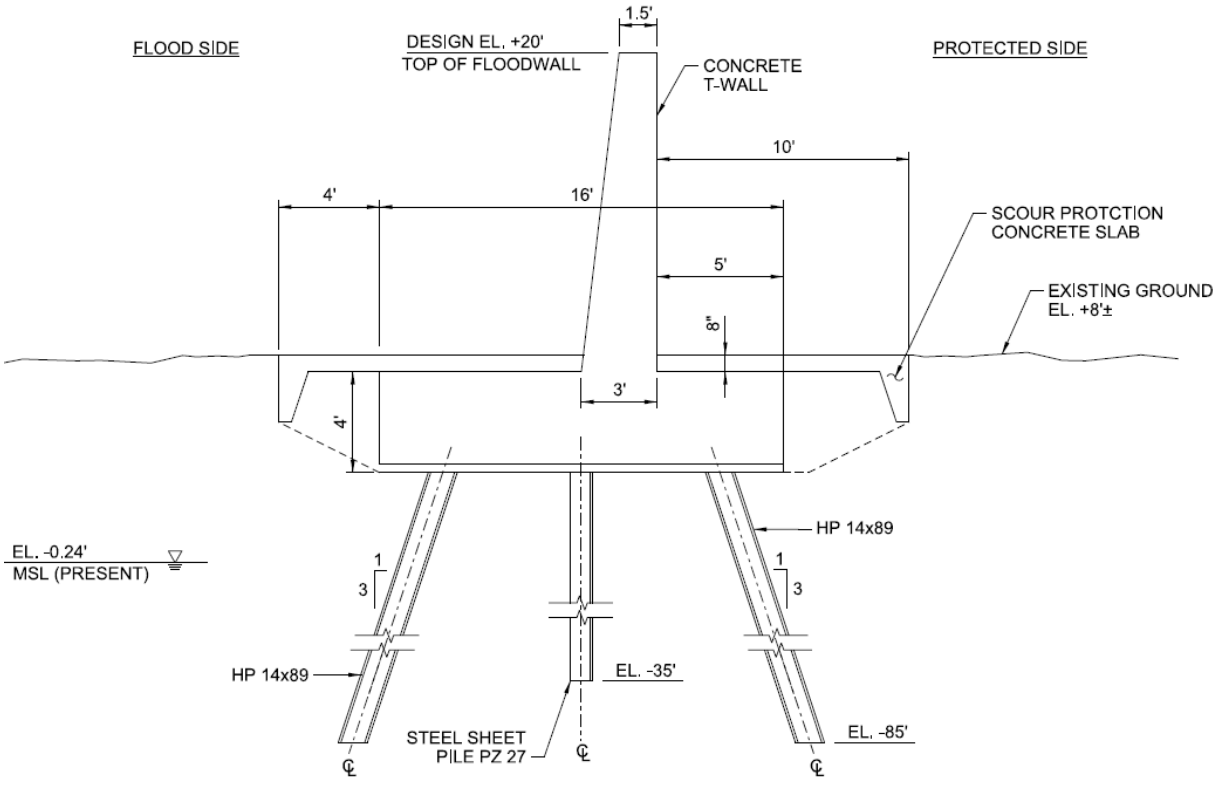

Figure 4-2: Large Floodwall Cross-Section

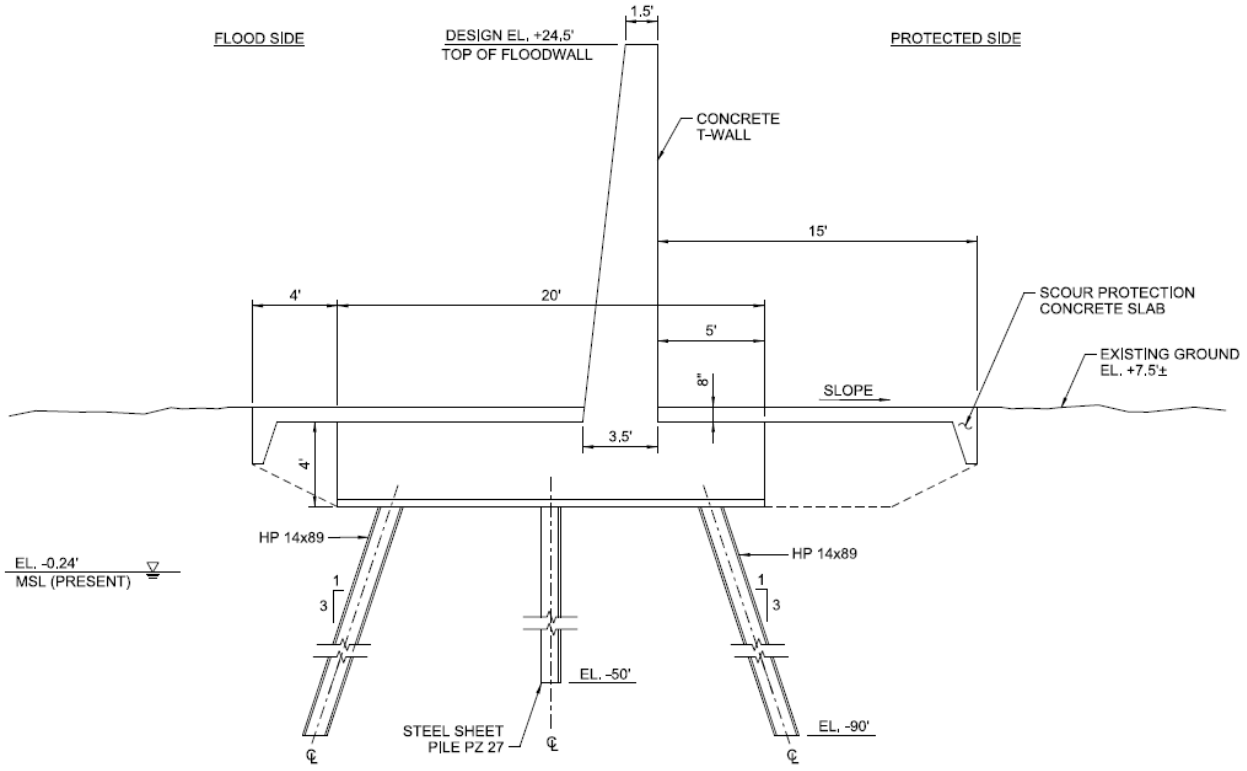

Figure 4-3: Extra-Large Floodwall Cross-Section

4.5.2 Medium Levee and Large Levee

Whereas floodwalls are made of materials such as reinforced concrete and steel, levees are made of compacted soil with grassy vegetation on top[3]. Levees are commonly used along rivers, coastlines and bodies of water to prevent inland flooding in the case of rising water levels.

Levees are typically constructed by placing engineering fill on a cleared and leveled surface; soil is compacted in layers into a large earthen structure that is wide at the base and tapers toward the top. The interior of the levee is a core composed of impervious material, usually a firm clay, to form a watertight barrier to prevent or minimize seepage, either through or beneath the section. Grass or some other type of non-woody vegetation are commonly planted on the levee to add stability and protection from erosion. The vegetation on the levee increases its aesthetic appeal. A photograph showing a levee after completion can be found in Photo 4-2.

Levees on poor soil are subject to instability, uncontrolled drainage flows, and settling, and therefore, require deeper excavation prior to construction. For this study, it was assumed the levee is founded on soil of medium quality. As discussed in Section 3.5.2, no site specific geotechnical

[3] Also considered to be a natural and nature based feature (NNBF)

analysis was completed. For the proto-typical design for the levee, six feet of material would be excavated from the ground for seepage control, and a side slope of 1 vertical on 3 horizontal was used to minimize erosion and scour potential. The medium levee has a height of 7 feet and the large levee has a height of 13 feet. To minimize seepage concerns, facilitate maintenance, and allow for ease of construction, a crest width of 10 feet and 15 feet were used for the medium and large levee design, respectively.

Due to the levee width, required setbacks and space need for smooth grade changes, relatively large tracts of real estate are typically required. For this reason, levees are best used as flood risk reduction measures along natural shoreline, or parallel to the course of streams, and set away some distance from the developed areas. Figure 4-4 and Figure 4-5 show the cross-section of a typical medium levee and large levee, respectively.



Photo 4-2: Levee in New Orleans (USACE, New Orleans District)

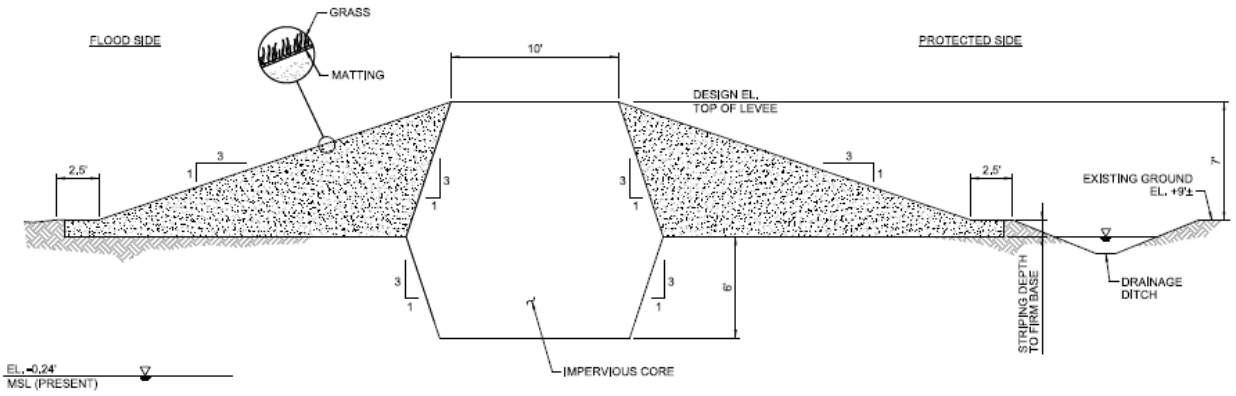

Figure 4-4: Medium Levee Cross-Section

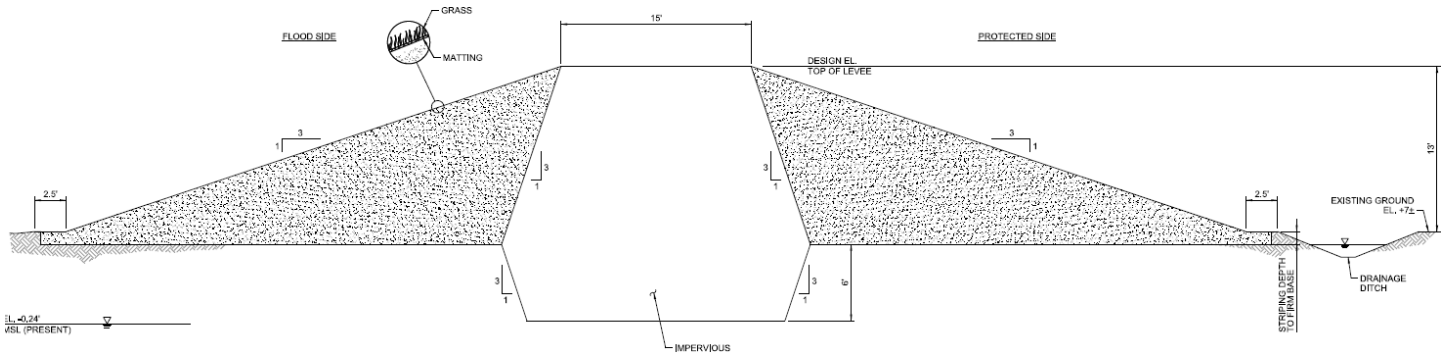

Figure 4-5: Large Levee Cross-Section

4.5.3 Stepped Promenade

Existing promenades in urban areas provide valuable community space and recreation opportunities along the waterfront. The existing promenade often occupies a relatively narrow strip along the shoreline with roads and buildings on the landward side. To improve the resiliency of existing promenades, preserve waterfront accessibility, and reduce run-up and flood risk for the roads and buildings behind it, a stepped promenade was designed as a conceptual SBM for the HAT Study. The stepped promenade has a top deck elevation varying +3 feet (waterfront edge) to +16 feet (landside limit of the promenade). A photograph showing a stepped seawall at Corpus Christi (Texas) can be found in Photo 4-3; the Corpus Christi stepped seawall bears some similarities to the proposed stepped promenade SBM.

The stepped promenade was assumed to be constructed along the shoreline to replace the existing promenade. The promenade is comprised of a stepped reinforced concrete deck supported on steel pipe piles and steel sheet pile that retains the soil. The existing mudline and promenade elevations

were assumed to be El. -40’ and El. 4’, respectively. A typical cross-section for the stepped promenade is shown in Figure 4-6. It should be noted that temporary structures required to shore up the existing promenade are likely to be required during demolition and construction of this SBM.



Photo 4-3: The Corpus Christi Seawall (Library of Congress/LC-USF34-038306)

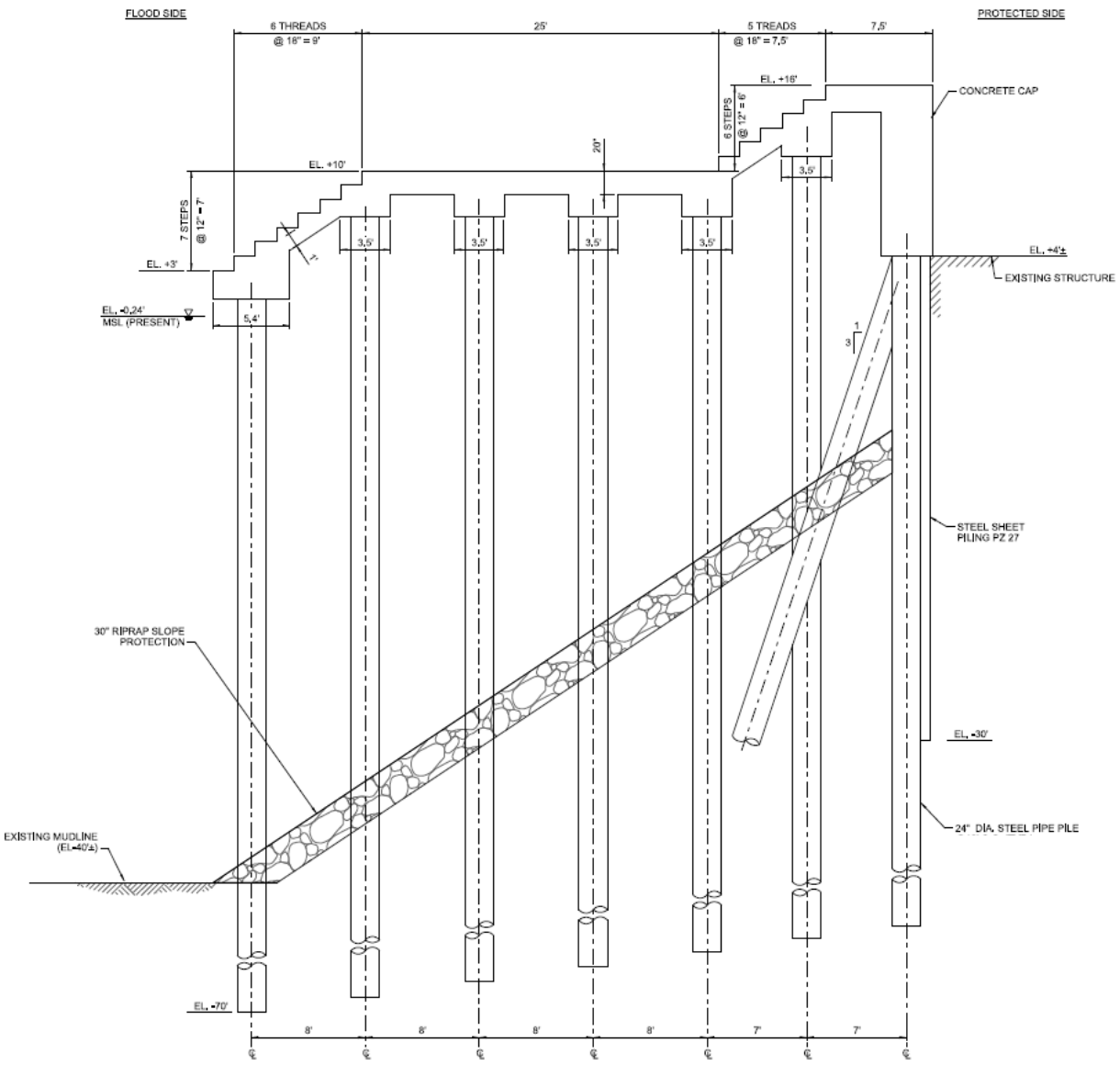

Figure 4-6: Stepped Promenade Cross-Section

4.5.4 Elevated Promenade

Similar to the stepped promenade, the prototypical elevated promenade concept seeks to preserve a waterfront space and view that is available to the public, but yet still able to provide flood risk reduction. The elevated promenade consists of a 60-foot diameter steel flat sheet cofferdam with sand backfill inside, and a reinforced concrete cap of 18-inch thick. The existing ground and existing mudline were assumed to be El. 8' and El. -5', respectively.

The elevated promenade was assumed to be constructed along the shoreline to replace an existing promenade. It should be noted that temporary structures to shore up the existing promenade are likely required during demolition and construction. Detailed design would be completed during at later stages of the study when site specific parameters are available.

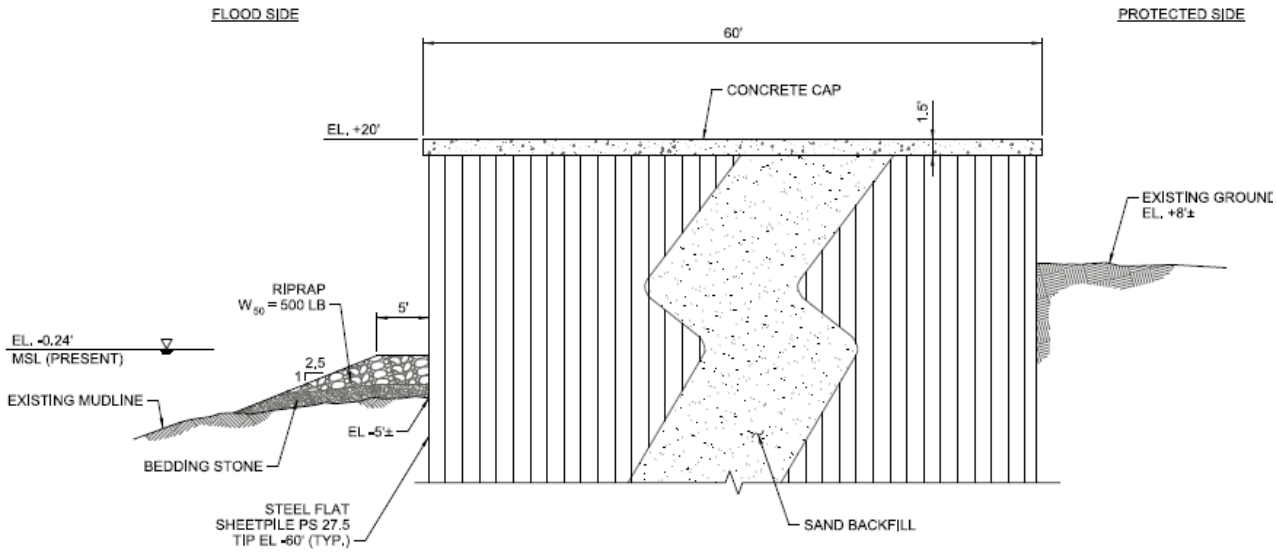

Figure 4-7: Elevated Promenade Cross-Section

4.5.5 Floodwall with Park Integration

A variation of the floodwall measures described in Section 4.5.1, was developed to provide flood risk reduction to urban areas behind the floodwall while also providing or maintaining a public amenity and open space along the waterfront, whilst at the same time reduce the incoming wave height and wave loading on the floodwall structure

The floodwall with park integration SBM is composed of a H-piles supported T-shape reinforced concrete floodwall with a vertical sheet pile cut-off wall (see Section 4.5.1), and an elevated park on berm. The existing ground elevations was assumed to be at +9 feet; the top of the wall elevation was set at +19 feet. The elevated park is located on the flood side of the floodwall and is supported on berm made of compacted soil and vegetation. The park on berm is considered to be a nature-based feature. Typical cross-section for the floodwall with park integration is shown in Figure 4-8.

The measure integrates well with existing urban waterfront areas; however, the park requires a relatively large footprint. It should be noted that the existing promenade was assumed to have a soil retaining structure and require some strengthening. Detailed design would be completed during later stages of the study when site specific parameters are available.

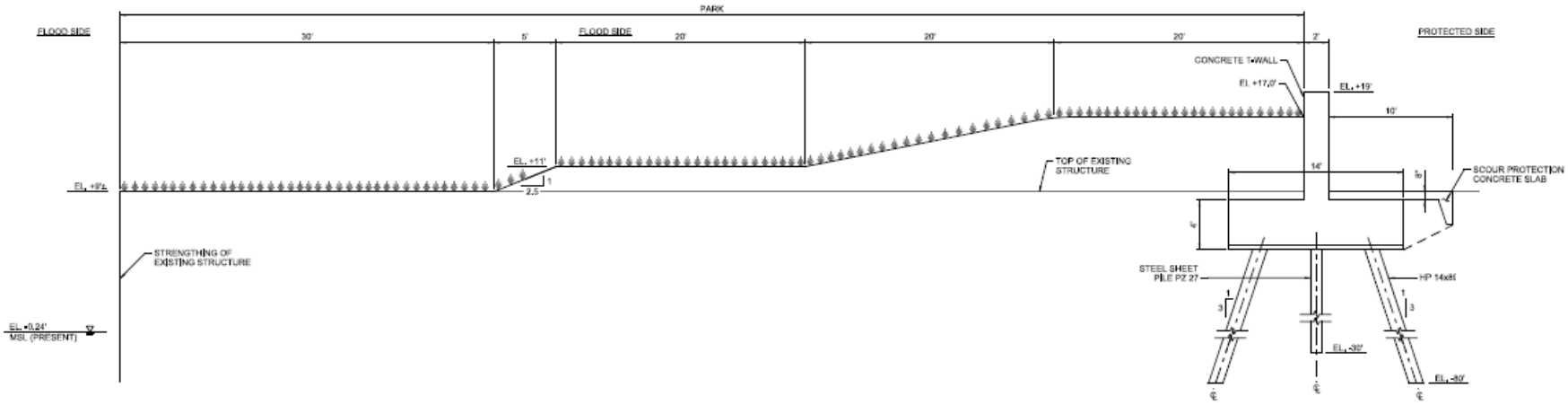

Figure 4-8: Floodwall with Park Integration Cross-Section

4.5.6 Seawall

The seawall is a composite structure and is comprised of a rubble mound structure and a H-pile supported T-shape reinforced concrete floodwall with a vertical sheet pile cut-off wall. Photo 4-4 shows an example of a rubble mound structure in the Bronx, New York.



Photo 4-4: Rubble mound structure in Bronx, New York (M&N, 2017)

It should be noted that a rubble mound structure[4], whereas effective at dissipating wave energy, cannot prevent coastal flooding since it is porous. The impervious concrete floodwall would be installed in combination with the rubble mound structure to prevent flooding, and reduce wave run-up. The prototypical design for the seawall is composed of a rubble mound structure on the seaward side and a pile supported concrete floodwall on the landward side. For this study, it was assumed that a rubble mound with two layers of 2.3-foot diameter armor stone, two layers of 1.1-foot diameter underlayer stone, two layers of 1.4-foot diameter toe armor stone and a slope of 2

[4] Rubble mound structures can be symmetrical or asymmetrical. They are typically made of erosion resistant material such as stone or concrete to protect the shoreline against wave action. Rubble mound is typically composed of an armor layer, filter layer(s), and toe protection. The armor layer is designed to dissipate wave energy; as a result, the core can be protected from direct wave attack. The filter layer supports the armor, and it allows passage of water while retaining the underlying soil. The toe is to provide stability against undermining at the bottom of the structure.

(Horizontal):1(Vertical) would provide sufficient stability. The underlayer would be on top of a geotextile; the geotextile would protect the underlaying base material or soil from erosion by waves and currents. The toe has a width of 4.2 feet and a crest width of 13.4 feet. The floodwall has an inverted T-shape reinforcement concrete structure with a base of 4-foot thick, battered H-piles and vertical steel sheet pile cut-off wall. The top of the floodwall is at El. 17' and the design existing ground elevation is at El. 6'.

One of the more important variables of the rubble mound design is the seaward side slope which, together with the crest height, is generally dictated by soil conditions and construction methods. For the purposes of this study, it was assumed that the rubble mound was founded on soils that are of moderate quality which is common in the study area per Section 3.5.2 and therefore would not require foundation/ground improvements. Bottom elevation of the rubble mound toe was assumed to be at Elevation -8'. Actual elevations will vary across the study area, but for feasibility level analysis, it was considered a reasonable elevation representative of the conditions of application within the study area. A typical cross-section for the seawall is shown in Figure 4-8. Finally, rubble mound structures, especially the ones with natural stone armor, integrate well with the natural shoreline. The natural look of the armor rock, in particular, has a relatively higher aesthetic appeal.

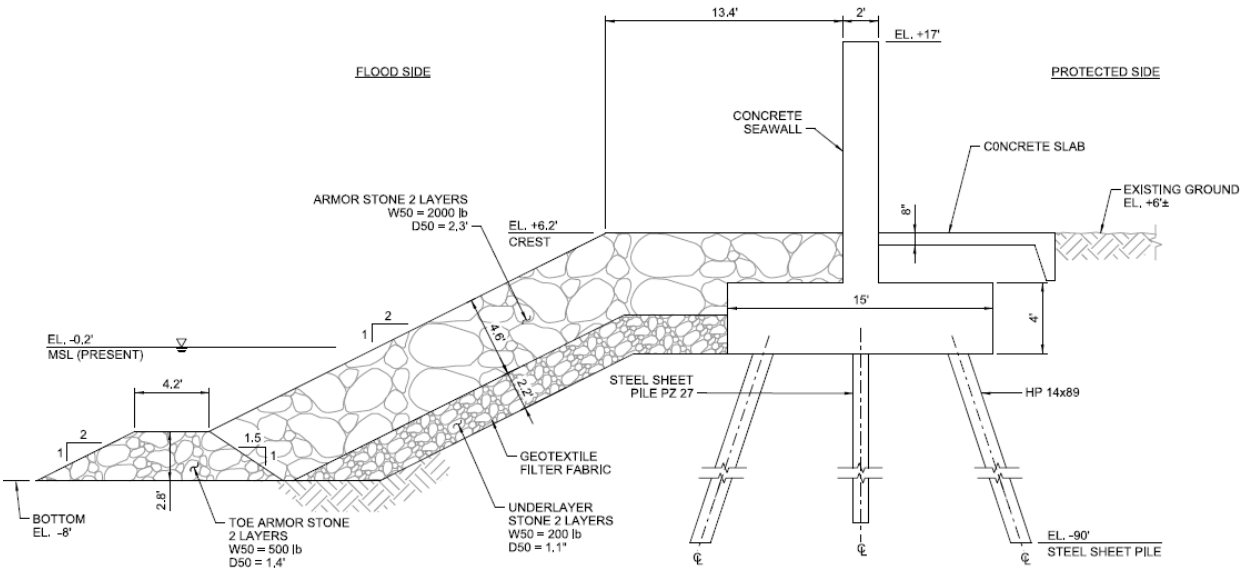

Figure 4-9: Seawall

4.5.7 Composite Seawall with Beach and Dune

A horizontally composite seawall similar to the design developed for the *Atlantic Ocean Shorefront Reach of the Rockaway Peninsula* (USACE, 2018a), in combination of beach and dune, is included within the HAT Study as SBM to provide a reduction in flood risk. Two measures were developed, namely the composite seawall with beach and dune (urban application) as shown in

Figure 4-10 and composite seawall with beach and dune (NNBF application) as shown in Figure 4-11. The Urban variation to this SBM is proposed to be applied in more developed beach areas where spatial constraints are more apparent. The double retaining wall limits the extent of the splash apron and thereby the footprint of the measure. The NNBF variation of this SBM has structural core but is completely covered in sand and has a larger footprint.

Both composite seawall configurations are based on the same general horizontal composite seawall design developed for the *Atlantic Ocean Shorefront Reach of the Rockaway Peninsula* (USACE, 2018a). For the composite seawall with beach and dune, an overtopping rate of 1 cfs per foot or 93 L/s/m was adopted. The structure consists of a 1-layer rubble mound structure with an impermeable core, a 1V:2H slope and a steel sheet pile with a concrete cap that functions as a flood risk reduction barrier. A concrete cap top elevation of +17 feet was provided; a dune crest elevation of +19 feet with a crest width of 25 feet was used. The existing grade behind the composite was assumed to be at an elevation of + 9'. For composite seawall with NNBF application, a layer of sand was on top of the rubble mound and concrete cap to enhance visual aesthetic and integration to the existing environment.

A typical cross-section for the overall composite seawall with beach and dune is shown in Figure 4-12.

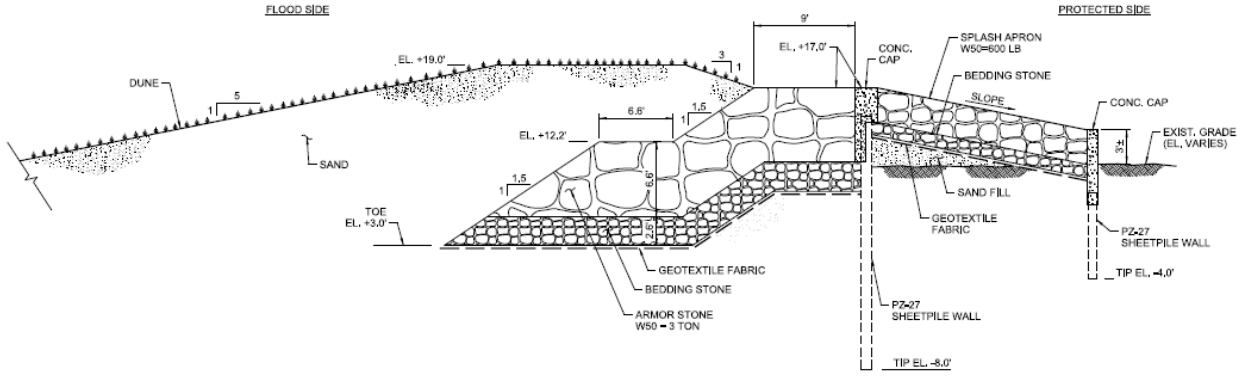

Figure 4-10: Composite Seawall with Beach and Dune (Urban Application) Cross-Section

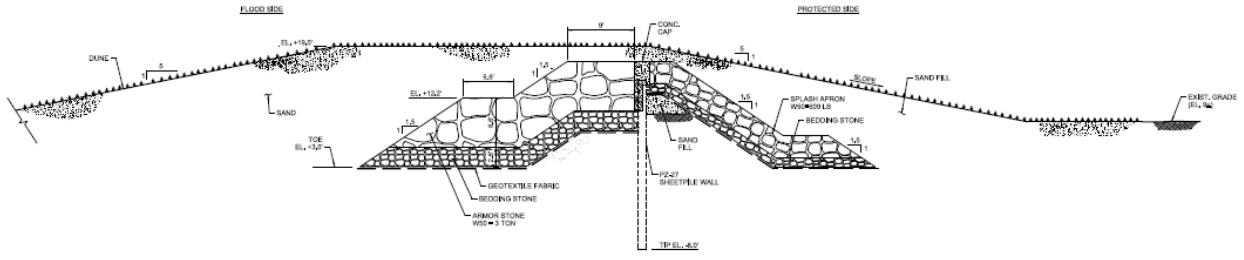

Figure 4-11: Composite Seawall with Beach and Dune (NNBF Application) Cross-Section

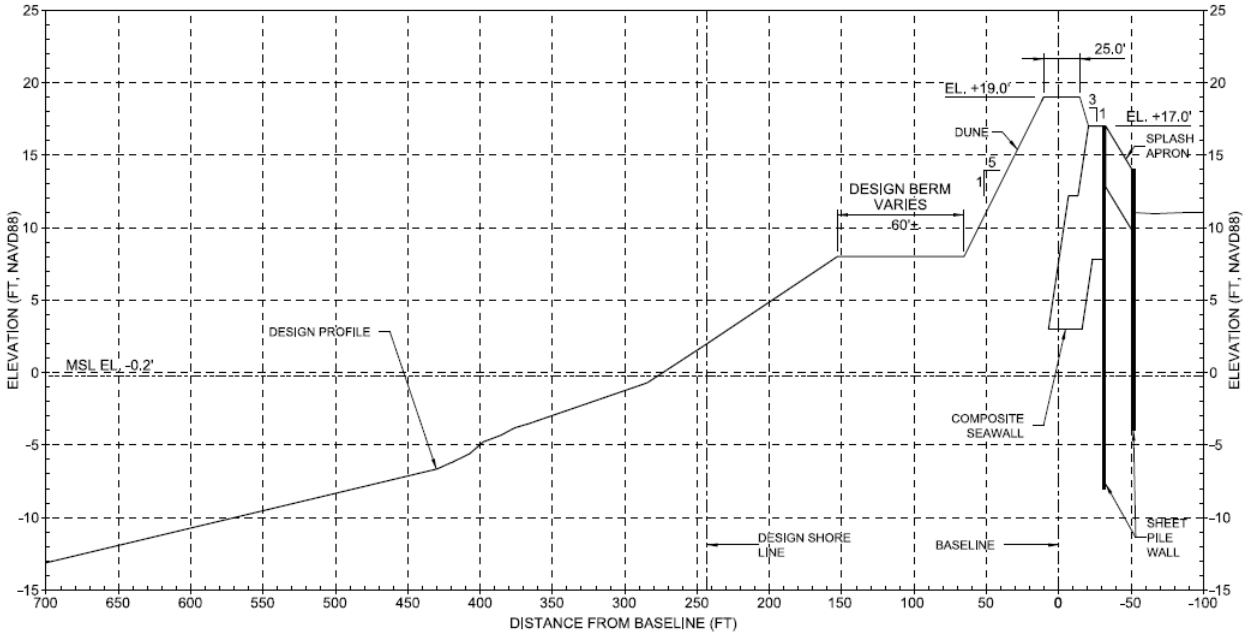

Figure 4-12: Typical Overall Cross-Section for Composite Seawall with Beach and Dune (Urban Application), NNBF Application Similar

4.5.8 Flip-up Barrier

At certain locations where SBMs are considered as part of the HATS alternatives, the need to preserve viewsheds and maintain a level and unimpeded access to the waterfront is critical. For those locations, a deployable flood barrier, i.e. the flip-up barrier, was considered and included as a SBM under the HAT Study. The flip-up barrier is currently under consideration in the Lower Manhattan Coastal Resiliency (LMCR) study (Ref. 2).

The Flip-up barrier is a passive deployable flood barrier. The passive deployment mechanism allows deployment of flip-up barrier without any involvement from operation personnel and is operated by physics (i.e. water pressure) and activated when the design conditions are met (i.e at the onset of submergence of the base). Passive systems allow for operational flexibility and will ensure the flip-up barrier be deployed when needed to provide flood risk reduction. For the HAT Study, a flip-up barrier similar to the design developed for the LMCR study was considered. The LMCR flip-up barrier is designed to include a dual operating mechanism. It can be manually operated, but also includes it standard passive deployment mechanism. Manually operated gates require operation personnel to physically go to the location of the gate and close it during storm conditions. The gate would then be locked into place to prevent tampering with. After deployment access to the flood side of the line of protection would be impeded.

A schematic view of the flip-up barrier is shown in Figure 4-13. A deployed flip-up barrier is shown in Photo 4-5. The flip-up barrier for the HAT Study includes a barrier with a height of 10 feet. The advantage of a flip-up barrier is a small structural footprint and no to little viewshed impacts when the barrier is not deployed. Although most of the structure is housed underground

when not deployed, public amenities cannot be permanently installed on top of the flip-up barrier and the barrier will need to be inspected prior to the onset of a storm to make sure to obstacles are blocking a safe deployment.

The flip-up barrier was assumed to be supported on piles. Pile design depends on design loads and soil parameters. Since flood-prone waterfront areas are likely to have poor soil conditions, excavation and backfilling prior to construction may be required. Detailed design would be completed during PED when site specific parameters are available.

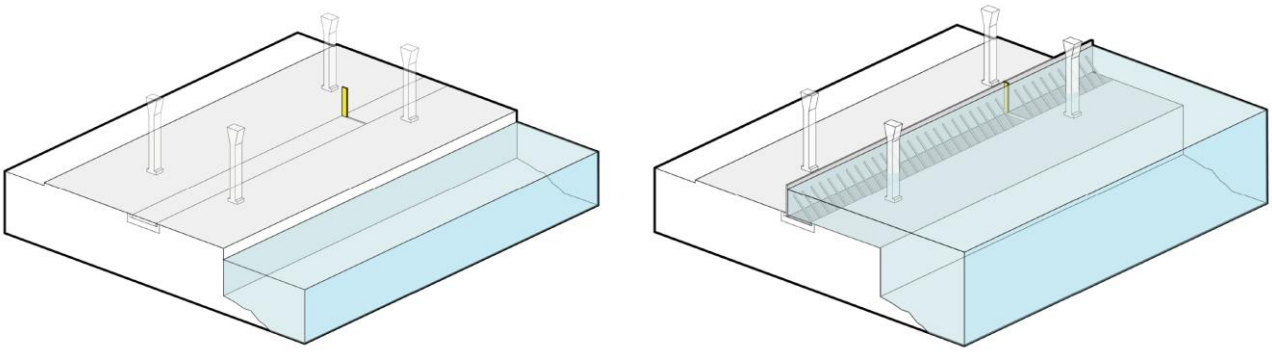

Figure 4-13: Schematic View of a Flip-up Barrier from LMCR



Photo 4-5: Flip-up barrier at Lourdes Hospital deployed automatically during a recent flood event (Floodbreak)

4.5.9 Operable Floodgate

Operable floodgates are measures added to a line of coastal storm damage risk reduction, across a road or driveway, which allows for unimpeded access across the alignment during normal day-to-

day conditions. Operable floodgates can be either manually or automatically operated. The prototypical operable floodgate developed for this study is designed to be manually operated. Manually operated gates require operation personnel to physically go to the location of the gate and close it during storm conditions. The gate would then be locked into place to prevent tampering with and access to the flood side of the line of protection would be impeded.

Both swing gates and roller gates were considered initially, the choice of gate depends on the orientation and space available. In general, a roller gate can slide into place along a track (Photo 4-6 and Figure 4-14) and a swing gate is supported on one side by top and bottom hinges attached to a support structure (Photo 4-7 and Figure 4-16).

Both gates have the advantage of being simple, with quick operation, where no special skill or equipment are required. However, swing gates will require a relatively large right-of-way area for operating while roller gates will require a level track surface.

The floodgates were assumed to be supported on piles. Pile design depends on design loads and soil parameters. Since flood-prone waterfront areas are likely to have poor soil conditions, excavation and backfilling prior to construction may be required. Gate widths of 30 feet and 60 feet were assumed. At this stage of the study no preferred floodgate type has been selected and only an inventory of total number of gates will be completed. Floodgate type evaluation and associated detailed design would be completed during the next phases of the study when site specific parameters are available.



Photo 4-6: Roller Gate (ETL-1110-2-2105)

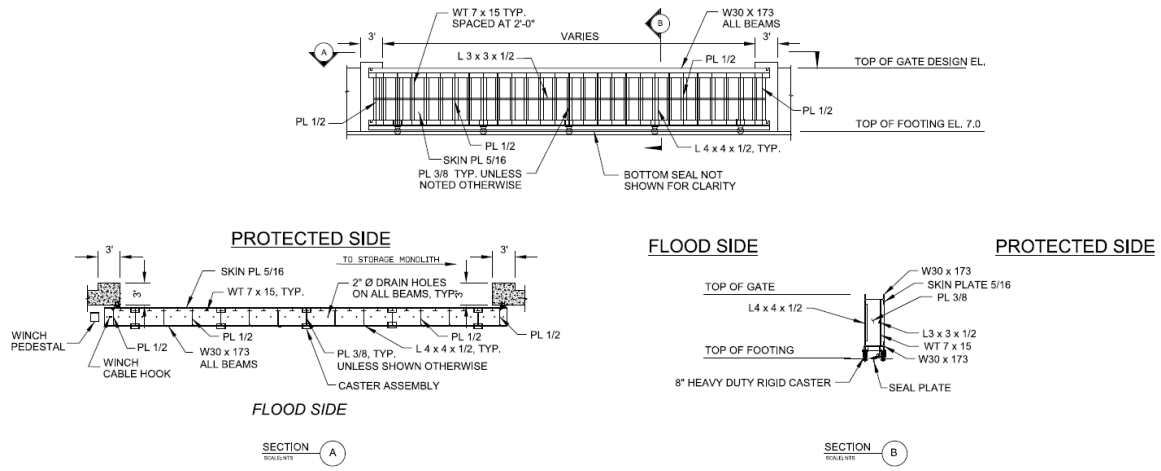

Figure 4-14: Roller Gate Cross-Section (USACE, 2016)



Figure 4-15: Steel Swing Gate at E Rodney French Boulevard, New Bedford



Photo 4-7: Single-leaf swing gate in New Orleans at a railroad crossing (M&N)

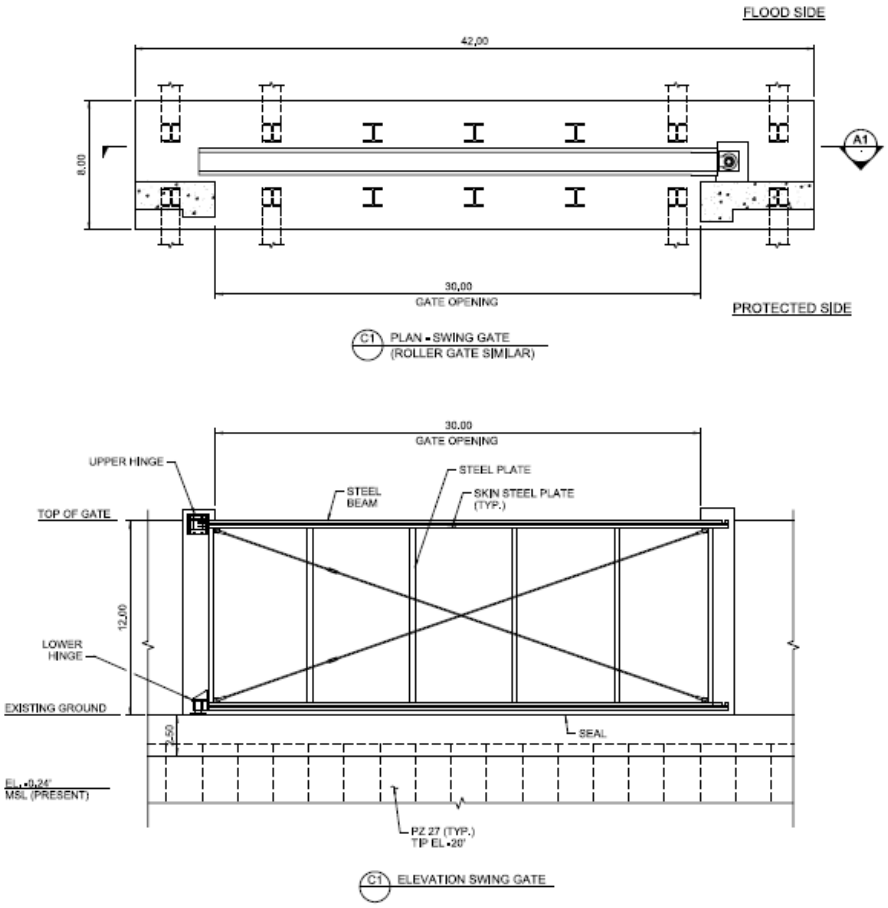

Figure 4-16: Swing Gate Cross-Section (USACE, 2018b)

4.5.10 Tide Gates

Tide gates are coastal storm flood risk reduction measures that stay open under normal conditions to let tidal flow pass but are closed when water levels are expected to exceed a certain level. Tide gates do not allow for navigation or passage of vessels or small boats. A tide gate is typically a reinforced concrete superstructure supported on steel pipe piles, with a steel sheet pile cut-off wall as a seepage control measure. The tide gate has a sill elevation of -5 feet and a top of wall elevation of +17 feet. The prototypical tide gate developed for this study is designed to be provided with an electric winch and to be manually operated remotely. Figure 4-17 shows an elevation of a tide gate at Oakwood Beach, New York.

The tide gates were assumed to be supported on piles. Pile design depends on design loads and soil parameters. Since flood-prone waterfront areas are likely to have poor soil conditions, excavation and backfilling prior to construction may be required. At this stage of the study, no preferred tide gate type has been selected and only an inventory of total number of gates was completed and shown in Appendix B. Tide gate type evaluation and associated detailed design would be completed during the next phases of the study when site specific parameters are available.

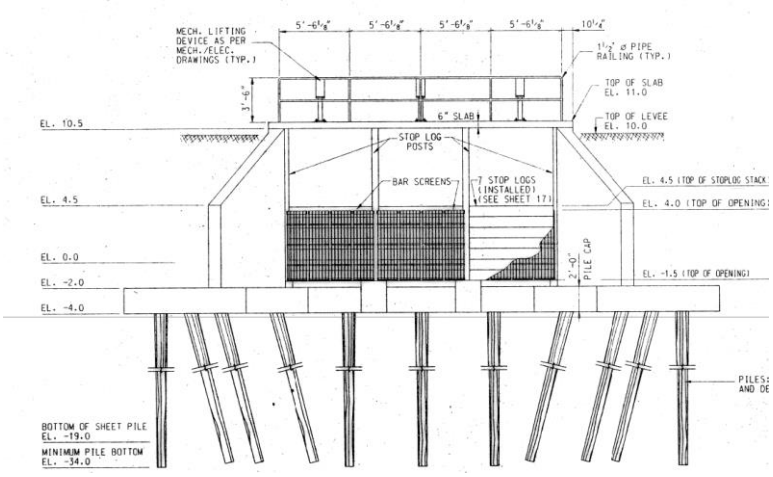

Figure 4-17: Tide Gate at Oakwood Beach, New York (USACE)

4.5.11 Road Ramp

Road ramps are not a SBM, but are a means of allowing vehicular access across the flood risk reduction alignment. For the HAT Study it is assumed that road ramps consist of two traffic lanes developed to allow for safe vehicular access. The road ramps for this project were designed to be used in conjunction with the large levees.

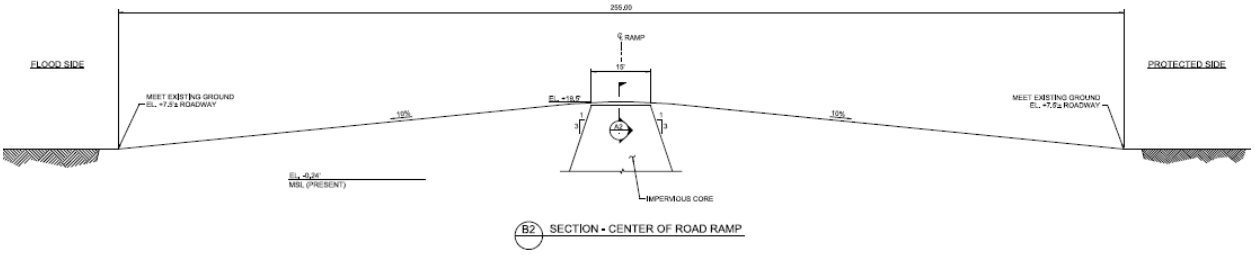

Figure 4-18: Road Ramp Cross-Sections

4.6 Limitations

Due to the large size of the study area and the absence of site-specific data at the feasibility phase, design of the prototypical SBMs was conceptual and generic. Site specific data such as bathymetric survey, topographic survey, condition survey and geotechnical investigation would be needed to refine the SBM concepts and SBM alignments. Due to the absence of site-specific data, site specific structural and non-structural work described in Section 3.3, such as access ramps and utility features, were captured qualitatively rather than quantitatively in this report. The detailing of such work was not assessed for this feasibility study but is recommended to be considered during next phases of the Study.

The SBM designs are based on publicly available data and desktop analyses, rather than site specific data. As a result, detailed site-specific work including additional accessories, upgrade and replacement of existing structural and non-structural items were captured qualitatively rather than quantitatively in this report. The detailing of such work as shown in Table 4-4 was not assessed for this feasibility stage but is recommended to be considered during next phases of the study.

Table 4-4: Detailed Design Work to be Considered During Next Phases of the Study

Detailed Design Work	Example
Utility relocation	Gas, water, electricity, cable, etc.
Additional drainage features	Storm water pipes, channels, etc.
Additional aesthetic features	Pavers, textured wall, etc.
Additional access features	Ramps, railing, stairs, etc.
Additional non-structural items	Lighting, conduits, emergency phones, real estate, right-of-way, easement quantities, environmental mitigation, etc.
Upgrade	Strengthening of existing structural elements, upgrade of non-structural items, etc.

5 INVENTORY OF SHORELINE BASED MEASURES

5.1 Introduction

The previous section presented the conceptual designs for the SBMs developed for the HAT Study. Due to the large area cover by the HAT Study and the varying coastal edge conditions associated the design of the SBMs, this assessment is preliminary and conceptual in nature, yet sufficient to establish a quantity take-off of construction materials. The completion of the conceptual designs in the previous section, in concert with the refinement of the alignments, allows for an inventory of SBMs (total length and number of measures) per HAT Study Alternative.

The inventory is separated into two data sets

1) Quantity take-off per SBM and,
2) Inventory of number and length of SBMs per alternative

These two data sets will be used by USACE-NAN to complete cost estimates for the study alternatives and allow for a relative comparison amongst the alternatives.

5.2 Quantities and Inventory of Shore Based Measure

5.2.1 Quantity Take-offs

Initially, a quantity per linear foot was developed for each prototypical SBM, with the exception of the flip-up barriers and operable floodgates. The quantity per linear foot of medium floodwall is shown in Table 5-1 as an example; the quantity per linear foot of other SBMs can be found in Appendix B. For gates and pre-fabricated items such as flip-up barriers, operable floodgates and tide gates; the total quantity per each SBM alignment was provided instead.

Table 5-1: Quantity per Linear Foot of Medium Floodwall

Medium Floodwall		
Item	Quantity per linear foot	Unit
Reinforced Concrete for Flood Wall and Splash Aprons	2.85	CY
PZ-27 Sheetpile Wall	0.36	TN
HP 14x89 Piles (3.03 TN, 83' Long)	0.20	EA
Excavation	2.41	CY
Repair Disturbed Pavement	0.11	SY

As discussed in Section 3.3, additional appurtenances were described qualitatively instead of quantitively and additional caveats and notes regarding the items not included within the quantity take-offs are listed in Appendix B.

5.2.2 Inventory of SBMs per Alternative

The HAT Study Alternatives were described in Section 2, and each of the Alternatives has a number of reaches that include SBMs. Each SBM alignment, depending on the existing site conditions, may comprise one or more conceptual SBMs. An inventory of SBMs and total length of SBMs for each Alternative was determined and is shown on Table 5-3. A diagram depicting the information from Table 5-2 is provided in Figure 5-1.

Table 5-2: Summary Table of SBM Length for Each Alternative in feet

	Alt. 2	Alt. 3A	Alt. 3B	Alt. 4	Alt. 5
Medium Floodwall	994	8,728	32,472	39,227	15,282
Large Floodwall	142	11,853	46,578	52,590	22,586
Extra Large Floodwall	414	7,236	36,356	34,647	20,804
Medium Levee	2,254	17,876	26,414	32,597	52,454
Large Levee	0	6,517	27,681	31,774	35,518
Buried Seawall/Dune - Natural Dune	51,064	3,970	3,970	3,970	0
Buried Seawall/Dune - Urban Dune	37,045	36,667	36,667	36,667	0
Deployable Flood Barrier - Flip Up Barrier	0	0	1,456	1,456	1,456
Deployable Flood Barrier - Pedestrian Gate	0	50	1,586	1,647	1,407
Deployable Flood Barrier - Railroad Gate	0	0	571	635	619
Deployable Flood Barrier - Vehicle Gate	0	778	5,725	5,551	3,168
Elevated Promenade	0	16,953	61,476	61,476	42,096
Floodwall with Park	0	0	6,963	6,963	3,355
Levee - - Road Ramp	0	629	3,277	3,344	3,429
Seawall	0	9,832	54,897	54,897	30,004
Stepped Promenade	0	0	4,329	4,329	4,329
Tide Gate	0	552	1,105	1,279	149
Total	91,914	121,640	351,522	373,049	236,656

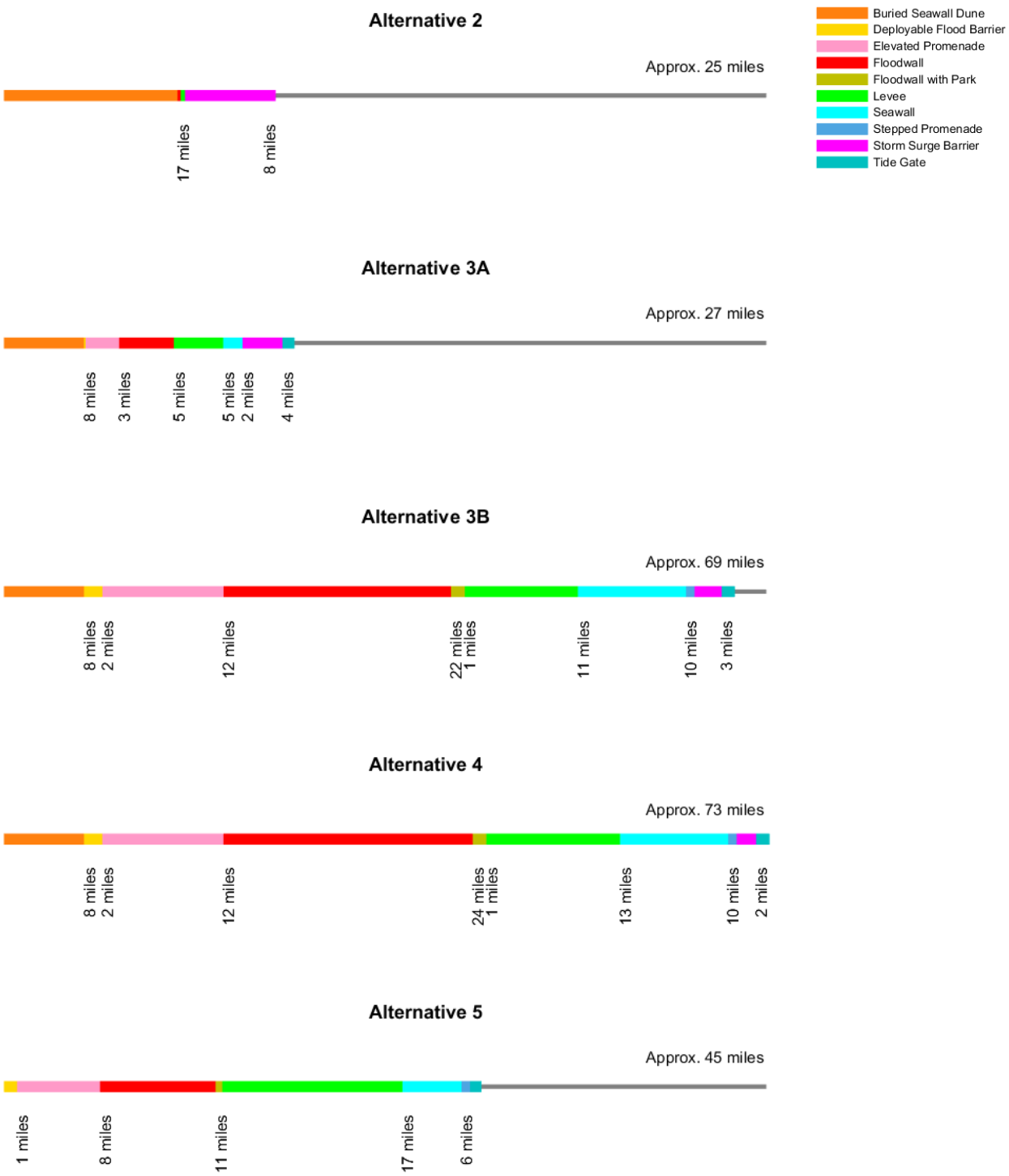

Figure 5-1: Diagram depicting SBM Length for HAT Study Alternative

Based on the SBM type and the proposed length for each SBM included within the reaches an inventory of SBMs and total length of SBMs within the Alternative's reaches was also determined.

Table 5-3 shows that, for example, Haverstraw SBM includes 1,345 feet of medium floodwall, 289 feet of stepped promenade and 1,857 feet of seawall.

Table 5-3: Summary Table of SBM Length for Haverstraw SBM

Location	Length of Measure by Type												
	Medium Floodwall	Large Floodwall	Extra Large Floodwall	Medium Levee	Large Levee	Composite Seawall/ Dune - NNBF	Composite Seawall/ Dune - Urban	Deployable Flood Barrier - Flip Up Barrier	Elevated Promenade	Floodwall with Park	Levee - Road Ramp	Seawall	Stepped Promenade
Haverstraw SBM	1,345	0	0	0	0	0	0	0	0	0	0	1,857	289

Table 5-4 through Table 5-8 show the SBM length for each reach that is included within the HAT Study Alternatives. These tables only include the inventory of SBMs, an inventory of operable gates is included in Appendix B.

From Table 5-4 through Table 5-8, it is clear that Alternative 4 has the greatest total length of SBMs.

Table 5-4: Length in feet of Shoreline Based Measures by Type and Location – Alternative 2

	Medium Floodwall	Large Floodwall	Extra Large Floodwall	Medium Levee	Large Levee	Composite Seawall/ Dune - NNBF	Composite Seawall/ Dune - Urban	Deployable Flood Barrier - Flip Up Barrier	Elevated Promenade	Floodwall with Park	Levee - Road Ramp	Seawall	Stepped Promenade	Total
Arthur Kill Barrier Tie-In	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Astoria SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Bronx River Westchester Creek Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
East Harlem SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Flushing Creek Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Gowanus Canal Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack Perimeter Lower Area - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack Perimeter Middle Area - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack Perimeter Upper Area - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack River Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Haverstraw Perimeter - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Haverstraw SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Jamaica Bay Barrier Tie-In	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Kill Van Kull Barrier Tie-In	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Kill Van Kull Barrier Tie-In Separated	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Long Island City Astoria SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
New Jersey along Hudson River SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
New York City West Side SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Newtown Creek Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Ossining SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Pelham Barrier Tie-In	158	142	0	2,254	0	0	0	0	0	0	0	0	0	**2,555**
Sandy Hook-Rockaway Barrier Tie-in	0	0	0	0	0	51,064	37,045	0	0	0	0	0	0	**88,109**
Tarrytown SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Throgs Neck Barrier Tie-in	836	0	414	0	0	0	0	0	0	0	0	0	0	**1,250**
Verrazano Narrows Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Yonkers North SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Yonkers South SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
TOTAL	**994**	**142**	**414**	**2,254**	**0**	**51,064**	**37,045**	**0**	**0**	**0**	**0**	**0**	**0**	**91,914**

Table 5-5: Length in feet of Shoreline Based Measures by Type and Location – Alternative 3A

	Medium Floodwall	Large Floodwall	Extra Large Floodwall	Medium Levee	Large Levee	Composite Seawall/ Dune - NNBF	Composite Seawall/ Dune - Urban	Deployable Flood Barrier - Flip Up Barrier	Elevated Promenade	Floodwall with Park	Levee - Road Ramp	Seawall	Stepped Promenade	Total
Arthur Kill Barrier Tie-In	726	0	0	0	0	0	0	0	0	0	0	0	0	**726**
Astoria SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Bronx River Westchester Creek Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
East Harlem SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Flushing Creek Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Gowanus Canal Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack Perimeter Lower Area - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack Perimeter Middle Area - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack Perimeter Upper Area - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack River Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Haverstraw Perimeter - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Haverstraw SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Jamaica Bay Barrier Tie-In	7,007	10,538	6,822	15,621	6,517	3,970	36,667	0	16,953	0	526	9,832	0	**114,454**
Kill Van Kull Barrier Tie-In	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Kill Van Kull Barrier Tie-In Separated	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Long Island City Astoria SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
New Jersey along Hudson River SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
New York City West Side SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Newtown Creek Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Ossining SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Pelham Barrier Tie-In	158	142	0	2,254	0	0	0	0	0	0	0	0	0	**2,555**
Sandy Hook-Rockaway Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Tarrytown SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Throgs Neck Barrier Tie-in	836	0	414	0	0	0	0	0	0	0	0	0	0	**1,250**
Verrazano Narrows Barrier Tie-in	0	1,173	0	0	0	0	0	0	0	0	103	0	0	**1,276**
Yonkers North SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Yonkers South SBM	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
TOTAL	**8,728**	**11,853**	**7,236**	**17,876**	**6,517**	**3,970**	**36,667**	**0**	**16,953**	**0**	**629**	**9,832**	**0**	**120,261**

Table 5-6: Length in feet of Shoreline Based Measures by Type and Location – Alternative 3B

	Medium Floodwall	Large Floodwall	Extra Large Floodwall	Medium Levee	Large Levee	Composite Seawall/ Dune - NNBF	Composite Seawall/ Dune - Urban	Deployable Flood Barrier - Flip Up Barrier	Elevated Promenade	Floodwall with Park	Levee - Road Ramp	Seawall	Stepped Promenade	Total
Arthur Kill Barrier Tie-In	726	0	0	0	0	0	0	0	0	0	0	0	0	**726**
Astoria SBM	1,603	1,659	3,523	918	0	0	0	0	4,034	0	0	7,149	1,877	**20,761**
Bronx River Westchester Creek Barrier Tie-in	2,144	4,860	6,136	2,606	0	0	0	0	949	1,767	0	7,532	0	**25,994**
East Harlem SBM	411	6,431	0	0	0	0	0	0	14,038	0	0	3,907	0	**24,787**
Flushing Creek Barrier Tie-in	457	3,980	50	0	0	0	0	0	968	1,842	0	3,009	0	**10,306**
Gowanus Canal Barrier Tie-in	3,017	186	179	0	0	0	0	0	0	0	0	0	0	**3,382**
Hackensack Perimeter Lower Area - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack Perimeter Middle Area - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack Perimeter Upper Area - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack River Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Haverstraw Perimeter - Polygon	2,370	0	1,848	1,366	0	0	0	0	0	0	47	1,146	0	**6,777**
Haverstraw SBM	1,345	0	0	0	0	0	0	0	0	0	0	1,857	289	**3,490**
Jamaica Bay Barrier Tie-In	7,007	10,538	6,822	15,621	6,517	3,970	36,667	0	16,953	0	526	9,832	0	**114,454**
Kill Van Kull Barrier Tie-In	0	0	2,363	0	0	0	0	0	0	0	0	0	0	**2,363**
Kill Van Kull Barrier Tie-In Separated	178	2,009	0	0	0	0	0	0	0	0	0	0	0	**2,187**
Long Island City Astoria SBM	0	1,031	750	0	1,255	0	0	0	3,790	1,967	139	6,019	0	**14,952**
New Jersey along Hudson River SBM	1,670	7,662	3,470	1,190	16,223	0	0	0	9,278	0	2,565	182	0	**42,240**
New York City West Side SBM	1,394	5,658	10,294	892	783	0	0	1,456	6,257	1,387	0	1,584	0	**29,705**
Newtown Creek Barrier Tie-in	3,502	2,277	0	1,327	947	0	0	0	509	0	0	4,521	0	**13,083**
Ossining SBM	484	0	0	0	0	0	0	0	0	0	0	2,364	740	**3,588**
Pelham Barrier Tie-In	158	142	0	2,254	0	0	0	0	0	0	0	0	0	**2,555**
Sandy Hook-Rockaway Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Tarrytown SBM	2,737	0	563	0	1,956	0	0	0	1,037	0	0	474	413	**7,180**
Throgs Neck Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Verrazano Narrows Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Yonkers North SBM	2,180	146	36	240	0	0	0	0	1,405	0	0	2,407	1,011	**7,424**
Yonkers South SBM	1,089	0	321	0	0	0	0	0	2,257	0	0	2,915	0	**6,581**
TOTAL	**32,472**	**46,578**	**36,356**	**26,414**	**27,681**	**3,970**	**36,667**	**1,456**	**61,476**	**6,963**	**3,277**	**54,897**	**4,329**	**342,536**

Table 5-7: Length in feet of Shoreline Based Measures by Type and Location – Alternative 4

	Medium Floodwall	Large Floodwall	Extra Large Floodwall	Medium Levee	Large Levee	Composite Seawall/ Dune - NNBF	Composite Seawall/ Dune - Urban	Deployable Flood Barrier - Flip Up Barrier	Elevated Promenade	Floodwall with Park	Levee - Road Ramp	Seawall	Stepped Promenade	Total
Arthur Kill Barrier Tie-In	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Astoria SBM	1,603	1,659	3,523	918	0	0	0	0	4,034	0	0	7,149	1,877	**20,761**
Bronx River Westchester Creek Barrier Tie-in	2,144	4,860	6,136	2,606	0	0	0	0	949	1,767	0	7,532	0	**25,994**
East Harlem SBM	411	6,431	0	0	0	0	0	0	14,038	0	0	3,907	0	**24,787**
Flushing Creek Barrier Tie-in	457	3,980	50	0	0	0	0	0	968	1,842	0	3,009	0	**10,306**
Gowanus Canal Barrier Tie-in	3,017	186	179	0	0	0	0	0	0	0	0	0	0	**3,382**
Hackensack Perimeter Lower Area - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack Perimeter Middle Area - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack Perimeter Upper Area - Polygon	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack River Barrier Tie-in	7,659	8,021	654	6,183	4,093	0	0	0	0	0	67	0	0	**26,678**
Haverstraw Perimeter - Polygon	2,370	0	1,848	1,366	0	0	0	0	0	0	47	1,146	0	**6,777**
Haverstraw SBM	1,345	0	0	0	0	0	0	0	0	0	0	1,857	289	**3,490**
Jamaica Bay Barrier Tie-In	7,007	10,538	6,822	15,621	6,517	3,970	36,667	0	16,953	0	526	9,832	0	**114,454**
Kill Van Kull Barrier Tie-In	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Kill Van Kull Barrier Tie-In Separated	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Long Island City Astoria SBM	0	1,031	750	0	1,255	0	0	0	3,790	1,967	139	6,019	0	**14,952**
New Jersey along Hudson River SBM	1,670	7,662	3,470	1,190	16,223	0	0	0	9,278	0	2,565	182	0	**42,240**
New York City West Side SBM	1,394	5,658	10,294	892	783	0	0	1,456	6,257	1,387	0	1,584	0	**29,705**
Newtown Creek Barrier Tie-in	3,502	2,277	0	1,327	947	0	0	0	509	0	0	4,521	0	**13,083**
Ossining SBM	484	0	0	0	0	0	0	0	0	0	0	2,364	740	**3,588**
Pelham Barrier Tie-In	158	142	0	2,254	0	0	0	0	0	0	0	0	0	**2,555**
Sandy Hook-Rockaway Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Tarrytown SBM	2,737	0	563	0	1,956	0	0	0	1,037	0	0	474	413	**7,180**
Throgs Neck Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Verrazano Narrows Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Yonkers North SBM	2,180	146	36	240	0	0	0	0	1,405	0	0	2,407	1,011	**7,424**
Yonkers South SBM	1,089	0	321	0	0	0	0	0	2,257	0	0	2,915	0	**6,581**
TOTAL	**39,227**	**52,590**	**34,647**	**32,597**	**31,774**	**3,970**	**36,667**	**1,456**	**61,476**	**6,963**	**3,344**	**54,897**	**4,329**	**363,937**

Table 5-8: Length in feet of Shoreline Based Measures by Type and Location – Alternative 5

	Medium Floodwall	Large Floodwall	Extra Large Floodwall	Medium Levee	Large Levee	Composite Seawall/ Dune - NNBF	Composite Seawall/ Dune - Urban	Deployable Flood Barrier - Flip Up Barrier	Elevated Promenade	Floodwall with Park	Levee - Road Ramp	Seawall	Stepped Promenade	Total
Arthur Kill Barrier Tie-In	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Astoria SBM	1,603	1,659	3,523	918	0	0	0	0	4,034	0	0	7,149	1,877	**20,761**
Bronx River Westchester Creek Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
East Harlem SBM	411	6,431	0	0	0	0	0	0	14,038	0	0	3,907	0	**24,787**
Flushing Creek Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Gowanus Canal Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Hackensack Perimeter Lower Area - Polygon	0	0	0	6,294	7,353	0	0	0	0	0	85	0	0	**13,732**
Hackensack Perimeter Middle Area - Polygon	0	0	0	10,046	0	0	0	0	0	0	152	0	0	**10,197**
Hackensack Perimeter Upper Area - Polygon	0	0	0	31,509	7,947	0	0	0	0	0	441	0	0	**39,898**
Hackensack River Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Haverstraw Perimeter - Polygon	2,370	0	1,848	1,366	0	0	0	0	0	0	47	1,146	0	**6,777**
Haverstraw SBM	1,345	0	0	0	0	0	0	0	0	0	0	1,857	289	**3,490**
Jamaica Bay Barrier Tie-In	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Kill Van Kull Barrier Tie-In	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Kill Van Kull Barrier Tie-In Separated	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Long Island City Astoria SBM	0	1,031	750	0	1,255	0	0	0	3,790	1,967	139	6,019	0	**14,952**
New Jersey along Hudson River SBM	1,670	7,662	3,470	1,190	16,223	0	0	0	9,278	0	2,565	182	0	**42,240**
New York City West Side SBM	1,394	5,658	10,294	892	783	0	0	1,456	6,257	1,387	0	1,584	0	**29,705**
Newtown Creek Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Ossining SBM	484	0	0	0	0	0	0	0	0	0	0	2,364	740	**3,588**
Pelham Barrier Tie-In	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Sandy Hook-Rockaway Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Tarrytown SBM	2,737	0	563	0	1,956	0	0	0	1,037	0	0	474	413	**7,180**
Throgs Neck Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Verrazano Narrows Barrier Tie-in	0	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Yonkers North SBM	2,180	146	36	240	0	0	0	0	1,405	0	0	2,407	1,011	**7,424**
Yonkers South SBM	1,089	0	321	0	0	0	0	0	2,257	0	0	2,915	0	**6,581**
TOTAL	**15,282**	**22,586**	**20,804**	**52,454**	**35,518**	**0**	**0**	**1,456**	**42,096**	**3,355**	**3,429**	**30,004**	**4,329**	**231,313**

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

This report documents the general design criteria and conceptual design for the shoreline based flood risk reduction measures which are components of the New York-New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study. The purpose of the study is to develop generalized SBMs that are comprehensive enough to be applicable and suitable for the entire study area yet not too detailed or site specific such that they could only be applied at one location. The development of the conceptual designs of the SBMs allows for quantity take-offs. Based on the alignments developed by USACE for the HAT Study Alternatives, the following SBMs were developed and conceptual designed were presented:

1. Floodwalls
    a. Medium Floodwall,
    b. Large Floodwall,
    c. Extra Large Floodwall,
2. Levees
    a. Medium Levee,
    b. Large Levee,
3. Promenades with Flood Risk Reduction Function
    a. Stepped Promenade, and
    b. Elevated Promenade,
4. Seawall,
5. Composite Seawall with Beach and Dune,
    a. Composite Seawall with Beach and Dune for Urban Application, and
    b. Composite Seawall with Beach and Dune for Natural Shorelines Application (NNBF),
6. Deployable Flood Barriers
    a. Flip-up Barrier,
    b. Operable Flood Gate,
7. Tide Gate.

The completion of the conceptual designs allowed for an inventory of shoreline based measures (total length and number of measures) per HAT Study Alternative and for a quantity take-off per SBM. This data set can used by USACE-NAN as input to complete cost estimates for the study alternatives and allow for a relative comparison amongst the alternatives on cost and benefits.

6.2 Recommendations

6.2.1 Recommendations for Alignment Refinements

The study alternatives were based on the alignments provided by USACE-NAN. At a number of locations small alignment adjustments were made to allow for the implementation of SBMs. In many instances the change in alignment or the selection of the SBM relied on a cursory review of available data. It should be reiterated that no site-specific topographic survey, bathymetric survey, condition survey and/or geotechnical analysis have been completed. Instead, in accordance with USACE's SMART planning principles, changes in alignment and the selection of SBM type was based on qualitative data and desktop level analysis which resulted in a generalization of existing conditions with the understanding that further refinements would be completed at later stages of the study when additional time and resources can be focused on the most viable alternatives. The implications of these assumptions are that further optimization of the alignment is possible and that for reaches where conflicts are most apparent, an alternative comparison on a reach by reach basis is recommended. In such a study, alternate alignments would be compared amongst each other and evaluated and screened using criteria such as, but not limited to, cost, constructability and impacts.

In general, when further refinements to the alignments of the HAT Study Alternatives are made additional data and studies are recommended to be incorporated. Such studies and data include:

1. Site topographic survey,
2. Existing structure condition survey,
3. Site-specific geotechnical data,
4. Bathymetric survey for alignments following existing bulkhead lines,
5. Site use and traffic studies,
6. Wetland survey and mapping,
7. Comprehensive interior drainage modeling, including combined probability analysis of storm surge and rainfall events.,
8. Continuation and furthering stakeholder and public outreach such that input and comments from stakeholders including city, state agencies and the public can further inform alignment alternatives to be evaluated,
9. An analysis of easement delineation and real estate studies such that impacts beyond the footprint of the measures can be preliminarily made assessed,
10. Utility investigations and as-needed service diversions or relocations studies,
11. Cost Estimates and Impacts for alignment alternatives

6.2.2 Recommendations for Further Design Refinements of SBMs

Due to the vast area covered by the HAT Study and preliminary nature of the study, generalized assumptions were made, including but not limited to, existing site topography, bathymetry, soil

conditions, and site conditions such that a set of CSRM measures could be developed and conceptually designed. Publicly available qualitative data and results from desktop analyses were used to develop a generic basis of design which serves as a common origin for all SBMs designs independent of their location of application throughout the Study Area. The conceptual designs for the SBMs are of sufficient detail to support a quantity take-off. The SBMs used here are limited to a total of thirteen (13) SBMs and are at a level of detail commensurate with a feasibility study.

Assumptions as discussed in the report have been made to advance the design; but it should be noted that additional data and studies are needed for the next design phase to refine the SBM designs such that more site-specific measures can be developed for the recommended alternative(s). Recommendations for next phase of the project include:

1. Setting the overtopping criterion and optimize it for the study. Ideally the overtopping criterion is informed by the two main considerations:
    a. The ability of the risk reduction system to handle the volume of overtopping (i.e. pumping or storage on the protected side of the risk reduction alignment may allow for accepting large overtopping volumes), and
    b. The type of construction on the protected side of the alignment. E.g. grey infrastructure has a relative high tolerance for large overtopping discharges prior to the onset of structural failure while levees have a lower tolerance. Given the urban nature of the study are and relatively high portion of grey SBMs a higher overtopping criterion could be considered
2. Refine the SBMs on a site-specific basis.
3. Evaluate the need for additional SBMs or site-specific SBMs.
4. Assess and design the transitions between various SBMs
5. Complete a gate type evaluation. E.g. for locations where deployable flood gates and tide gates are required, types, sizes and configurations should be evaluated

Finally, albeit that major construction items have been accounted for in the quantity take-off, a number of items could only be qualitatively discussed as insufficient information or data is available at this stage in the study to provide meaningful quantitative data. For the SBMs additional data collection and studies should be completed such that existing data gaps can be filled and a more complete inventory of items and work form the basis the cost estimates. Such items are detailed in Table 6-1.

Table 6-1: Detailed Design Work to be Considered During Next Phases of the Study

SBM Appurtenances and Construction Related Work Items	Example
Utility relocation	Gas, water, electricity, cable, etc.
Additional drainage features	Storm water pipes, channels, etc.
Additional aesthetic features	Pavers, textured wall, etc.
Additional access features	Ramps, railing, stairs, etc.
Additional non-structural items	Lighting, conduits, emergency phones, real estate, right-of-way, easement quantities, environmental mitigation, etc.
Upgrade	Strengthening of existing structural elements, upgrade of non-structural items, etc.

7 REFERENCES

1. Booij, N, Holthuijsen, L.H., and Ris, R.C., 1996: The "SWAN" wave model for shallow water.
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3. USACE (2015). North Atlantic Coast Comprehensive Study: Resilitent Adaptation to Increasing Risk, 2015.
4. USACE (2016). Atlantic Coast of New York, East Rockaway Inlet to Rockaway Inlet Draft Hurricane Sandy General Revaluation Report and Environmental Impact Statement Engineering Appendix A-2.
5. USACE (2018a). DRAFT FINAL Integrated Hurricane Sandy General Reevaluation Report and Environmental Impact Statement Atlantic Coast of New York, East Rockaway Inlet to Rockaway Inlet and Jamaica Bay, Appendix A1 (Shorefront Engineering and Design Appendix)
6. USACE (2018b). DRAFT FINAL Integrated Hurricane Sandy General Reevaluation Report and Environmental Impact Statement Atlantic Coast of New York, East Rockaway Inlet to Rockaway Inlet and Jamaica Bay, Appendix A2 (Jamaica Bay High Frequency Flood Risk Reduction Features Engineering and Design Appendix)
7. USACE (2018c). NY/NJ Harbor Wide Gate/Beach Restoration Alternatives. October 2018.

A. APPENDIX A: ANALYSIS FOR REQUIRED FREEBOARD OF CSRM MEASURES

The analysis to determine the required freeboard for the land-based coastal storm risk management (CSRM) measures for HATS is described in this appendix.

An allowable overtopping threshold of 0.0108 cubic foot per second per foot (cfs/ft) or 1 liter per second per meter (l/s/m) was adopted for all land-based structural features, with the exception of the composite seawall sections in the Rockaway peninsula (USACE, 2016). The 100-year (1% Annual Exceedance Probability) still water levels and wave conditions were used to inform the determination of the design freeboard for the individual project features. These input conditions were derived from the corresponding statistics obtained from the USACE (2015) North Atlantic Comprehensive Coastal Study (NACCS) database.

North Atlantic Comprehensive Coastal Study

The NACCS (2015) was conducted to provide information for computing the joint probability of coastal storm forcing parameters for the U.S. North Atlantic Coast, which is critical for effective flood risk management. As part of the NACCS, estimates of nearshore winds, waves, and water-levels, as well as the associated marginal and joint probabilities were evaluated. This was done by simulating a selected suite of tropical and extra-tropical storms to characterize the regional storm hazard. The modeling suite consisted of an offshore wave model (WAM) for simulation of deep-water waves, which were subsequently used to generate boundary conditions for a near-shore steady state wave model STWAVE. The STWAVE model for near-shore waves also allowed for simulation of local wind-generated waves, and was paired with the hydrodynamic circulation model ADCIRC to allow for dynamic interaction between surge and waves. While the ADCIRC model mesh extends across the western North Atlantic with approximately 3.1 million nodes, the nearshore wave model STWAVE is applied over ten domains from coastal Virginia to Maine, including one in the upper New York Bight area. A suite of 1150 storms including 100 extratropical events, and 1050 synthetic tropical events were simulated for the NACCS production. The high-frequency outputs and statistical products from the modeling are publicly archived for a relatively small number of 18000 Save Points.

A.1. Still Water Levels

The still water levels corresponding to the 100-year return period (i.e. 1% AEP) vary across the New York Harbor area. The NACCS results which include the computed water-level statistics corresponding to different confidence intervals based on ADCIRC model results at several output locations (Save Points) were used to inform the design water-levels for HATS. The extracted 100-year (1% AEP) still water levels corresponding to the 50% confidence limit within the Project Area are shown in Figure A- 1. These still water levels were subsequently updated to include the projected Sea Level Change until the start date of the Project (2030), and up to the end of the functional design evaluation period (2080) based on the USACE intermediate estimate of Sea

Level Change, which correspond to approximately 0.6 feet and 1.8 feet respectively based on available estimate from the Sandy Hook tidal gage.

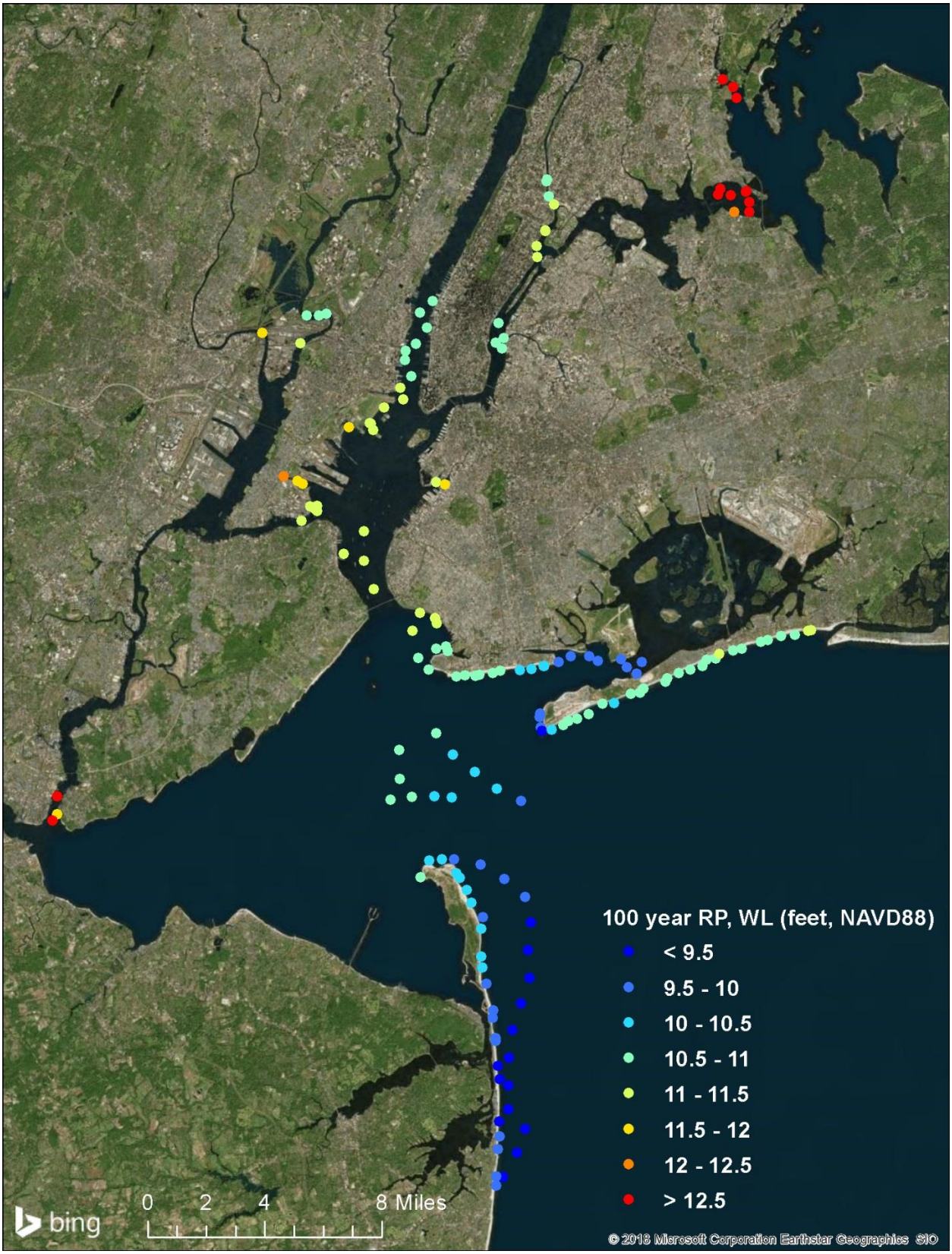

Figure A- 1: 100-year Still Water Level extracted from NACCS Save Points

A.2. Wave Conditions

The expected significant wave heights for the 100-year return period (i.e. 1% AEP) were extracted from NACCS at several Save Points within New York Harbor as shown in Figure A- 2. The expected wave-heights at the project features were further updated to account for the transformation of the wave from the NACCS Save Point located within the Harbor, to the individual project features located at the shoreline, where the waves are expected to be depth-limited.

The Simulation of Waves Nearshore (SWAN) model (Booij et al, 1996) was used to simulate the transformation of waves along 1-D transects from boundary points near NACCS save points to the corresponding project alignment features. The model as applied here accounts for the wave transformation over nearshore bathymetry due to shoaling, bottom-friction, and depth-induced wave-breaking. The bathymetric data for the modeling was derived from high-resolution (1/9 arc seconds or 10 feet) resolution topo-bathy Digital Elevation Models (DEMs) developed by NOAA, post- Hurricane Sandy in 2012. The map of the DEM in the HATS Study Area is shown in Figure A- 3.

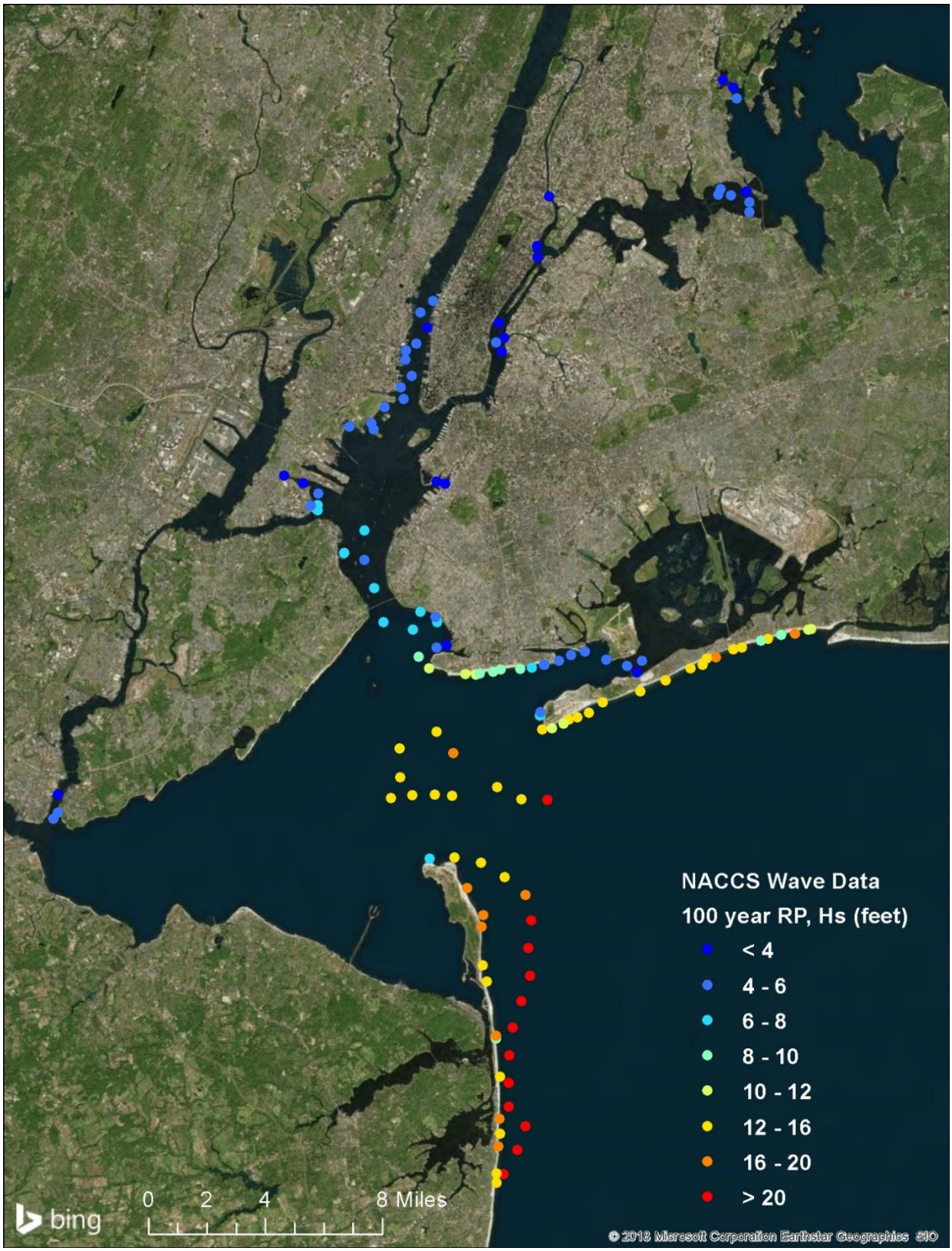

Figure A- 2: 100-yr wave-height statistic extracted from NACCS model Save Points

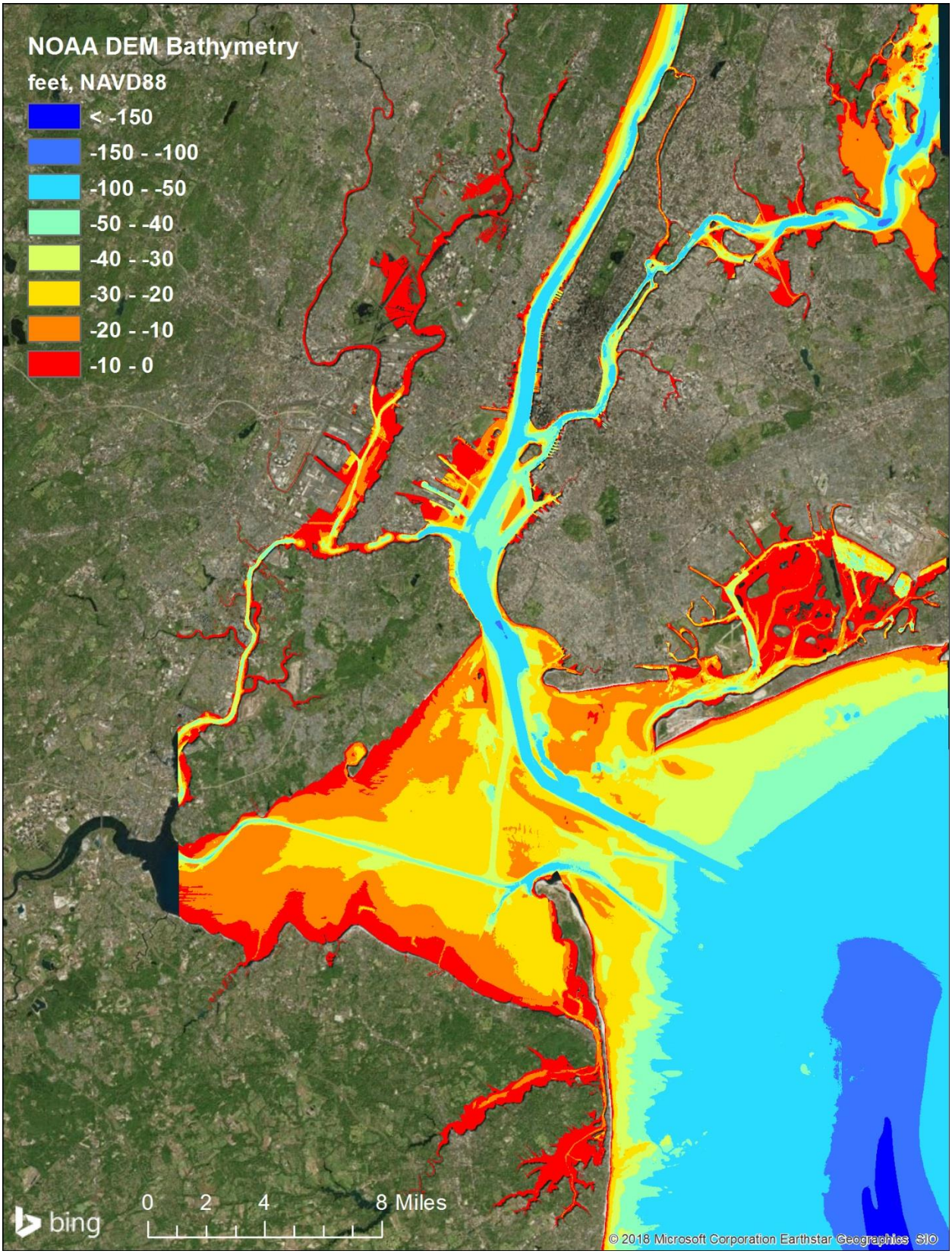

Figure A- 3: NOAA High-resolution DEM in HATS Study Area

Modeling wave-heights for project features

Based on the available resolution of NACCS wave statistics across the study area, transects were drawn to model wave-transformation at several HATS locations using the 100-year return period (1% AEP) wave-height as boundary condition. Figure A- 4 shows these transect locations with respect to the HATS alignments for the shoreline based measures (SBM).

At each of these transects, the bottom elevation profile was extracted from the DEM every 6 feet to specify the model bathymetry. A model still-water elevation corresponding to the respective 100-year return period (1% AEP) Still Water-Level plus the Sea Level Rise (SLR) corresponding to the USACE intermediate projection for 2080 was applied. A typical JONSWAP wave spectrum centered on the 100-year NACCS wave-height at the boundary point, and a corresponding peak wave-period according to typical fetch and depth limited wave growth (CERC, 1984) was assumed. The SWAN model was run in stationary mode, which means that the wave conditions within the 1-D model domain were allowed to evolve to a steady-state with the input conditions.



Figure A- 4: 1-D wave model transects to estimate wave conditions at SBMs

Wave Model Results

Figure A- 5 shows an overview of the model outputs with colored transects representing the magnitude of the simulated wave-heights. Profiles of bottom elevation, water-level, and predicted wave-height at a few selected transects are shown in Figure A- 6 as an example. From each model transect, the results at point closest to the corresponding shore based measure was then extracted as shown in Figure A- 7. These results were used as the estimated design wave-heights and periods for the respective project features. A map of the design wave-height at each HATS alignment based on the simulated wave-height seaward of the feature from the nearest model transect is shown in Figure A- 8, and the corresponding wave periods in Figure A- 9.

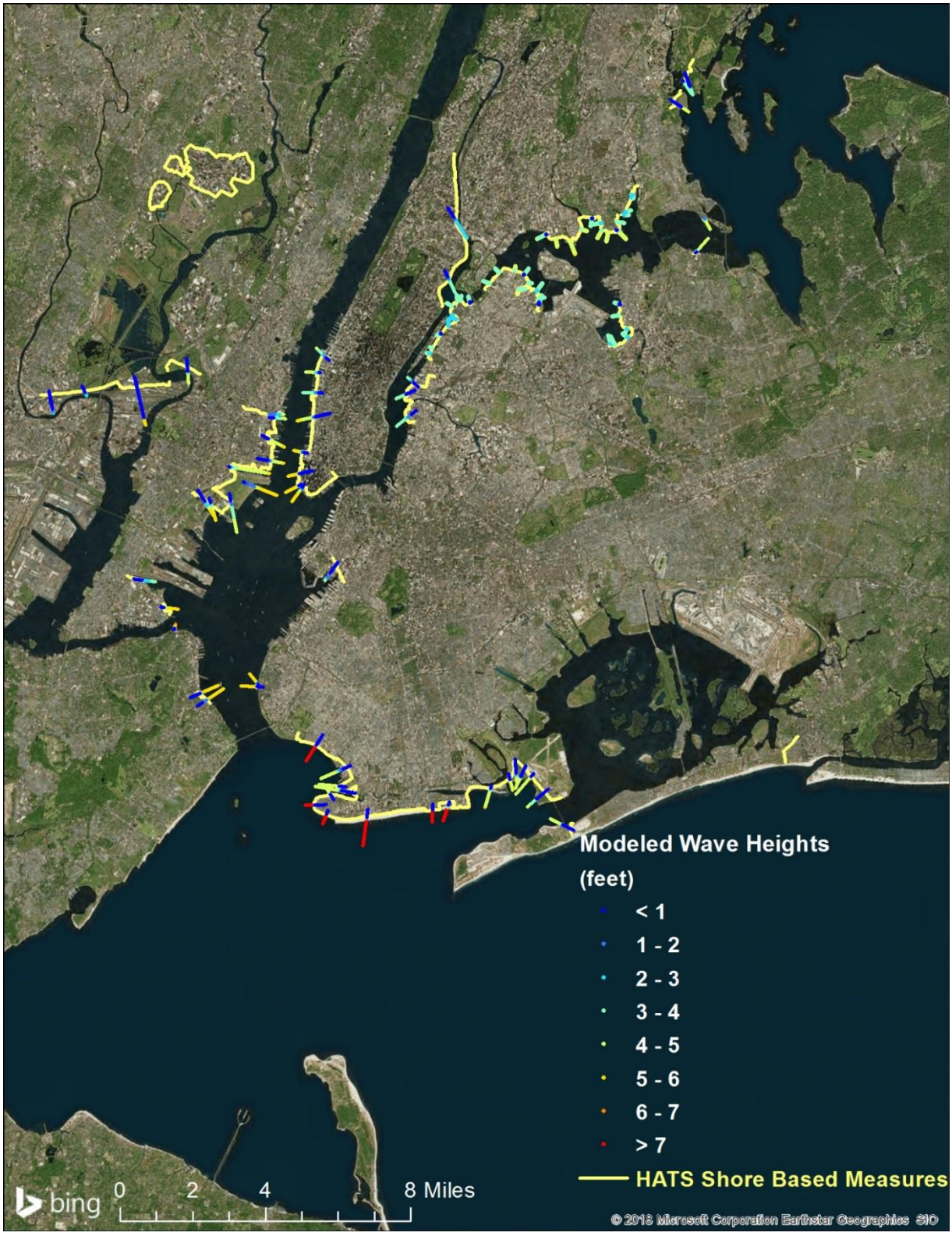

Figure A- 5: Overview of wave model output showing predicted wave-heights

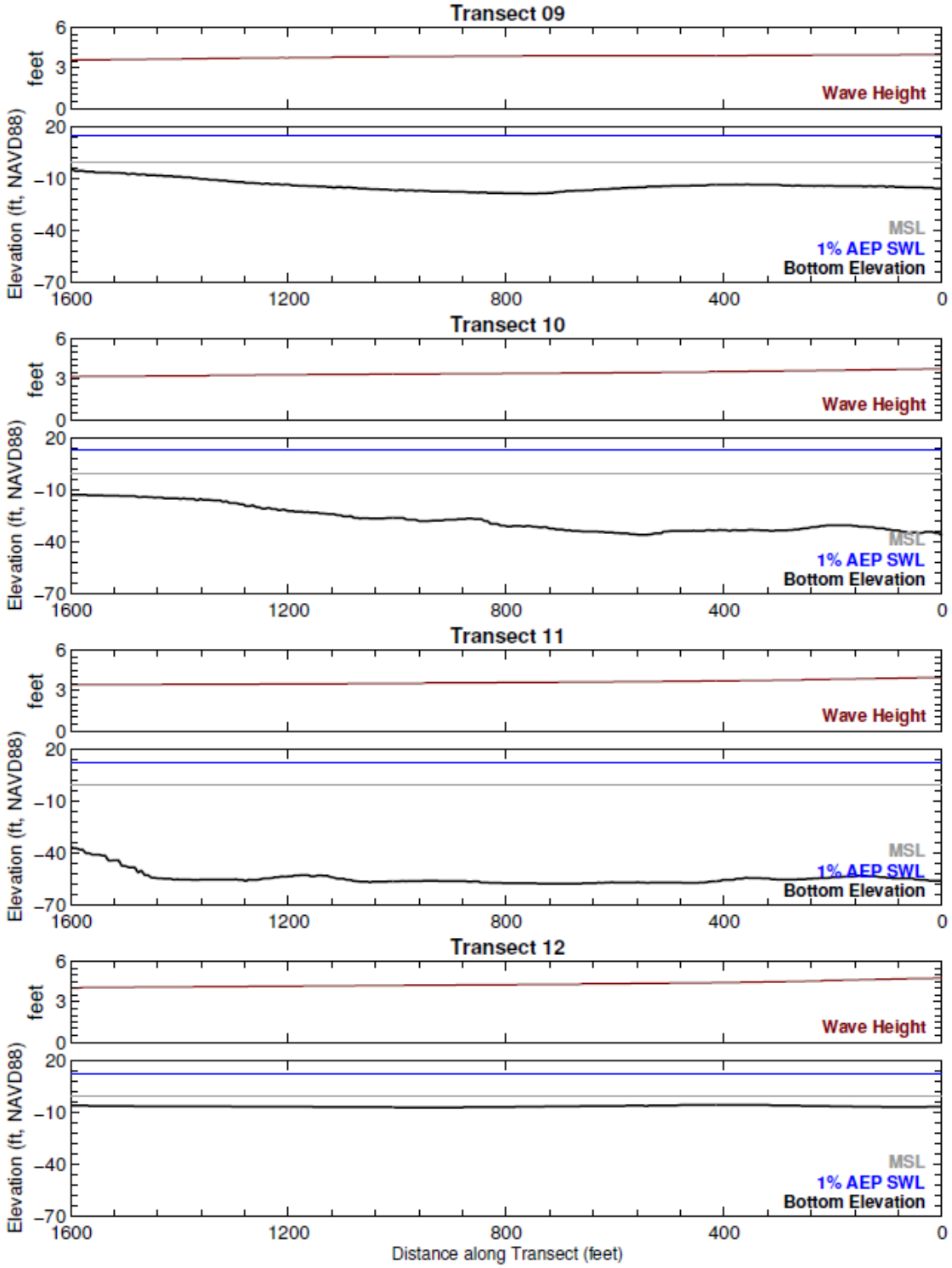

Figure A- 6: Transect elevation profiles at select locations showing 1-D model wave-height transformation

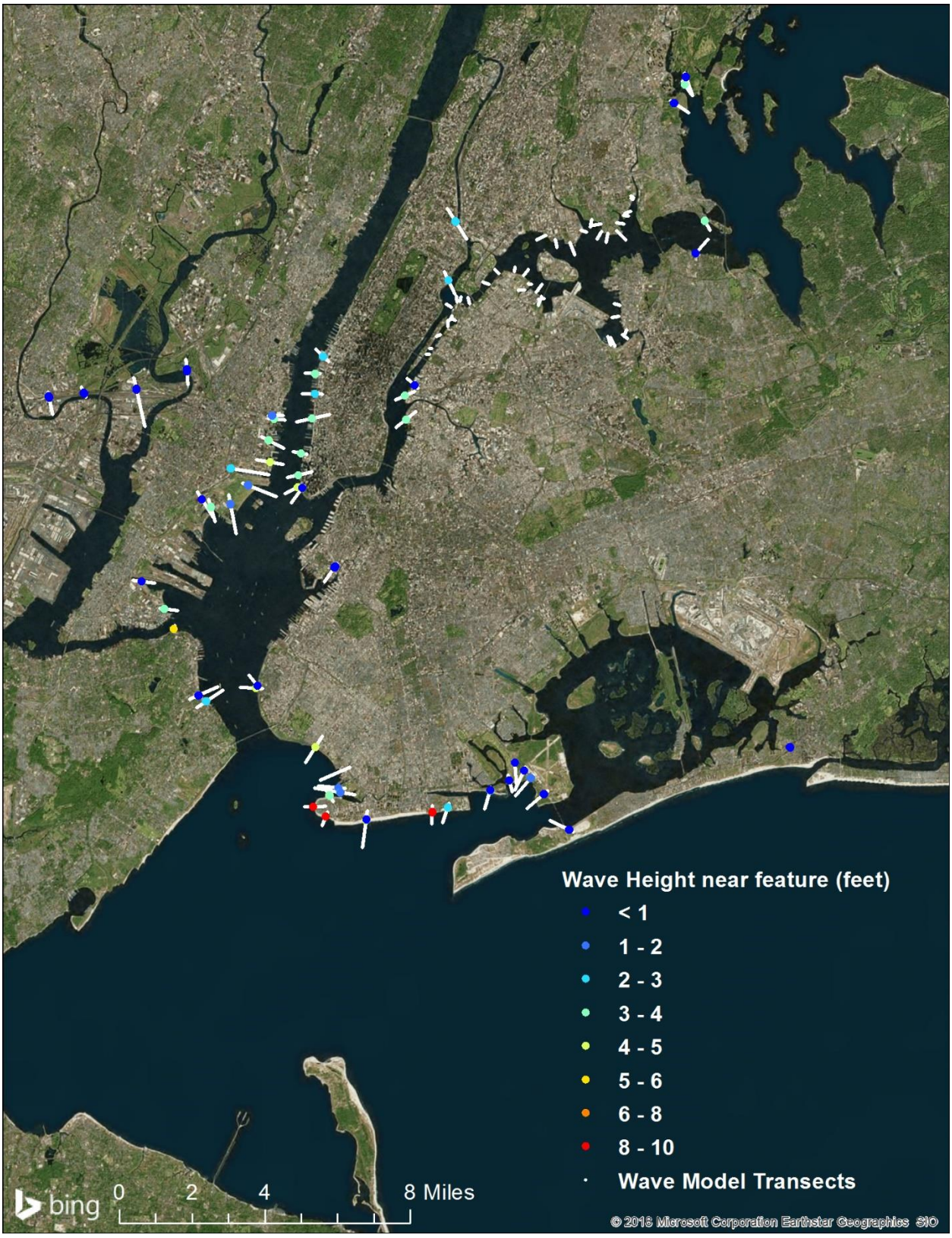

Figure A- 7: Extracted model wave-height near each shore-based-measure

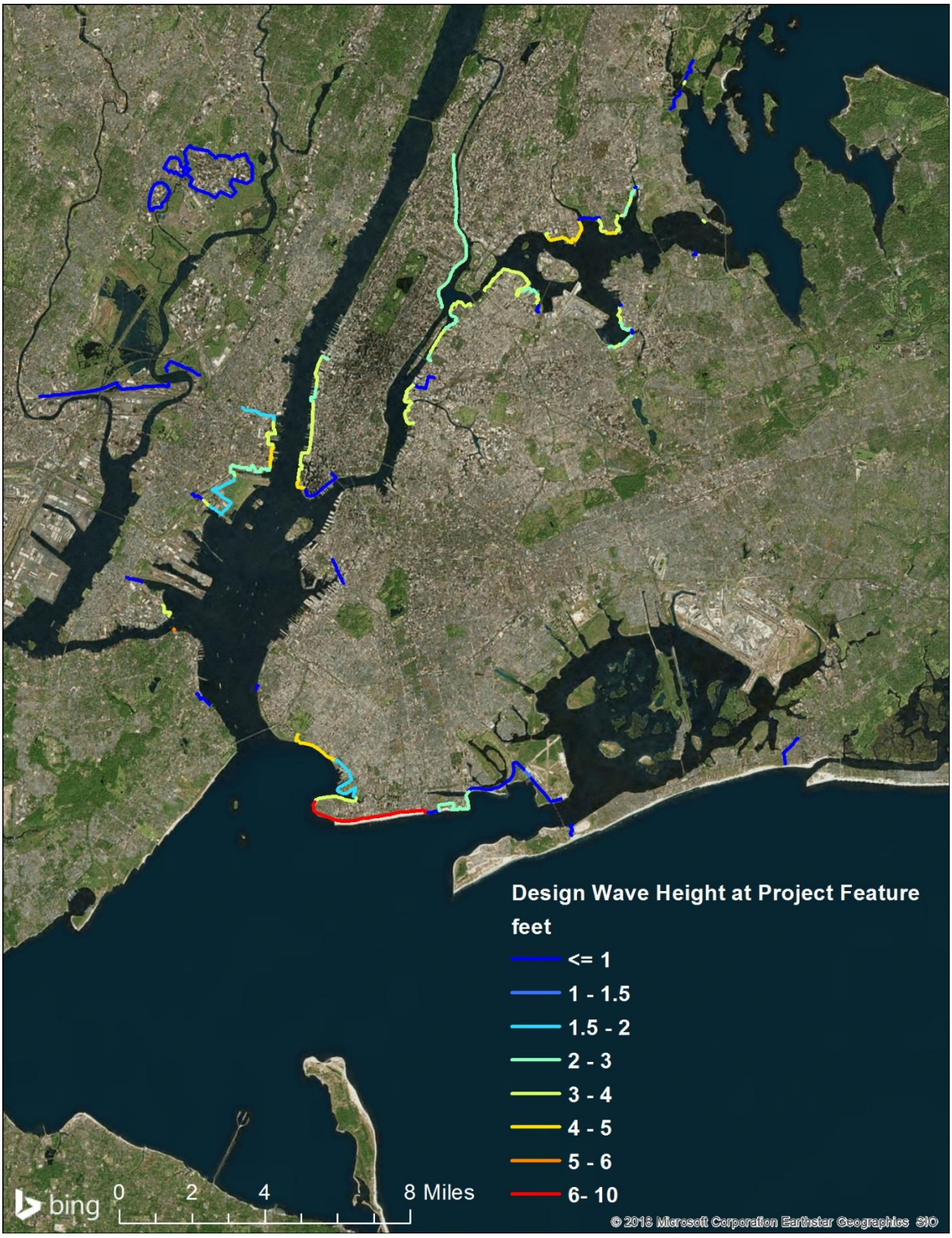

Figure A- 8: Design wave-heights at HATS alignments

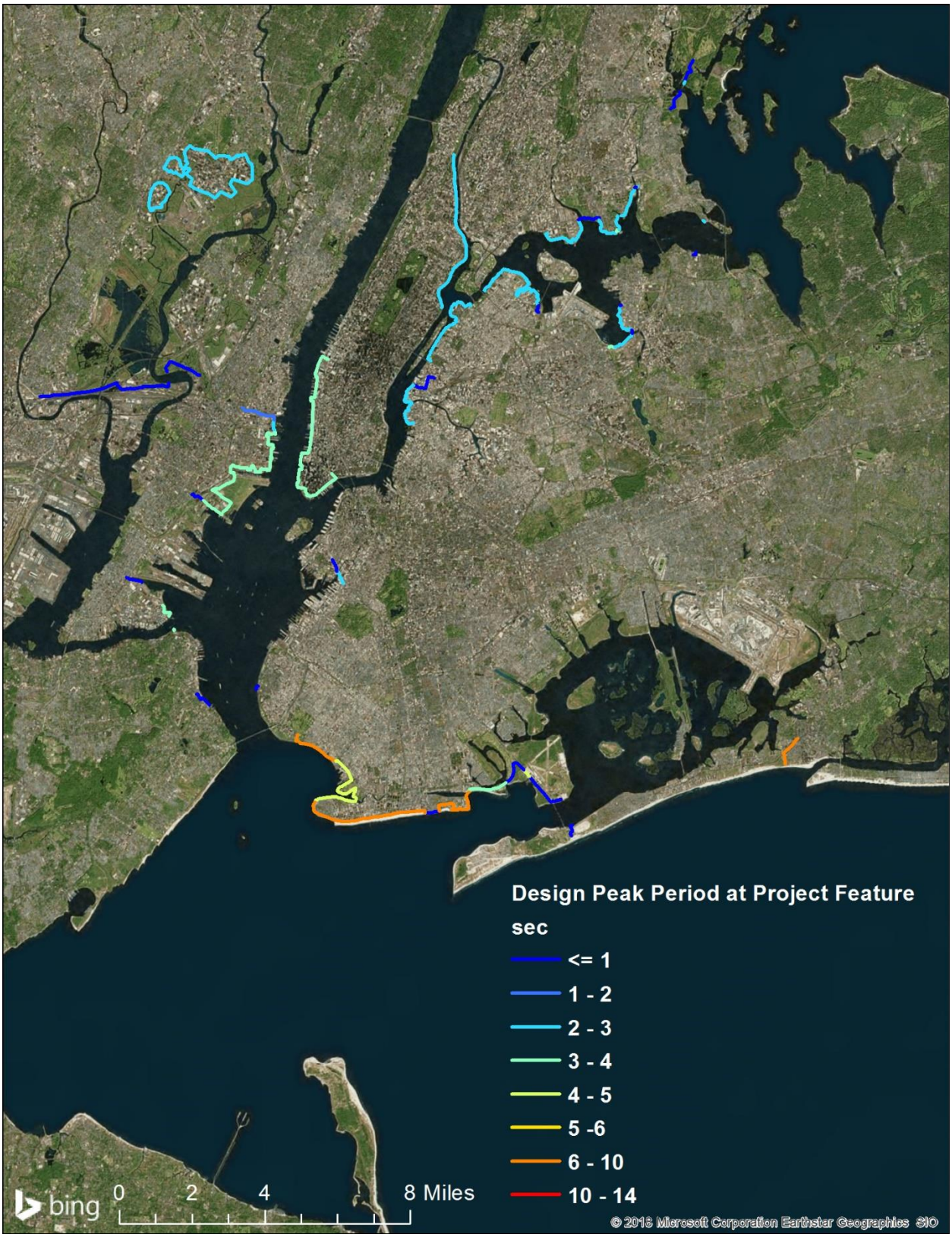

Figure A- 9: Design peak-periods at HATS alignments

A.3. Required Freeboard

The 1% AEP still water levels and waves estimated above were used as the basis for the calculation of the design freeboard for the individual project features. These calculations were based on the empirical formulae for overtopping volumes for different structure types described in EurOtop, 2016. Based on an assumed overtopping threshold of 1 l/s/m, the freeboards for three different structure types – vertical wall, sloping wall (1:3), and rubble armor – were calculated. Where the required freeboard was computed to be less than a foot, a minimum freeboard of 1 foot was used for that project feature. The elevation of the top of the wall for each feature was set as the 100-year (1% AEP) still water level plus the estimated required freeboard to meet the above described overtopping threshold. The calculated freeboards assigned to each feature based on their structure type is shown in Figure A- 10.

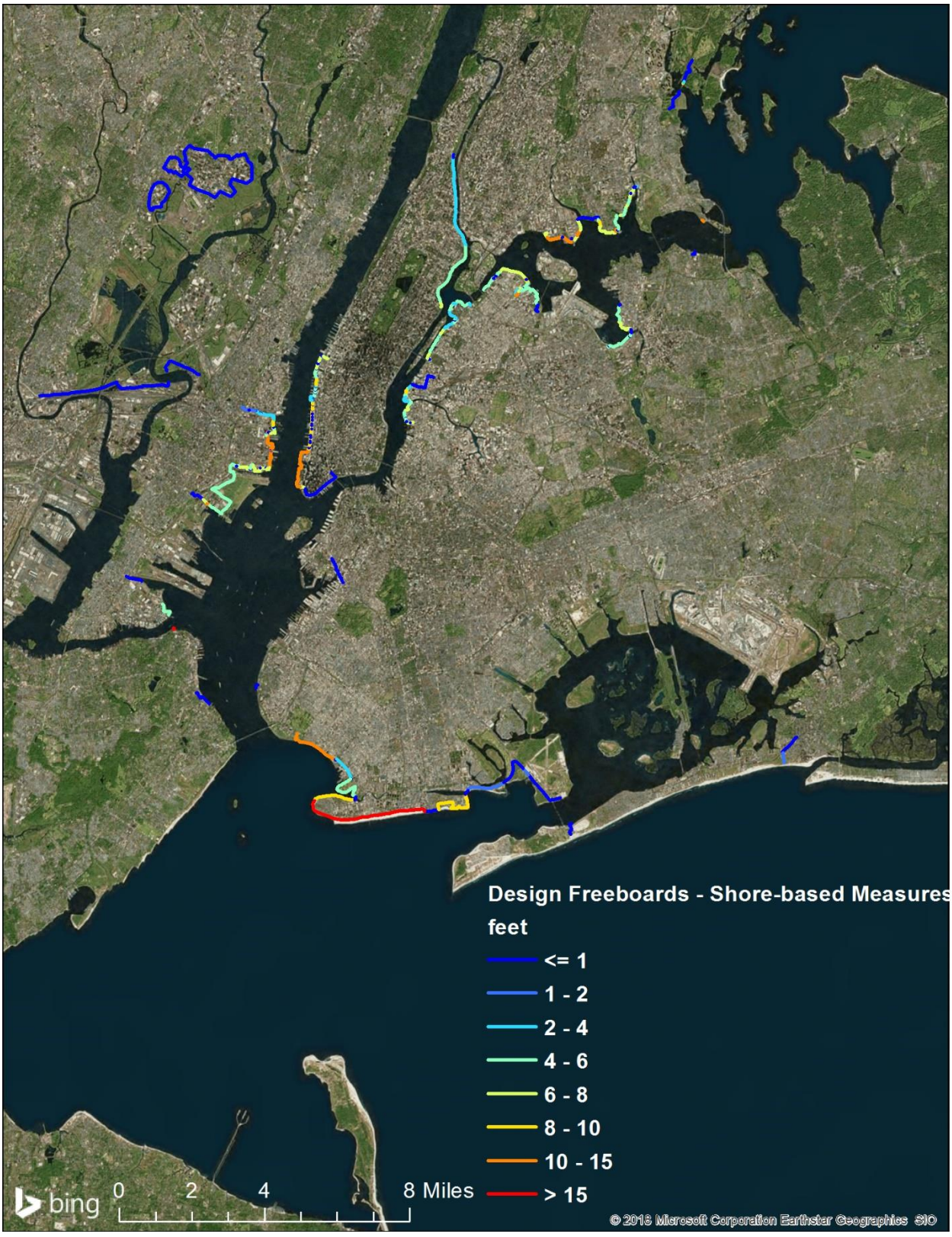

Figure A- 10: Calculated freeboards for the shoreline based measures (SBM) based on wave conditions and overtopping criteria

A.4. Conclusion

The water-level and wave conditions derived as summarized in this appendix are used as the basis for the calculation of the freeboard requirements of the shoreline based measures, to meet the specified overtopping thresholds.

The NACCS analysis of wave statistics forms the basis of the current study, as it is used to provide boundary conditions for the 1-D wave transformation to the shoreline features. The expected annual exceedance probabilities for different wave conditions for the shoreline based measures for HATS are therefore based on the NACCS analysis. However, the 1-D wave transformation model parameters were not calibrated or validated for lack of wave data at the shoreline. Additional sensitivity analysis to model parameters or more detailed analysis using a 2-D wave model could help further improve confidence in the model results.

An additional source of uncertainty in the wave transect modeling analysis derives from the limitations of the input bathymetric DEM from NOAA, which is based on interpolation of several data sources including some historical data.

B. APPENDIX B: QUANTITY TAKE-OFF

Table B-1: Operable Flood Gate Inventory – Alternative 2

Location	Number of Operable Flood Gate		
	Vehicle	Pedestrian	Rail
TOTAL COUNT	**0**	**0**	**0**

Table B-2: Tide Gate Inventory – Alternative 2

Location	Number of Tide Gate	Length of Tide Gate (feet)
TOTAL	**0**	**0**

Table B-3: Operable Flood Gate Inventory – Alternative 3A

Location	Number of Operable Flood Gate		
	Vehicle	Pedestrian	Rail
Jamaica Bay Barrier Tie-In	10	1	0
Verrazano Narrows Barrier Tie-in	1	0	0
TOTAL COUNT	**11**	**1**	**0**

Table B-4: Tide Gate Inventory – Alternative 3A

Location	Number of Tide Gate	Length of Tide Gate (feet)
Jamaica Bay Barrier Tie-In	1	552
TOTAL	**1**	**552**

Table B-5: Operable Flood Gate Inventory – Alternative 3B

Location	Number of Operable Flood Gate		
	Vehicle	Pedestrian	Rail
Astoria SBM	8	0	0
Bronx River Westchester Creek Barrier Tie-in	4	2	0
East Harlem SBM	3	0	0
Flushing Creek Barrier Tie-in	5	0	0
Gowanus Canal Barrier Tie-in	9	0	0
Haverstraw Perimeter - Polygon	7	0	0
Haverstraw SBM	1	0	0
Jamaica Bay Barrier Tie-In	10	1	0
Kill Van Kull Barrier Tie-In Separated	2	0	0
Long Island City Astoria SBM	3	0	0
New Jersey along Hudson River SBM	9	8	3
New York City West Side SBM	17	41	0
Newtown Creek Barrier Tie-in	14	3	0
Ossining SBM	1	0	2
Tarrytown SBM	3	0	1
Yonkers North SBM	2	0	1
Yonkers South SBM	2	0	1
TOTAL COUNT	**100**	**55**	**8**

Table B-6: Tide Gate Inventory – Alternative 3B

Location	Number of Tide Gate	Length of Tide Gate (feet)
Jamaica Bay Barrier Tie-In	1	552
Bronx River Westchester Creek Barrier Tie-in	1	511
Tarrytown SBM	1	42
TOTAL	**3**	**1105**

Table B-7: Operable Gate Inventory – Alternative 4

Location	Number of Operable Flood Gate		
	Vehicle	Pedestrian	Rail
Astoria SBM	8	0	0
Bronx River Westchester Creek Barrier Tie-in	4	2	0
East Harlem SBM	3	0	0
Flushing Creek Barrier Tie-in	5	0	0
Gowanus Canal Barrier Tie-in	9	0	0
Hackensack River Barrier Tie-in	1	1	1
Haverstraw Perimeter - Polygon	7	0	0
Haverstraw SBM	1	0	0
Jamaica Bay Barrier Tie-In	10	1	0
Long Island City Astoria SBM	3	0	0
New Jersey along Hudson River SBM	9	8	3
New York City West Side SBM	17	41	0
Newtown Creek Barrier Tie-in	14	3	0
Ossining SBM	1	0	2
Tarrytown SBM	3	0	1
Yonkers North SBM	2	0	1
Yonkers South SBM	2	0	1
TOTAL COUNT	**99**	**56**	**9**

Table B-8: Tide Gate Inventory – Alternative 4

Location	Number of Tide Gate	Length of Tide Gate (feet)
Hackensack River Barrier Tie-in	2	174
Jamaica Bay Barrier Tie-In	1	552
Bronx River Westchester Creek Barrier Tie-in	1	101
Tarrytown SBM	1	42
TOTAL	**5**	**869**

Table B-9: Operable Gate Inventory – Alternative 5

Location	Number of Operable Flood Gate		
	Vehicle	Pedestrian	Rail
Astoria SBM	8	0	0
East Harlem SBM	3	0	0
Hackensack Perimeter Lower Area - Polygon	2	0	0
Hackensack Perimeter Middle Area - Polygon	3	0	0
Hackensack Perimeter Upper Area - Polygon	3	0	1
Haverstraw Perimeter - Polygon	7	0	0
Haverstraw SBM	1	0	0
Long Island City Astoria SBM	3	0	0
New Jersey along Hudson River SBM	9	8	3
New York City West Side SBM	17	41	0
Ossining SBM	1	0	2
Tarrytown SBM	3	0	1
Yonkers North SBM	2	0	1
TOTAL COUNT	**64**	**49**	**9**

Table B-10: Tide Gate Inventory – Alternative 5

Location	Number of Tide Gate	Length of Tide Gate (feet)
Hackensack Perimeter Upper Area - Polygon	2	107
Tarrytown SBM	1	42
TOTAL	**3**	**149**

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Project Title:	New York/New Jersey Harbor & Tributaries CSRM Study	Update revision No. By:	M. Kluijver	Date:	2/15/2019
Consultant Name:	Moffatt & Nichol				
Consultant Contact:	M. Kluijver				
Revision:	D				

XL Floodwall		
Item	**Quantity per linear foot**	**Unit**
Reinforced Concrete for Flood Wall and Splash Aprons	5.48	CY
PZ-27 Sheetpile Wall	0.72	TN
HP 14x89 Piles (99' long, 3.61 TN)	0.29	EA
Excavation	3.93	CY
Repair Disturbed Pavement	0.11	SY

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Ladders/Stairs with handrail to provide access to flood side and aid inspection
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

Large Floodwall		
Item	**Quantity per linear foot**	**Unit**
Reinforced Concrete for Flood Wall and Splash Aprons	4.07	CY
PZ-27 Sheetpile Wall	0.53	TN
HP 14x89 Piles (3.43 TN, 93' Long)	0.24	EA
Excavation	2.41	CY
Repair Disturbed Pavement	0.11	SY

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Ladders/Stairs with handrail to provide access to flood side and aid inspection
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

Medium Floodwall		
Item	**Quantity per linear foot**	**Unit**
Reinforced Concrete for Flood Wall and Splash Aprons	2.85	CY
PZ-27 Sheetpile Wall	0.36	TN
HP 14x89 Piles (3.03 TN, 83' Long)	0.20	EA
Excavation	2.41	CY
Repair Disturbed Pavement	0.11	SY

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Ladders/Stairs with handrail to provide access to flood side and aid inspection
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

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Consultant Contact:	M. Kluijver				
Revision:	D				

Medium Levee		
Item	**Quantity per linear foot**	**Unit**
Impervious Core	6.70	CY
Fill Dirt	6.37	CY
Berm Excavation	5.04	CY
Drainage Ditch Excavation	0.45	CY
Grass Cover with Matting	7.89	SY

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Patrol/Inspection Roads
Access Ramps
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

Large Levee		
Item	**Quantity per linear foot**	**Unit**
Impervious Core	15.15	CY
Fill Dirt	19.41	CY
Berm Excavation	8.56	CY
Drainage Ditch Excavation	0.45	CY
Grass Cover with Matting	12.67	SY

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Patrol/Inspection Roads
Access Ramps
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

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Stepped Promenade		
Item	**Quantity per linear foot**	**Unit**
Reinforced Concrete for Promenade	5.89	CY
PZ-27 Sheetpile Wall	0.47	TN
Piles (24" Dia steel pipe, 77' long ave., 60 tons ave.)	0.67	#/FT
Riprap Slope Protection	8.00	TN
Concrete for Filling Piles	1.55	CY
Railings	1.00	LF
Stairs / Access Ramps	0.002	EA

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Benches
Emergency Phones
AEDs
Bike Racks
Bollards
Concrete Markings (Striping for Pedestrians / Bikes)
Signage
Lighting
Conduit (to power lighting)
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

Elevated Promenade		
Item	**Quantity per linear foot**	**Unit**
Sand Fill	33.33	CY
Reinforced Concrete	3.39	CY
Riprap	2.11	TN
Bedding Stone	1.68	TN
Steel Flat Sheet Piles PS 27.5	12.40	TN
Geotextile	2.11	SY
Railings	2.00	LF
Stairs / Access Ramps	0.002	EA

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Benches
Emergency Phones
AEDs
Bike Racks
Bollards
Concrete Markings (Striping for Pedestrians / Bikes)
Signage
Lighting
Conduit (to power lighting)
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

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Consultant Name:

Composit Seawall with Beach and Dune (Urban Application)		
Item	**Quantity per linear foot**	**Unit**
Bedding Stone	7.87	TN
Armor Stone W-50	10.44	TN
Splash Apron Stone	3.27	TN
Reinforced Concrete Pile Cap	0.36	CY
PZ-27 Sheetpile Wall	0.30	TN
Geotextile Fabric	6.56	SY
Planting Area	8.22	SY
Sand Fill	50.62	CY
Excavation	7.59	CY

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate:
Stairs and ramps with handrail
Dune Fence to protect plantings
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

Composit Seawall with Beach and Dune (NNBF Application)		
Item	**Quantity per linear foot**	**Unit**
Bedding Stone	8.87	TN
Armor Stone W-50	10.44	TN
Splash Apron Stone	4.99	TN
Reinforced Concrete Pile Cap	0.36	CY
PZ-27 Sheetpile Wall	0.30	TN
Geotextile Fabric	8.78	SY
Planting Area	18.22	SY
Sand Fill	35.33	CY
Excavation	14.56	CY

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate:
Stairs and ramps with handrail
Dune Fence to protect plantings
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

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Seawall		
Item	**Quantity per linear foot**	**Unit**
Reinforced Concrete	3.56	CY
Armor Stone D50 = 2.3'	8.56	TN
Excavation	5.37	CY
PZ-27 Sheetpile Wall	0.41	TN
HP 14x89 Piles (95', 3.47 TN)	0.29	EA
Geotextile	4.56	SY
Underlayer Stone D50 = 1.1'	3.14	TN
Toe Armor Stone D50 = 1.4'	1.44	TN

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Ladders/Stairs with handrail to provide access to flood side and aid inspection
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

Floodwall With Park Integration		
Item	**Quantity per linear foot**	**Unit**
Reinforced Concrete	3.04	CY
Fill Dirt	9.07	CY
Excavation	2.59	CY
PZ-27 Sheetpile Wall	0.47	TN
HP 14x89 Piles (84' long, 3.07 TN)	0.22	EA
Grass Cover With Matting	8.56	SY
Stairs / Access Ramps	0.002	EA

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Benches
Picnic Tables
Lighting
Landscaping
Stairs and ramps with handrail
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

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Levee with Road Ramp		
Item	**Quantity***	**Unit**
Impervious Core	2431.28	CY
Fill Dirt	6435.13	CY
Asphalt	469.20	TN
Sub Base	349.07	TN
Grass Cover with Matting	2488.22	SY
Excavation	1373.17	CY
Asphalt Removal	1159.78	SY
Guard Rail	616.00	LF

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Signs
Reflectors
Striping
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

*Note: Levee with Road Ramp quantites are on a per each basis.

Flip-up Barrier		
Item	**Quantity***	**Unit**
Flip-up Barrier	NA	NA

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Foundation including piles
Transitions/ additional structual supports due to change in directions or obstructions
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

*Note: The Flip-up barrier length was provided. Per discussion with USACE-NAN, only the length of the measure is provided here. Commensurate with the level of analysis for this feasibility study and nature of the project, material quantities per linear foot has not been obtained for this passive automatic flood barrier as it will be highly site dependent.

Operable Floodgates		
Item	**Quantity***	**Unit**
Operable Floodgates	1	Each

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Foundation including piles
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

*Note: Operable gate quantities are on a per each basis. Per discussion with USACE-NAN, only the total quantity/number per each SBM alignment will be provided. Commensurate with the level of analysis for this feasibility study and nature of the project, material quantities per linear foot has not been obtained for this passive automatic flood barrier as it will be highly site dependent.

Tide Gates		
Item	**Quantity***	**Unit**
Tide Gates	1	Each

Additional Appurtenances: The items below are outside of the core construction quantities but should still be considered in the cost estimate.
Foundation including piles
Transitions between feature types
Utility relocation, drainage features, aesthetic features, real estate, right-of-way, easement, environmental mitigation

*Note: Tide gate quantities are on a per each basis. Per discussion with USACE-NAN, only the total quantity/number per each SBM alignment will be provided. Commensurate with the level of analysis for this feasibility study and nature of the project, material quantities per linear foot has not been obtained for this passive automatic flood barrier as it will be highly site dependent.

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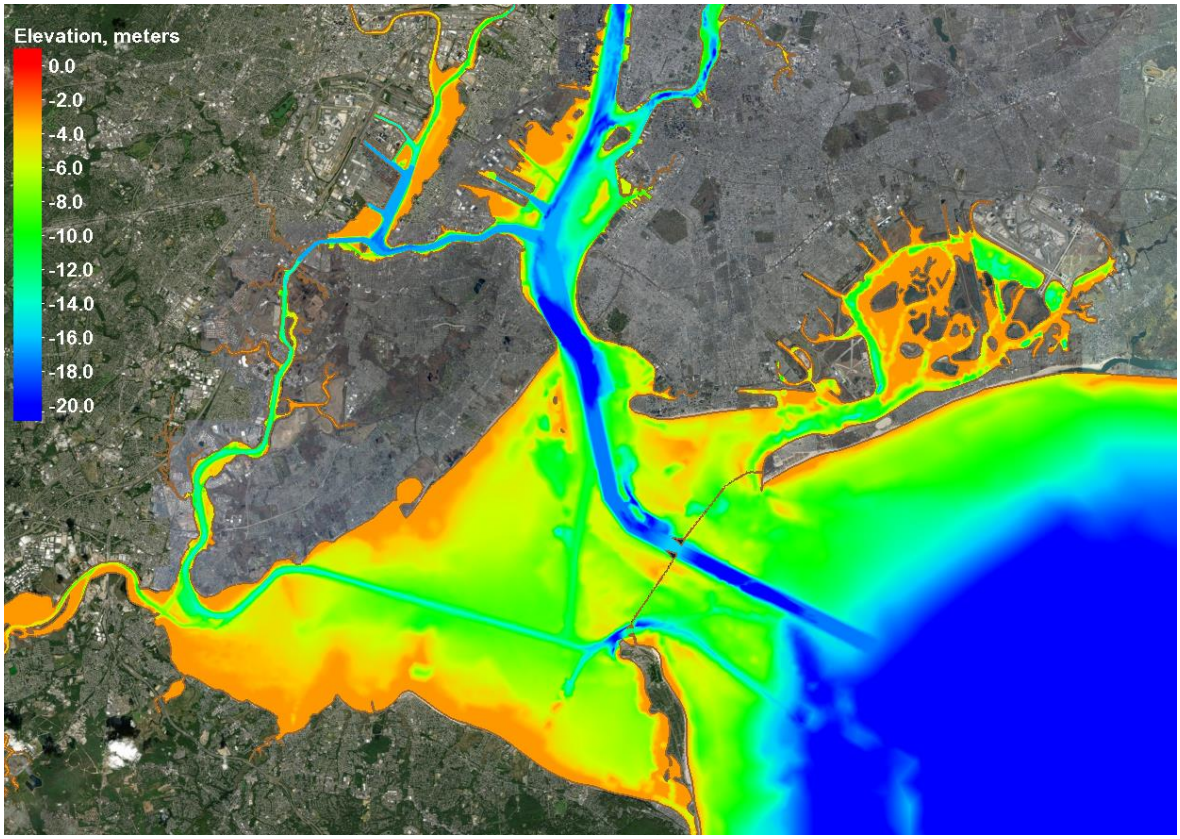
Analysis of Potential Storm Surge Barrier Impacts during Normal Tidal Conditions

Anthony G. Emiren and Tate O. McAlpin

September 2018



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Analysis of Potential Storm Surge Barrier Impacts during Normal Tidal Conditions

First M. Lastname1, First M. Lastname2, and First M. Lastname3

Coastal and Hydraulics Laboratory
US Army Engineer Research and Development Center
3909 Halls Ferry Road
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Abstract

This numerical modeling study investigates the impacts associated with proposed surge barriers in the New York/New Jersey Harbor during normal (non-storm) conditions with all the structures open to flow. Year-long Adaptive Hydraulics three dimensional numerical model simulations were performed for each barrier alternatives as well as a base model simulation without the structures in place. The results included from this study are point velocity and water surface elevation model comparisons, velocity percentiles, spatial figures of the 50th and 75th percentile velocities and salinity, and tidal prism percentiles for the areas impounded by the structures.

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Preface

This study was conducted for the US Army Corps of Engineers, New York District, under Project Number, "Title." The technical monitor was T.M. Name. This study was conducted by Mr. Tate McAlpin and Mr. Anthony Emiren.

The work was performed by the River and Estuarine Engineering Branch of the Flood and Storm Protection Division (CF), US Army Engineer Research and Development Center – Construction Engineering Research Laboratory (ERDC-CERL). At the time of publication, Keith Flowers was Chief, CEERD-DF-R; Cary Talbot was Chief, CEERD-HF. The Deputy Director of ERDC-CHL was COL Jeff Eckstein and the Director was Dr. Ty Wamsley.

COL Ivan Beckman was the Commander of ERDC, and Dr. David Pittman was the Director.

Unit Conversion Factors

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic inches	1.6387064 E-05	cubic meters
feet	0.3048	meters
inches	0.0254	meters
knots	0.5144444	meters per second
microns	1.0 E-06	meters
miles (nautical)	1,852	meters
miles (US statute)	1,609.347	meters
miles per hour	0.44704	meters per second
mils	0.0254	millimeters
ounces (US fluid)	2.957353 E-05	cubic meters
pints (US liquid)	4.73176 E-04	cubic meters
quarts (US liquid)	9.463529 E-04	cubic meters
square feet	0.09290304	square meters
square inches	6.4516 E-04	square meters
square miles	2.589998 E+06	square meters
square yards	0.8361274	square meters
yards	0.9144	meters

1 Introduction

The U.S. Army Corps of Engineers New York District (NAN) are considering proposed surge barrier configurations in the New York/New Jersey Harbor (NYNJH). This hydrodynamic and salinity intrusion numerical model study investigates the impacts associated with these proposed surge barriers in the NYNJH during normal conditions with all the structures open to flow. This study does not consider time periods and associated impacts during tropical or extratropical events.

Year-long Adaptive Hydraulics (AdH) three dimensional numerical model simulations were performed for each alternative as well as a base model simulation without the structures in place. The AdH mesh used and modified in these simulations is the "With Project" mesh from McAlpin et al, 2017. A mesh was made for each alternative by modifying the geometry of the original mesh to include the surge barriers. Water does not overtop the piers or barrier walls in these simulations. These areas are considered infinitely high. The inflow and tidal condition data are the 1995 conditions as detailed in McAlpin et al, 2017. There are no large storm events during this year, so the model is simulating normal or typical hydrologic conditions with only minor storm events making this an appropriate simulation time period.

It should be noted the McAlpin et al, 2017 study was to investigate the sediment transport behavior in the harbor and as such was not focused on resolving some peripheral areas. Examples of this is the salinity intrusion up the Hudson River. This was not a primary concern for the sedimentation study and was considered sufficient for the purposes of the previous study. This model should be reasonable for relative changes in salinity but will be deficient in predicting the absolute salinity values and salinity intrusion up the Hudson River. Since the NYNJH was the focus area of the previous study, some resolution and schematization was utilized away from the study area to reduce time and computational demands during the NYNJH sedimentation study. The representation of the New Jersey Meadowlands marsh areas and associated schematization are an example of an engineering decision made to reduce the computational burden that was appropriate for the sedimentation study but may not be for evaluating tidal exchange associated with a structure on the Hackensack River. So while the

model utilized in this study is appropriate for a feasibility level analysis, improvements may be required prior to completing full design level simulations.

Two sets of barrier alignments were simulated with each set consisting of four alternatives. The initial set of alignments were simulated with the model output being utilized along with ship sizes/frequency/transit path information to determine improved structure configurations in terms of gate sizes, locations, and elevations. The results from these simulations are presented in two sections of this report with the first section being the initial simulations and the second section being the "improved" structure configurations developed based on the initial model simulations and additional analysis of ship traffic data.

The emphasis of this study is to examine how each alternative will impact the system. Results will include point velocity and water surface elevation model comparisons, velocity percentiles, spatial figures of the 50th and 75th percentile velocities and salinity, and tidal prism percentiles for the areas impounded by the structures. The analysis is consistent between the two sets of alternatives.

2 Alternative Set 1 and Results

The first set of simulations evaluated four alternative alignments to investigate the hydrodynamic and salinity transport impacts due to proposed system modifications.

2.1 Alternatives

The four alternatives included alignments that ranged from large scale bay-wide protection (Sandy Hook to Rockaway structures in alignment 2) to smaller scale more localized protections. A profile view of the all structures for alternative set 1 are provided in Appendix A. It should be noted that structures present in multiple alternatives are equivalent in all of the alternatives. An example would be Jamaica Bay. This structure is consistent in Alternative 3a, 3b, and 4.

2.1.1 Alternative 2

Alternative 2 consisted of 2 structures with the alignment shown in Figure 1. The Sandy Hook to Rockaway structure is shown in Figure 2 and the Throgs Neck structure is shown in Figure 3.

Figure 1: Set 1 Alternative 2

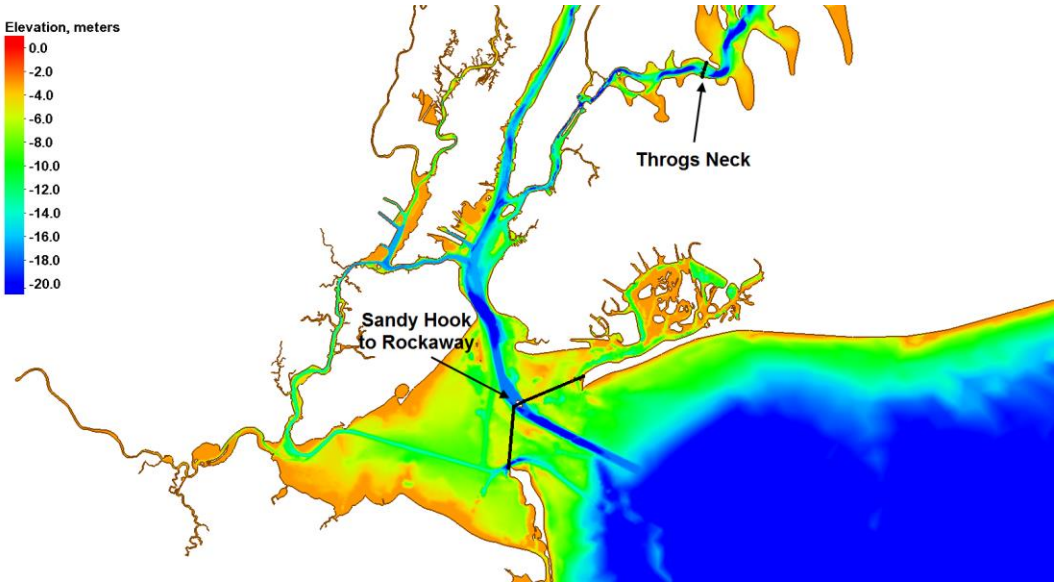


Figure 2: Set 1 Alternative 2 Sandy Hook to Rockaway Structure

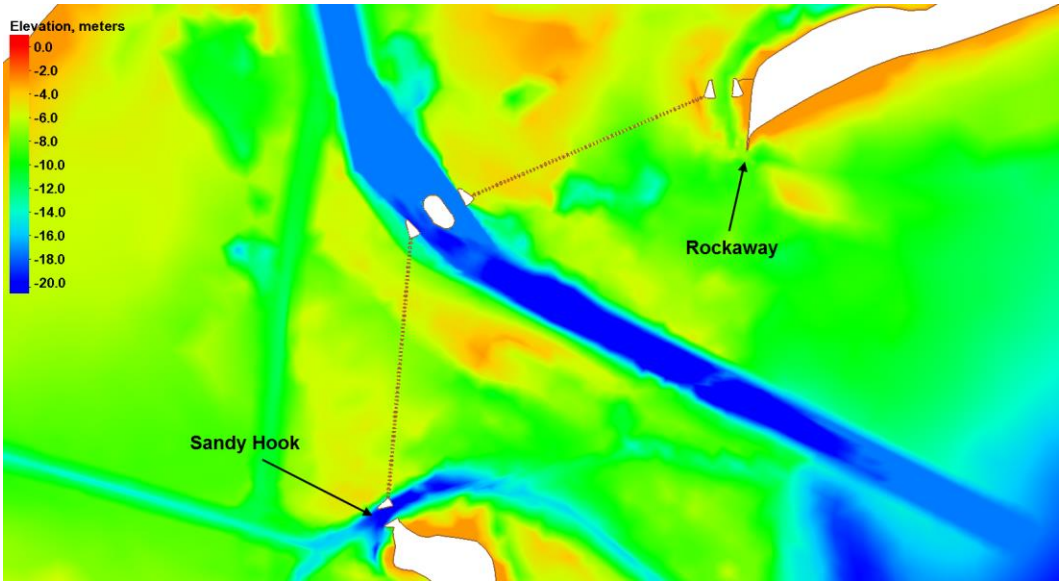
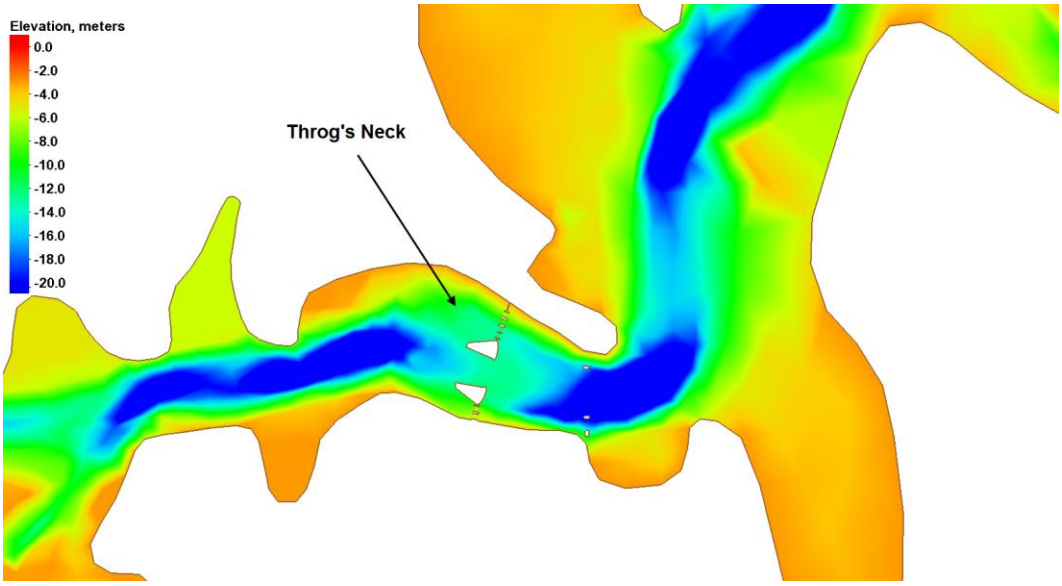


Figure 3: Set 1 Alternative 2 Throg's Neck Structure



2.1.2 Alternative 3a

Alternative 3a consisted of 4 structures with the alignment shown in Figure 4. The Throg's Neck Structure in Alternative 3a is aligned the same as Alternative 2's Throg's Neck Structure depicted in Figure 3.

Figure 4: Set 1 Alternative 3a

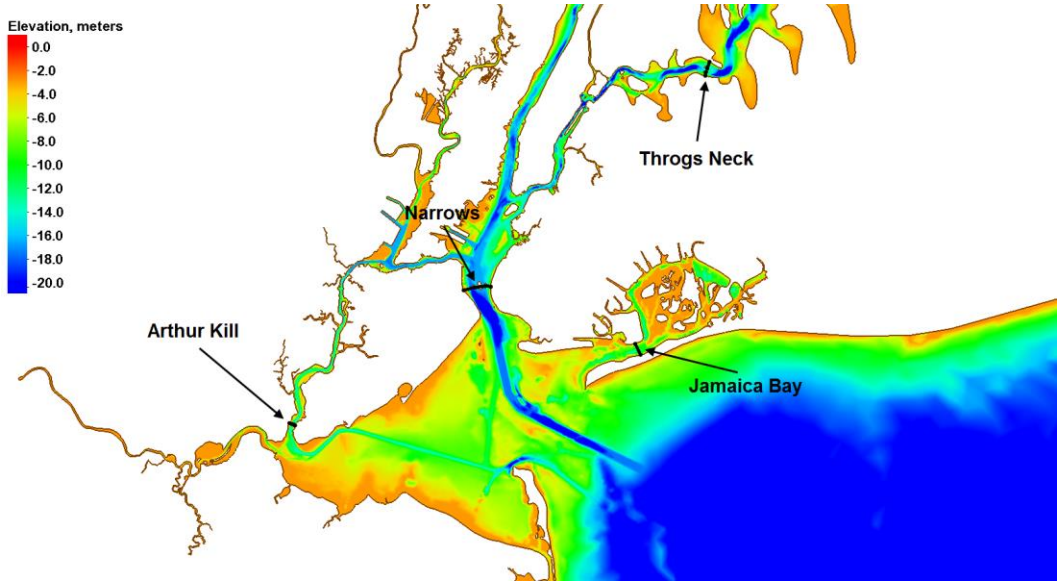


Figure 5: Set 1 Alternative 3a Arthur Kill Structure

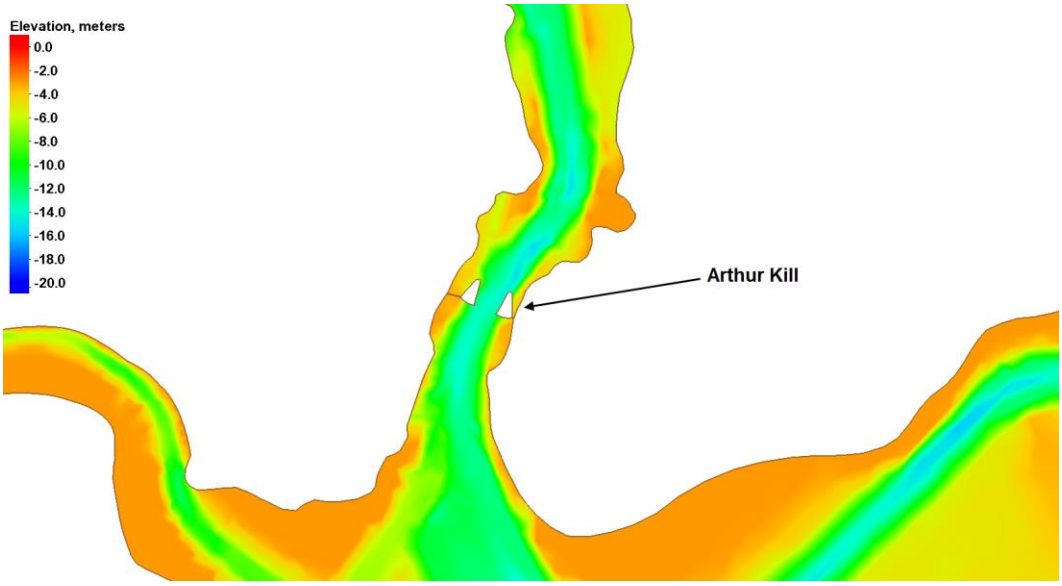


Figure 6: Set 1 Alternative 3a Narrows Structure

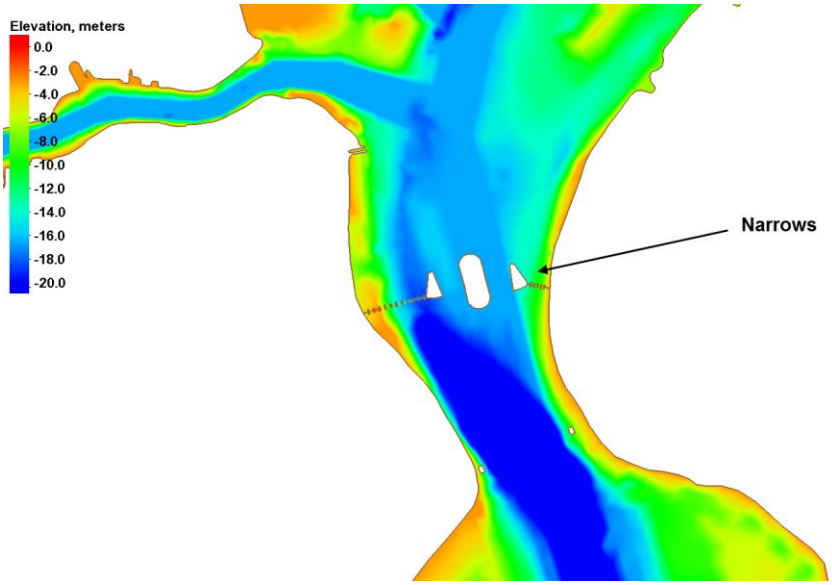
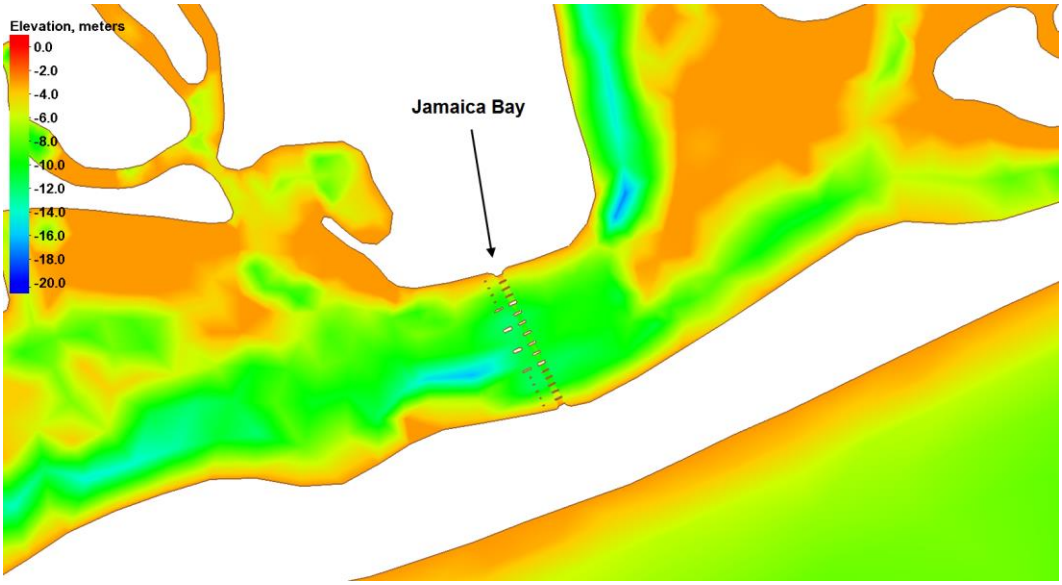


Figure 7: Set 1 Alternative 3a Jamaica Bay Structure



2.1.3 Alternative 3b

Alternative 3b consisted of 3 structures with the alignment shown in Figure 8. The Arthur Kill and Jamaica Bay Structures in Alternative 3b are aligned the same as Alternative 3a's Arthur Kill and Jamaica Bay structures which are depicted in Figure 5 and Figure 7 respectively.

Figure 8: Set 1 Alternative 3b

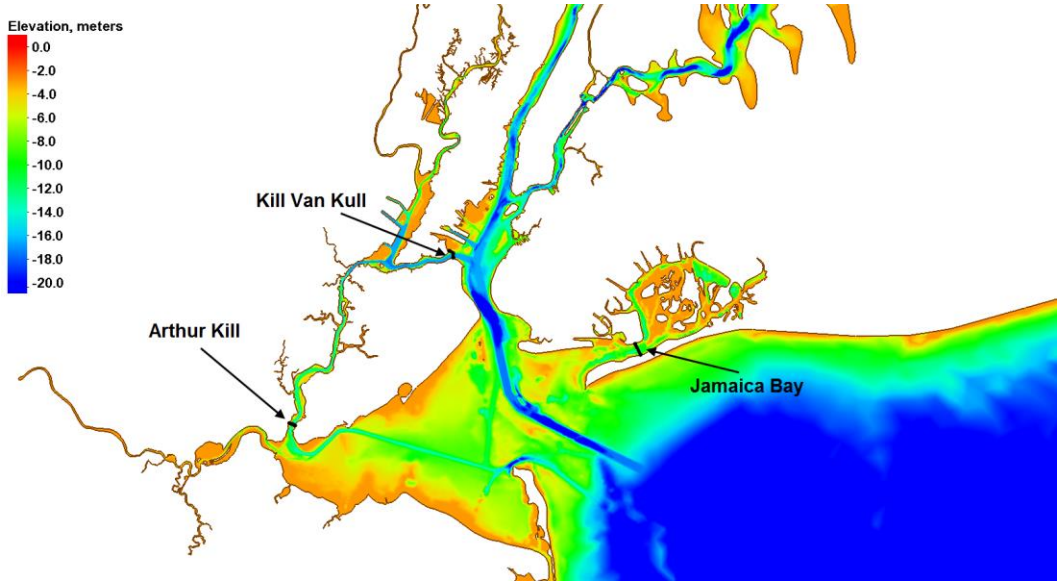
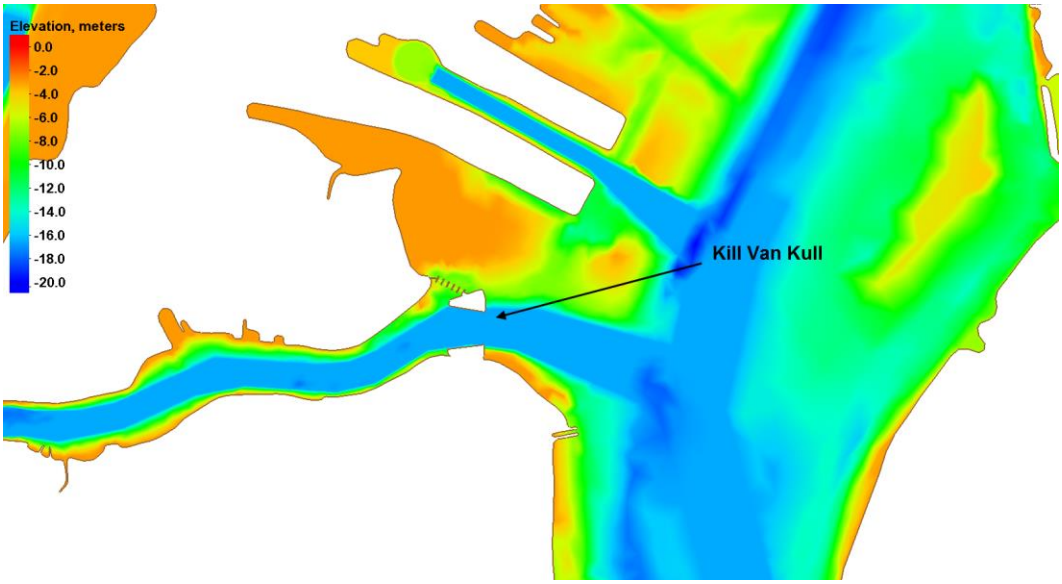


Figure 9: Set 1 Alternative 3b Kill van Kull Structure



2.1.4 Alternative 4

Alternative 4 consisted of 2 structures with the alignment shown in Figure 10. The Jamaica Bay Structure in Alternative 4 is aligned the same as Alternative 3a's Jamaica Bay Structure depicted in Figure 7.

Figure 10: Set 1 Alternative 4

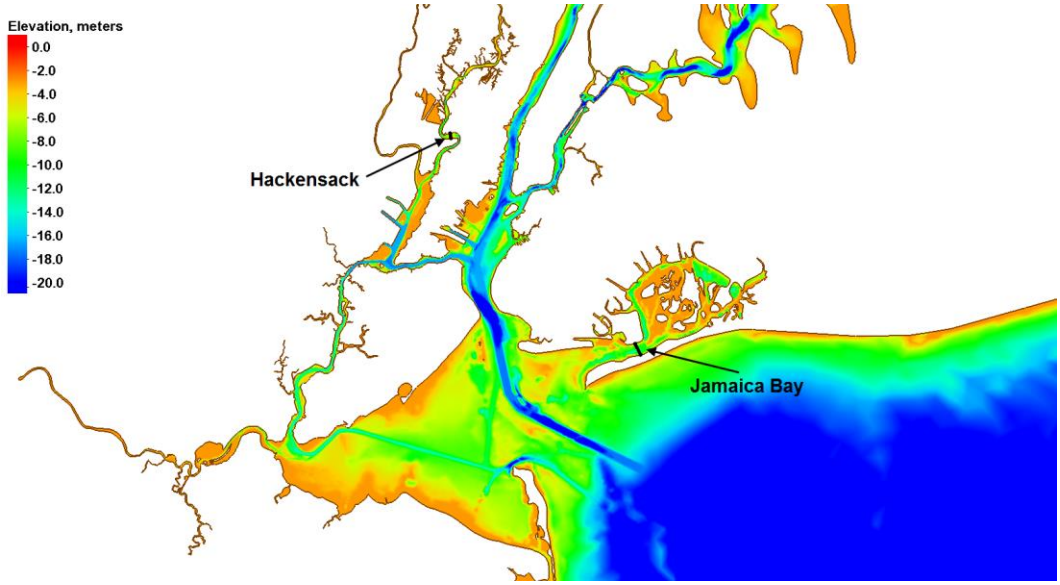
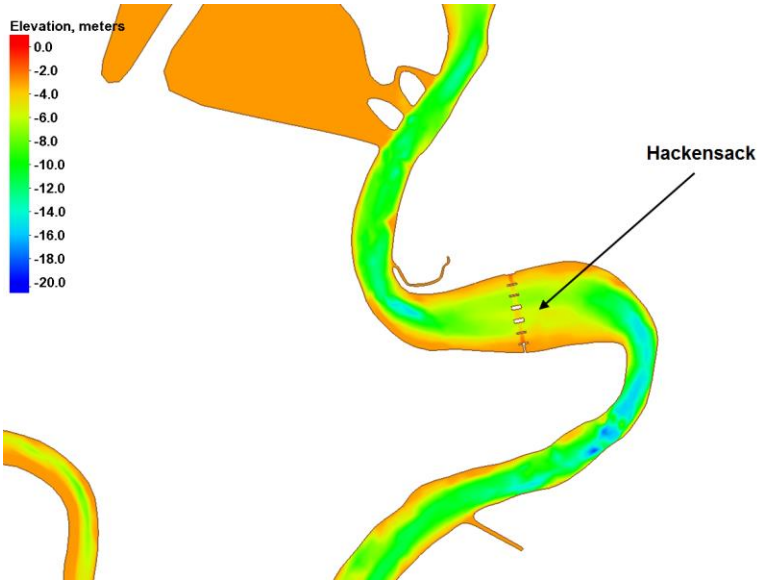


Figure 11: Set 1 Alternative 4 Hackensack Structure



2.2 Model Results and Analysis

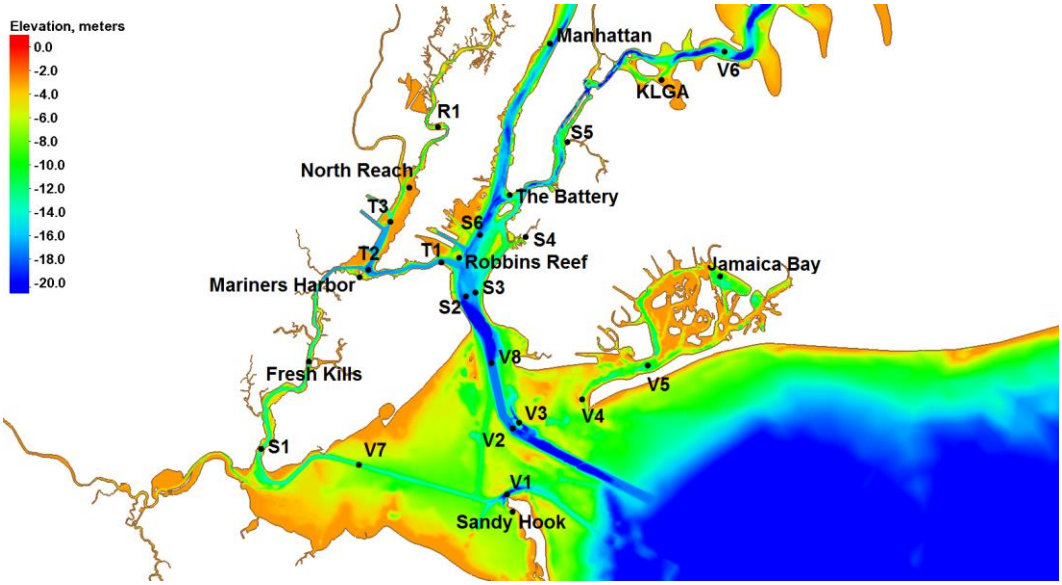
The four alternatives were compared to the without project simulation to determine the impact of the alternatives on water levels, velocities, and tidal prism. The analysis for the first set of alternatives are provided in the following sections and consist of point velocity and water surface elevation model comparisons, velocity percentiles, spatial figures of the 50th and 75th

percentile velocities and salinity, and tidal prism percentiles for the areas affected by the structures.

2.2.1 Point Comparisons

Point comparisons were extracted from the model results at the locations shown in Figure 12. These comparisons are shown in Figure 13 to Figure 93. Even though the AdH simulations are a yearlong, all of the time series plots are zoomed in to the first two weeks of the month of July so the impact of each alternative is more easily visible.

Figure 12: Set 1 Analysis Points



2.2.1.1 Water Surface Elevation Point Comparisons

In Figure 13 through Figure 39, the water surface elevation time series values are plotted for each alternative and the without project at the locations shown in Figure 12.

Figure 13: Set 1 Point V1 Water Surface Elevation

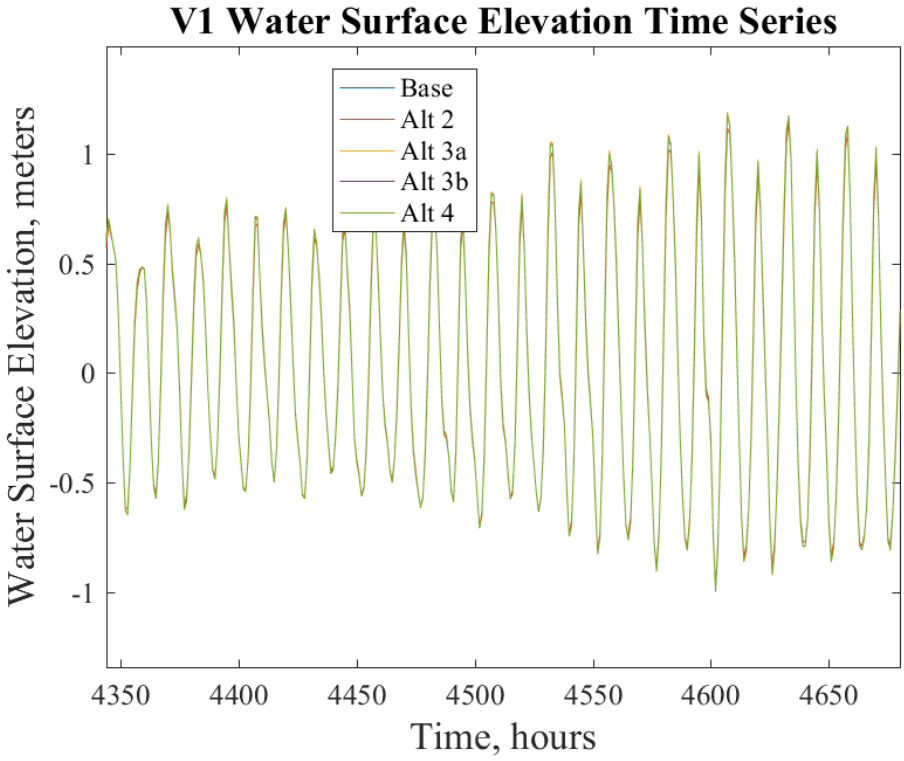


Figure 14: Set 1 Point V2 Water Surface Elevation

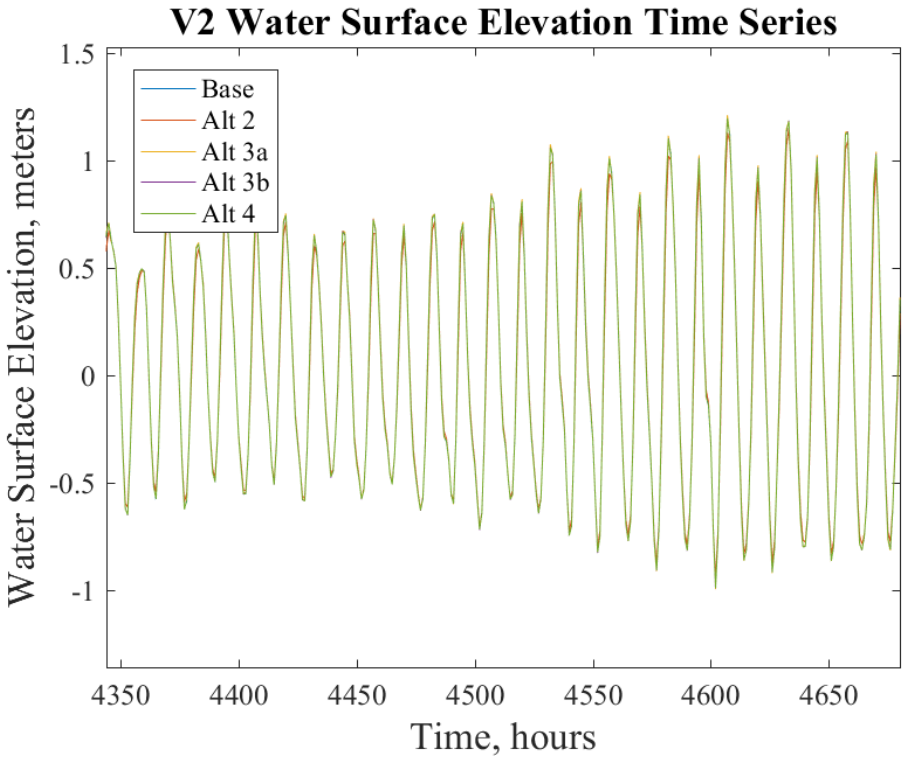


Figure 15: Set 1 Point V3 Water Surface Elevation

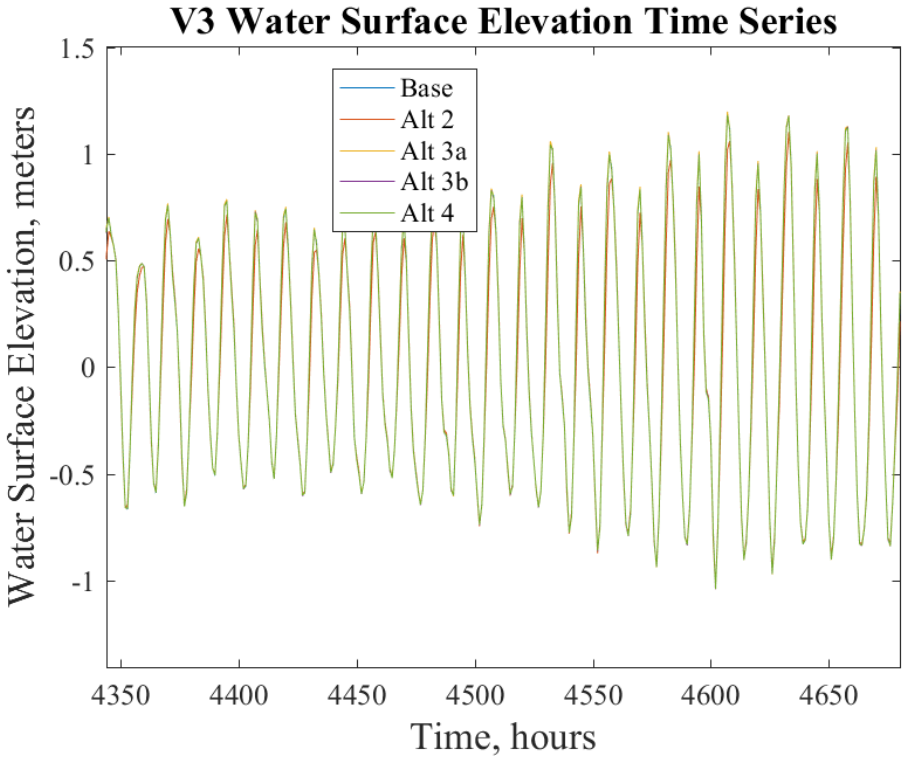


Figure 16: Set 1 Point V4 Water Surface Elevation

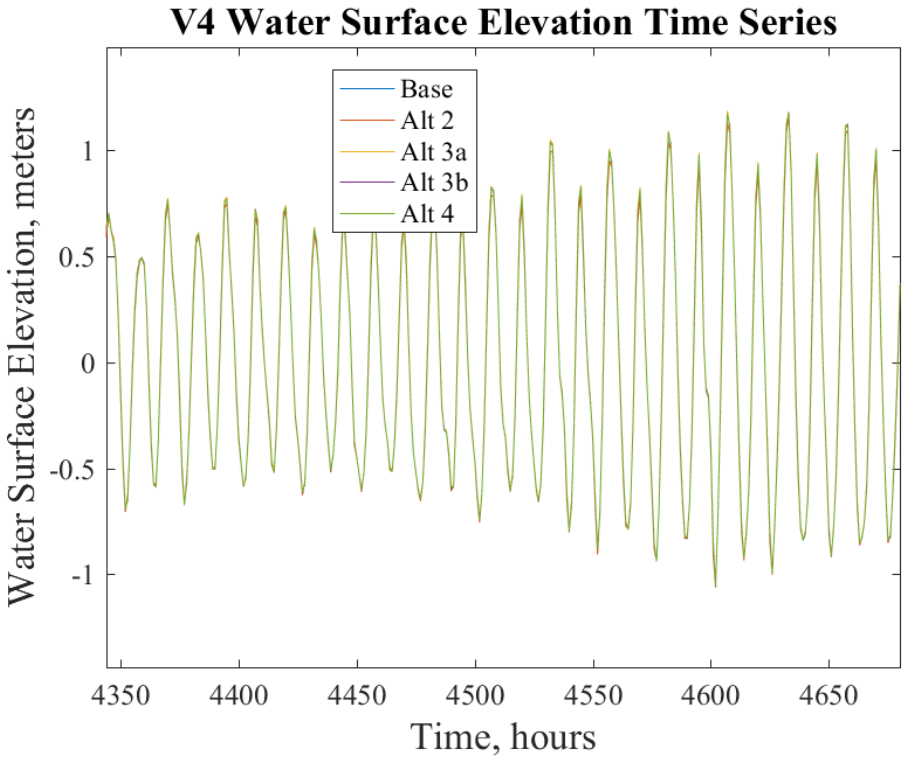


Figure 17: Set 1 Point V5 Water Surface Elevation

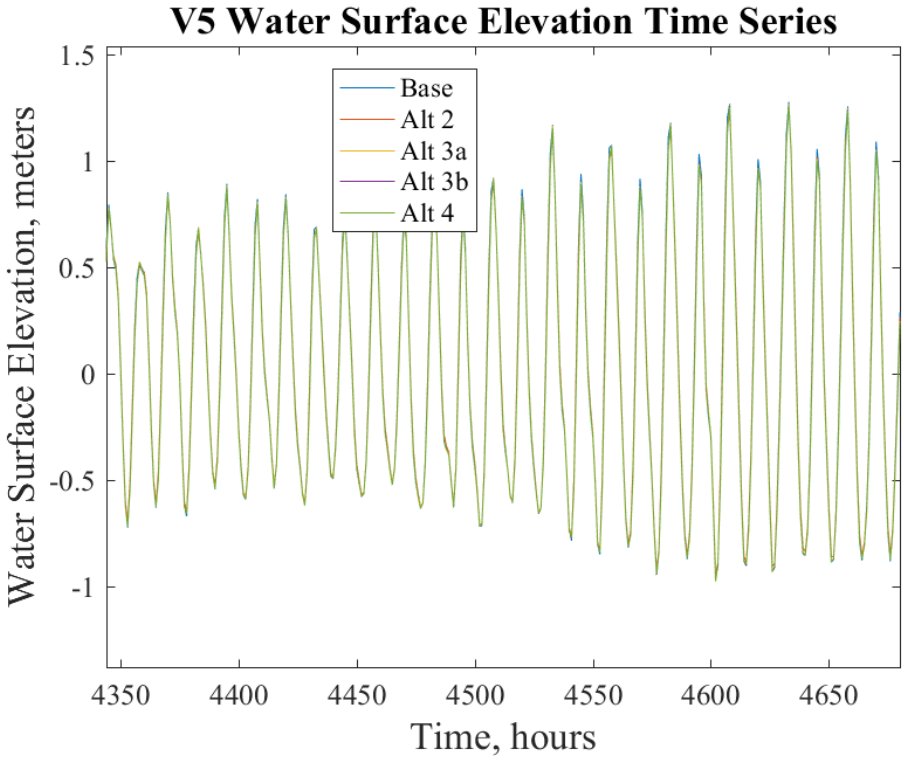


Figure 18: Set 1 Point V6 Water Surface Elevation

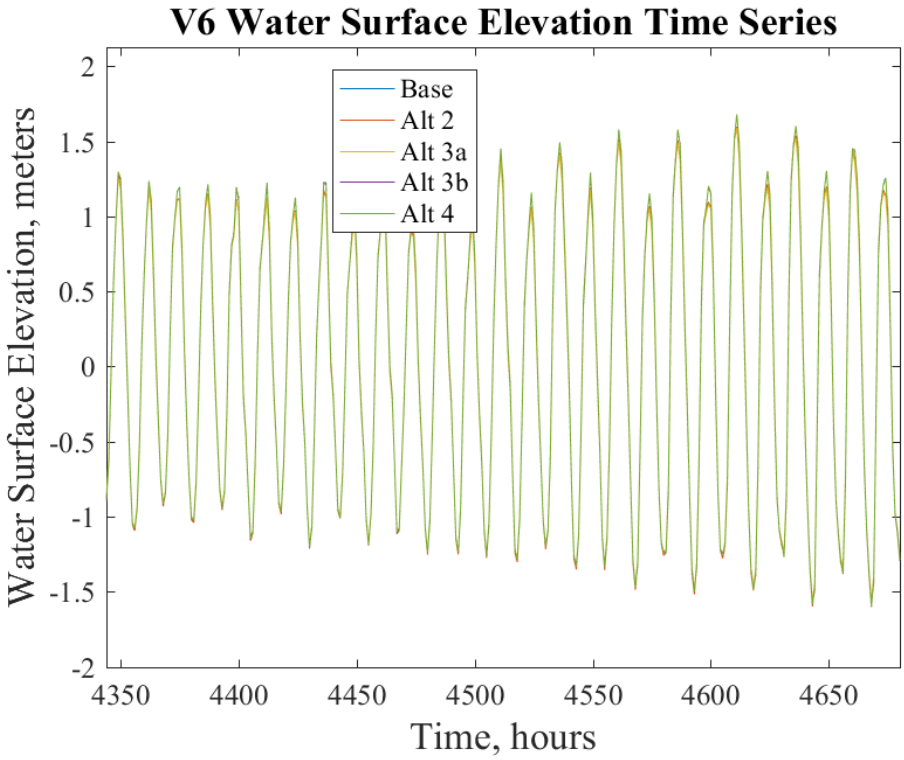


Figure 19: Set 1 Point V7 Water Surface Elevation

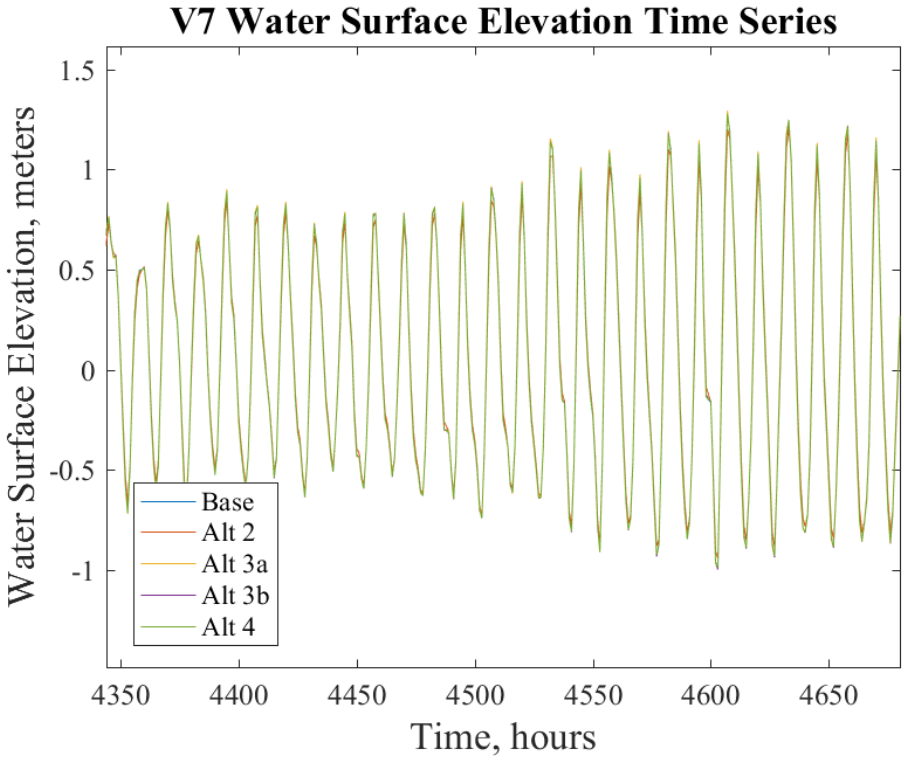


Figure 20: Set 1 Point V8 Water Surface Elevation

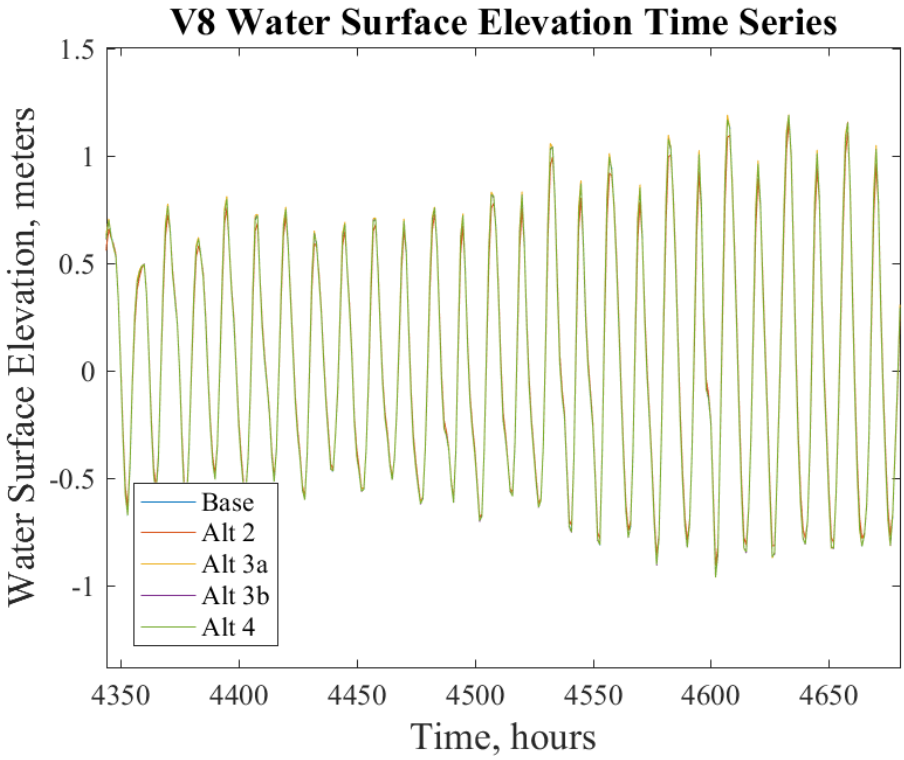


Figure 21: Set 1 Point S1 Water Surface Elevation

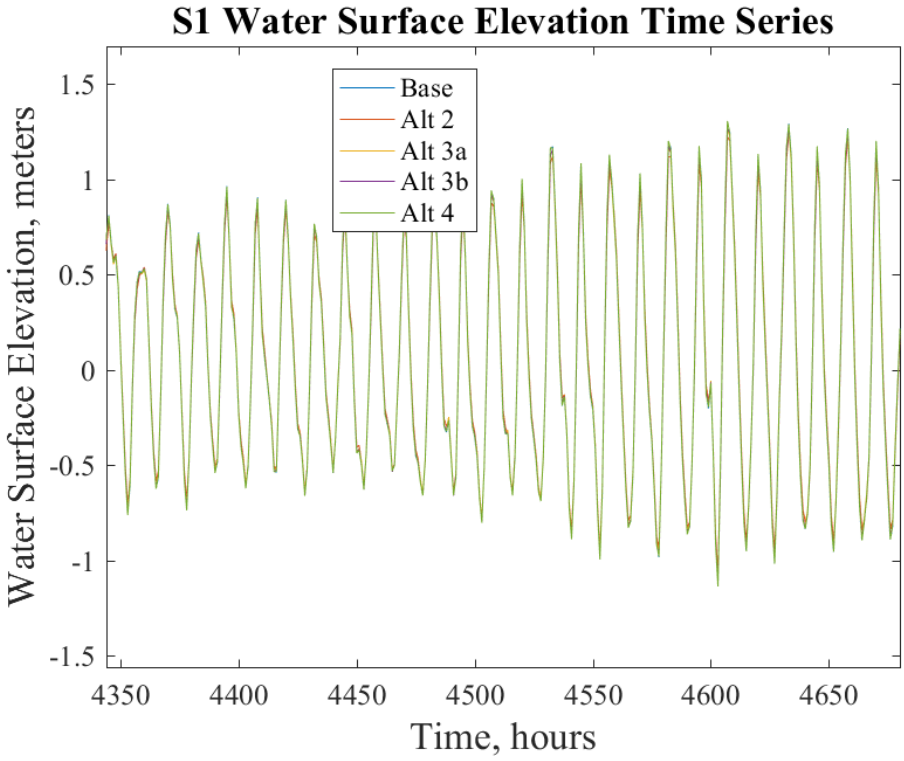


Figure 22: Set 1 Point S2 Water Surface Elevation

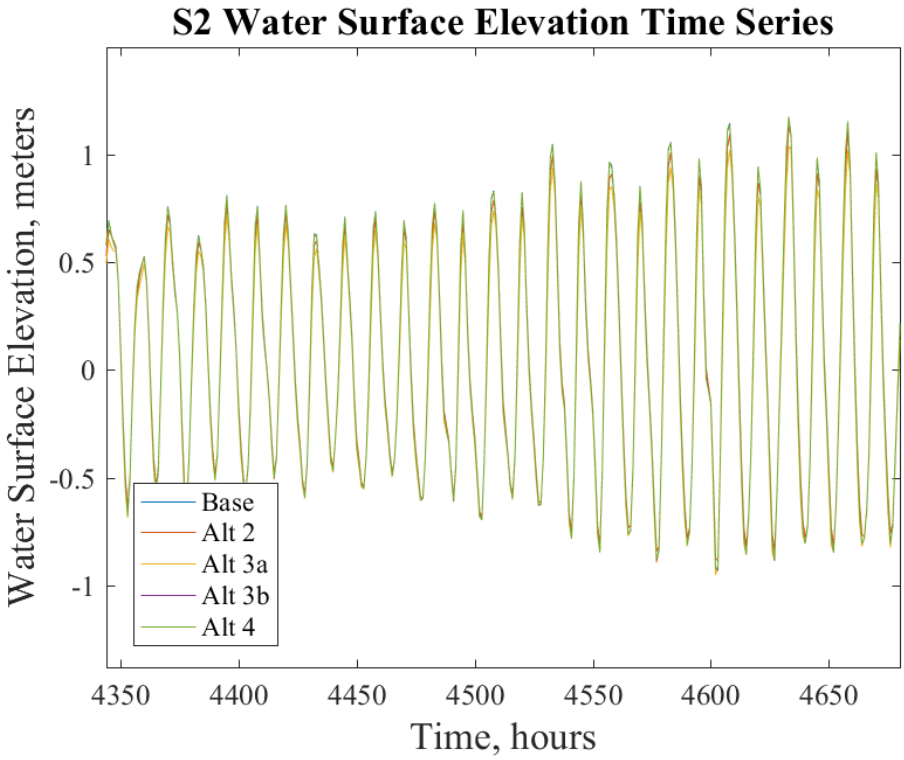


Figure 23: Set 1 Point S3 Water Surface Elevation

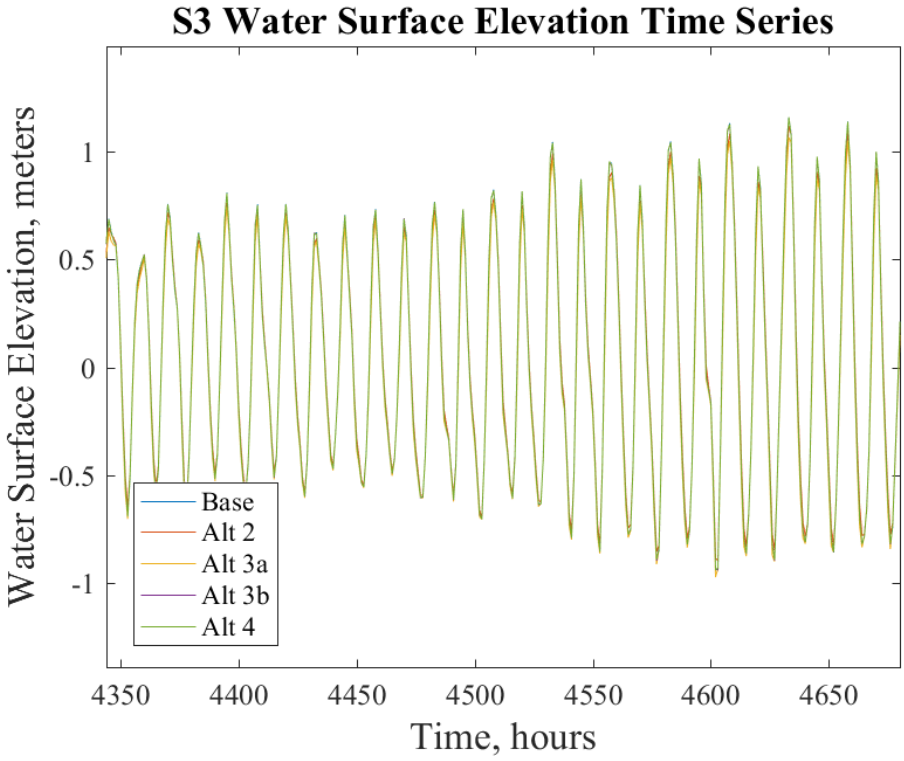


Figure 24: Set 1 Point S4 Water Surface Elevation

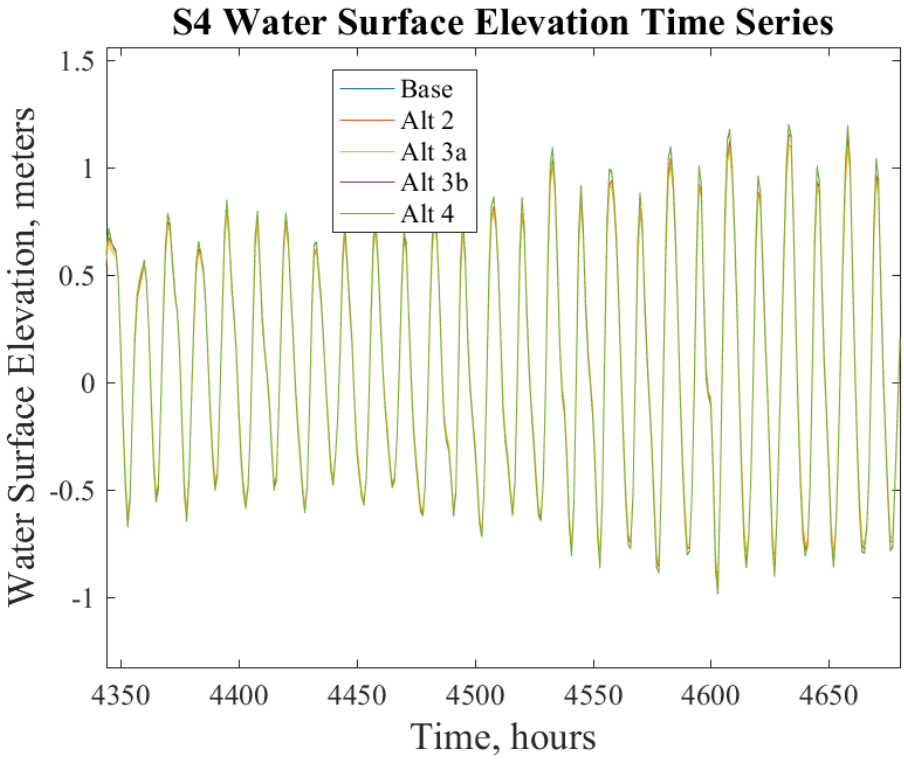


Figure 25: Set 1 Point S5 Water Surface Elevation

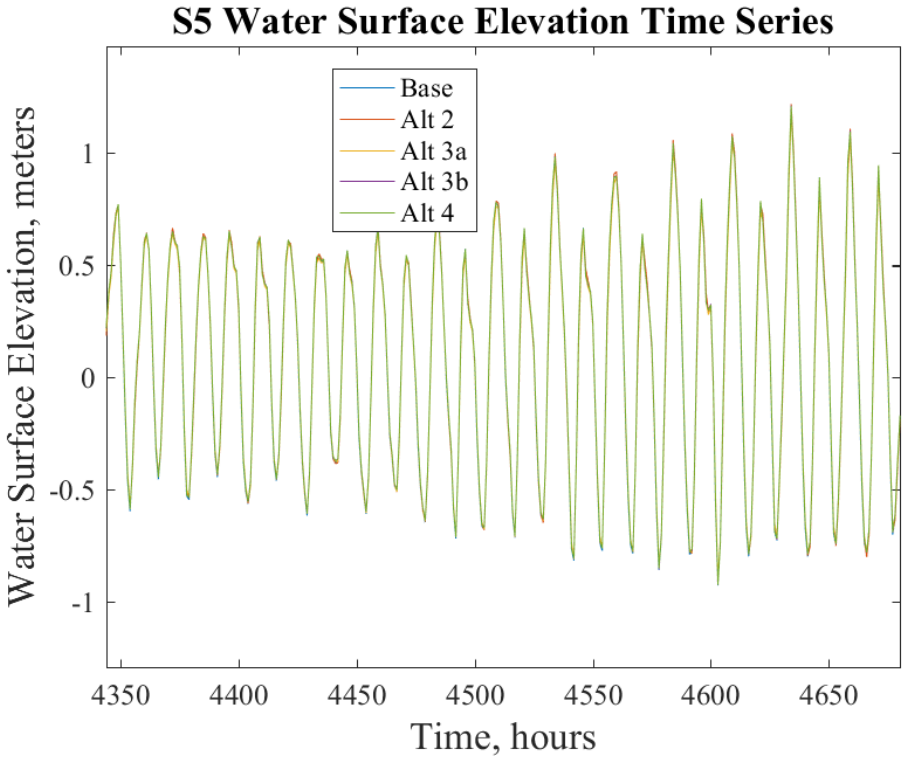


Figure 26: Set 1 Point S6 Water Surface Elevation

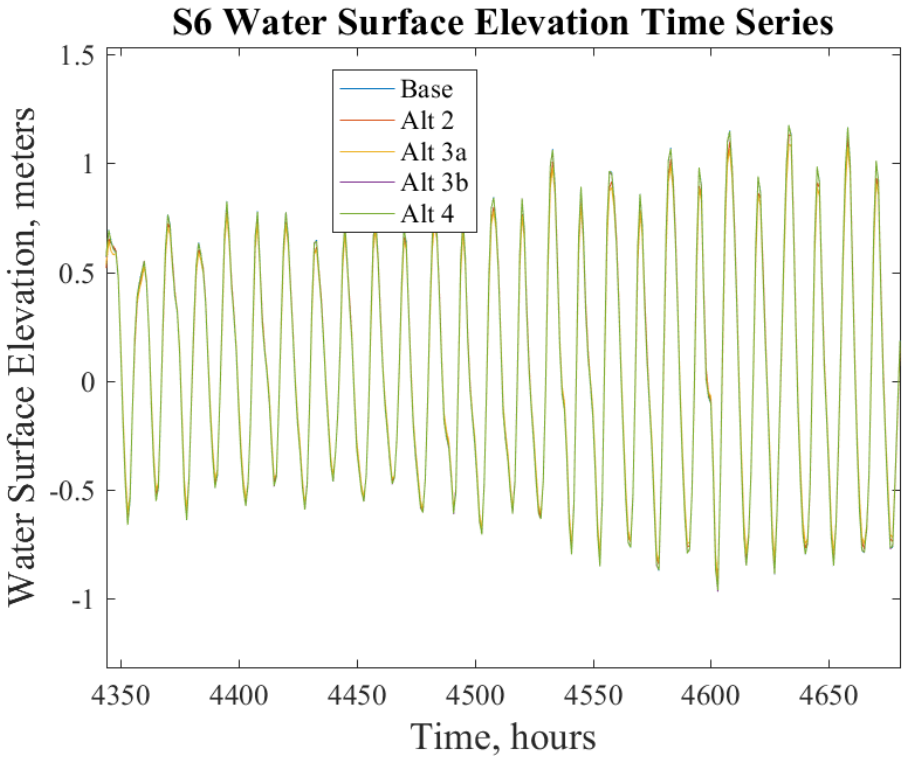


Figure 27: Set 1 Point T1 Water Surface Elevation

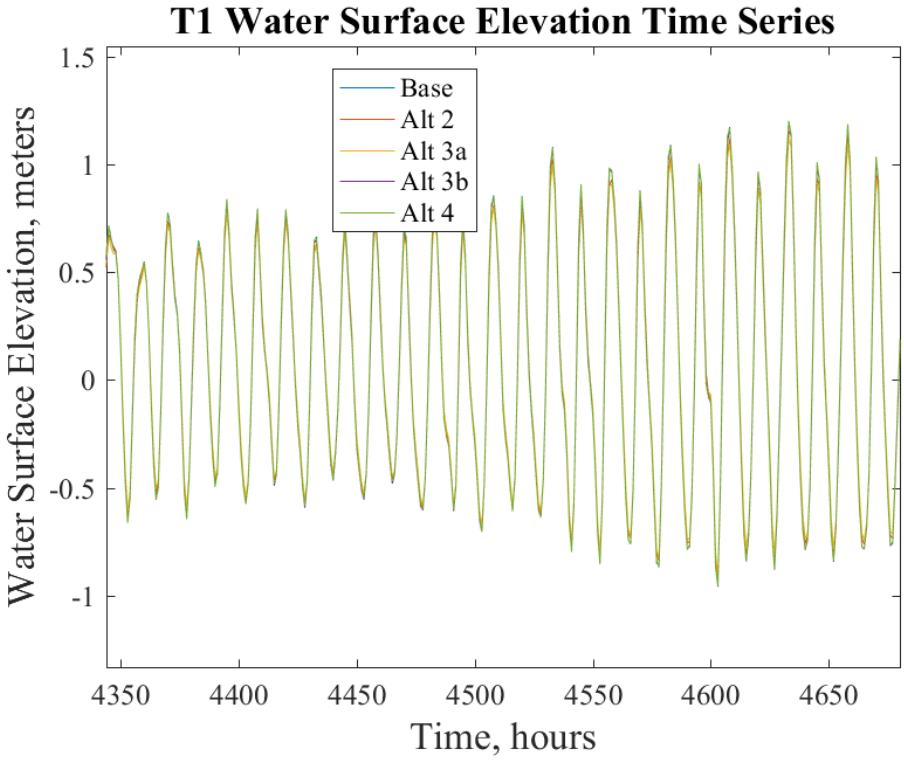


Figure 28: Set 1 Point T2 Water Surface Elevation

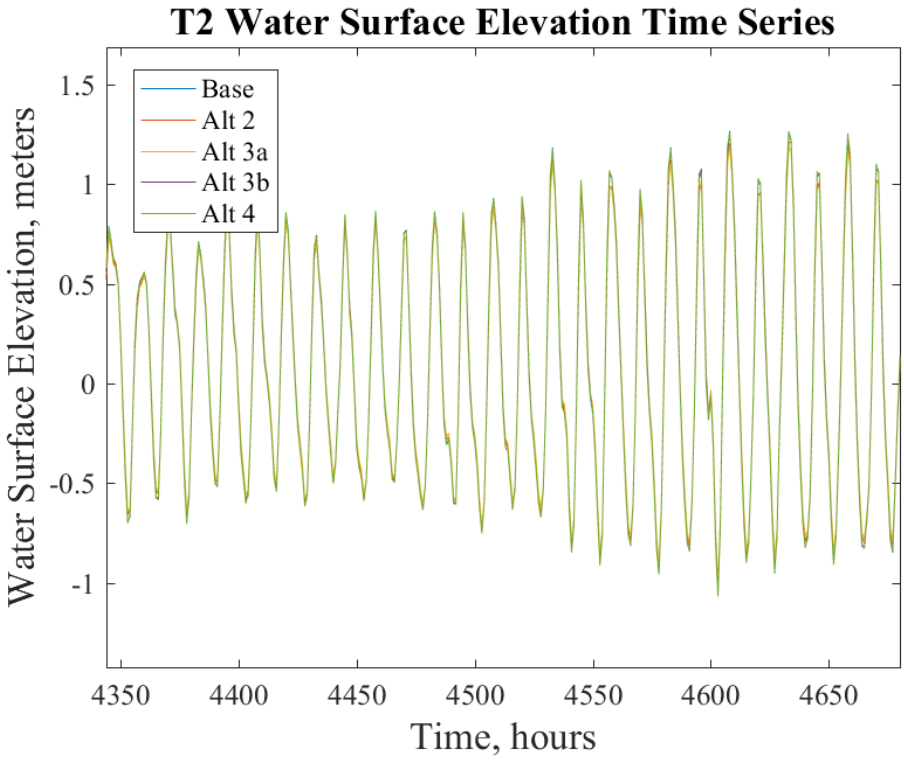


Figure 29: Set 1 Point T3 Water Surface Elevation

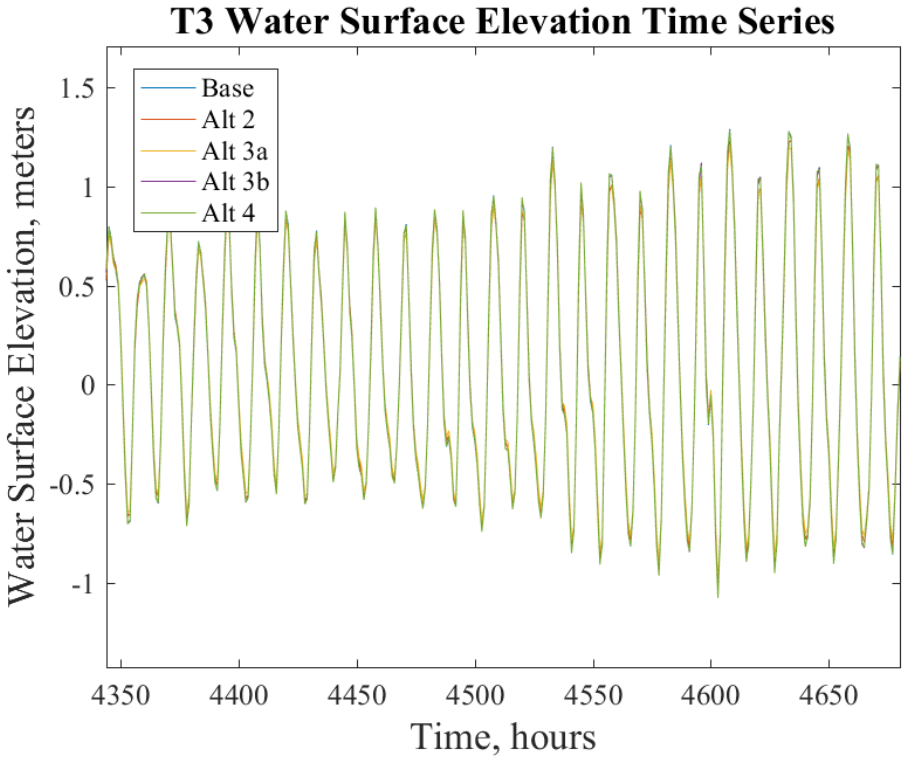


Figure 30: Set 1 Point R1 Water Surface Elevation

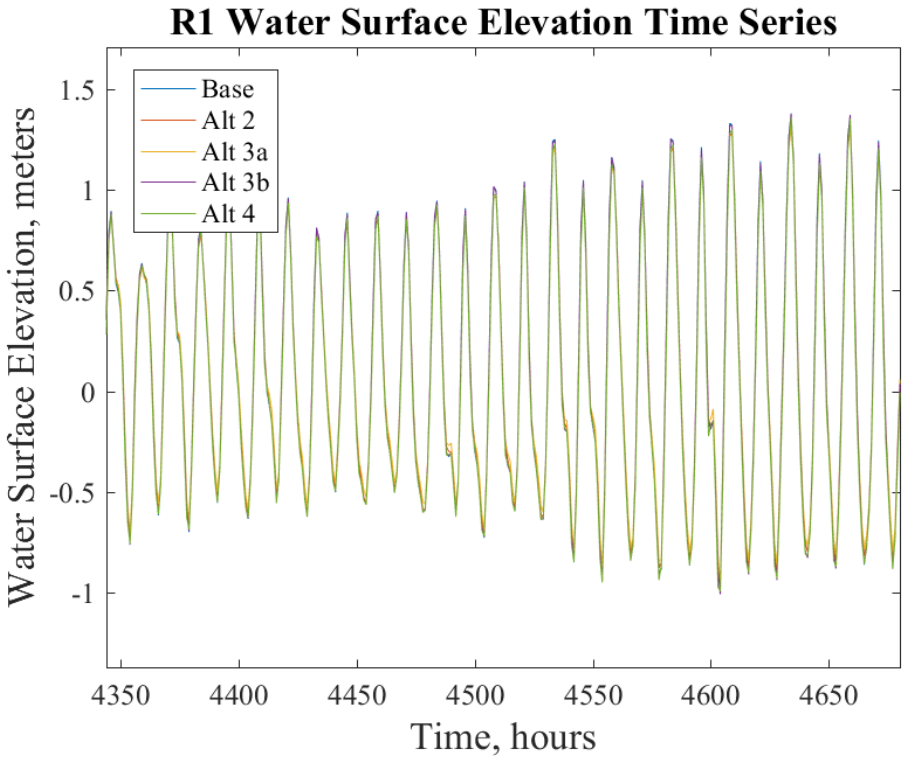


Figure 31: Set 1 Point Fresh Kills Water Surface Elevation

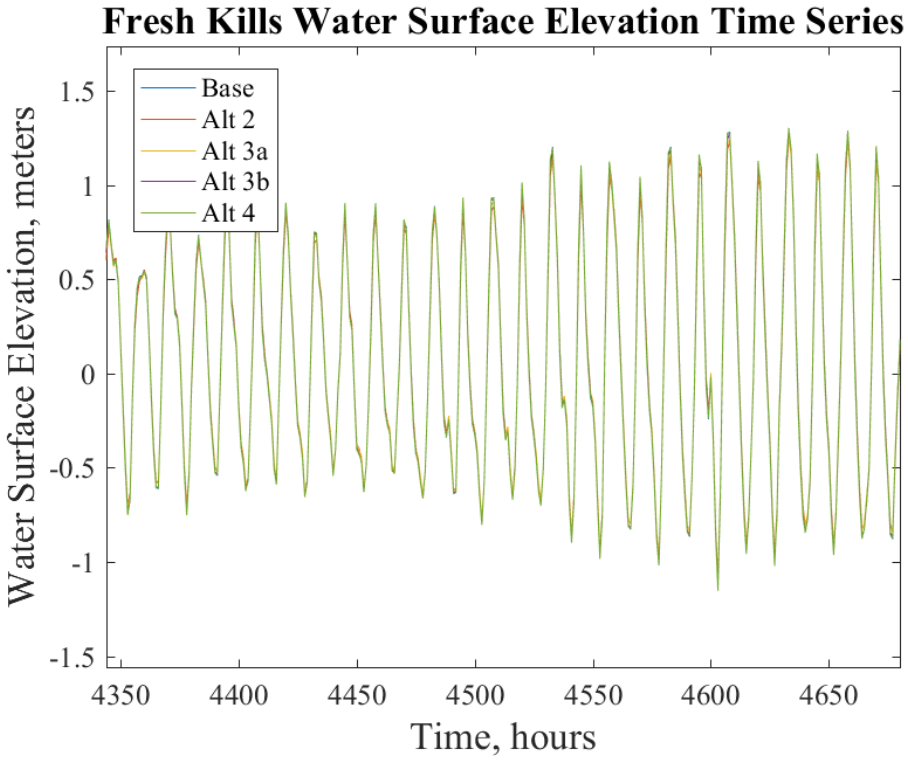


Figure 32: Set 1 Point Sandy Hook Water Surface Elevation

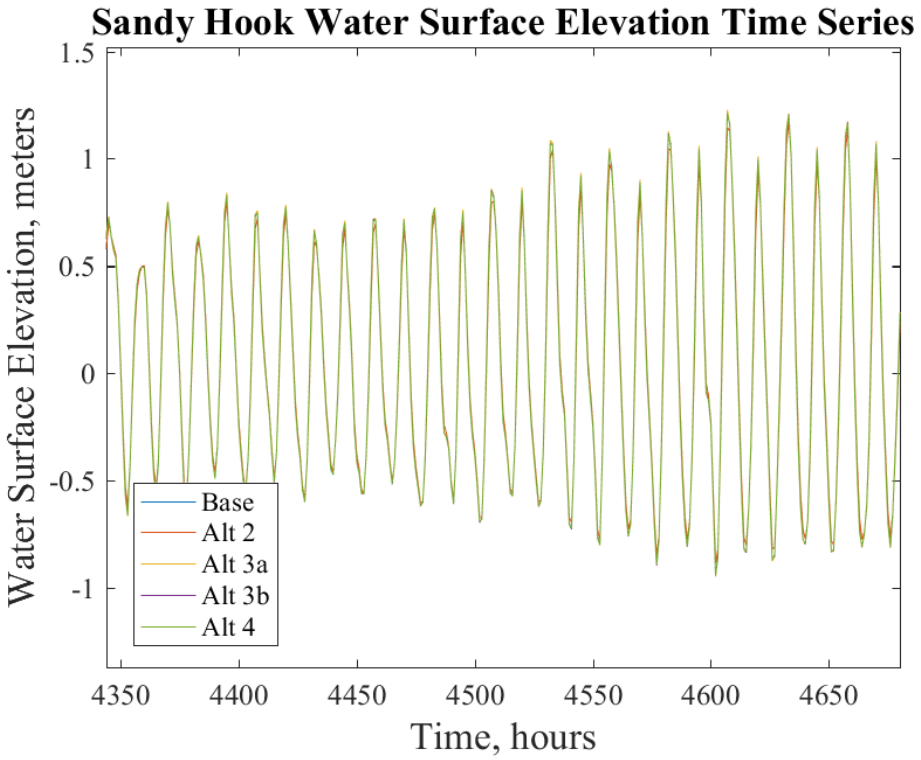


Figure 33: Set 1 Point Jamaica Bay Water Surface Elevation

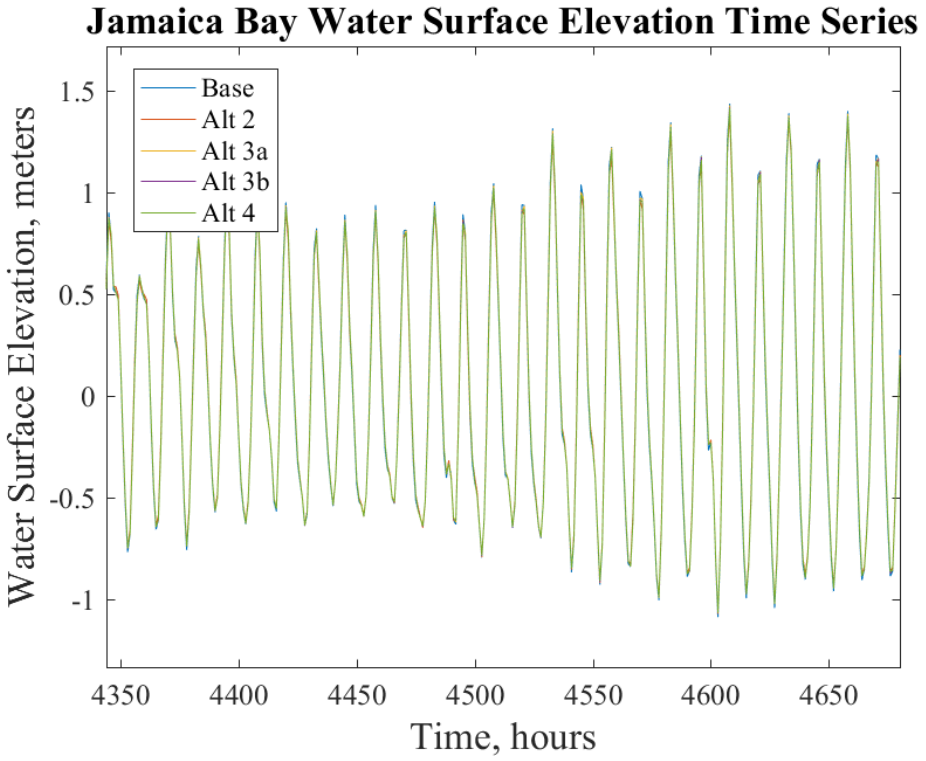


Figure 34: Set 1 Point Robbins Reef Water Surface Elevation

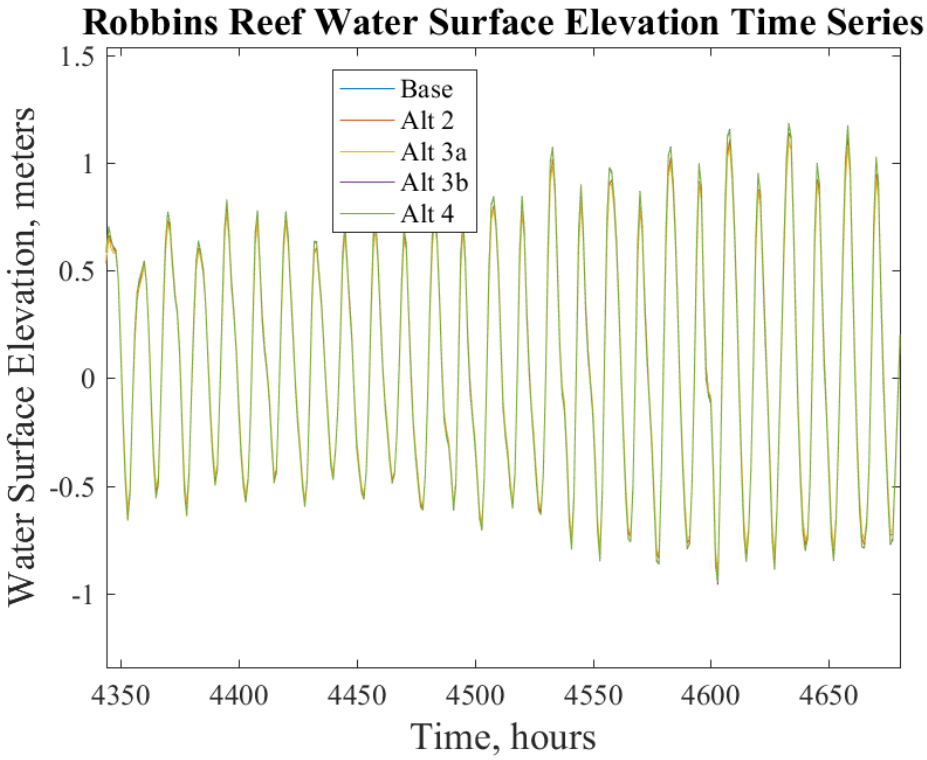


Figure 35: Set 1 Point Battery Water Surface Elevation

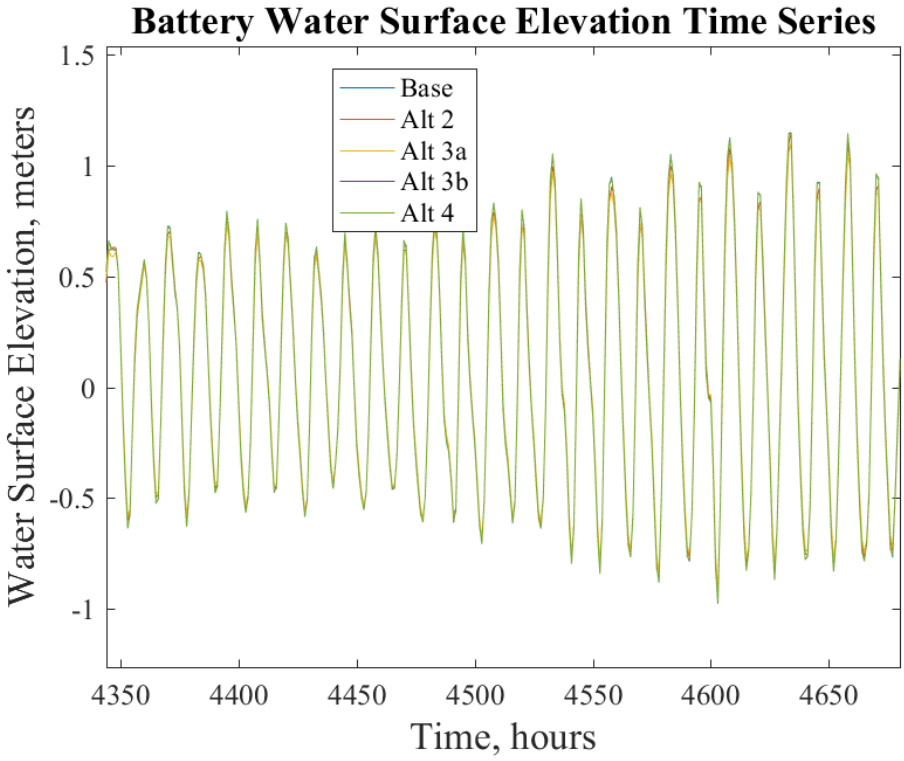


Figure 36: Set 1 Point KLGA Water Surface Elevation

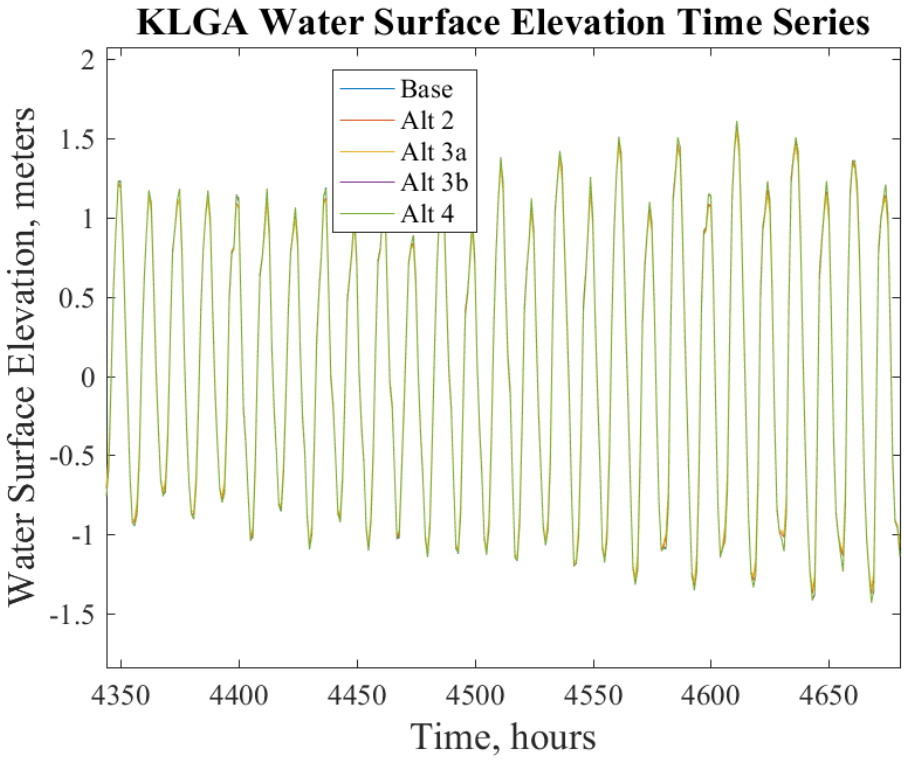


Figure 37: Set 1 Point Mariners Water Surface Elevation

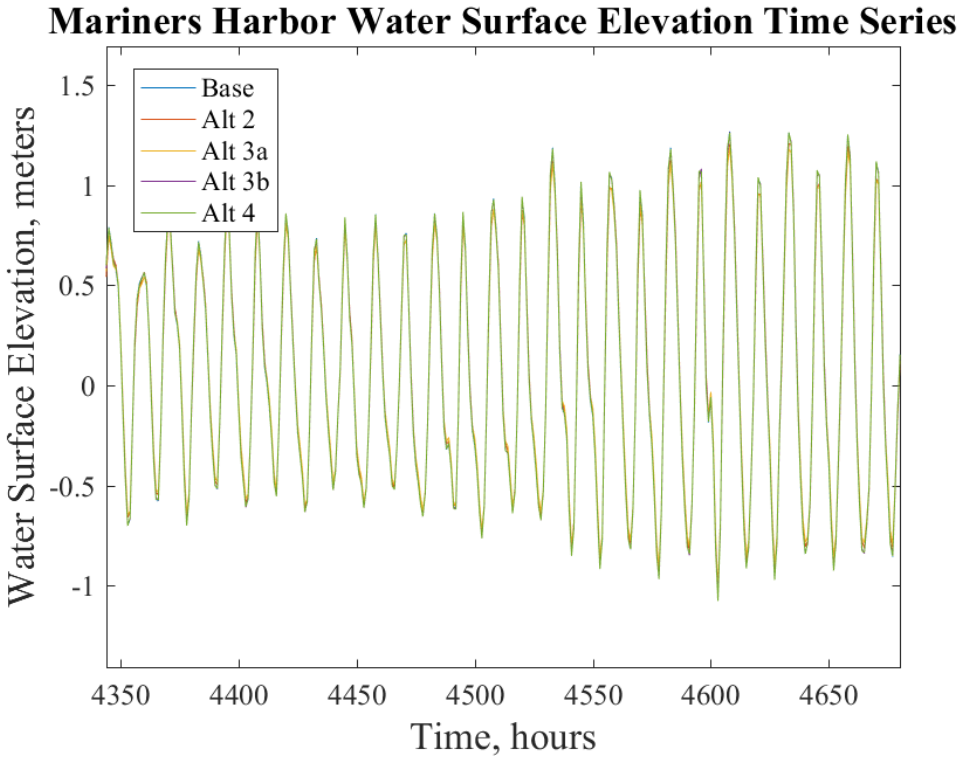


Figure 38: Set 1 Point North Reach Water Surface Elevation

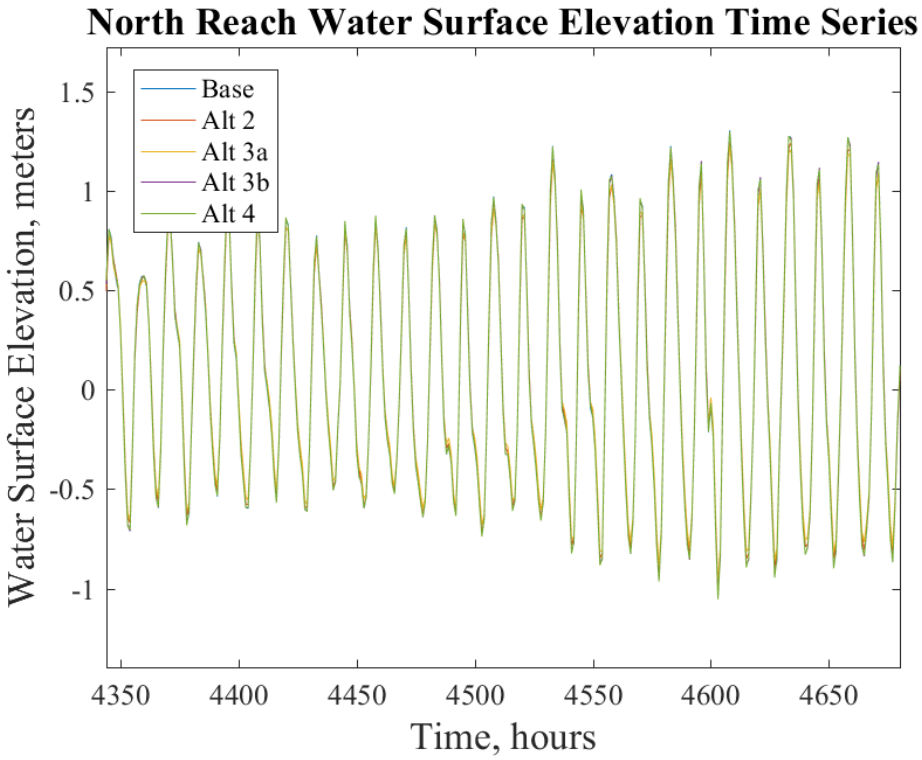
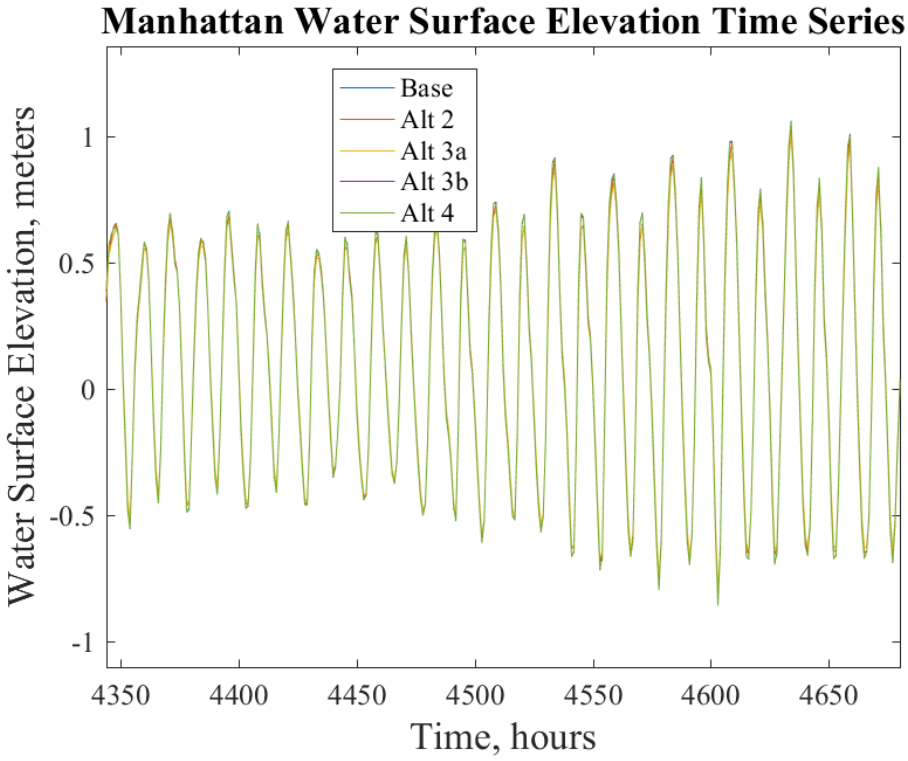


Figure 39: Set 1 Point Manhattan Water Surface Elevation



2.2.1.2 Velocity Point Comparisons

In Figure 40 through Figure 66, the depth average velocity magnitude time series values are plotted for each alternative and the without project for the locations shown in Figure 12.

Figure 40: Set 1 Point V1 Depth Averaged Velocity Magnitudes

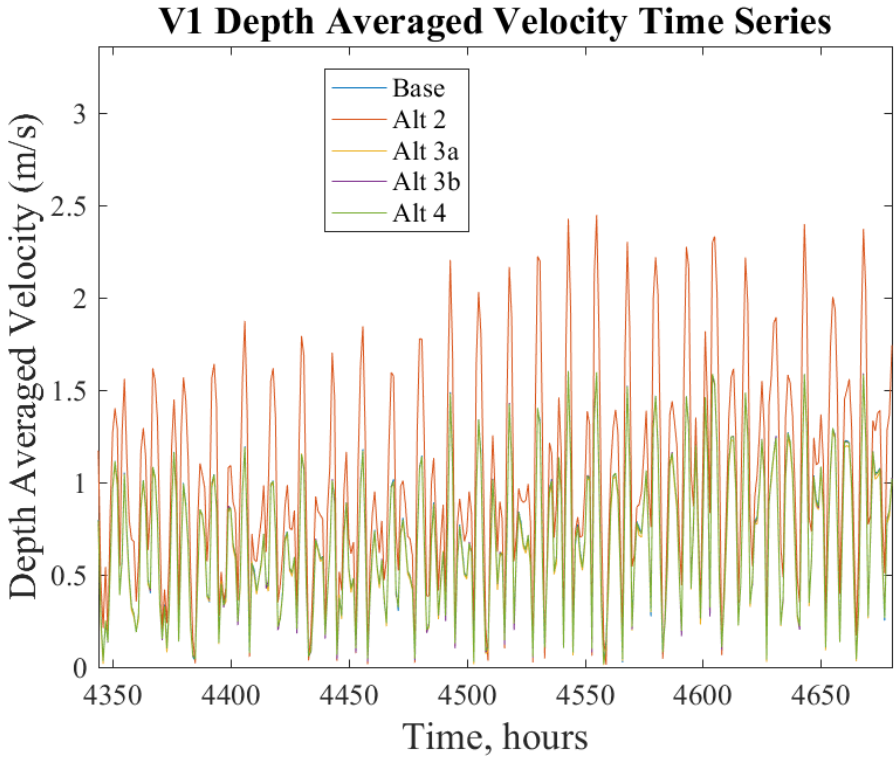


Figure 41: Set 1 Point V2 Depth Averaged Velocity Magnitudes

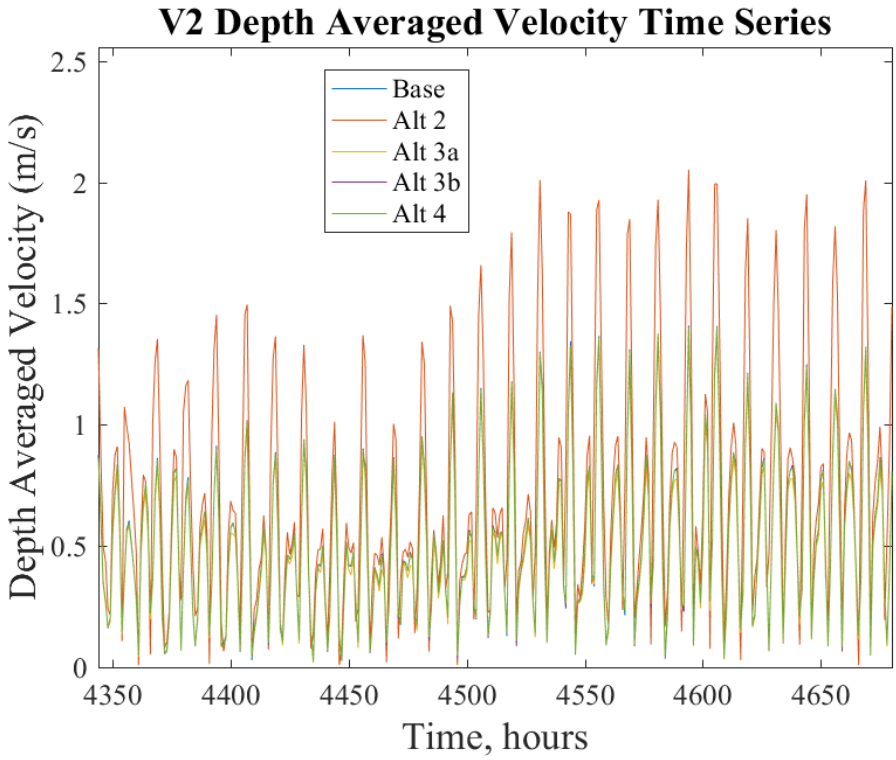


Figure 42: Set 1 Point V3 Depth Averaged Velocity Magnitudes

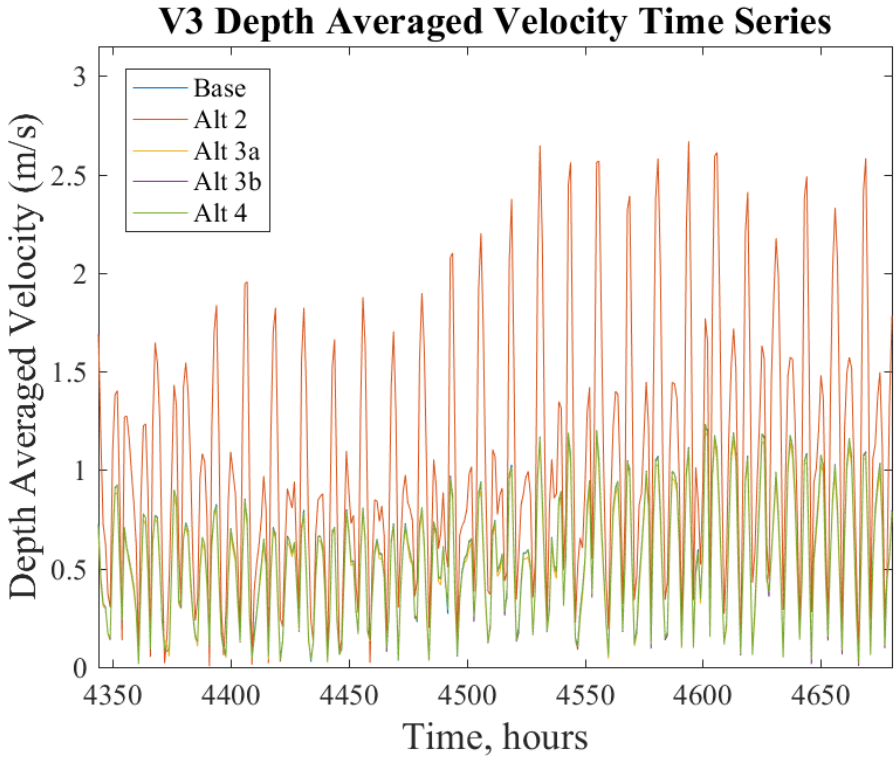


Figure 43: Set 1 Point V4 Depth Averaged Velocity Magnitudes

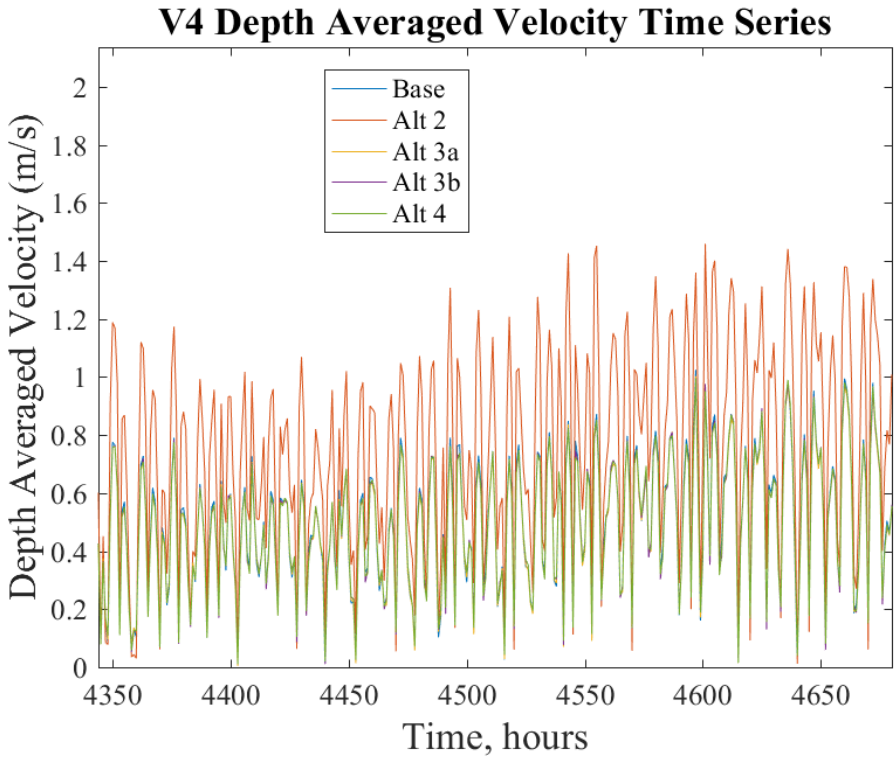


Figure 44: Set 1 Point V5 Depth Averaged Velocity Magnitudes

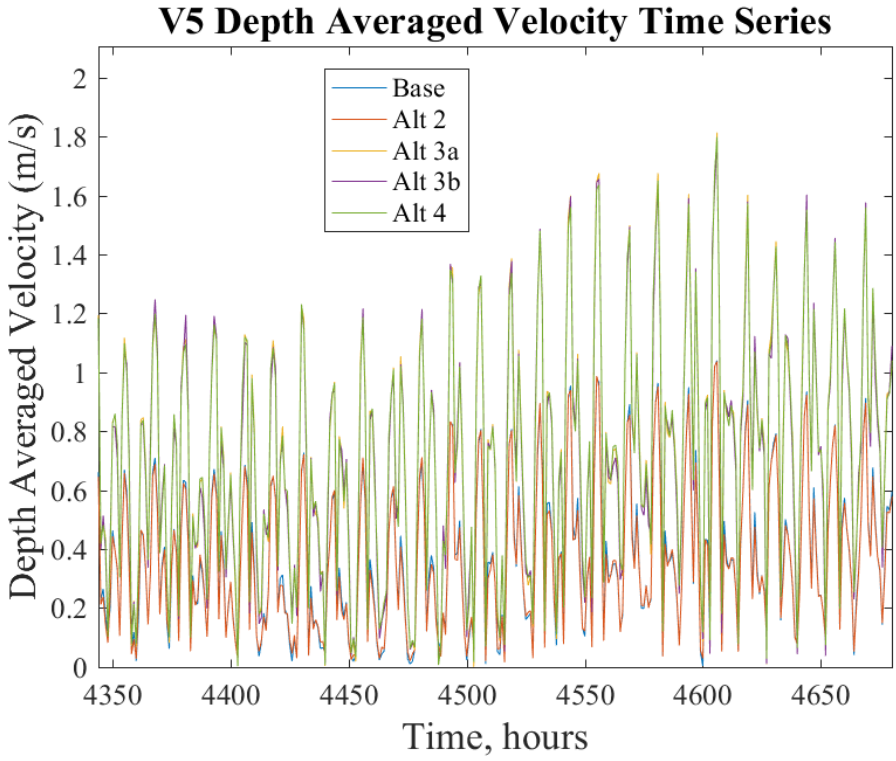


Figure 45: Set 1 Point V6 Depth Averaged Velocity Magnitudes

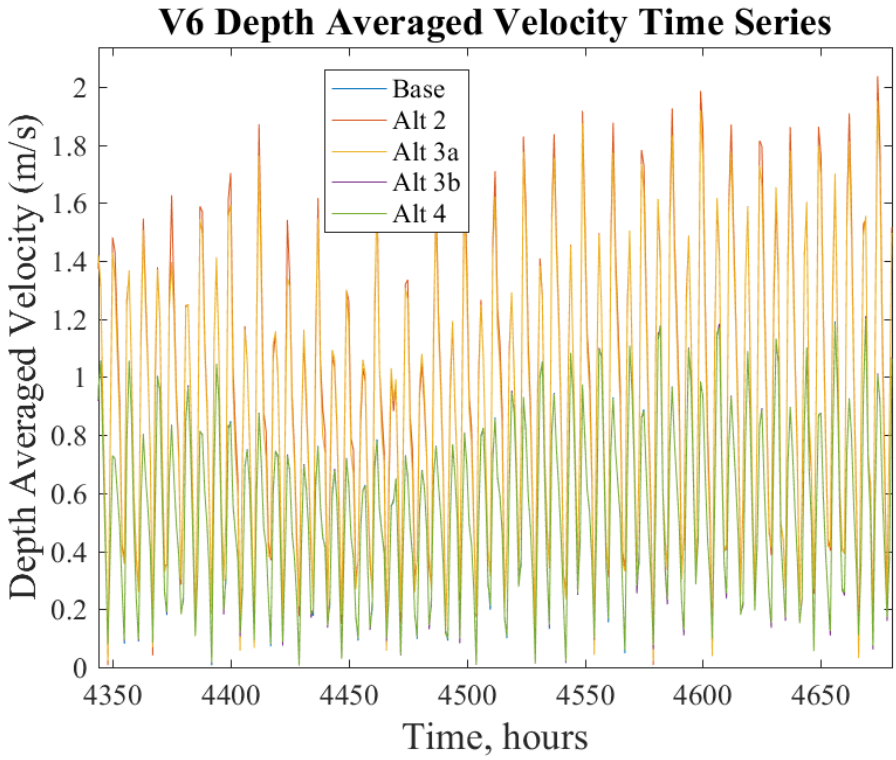


Figure 46: Set 1 Point V7 Depth Averaged Velocity Magnitudes

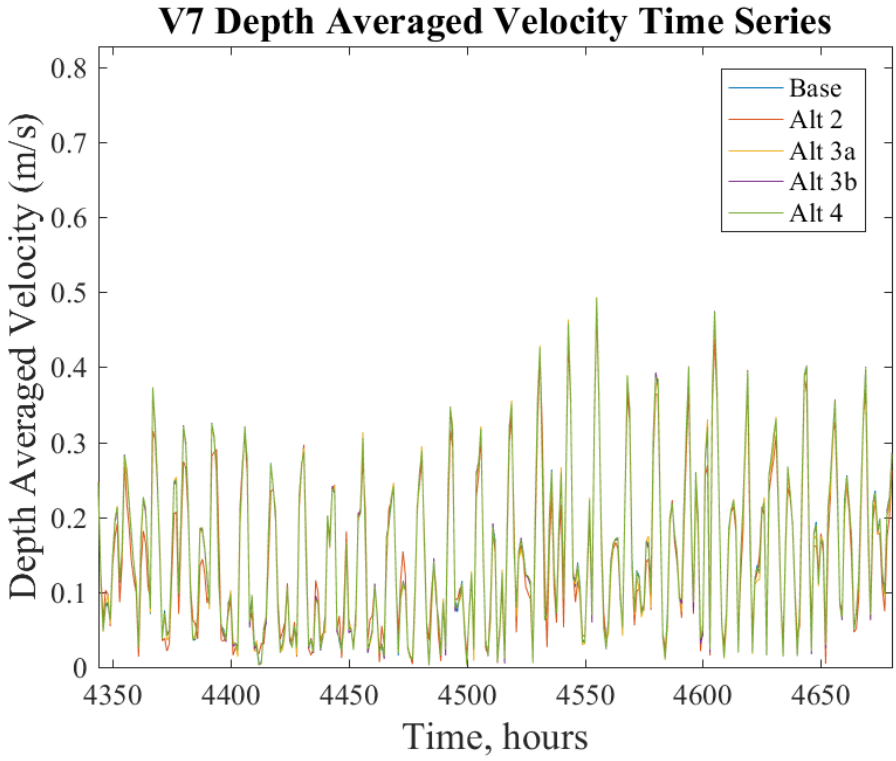


Figure 47: Set 1 Point V8 Depth Averaged Velocity Magnitudes

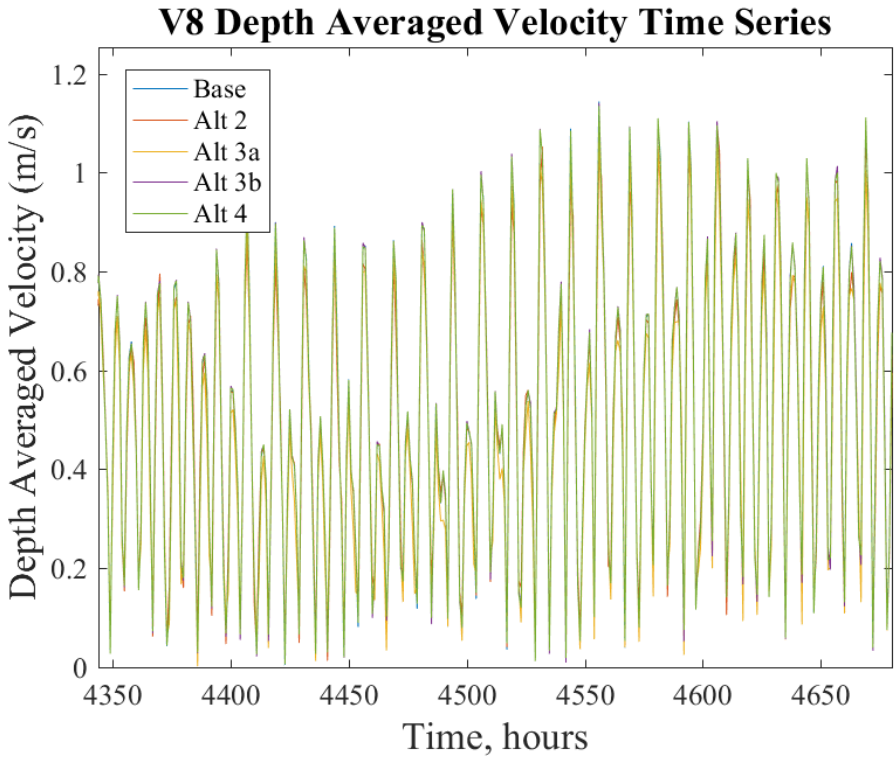


Figure 48: Set 1 Point S1 Depth Averaged Velocity Magnitudes

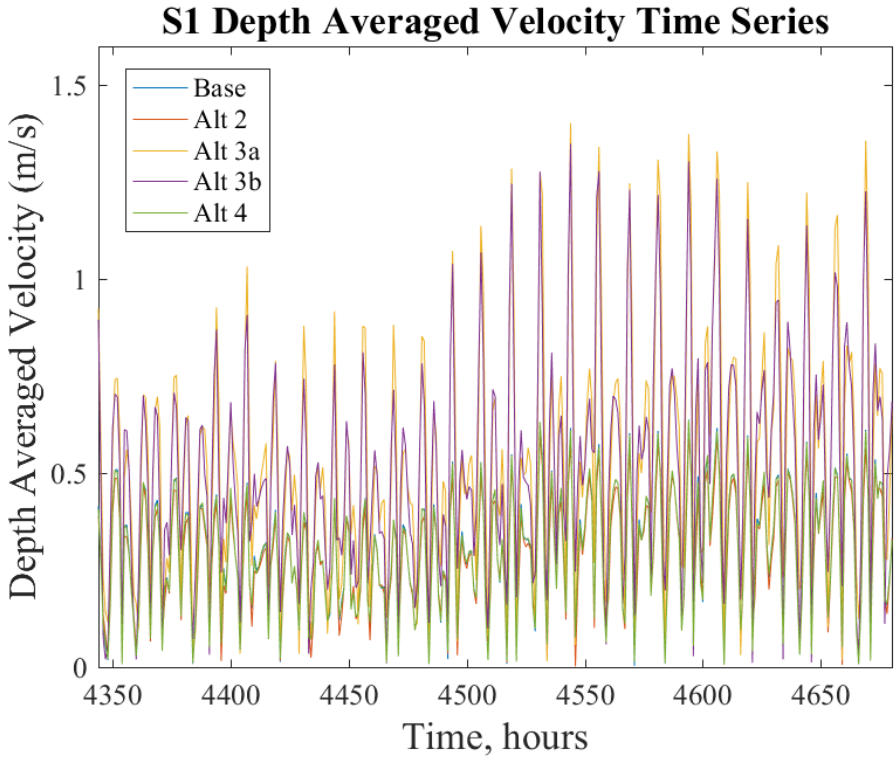


Figure 49: Set 1 Point S2 Depth Averaged Velocity Magnitudes

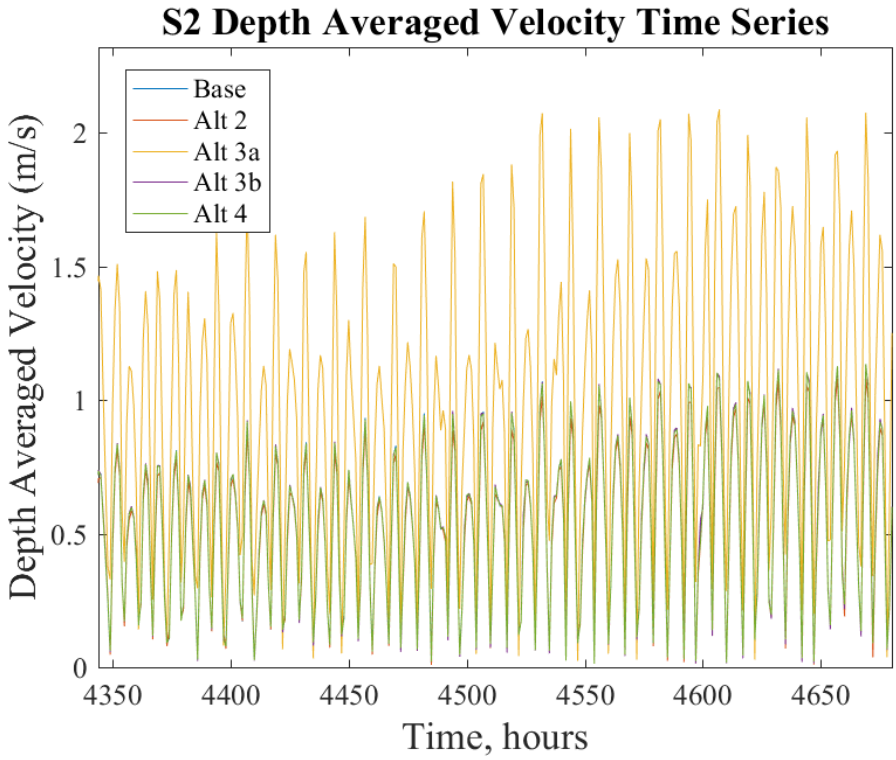


Figure 50: Set 1 Point S3 Depth Averaged Velocity Magnitudes

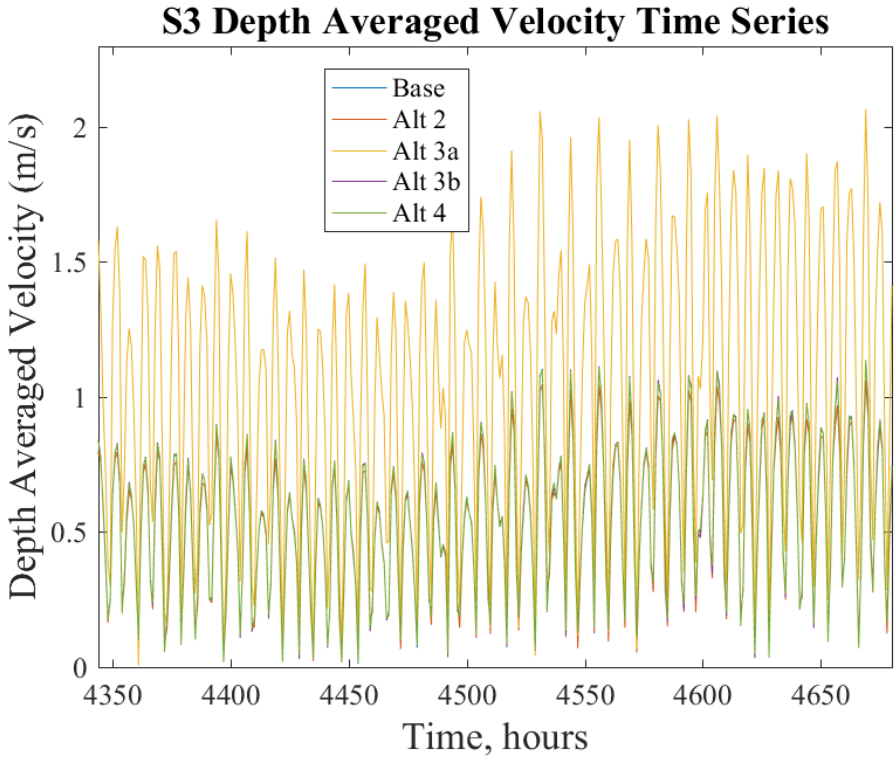


Figure 51: Set 1 Point S4 Depth Averaged Velocity Magnitudes

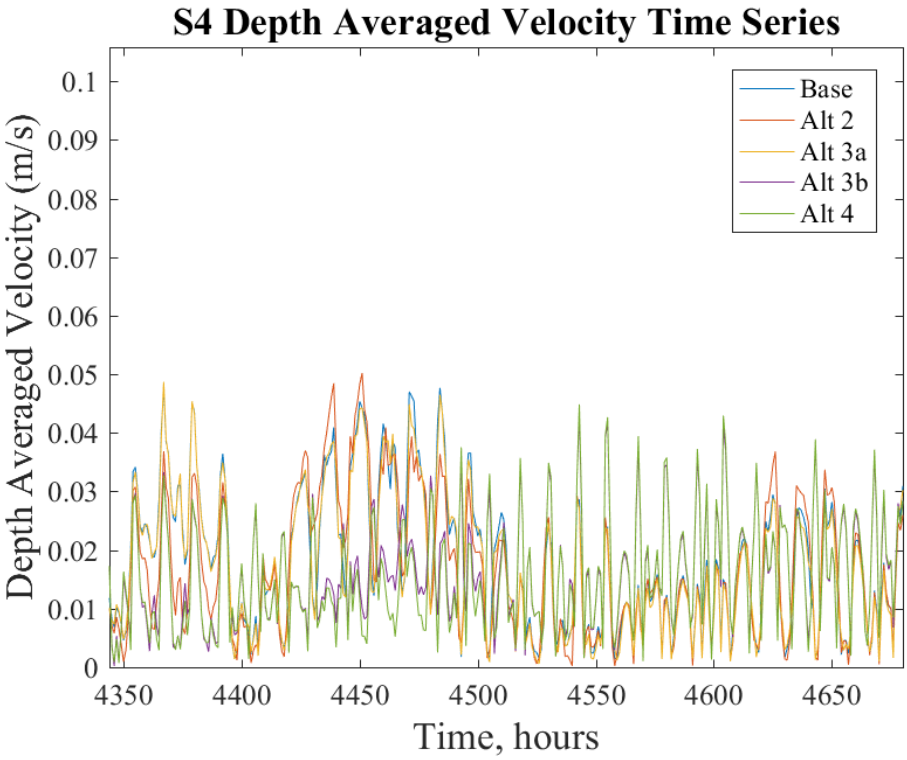


Figure 52: Set 1 Point S5 Depth Averaged Velocity Magnitudes

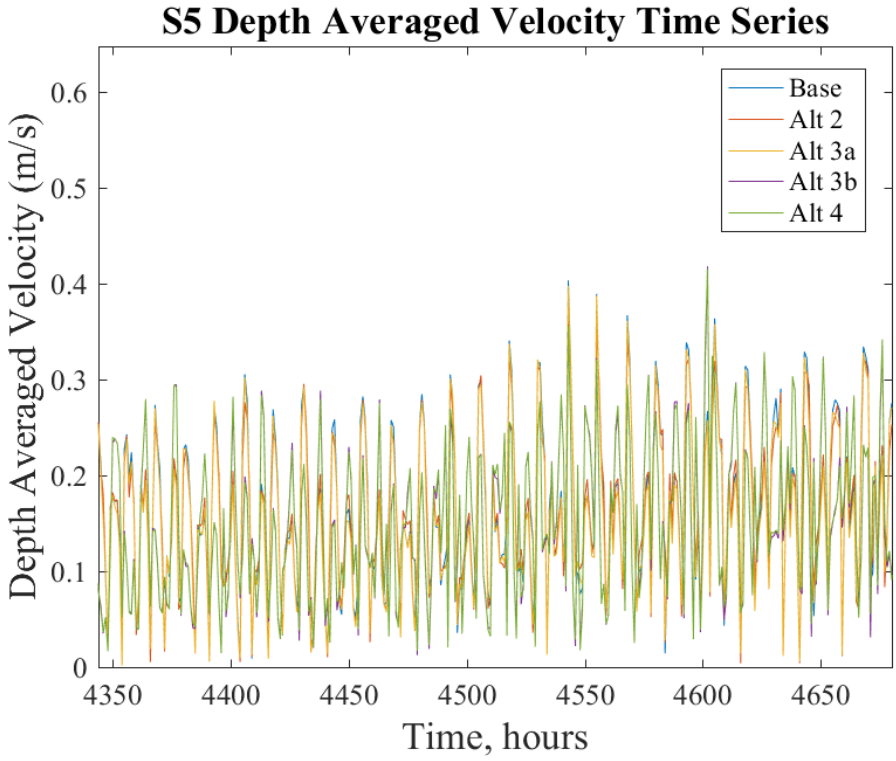


Figure 53: Set 1 Point S6 Depth Averaged Velocity Magnitudes

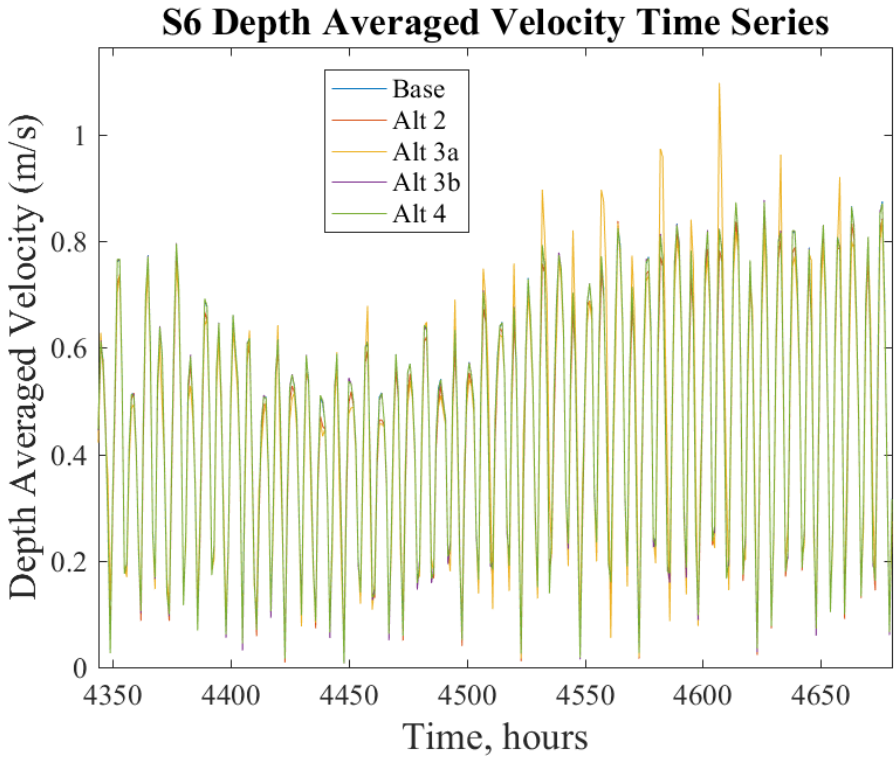


Figure 54: Set 1 Point T1 Depth Averaged Velocity Magnitudes

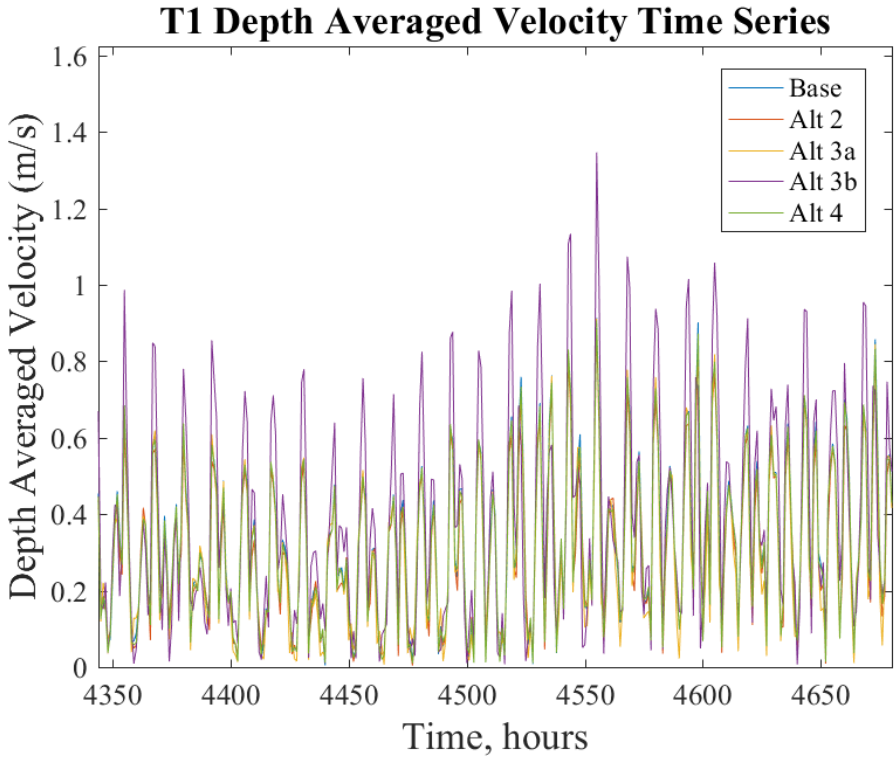


Figure 55: Set 1 Point T2 Depth Averaged Velocity Magnitudes

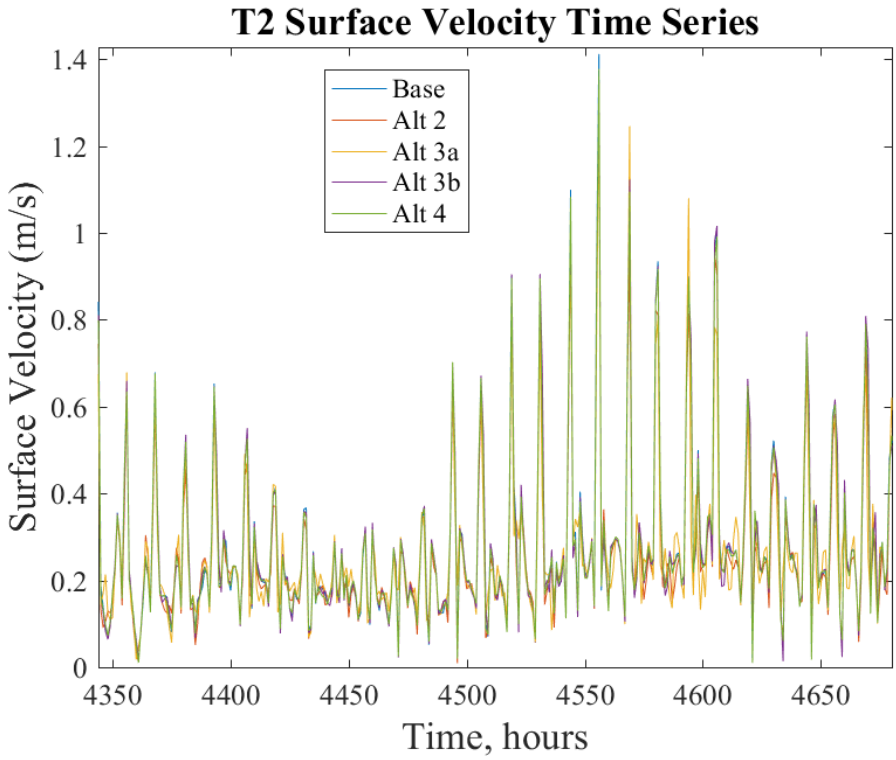


Figure 56: Set 1 Point T3 Depth Averaged Velocity Magnitudes

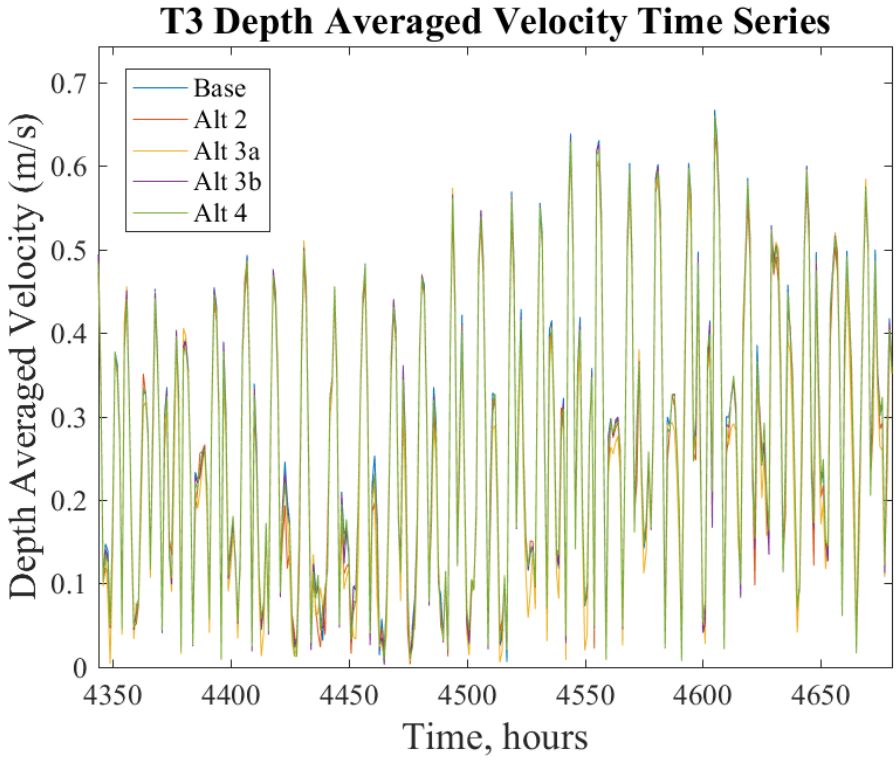


Figure 57: Set 1 Point R1 Depth Averaged Velocity Magnitudes

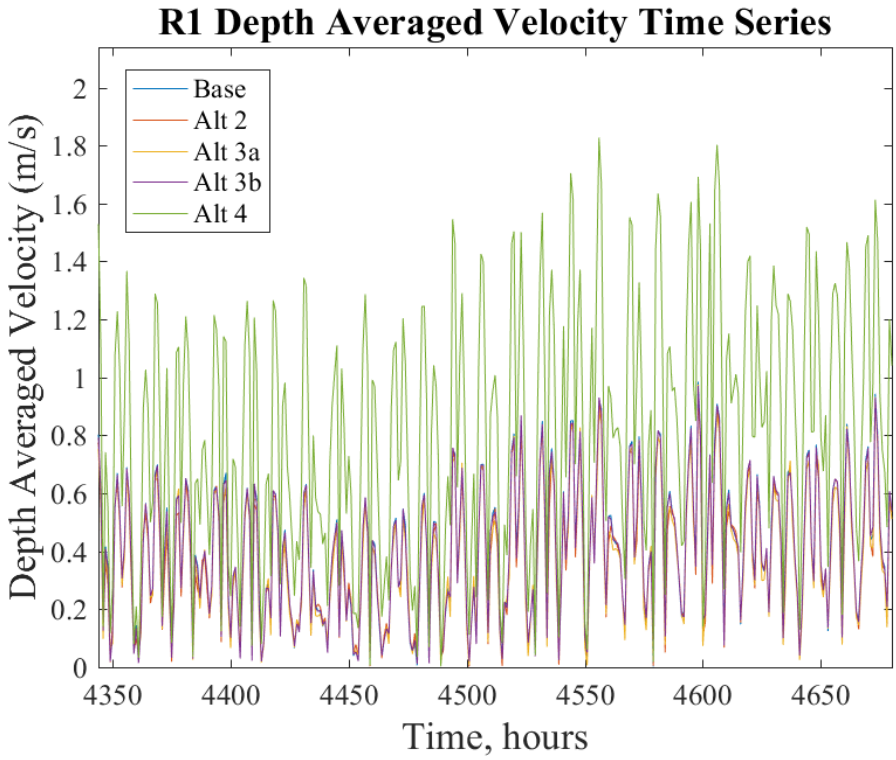


Figure 58: Set 1 Point Fresh Kills Depth Averaged Velocity Magnitudes

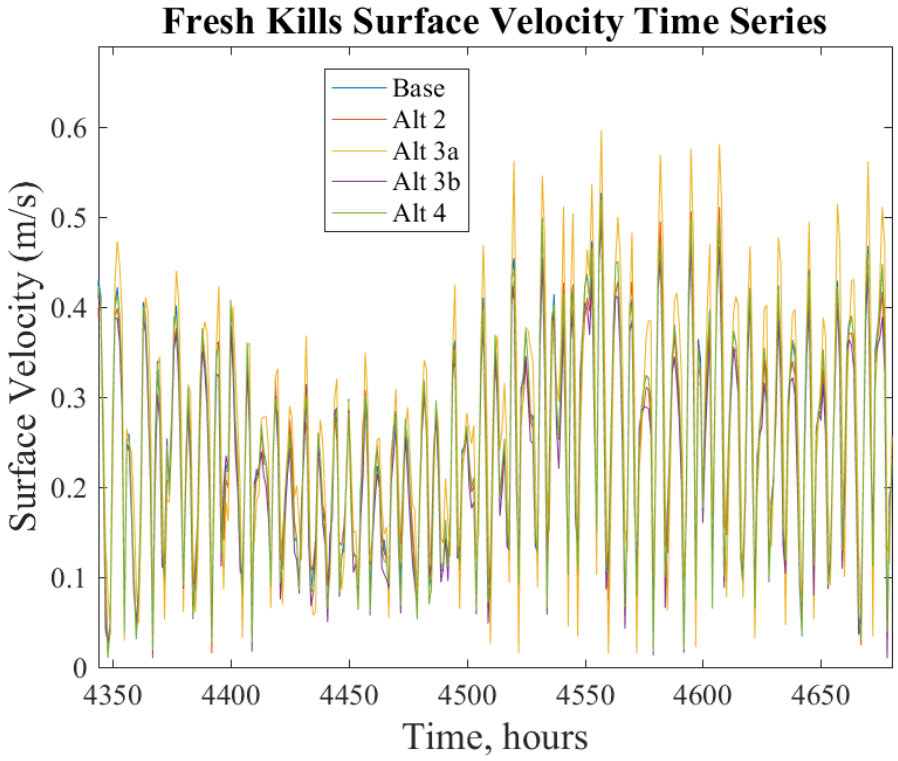


Figure 59: Set 1 Point Sandy Hook Depth Averaged Velocity Magnitudes

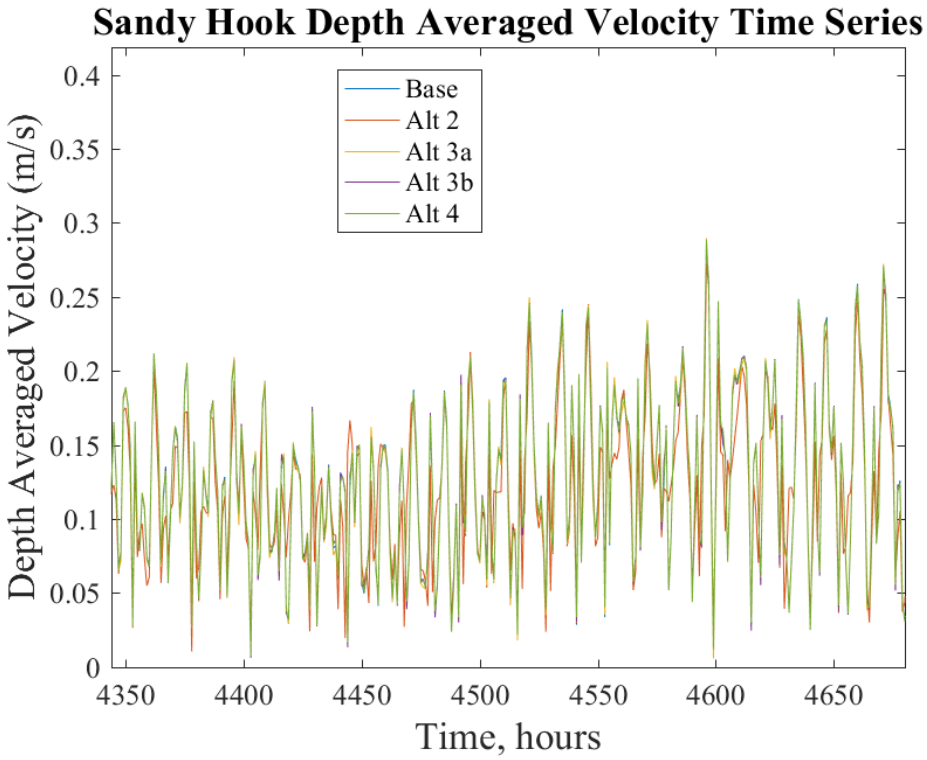


Figure 60: Set 1 Point Jamaica Bay Depth Averaged Velocity Magnitudes

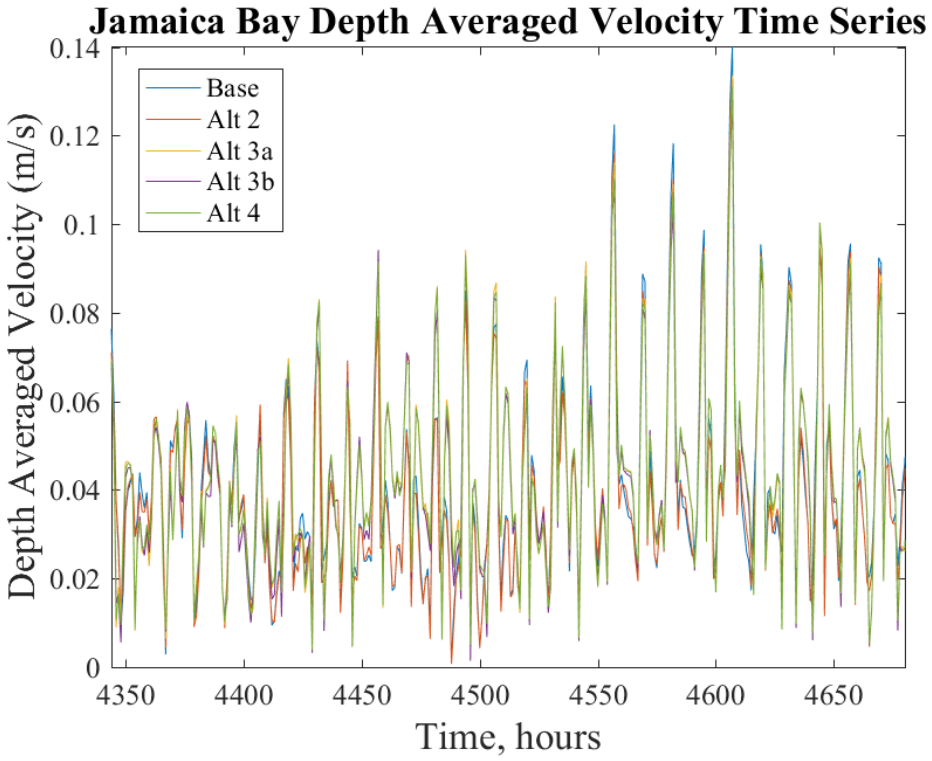


Figure 61: Set 1 Point Robbins Reef Depth Averaged Velocity Magnitudes

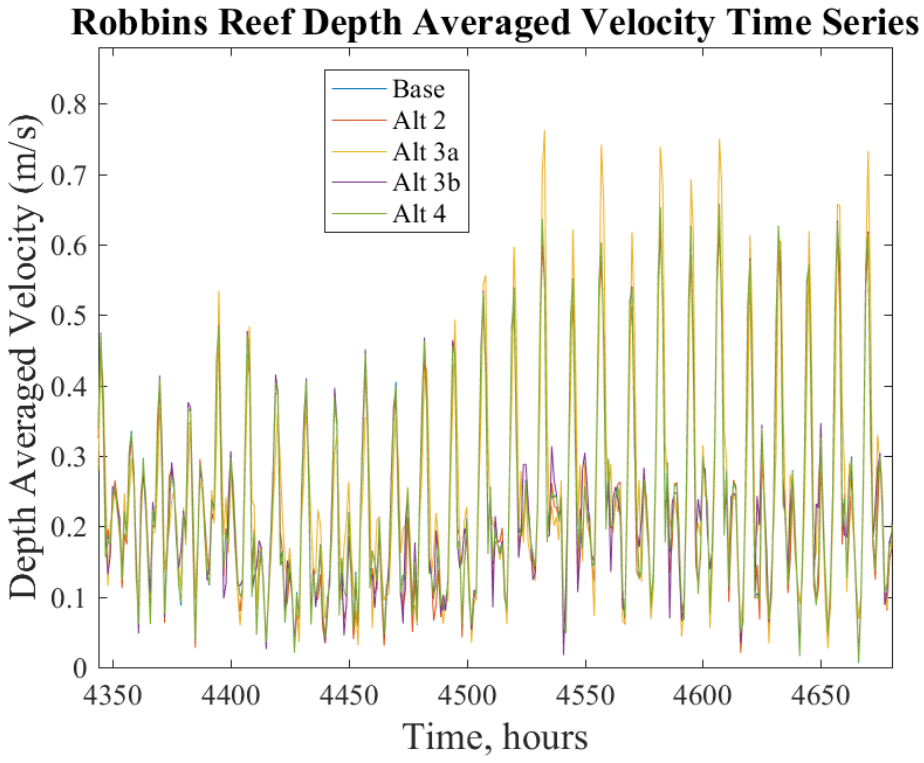


Figure 62: Set 1 Point Battery Depth Averaged Velocity Magnitudes

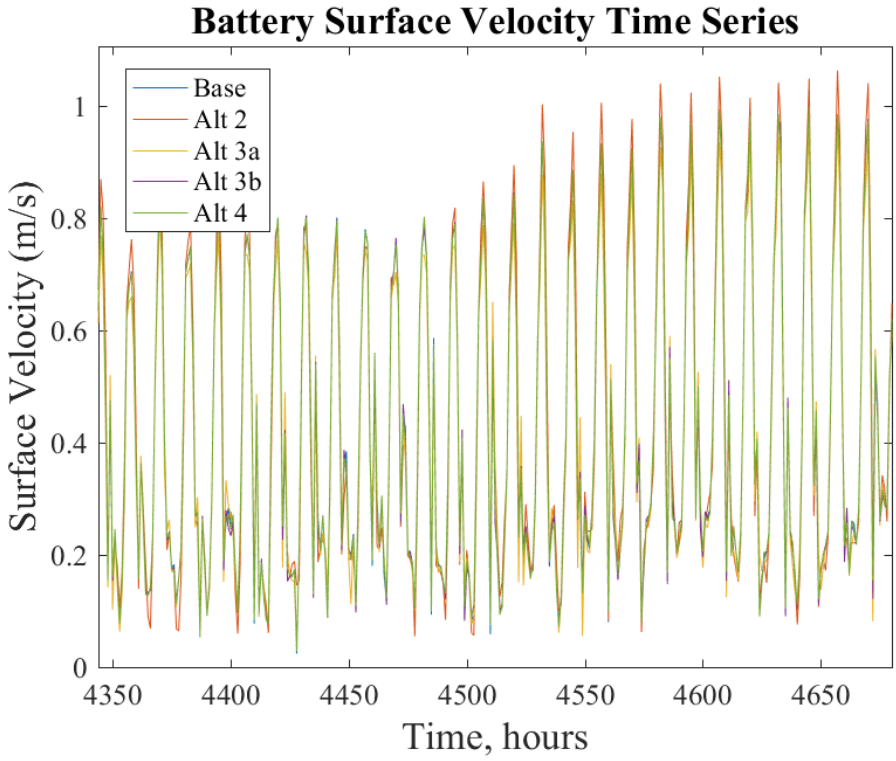


Figure 63: Set 1 Point KLGA Depth Averaged Velocity Magnitudes

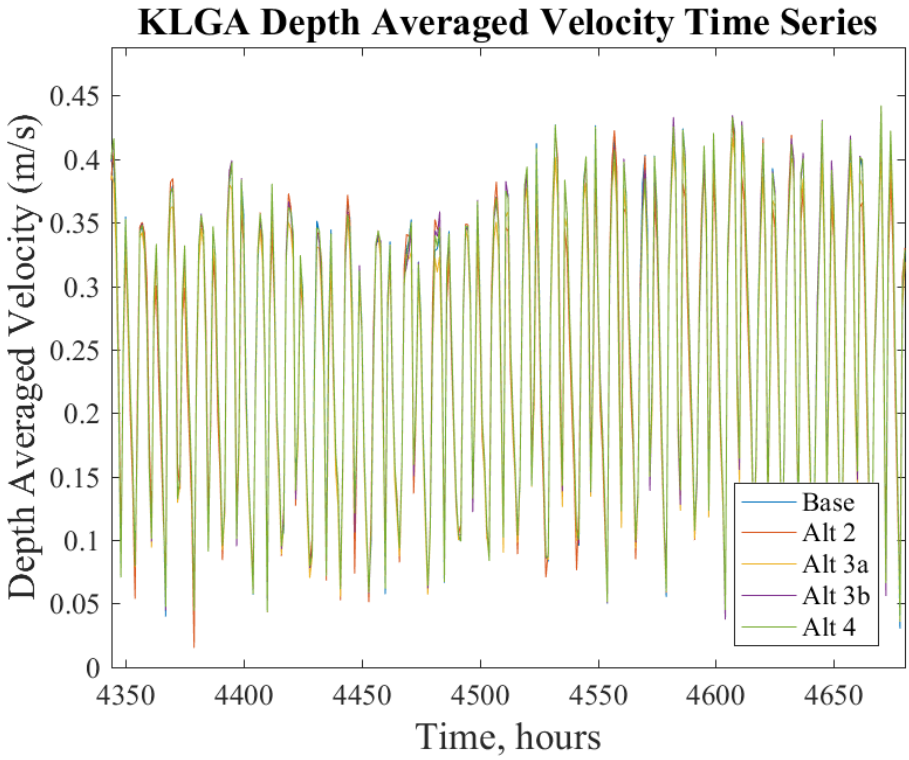


Figure 64: Set 1 Point Mariners Harbor Depth Averaged Velocity Magnitudes

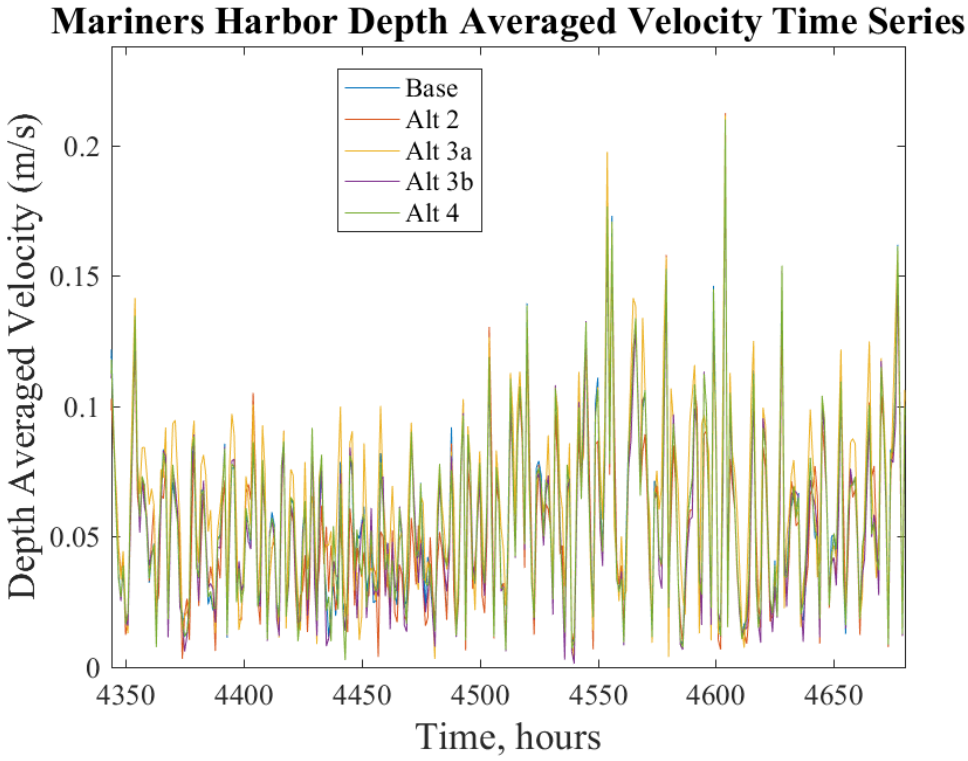


Figure 65: Set 1 Point North Reach Depth Averaged Velocity Magnitudes

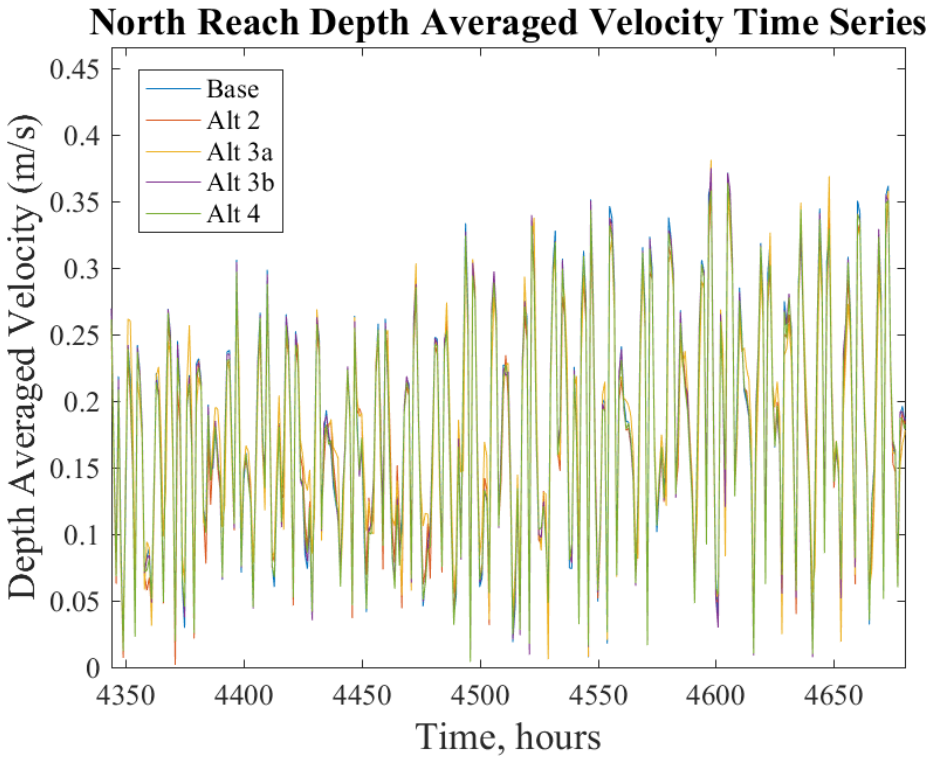
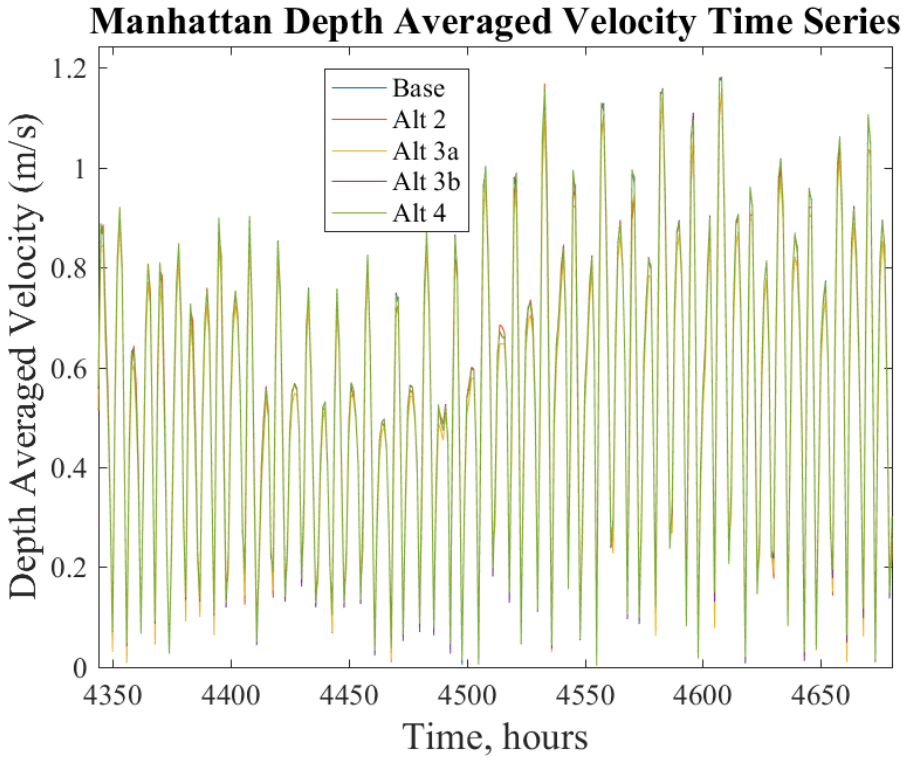


Figure 66: Set 1 Point Manhattan Depth Averaged Velocity Magnitudes



2.2.1.3 Velocity Percentiles

In Figure 67 through Figure 93, the depth average velocity percentiles are plotted for each alternative and the without project for the locations shown in Figure 12.

Figure 67: Set 1 Point V1 Depth Average Velocity Magnitude Percentile

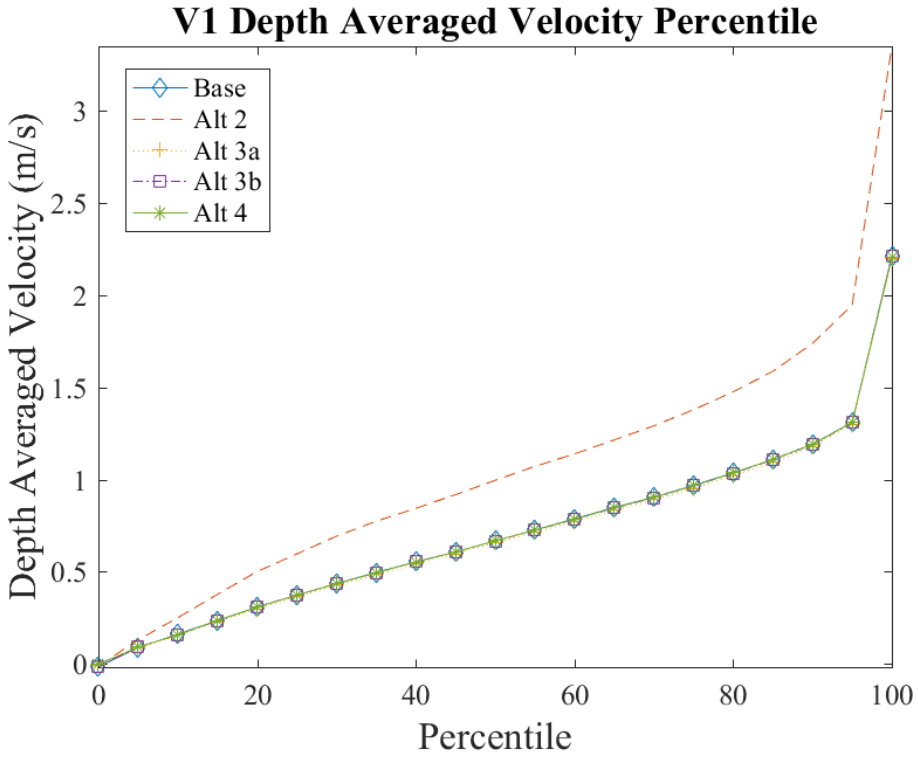


Figure 68: Set 1 Point V2 Depth Average Velocity Magnitude Percentile

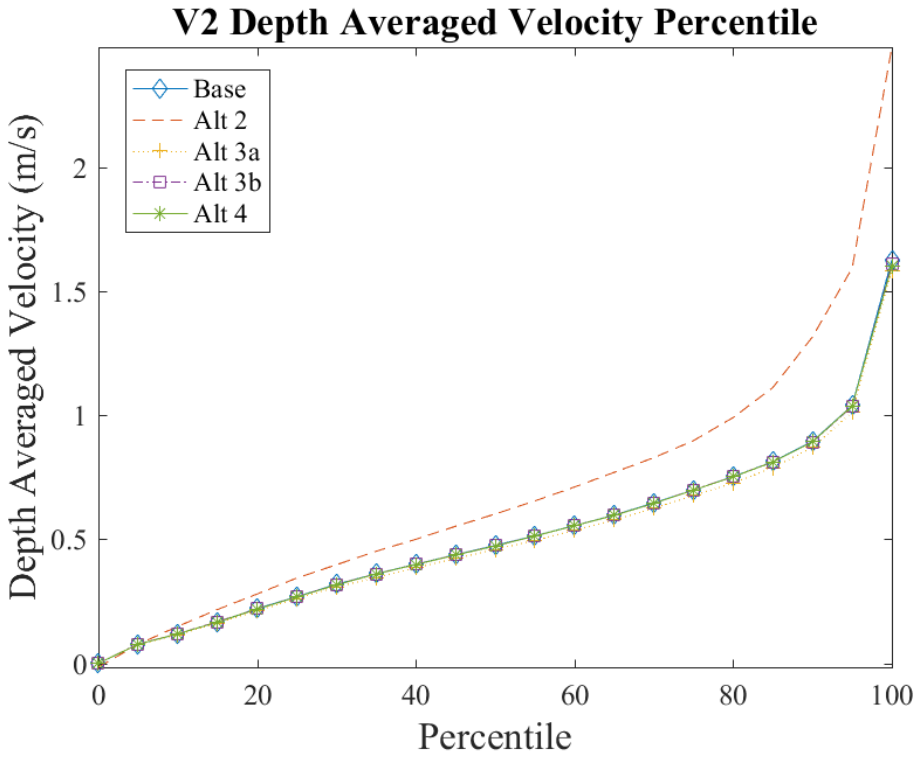


Figure 69: Set 1 Point V3 Depth Average Velocity Magnitude Percentile

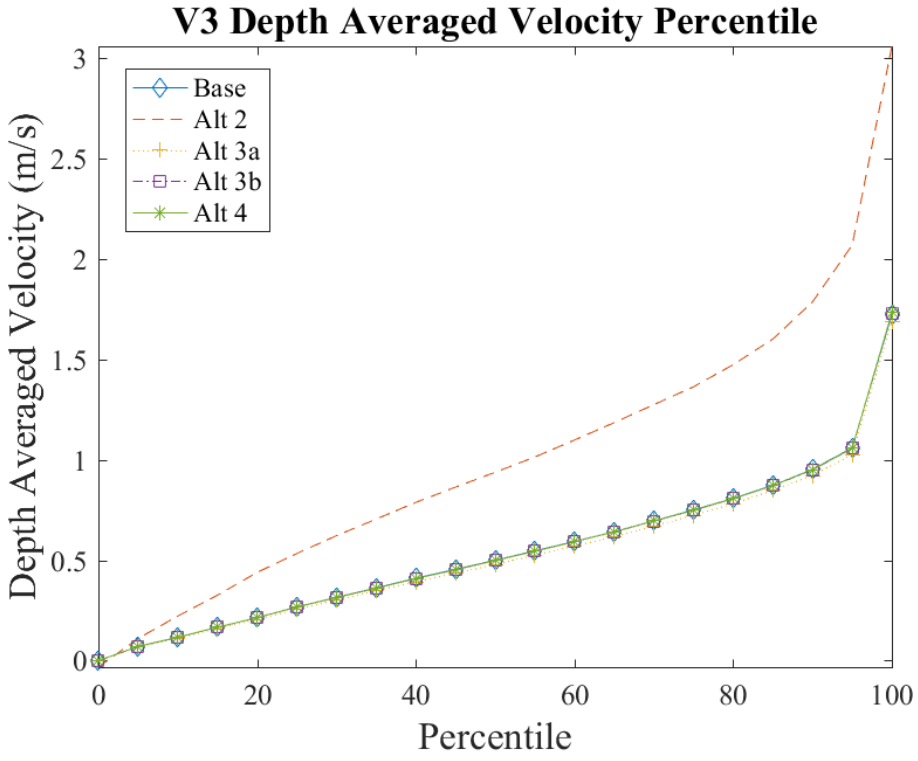


Figure 70: Set 1 Point V4 Depth Average Velocity Magnitude Percentile

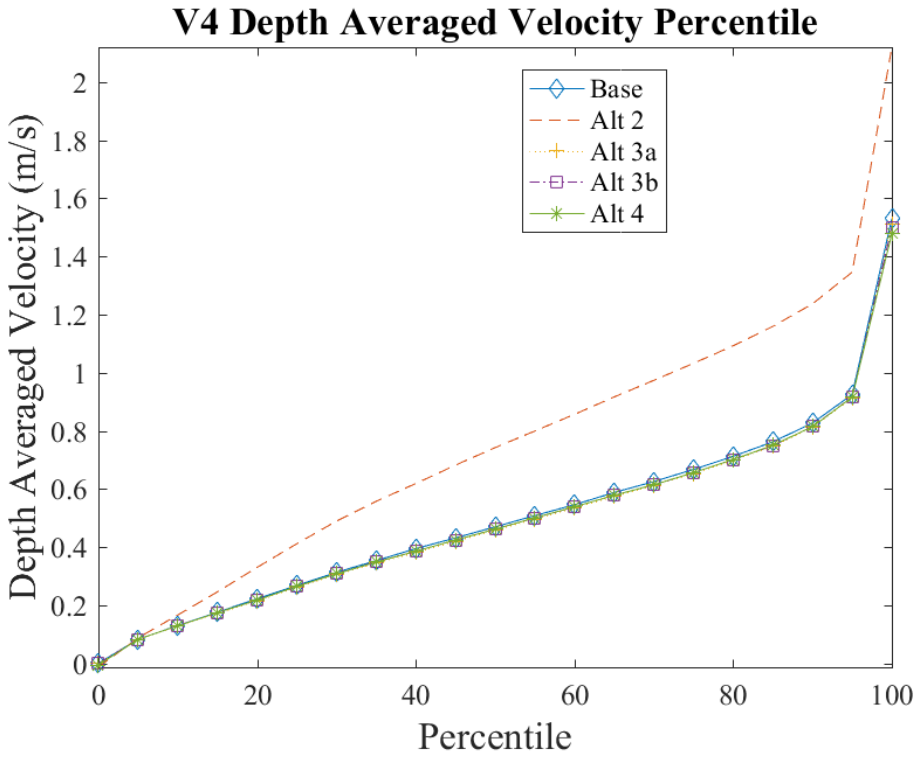


Figure 71: Set 1 Point V5 Depth Average Velocity Magnitude Percentile

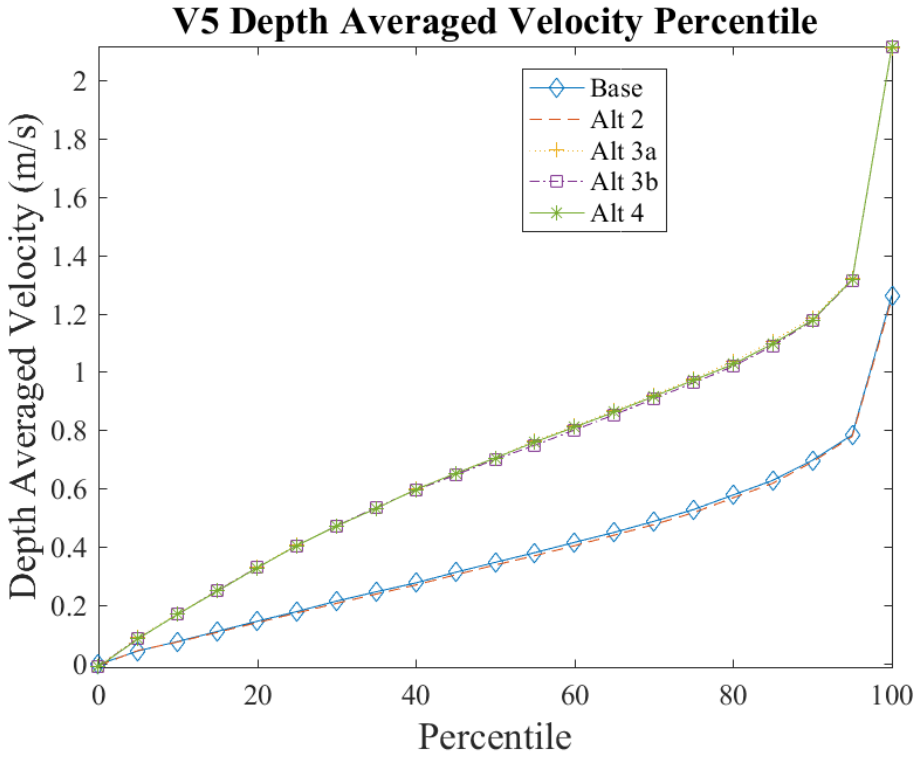


Figure 72: Set 1 Point V6 Depth Average Velocity Magnitude Percentile

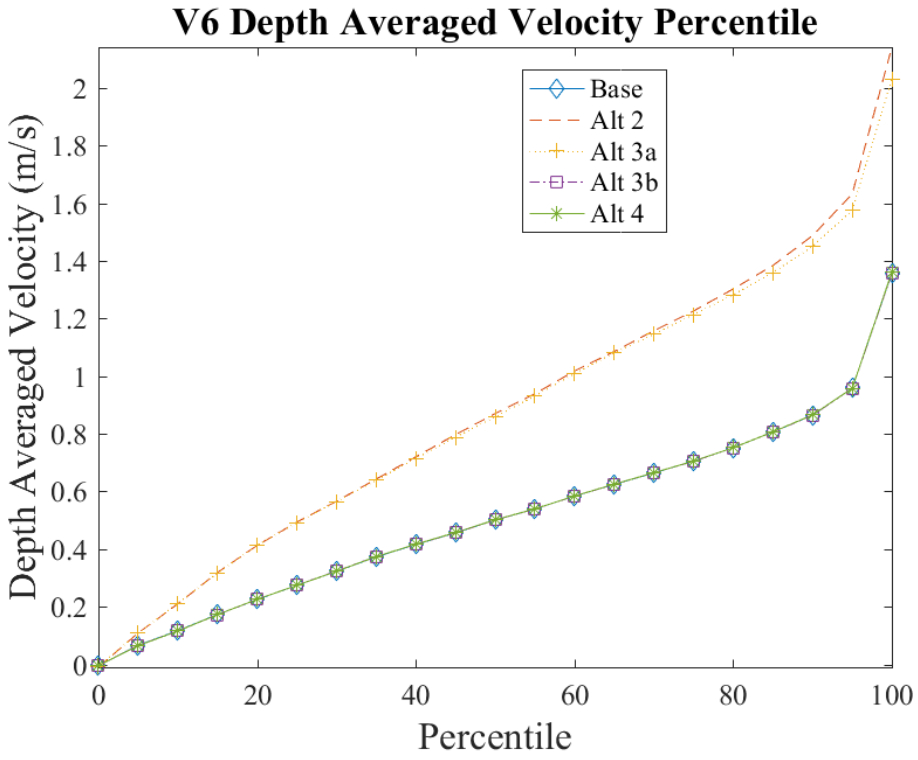


Figure 73: Set 1 Point V7 Depth Average Velocity Magnitude Percentile

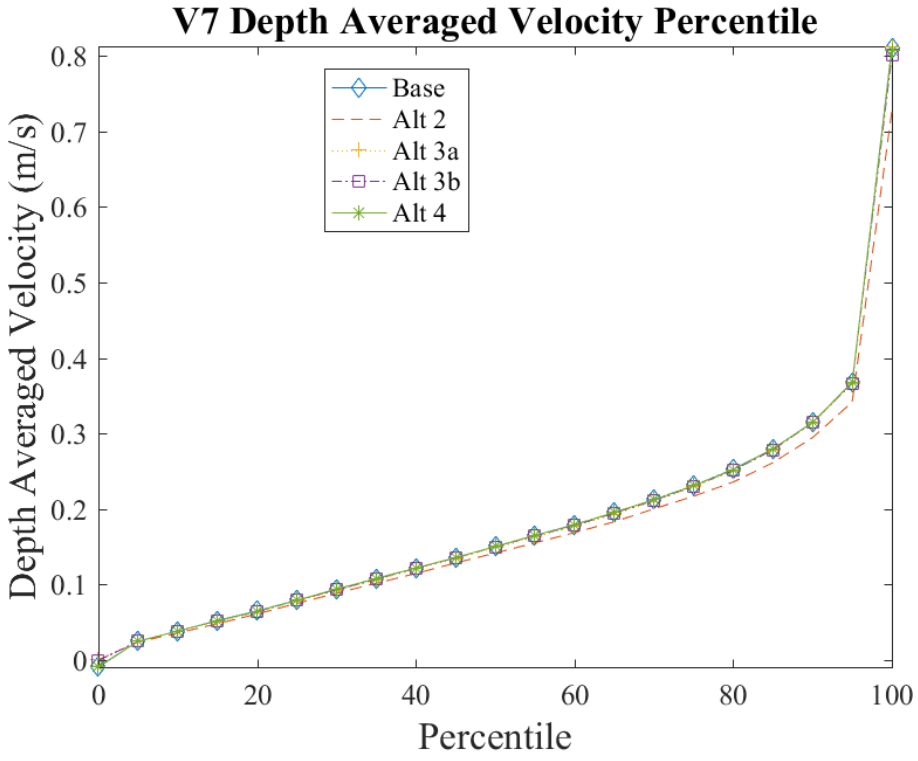


Figure 74: Set 1 Point V8 Depth Average Velocity Magnitude Percentile

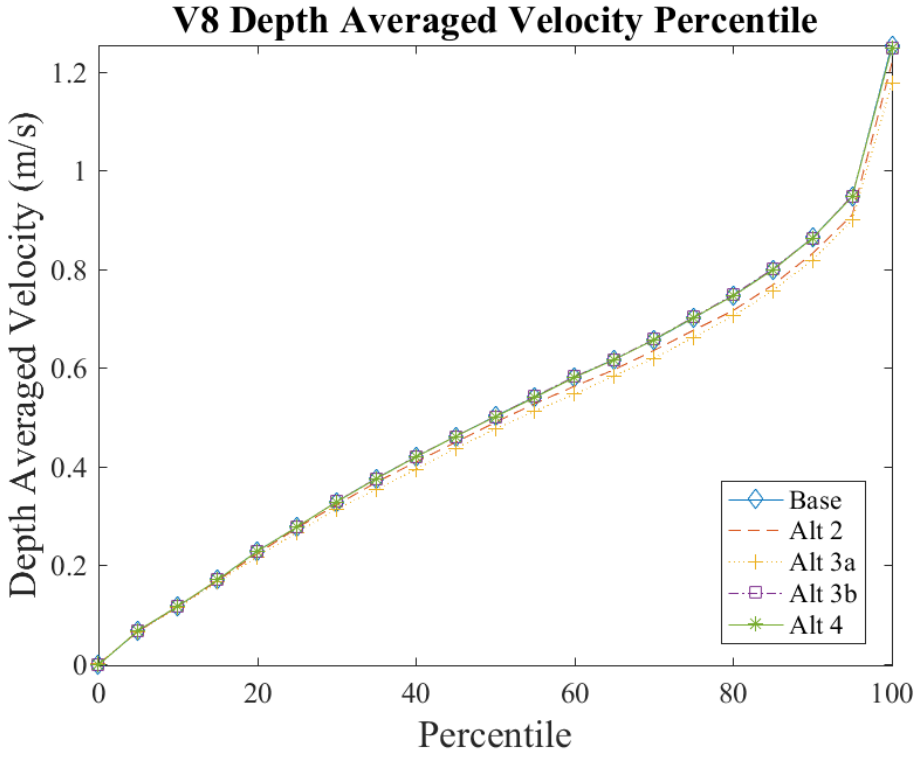


Figure 75: Set 1 Point S1 Depth Average Velocity Magnitude Percentile

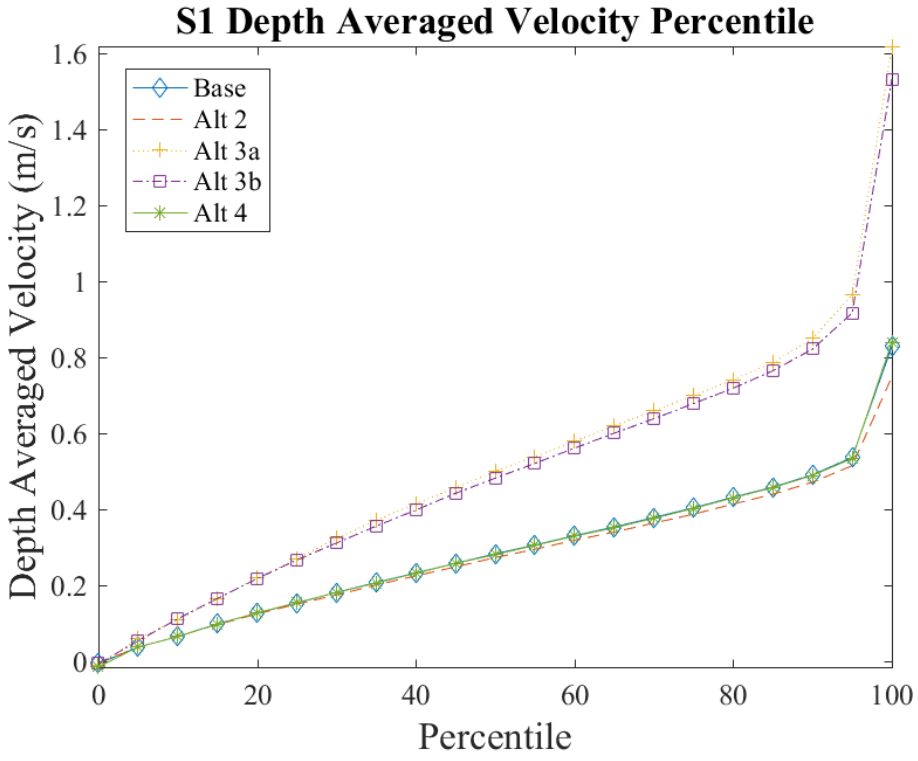


Figure 76: Set 1 Point S2 Depth Average Velocity Magnitude Percentile

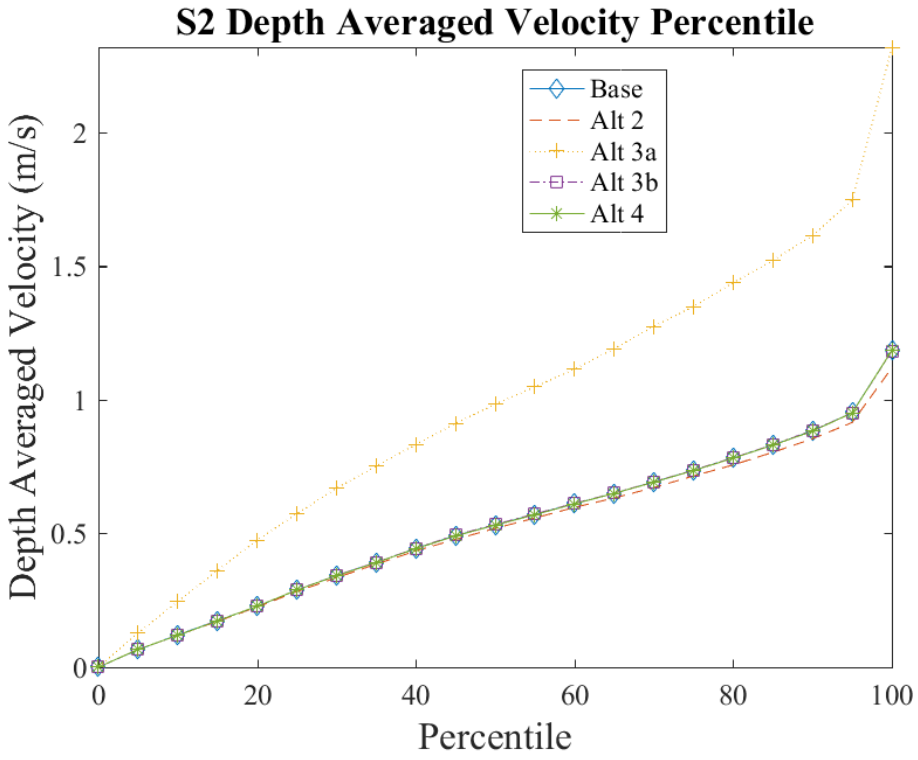


Figure 77: Set 1 Point S3 Depth Average Velocity Magnitude Percentile

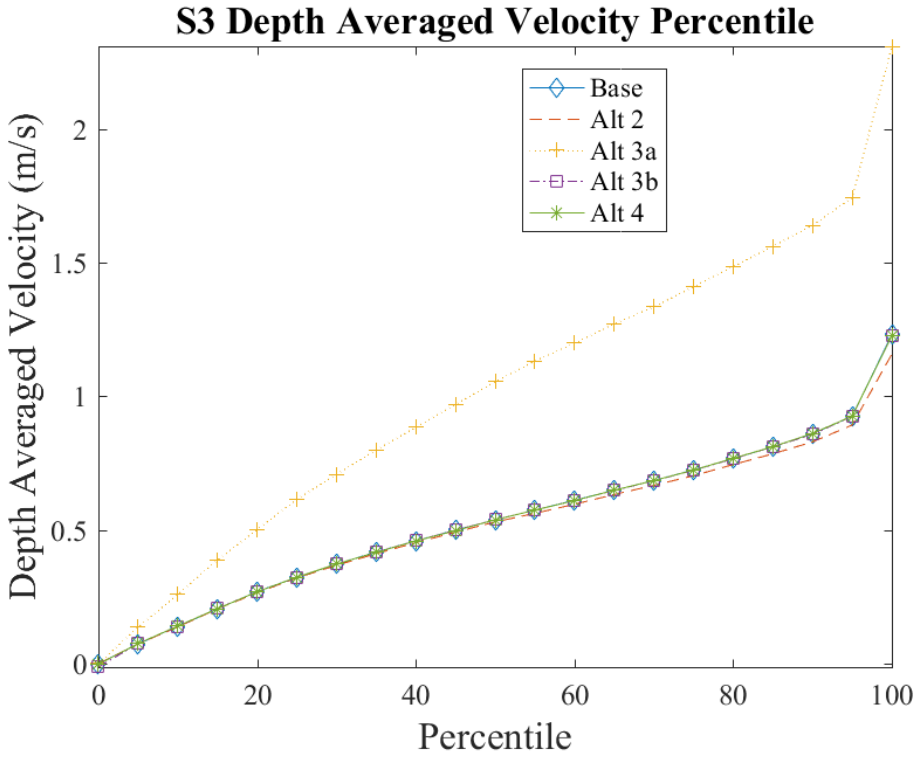


Figure 78: Set 1 Point S4 Depth Average Velocity Magnitude Percentile

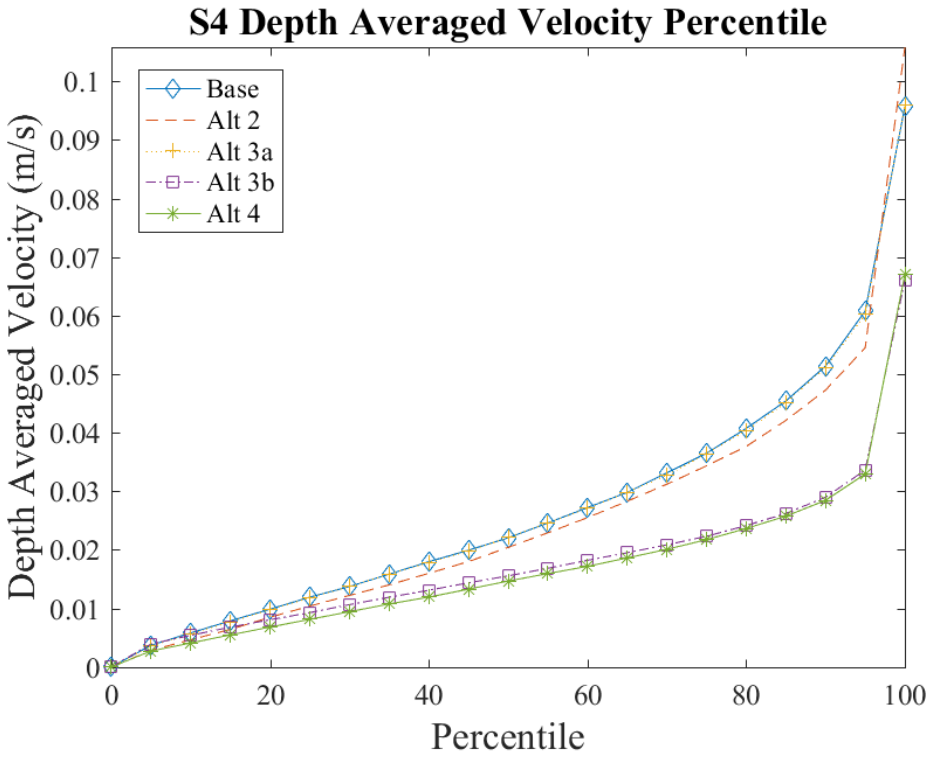


Figure 79: Set 1 Point S5 Depth Average Velocity Magnitude Percentile

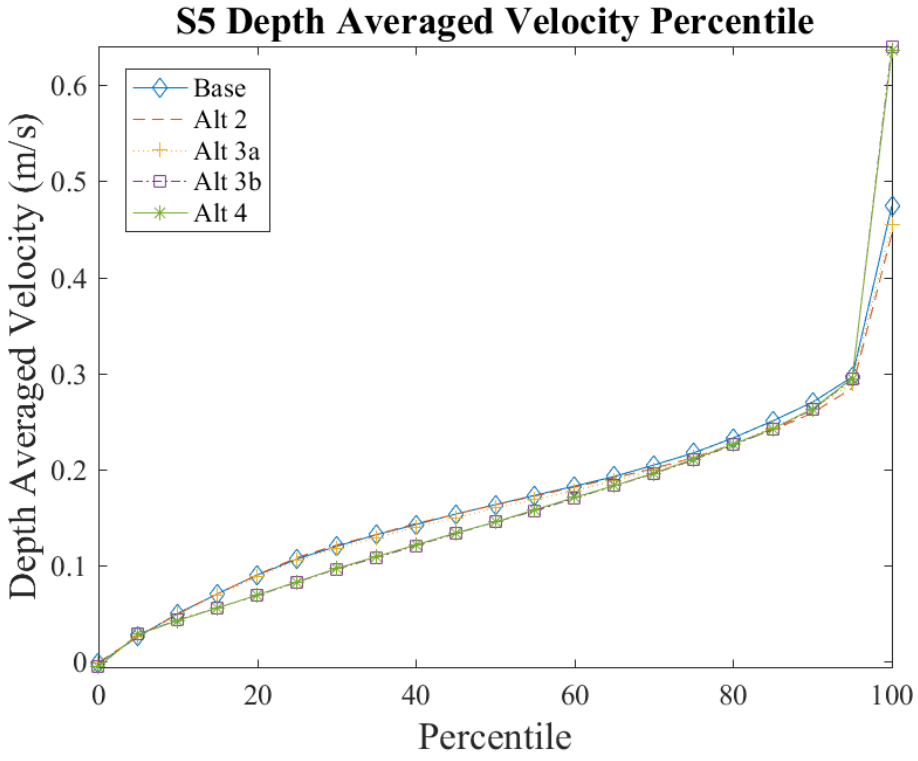


Figure 80: Set 1 Point S6 Depth Average Velocity Magnitude Percentile

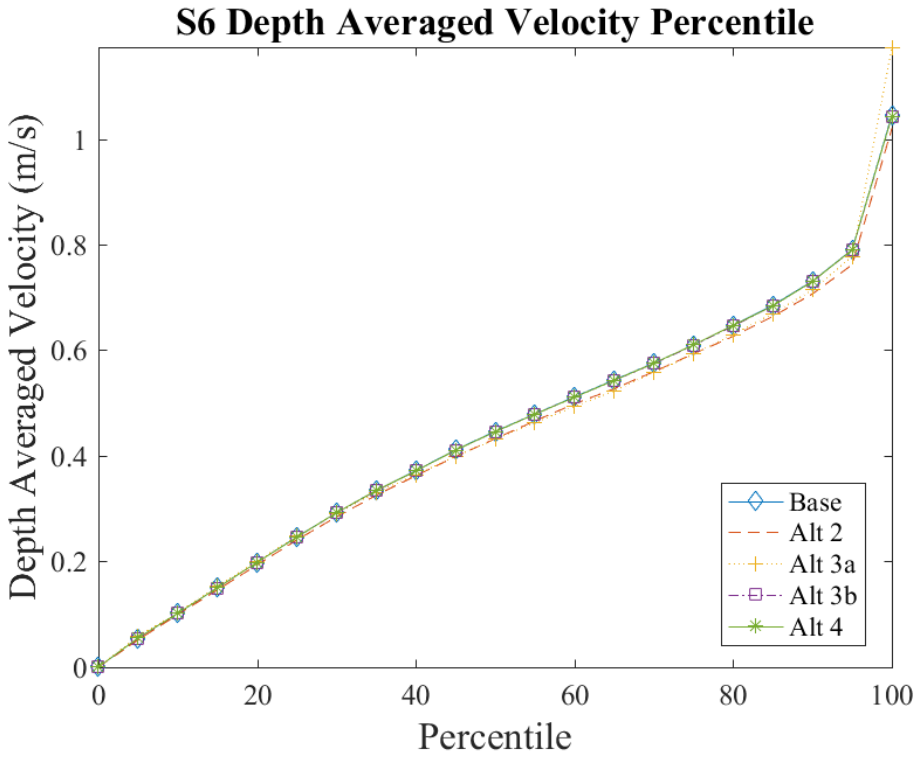


Figure 81: Set 1 Point T1 Depth Average Velocity Magnitude Percentile

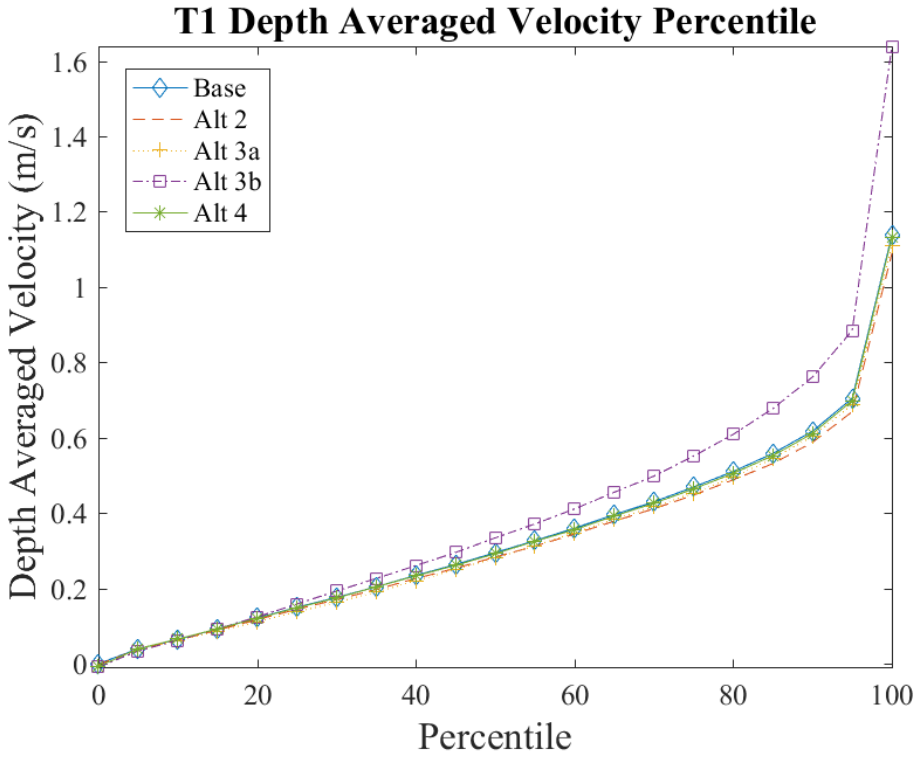


Figure 82: Set 1 Point T2 Depth Average Velocity Magnitude Percentile

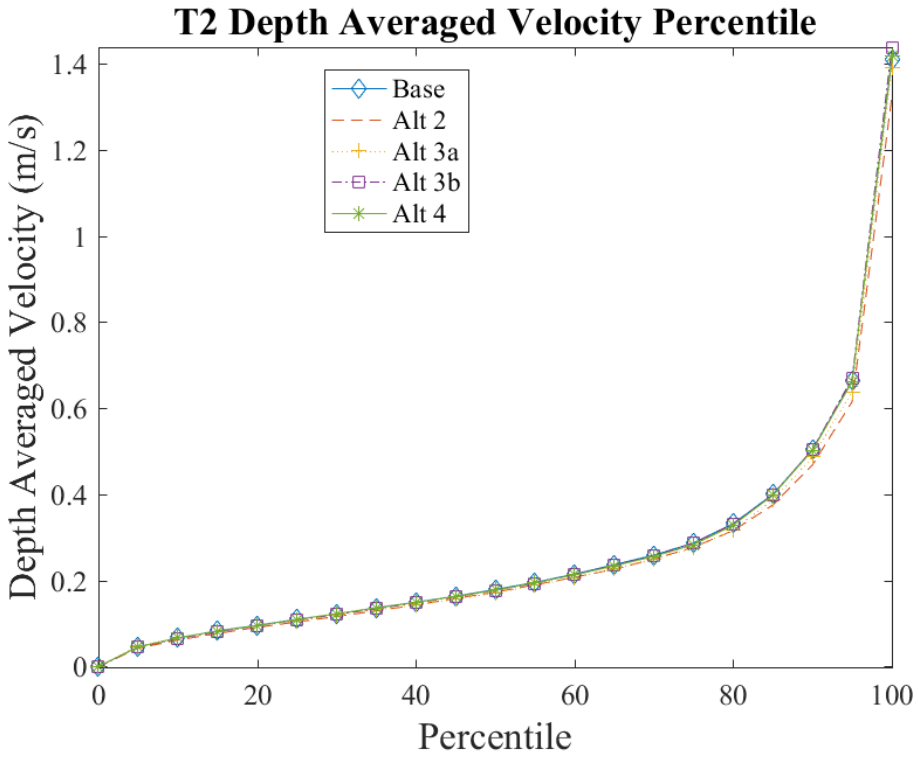


Figure 83: Set 1 Point T3 Depth Average Velocity Magnitude Percentile

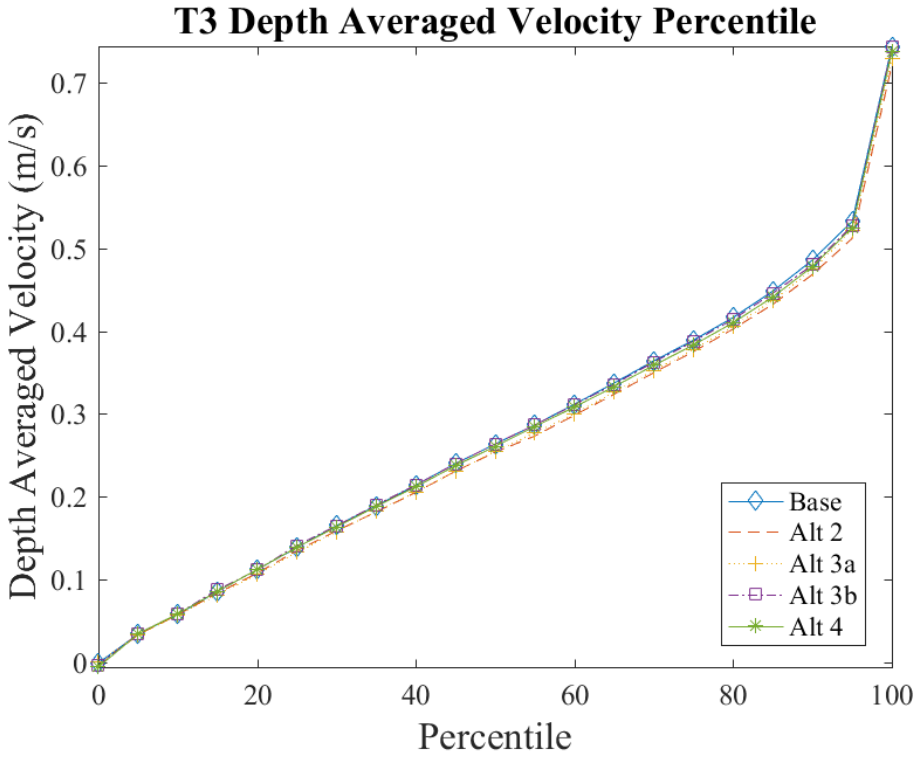


Figure 84: Set 1 Point R1 Depth Average Velocity Magnitude Percentile

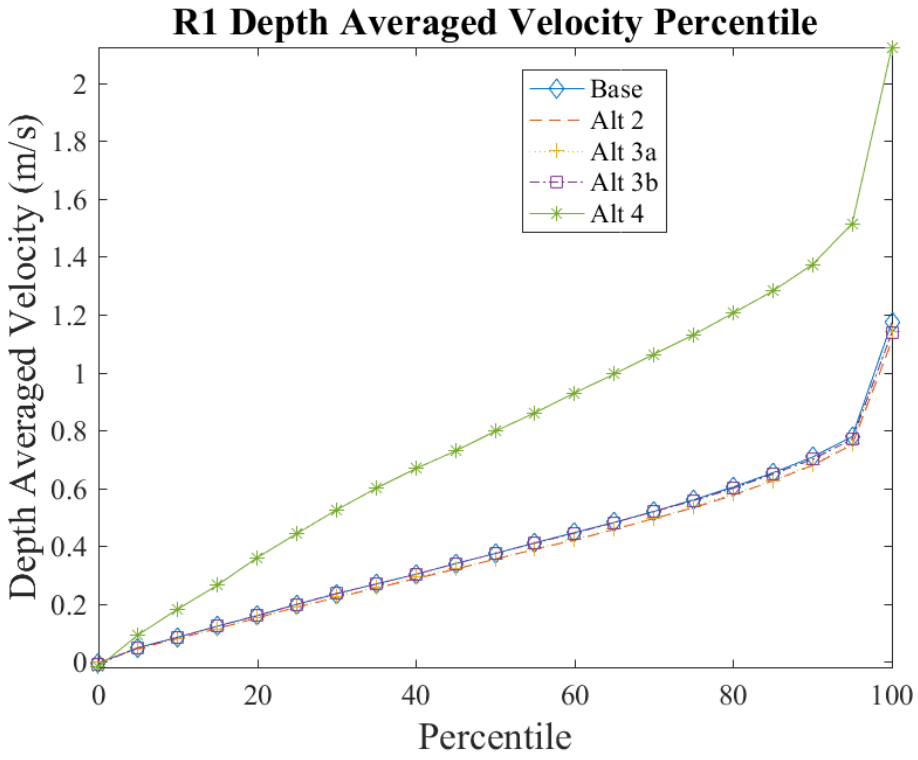


Figure 85: Set 1 Point Fresh Kills Depth Average Velocity Magnitude Percentile

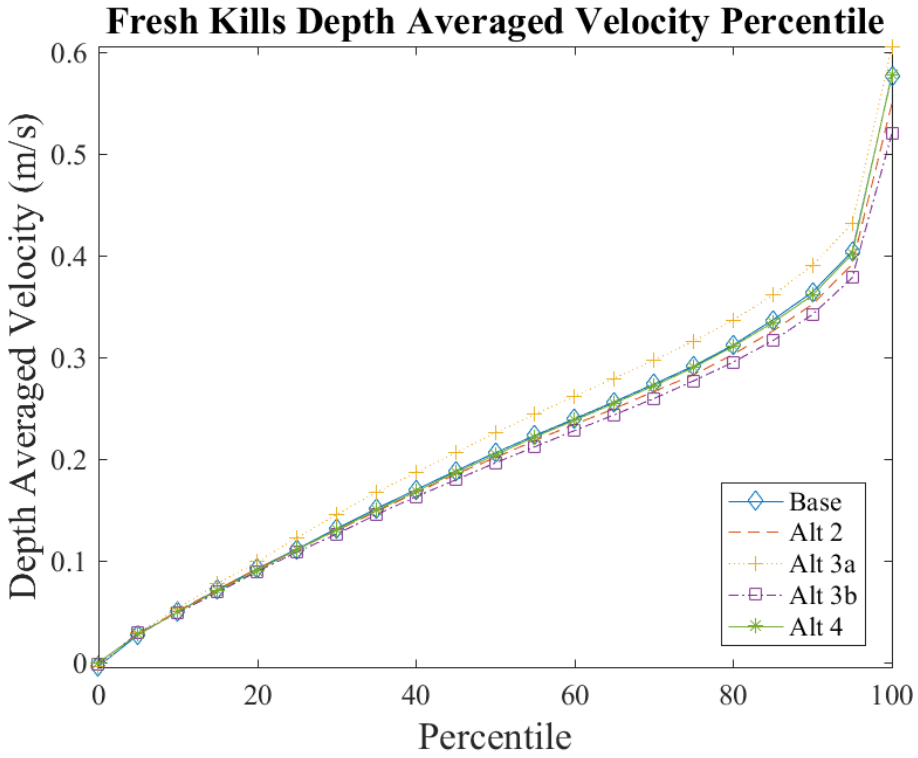


Figure 86: Set 1 Point Sandy Hook Depth Average Velocity Magnitude Percentile

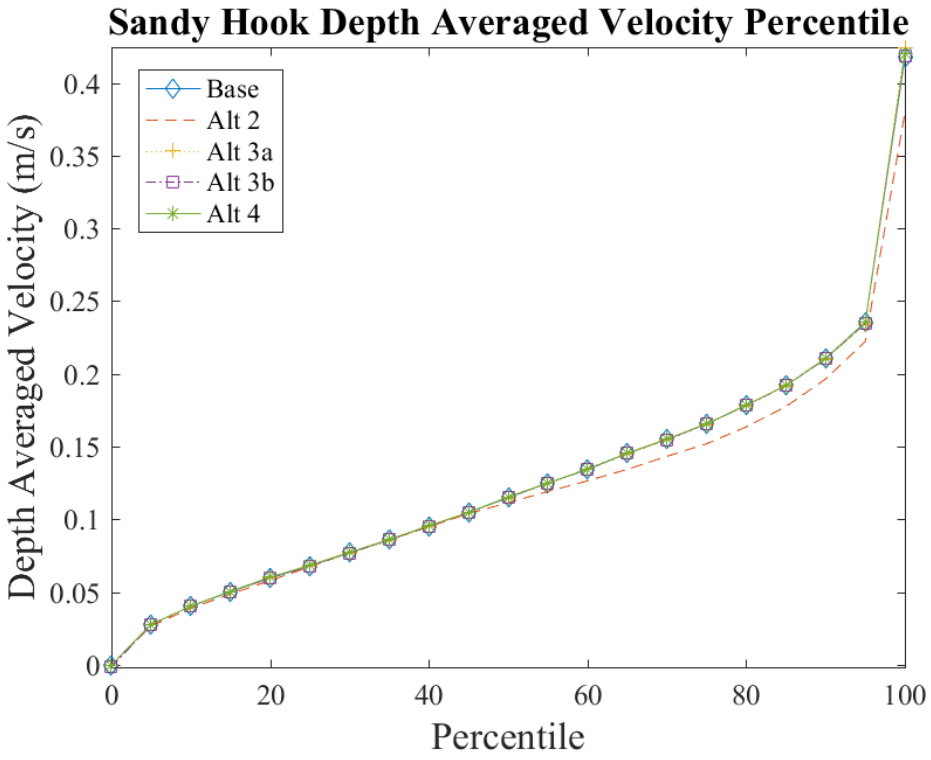


Figure 87: Set 1 Point Jamaica Bay Depth Average Velocity Magnitude Percentile

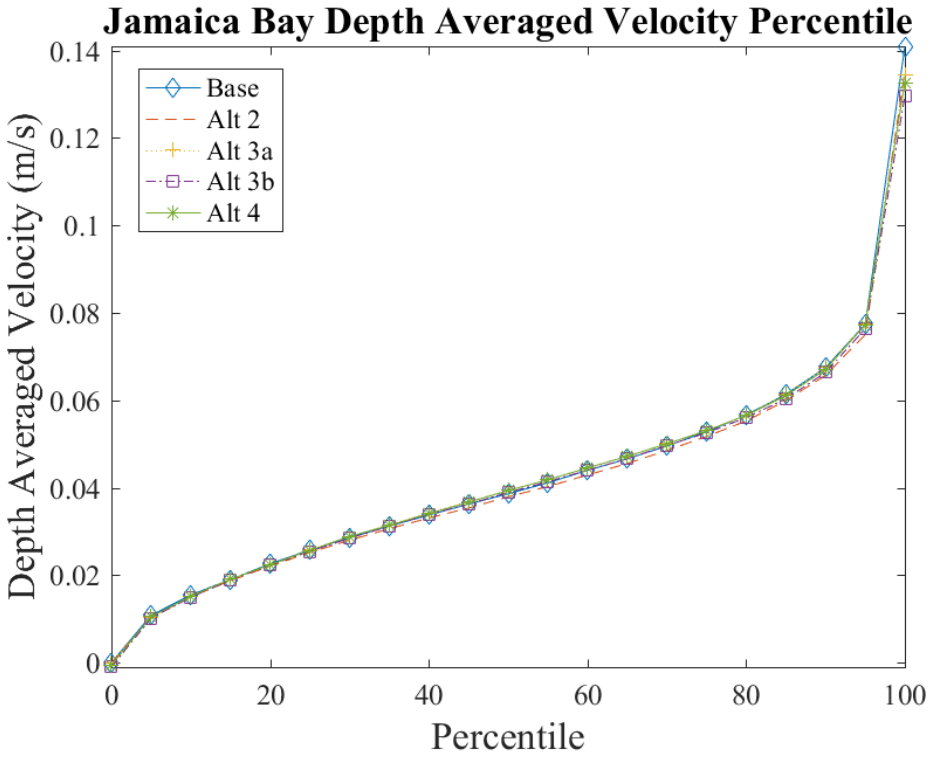


Figure 88: Set 1 Point Robbins Reef Depth Average Velocity Magnitude Percentile

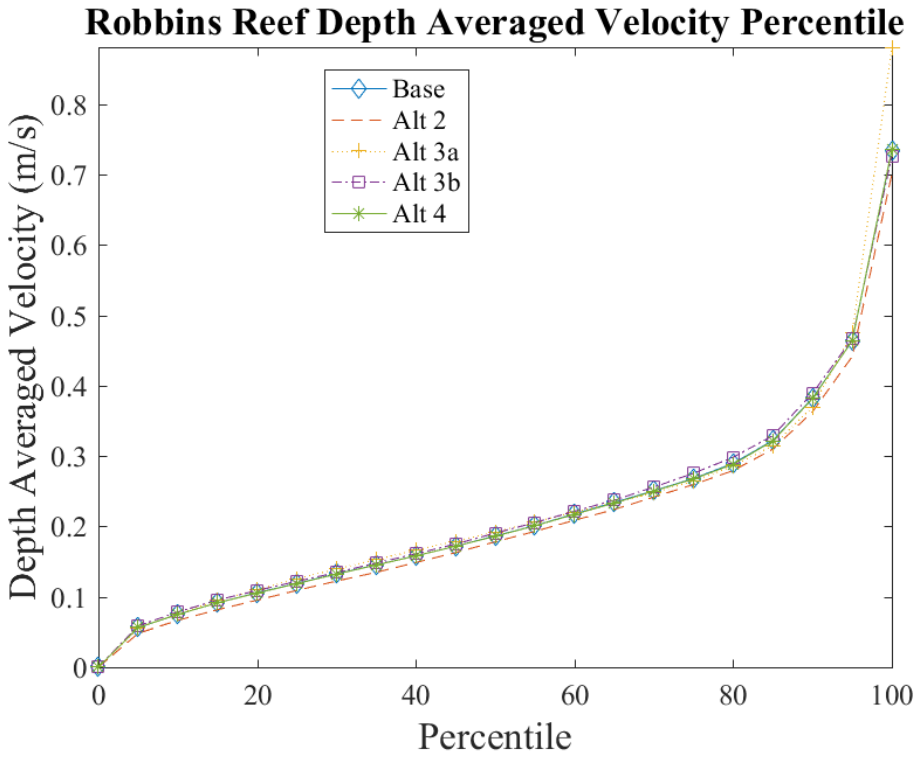


Figure 89: Set 1 Point Battery Depth Average Velocity Magnitude Percentile

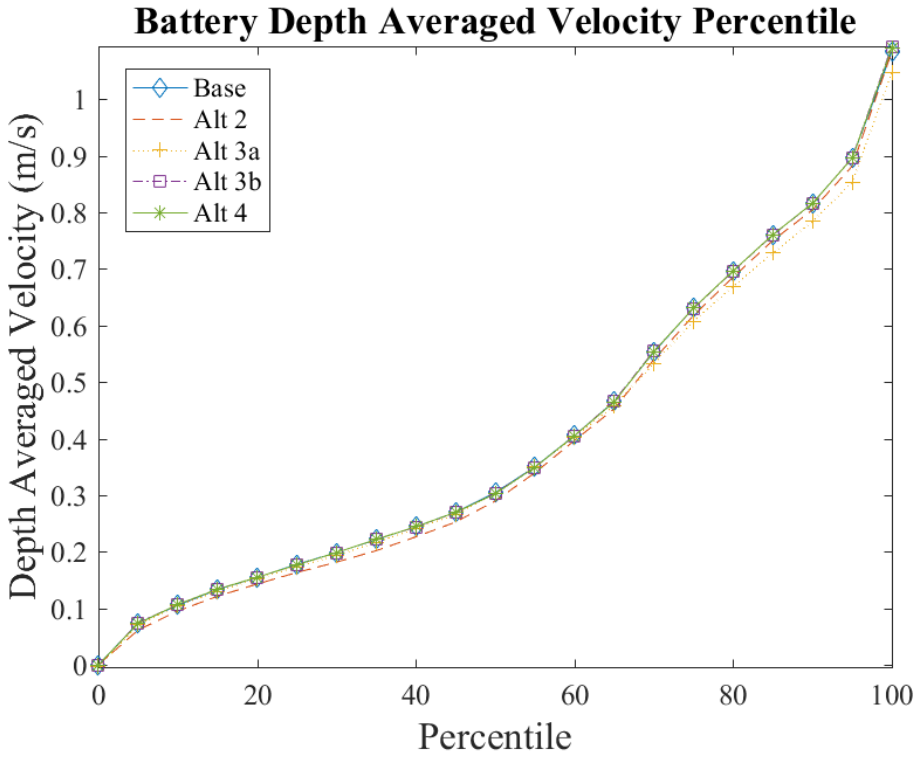


Figure 90: Set 1 Point KLGA Depth Average Velocity Magnitude Percentile

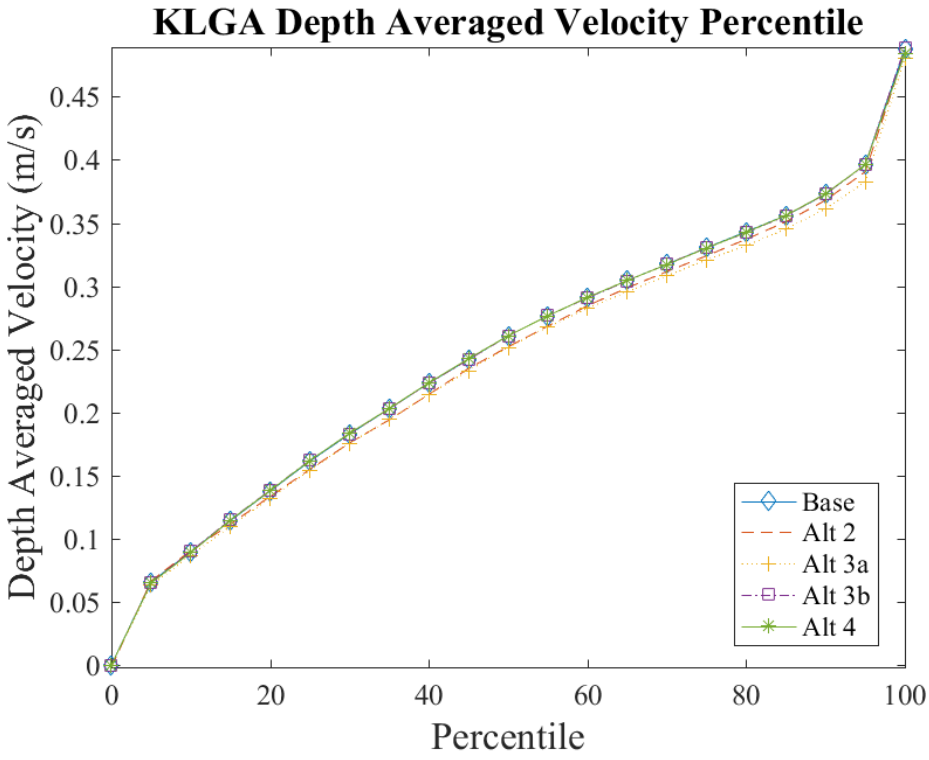


Figure 91: Set 1 Point Mariners Harbor Depth Average Velocity Magnitude Percentile

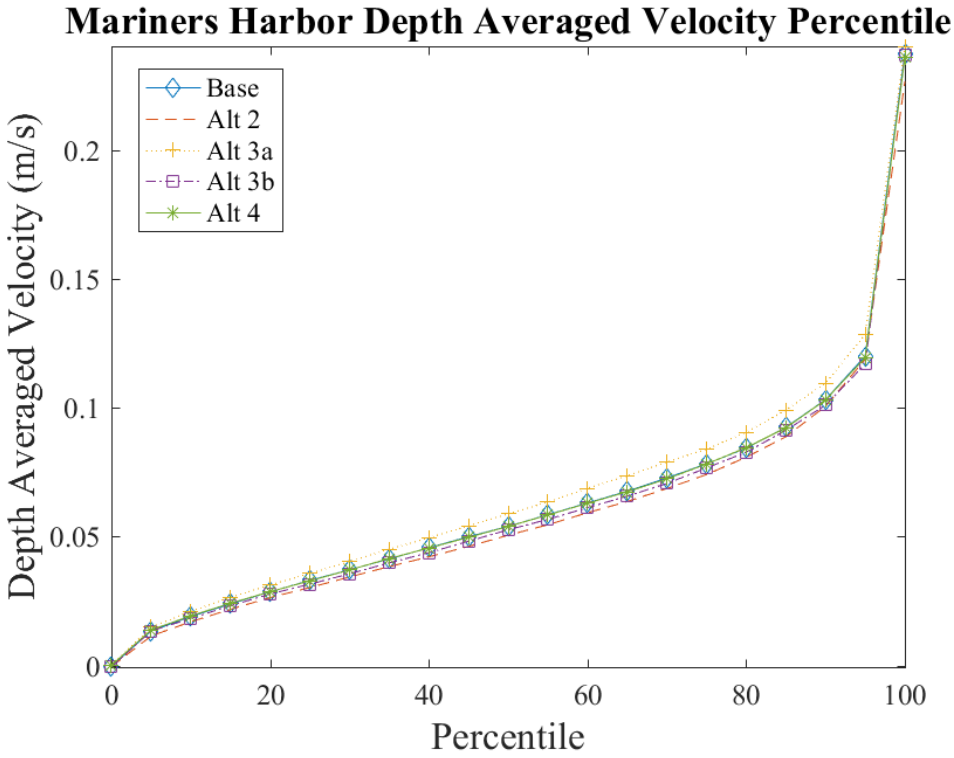


Figure 92: Set 1 Point North Reach Depth Average Velocity Magnitude Percentile

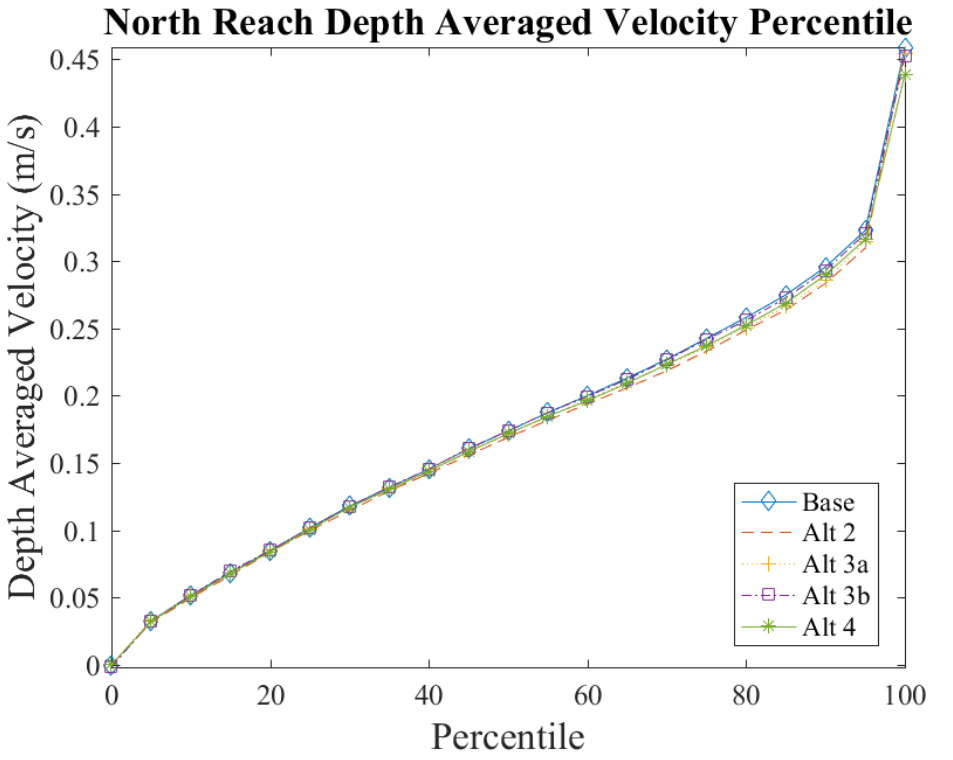
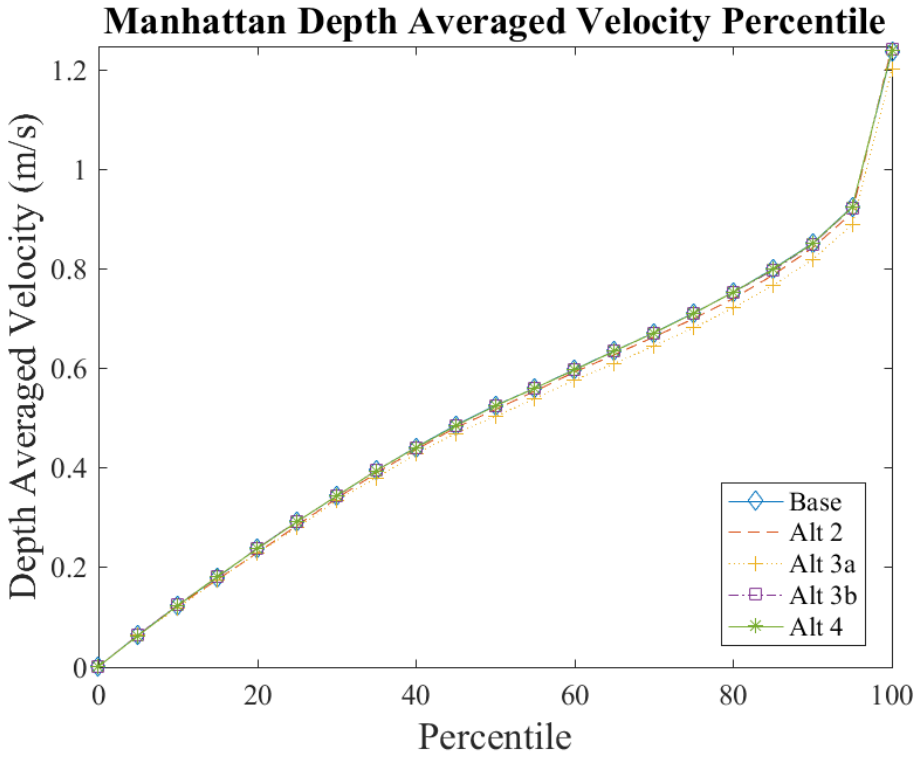


Figure 93: Set 1 Point Manhattan Depth Average Velocity Magnitude Percentile



2.2.2 Spatial Percentile Analysis

The 50th and 75th percentile depth average velocities and salinities throughout the entire mesh were calculated to visualize spatial distributions. This gives more insight and a better understanding into the barrier spatial impacts on the system.

2.2.2.1 Spatial Velocity Percentiles

The spatial depth average velocity percentile figures are depicted in Figure 94 through Figure 109.

Figure 94: Set 1 Depth Averaged Velocity 50th percentile in Lower Bay (Alternative 2 top, Base Bottom)

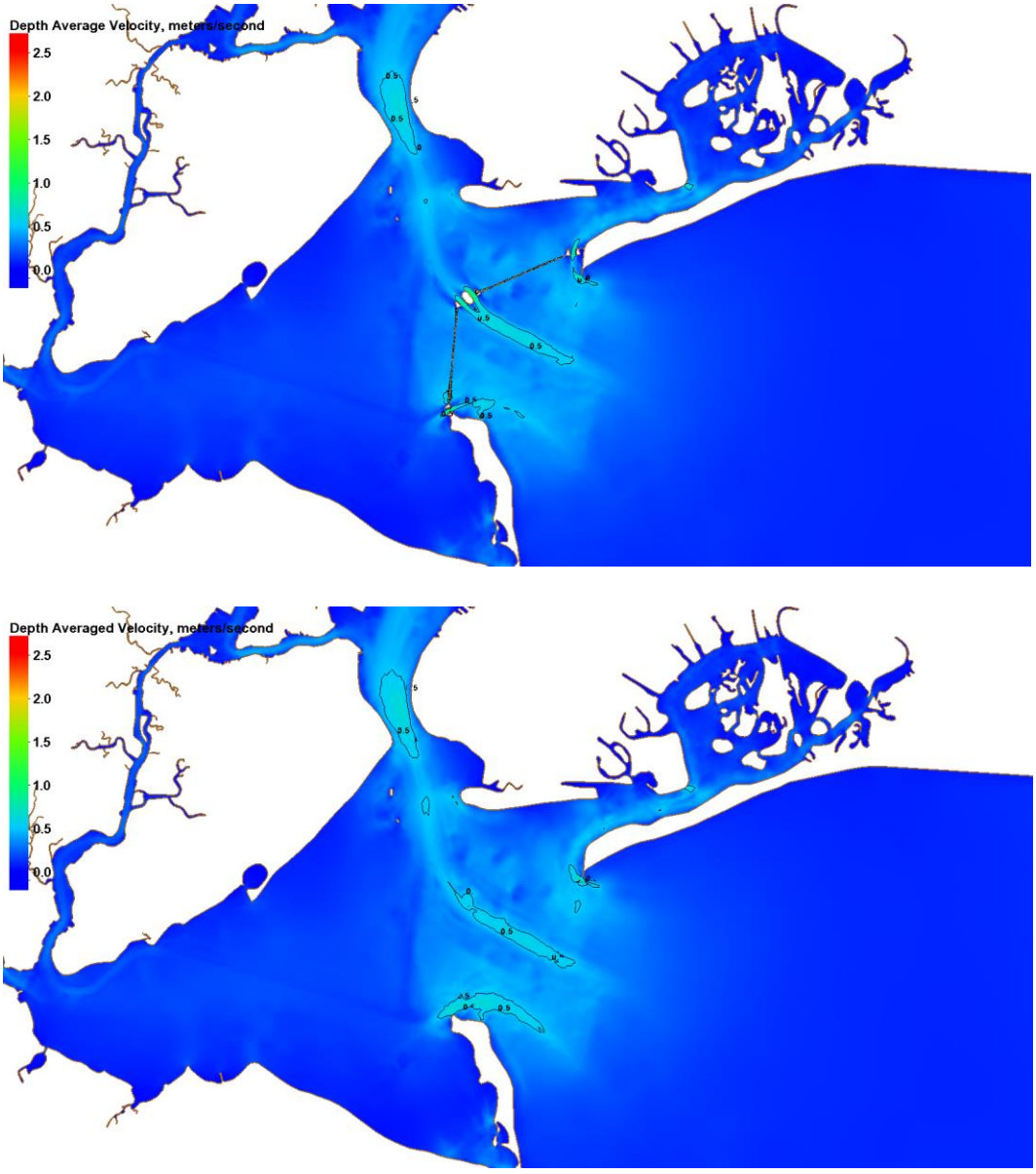


Figure 95: Set 1 Depth Averaged Velocity 75th percentile in Lower Bay (Alternative 2 top, Base Bottom)

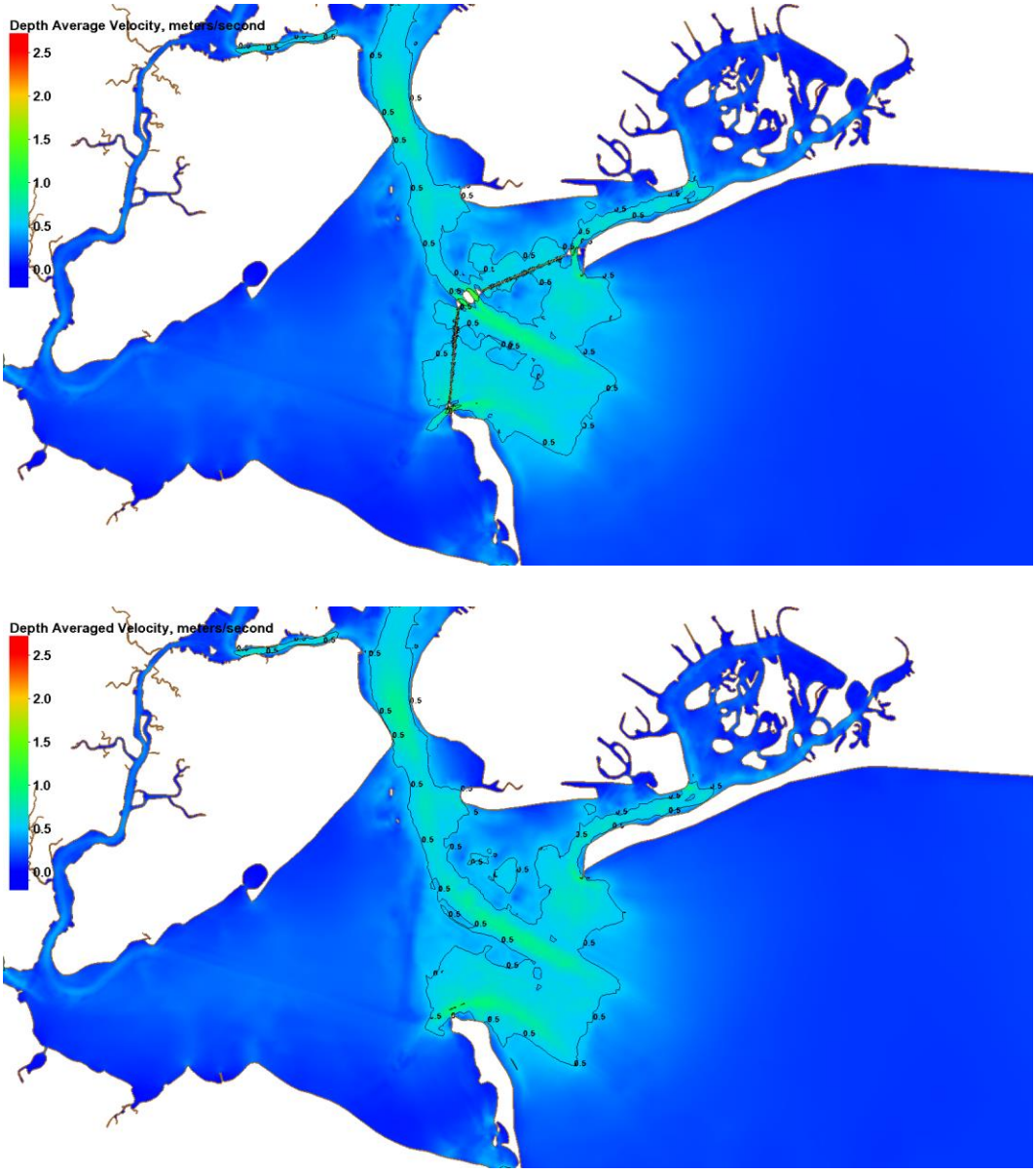


Figure 96: Set 1 Depth Averaged Velocity 50th percentile in Upper Bay (Alternative 2 top, Base Bottom)

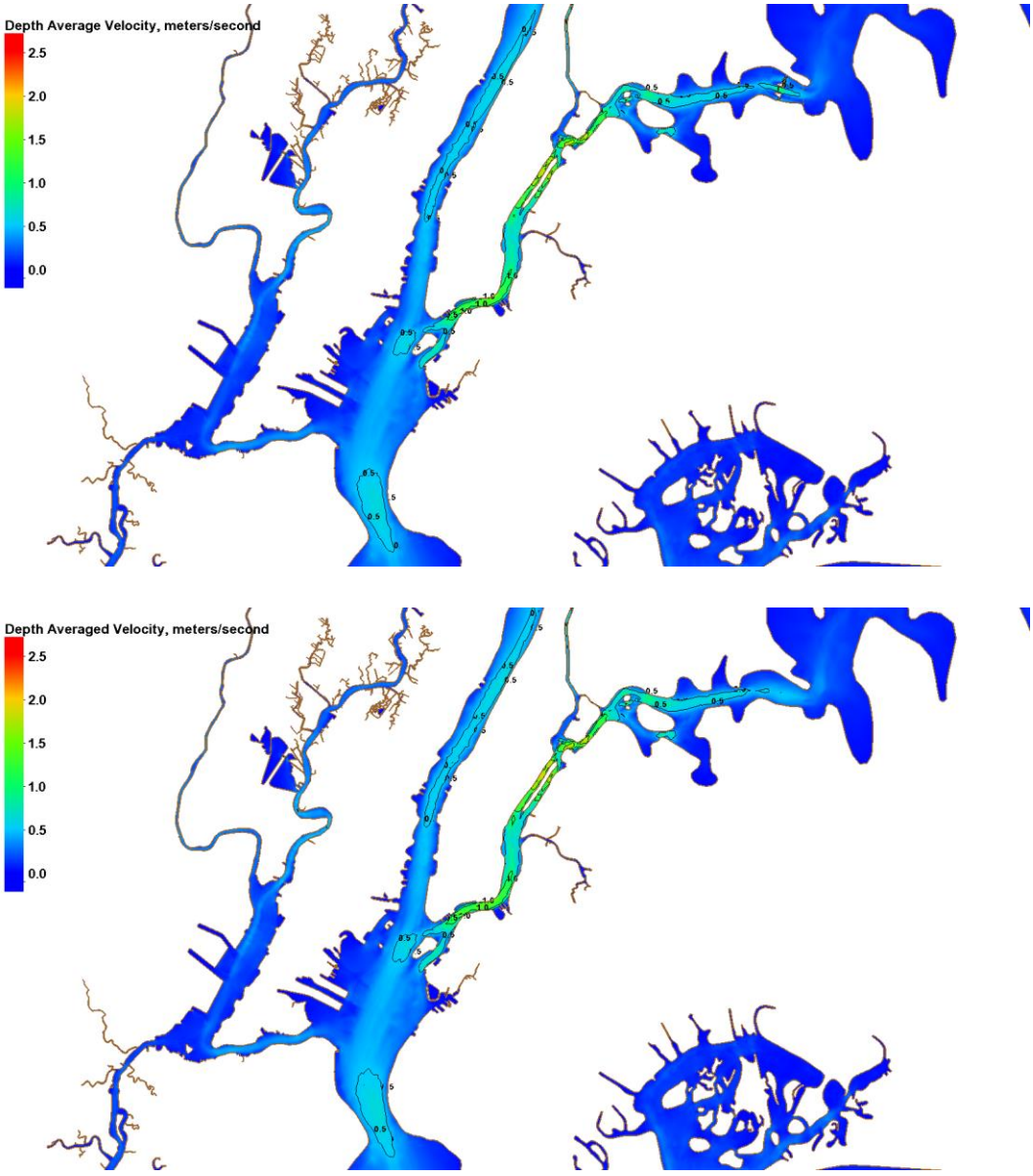


Figure 97: Set 1 Depth Averaged Velocity 75th percentile in Upper Bay (Alternative 2 top, Base Bottom)

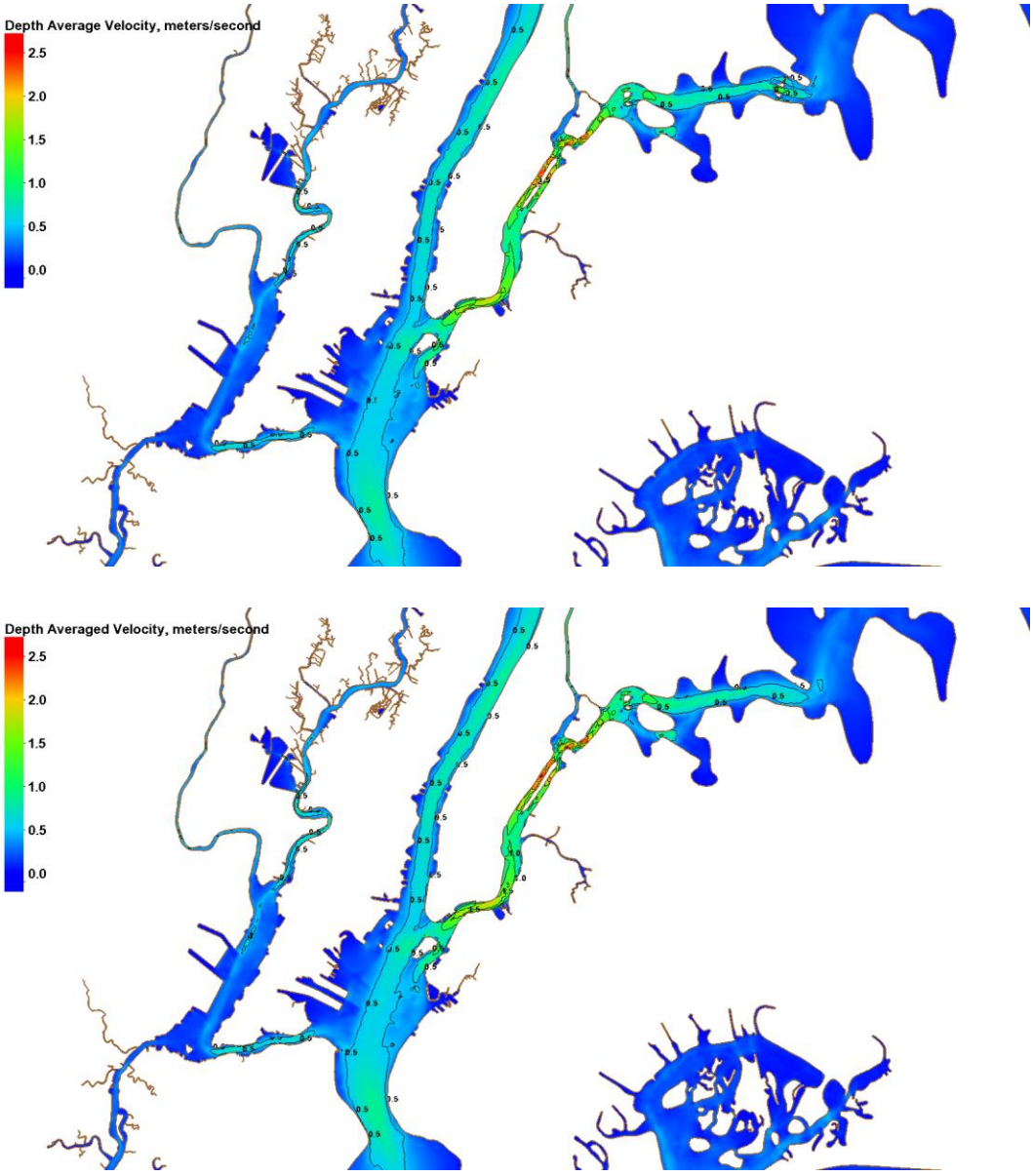


Figure 98: Set 1 Depth Averaged Velocity 50th percentile in Lower Bay (Alternative 3a top, Base Bottom)

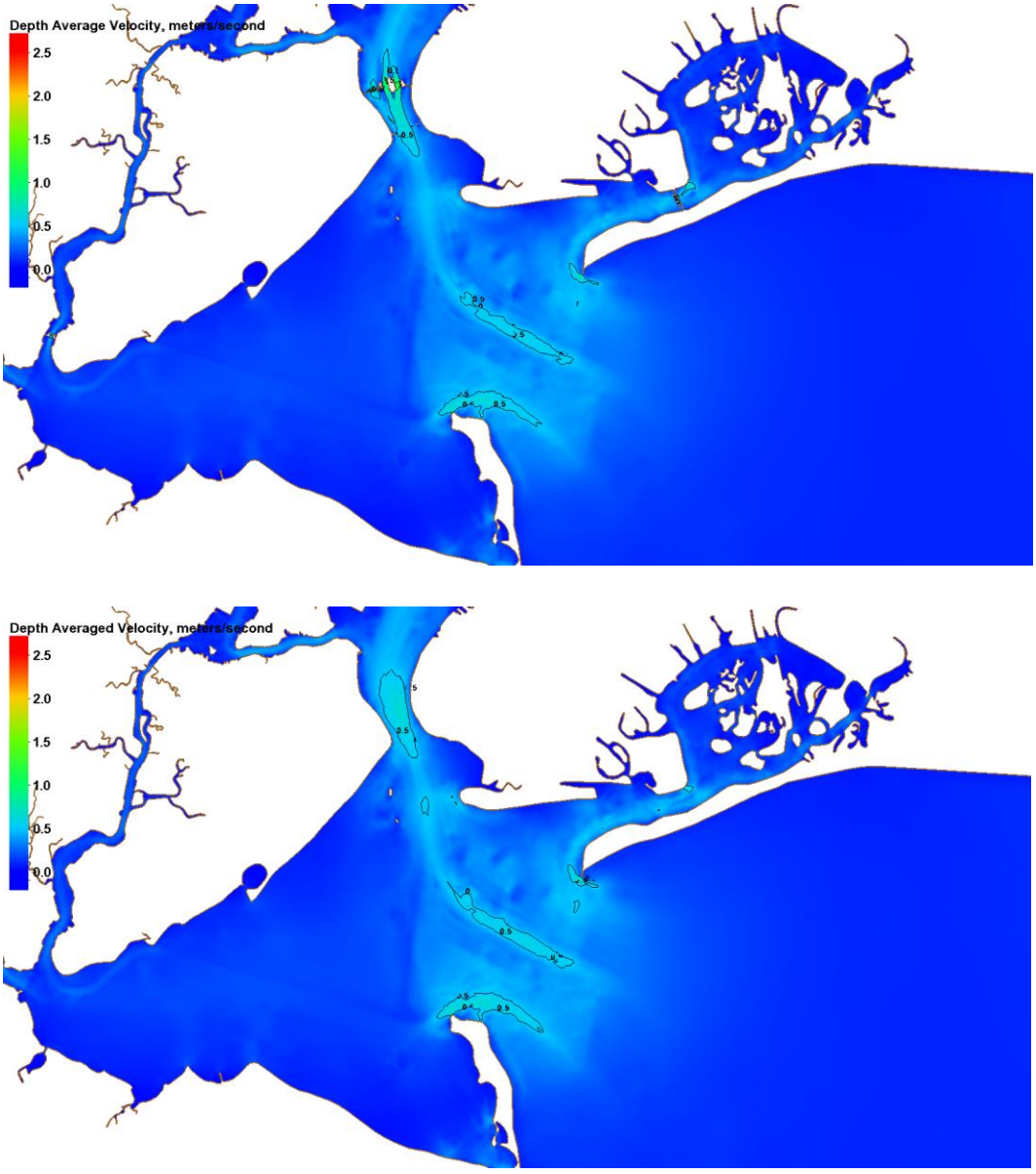


Figure 99: Set 1 Depth Averaged Velocity 75th percentile in Lower Bay (Alternative 3a top, Base Bottom)

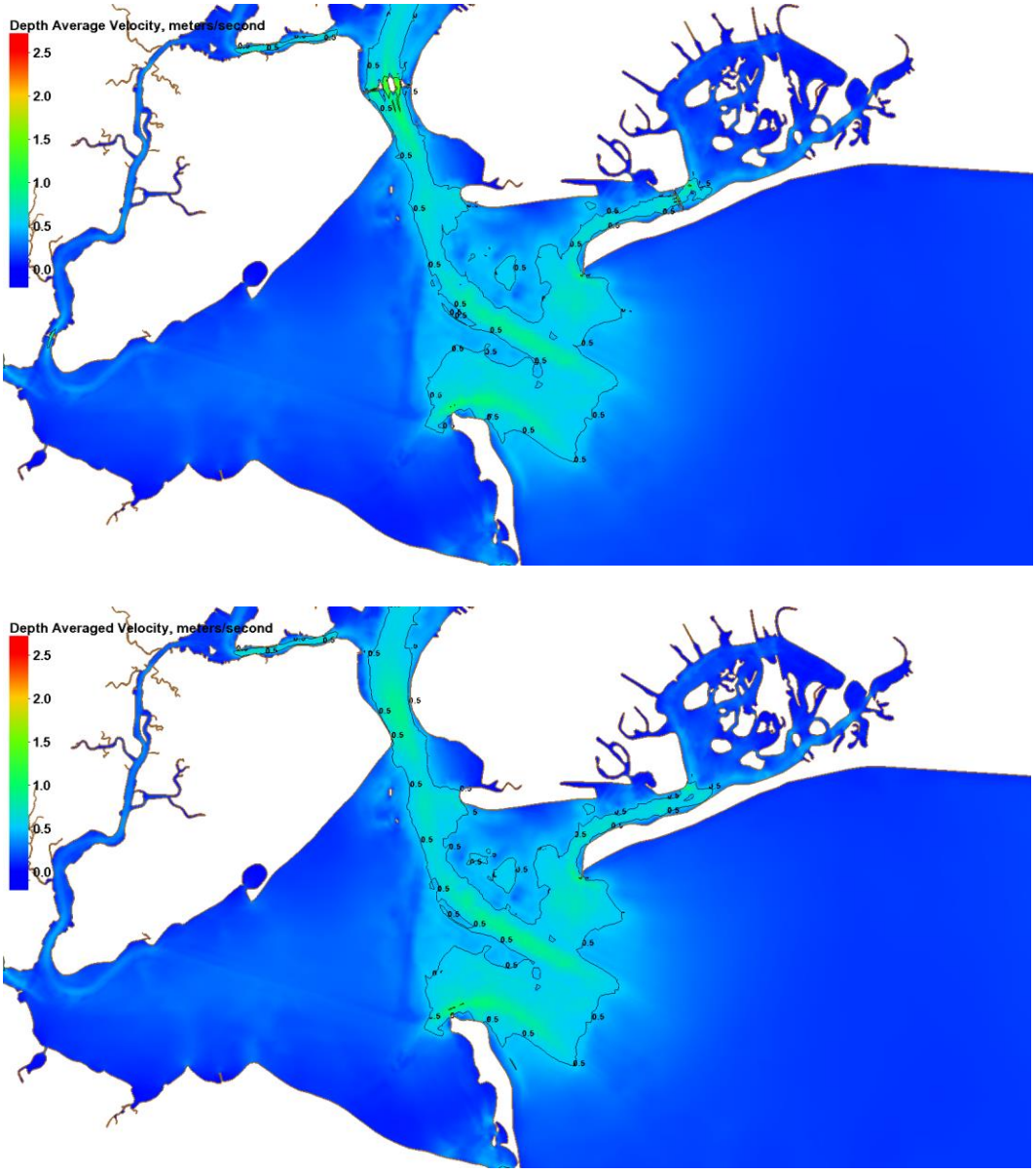


Figure 100: Set 1 Depth Averaged Velocity 50th percentile in Upper Bay (Alternative 3a top, Base Bottom)

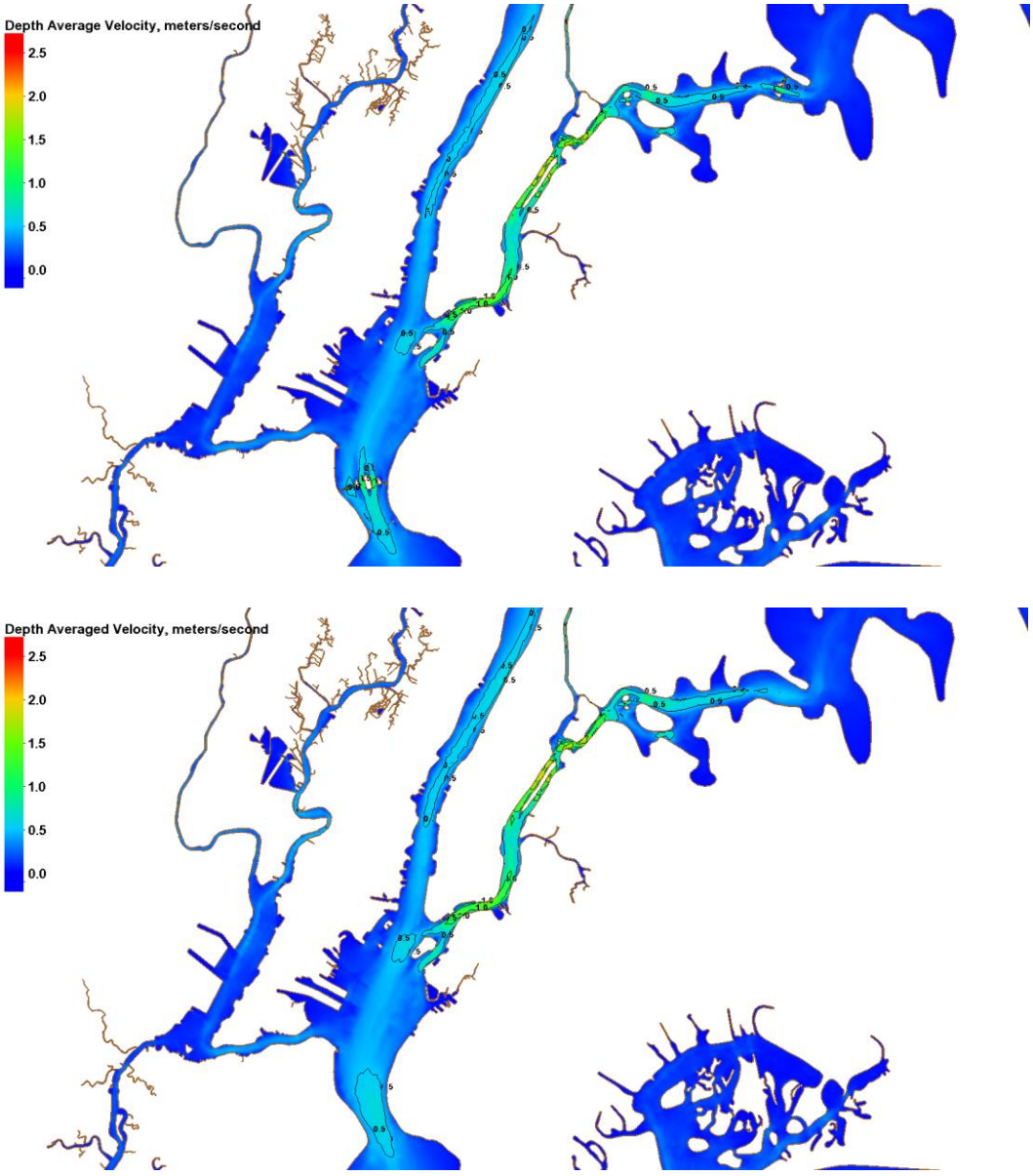


Figure 101: Set 1 Depth Averaged Velocity 75th percentile in Upper Bay (Alternative 3a top, Base Bottom)

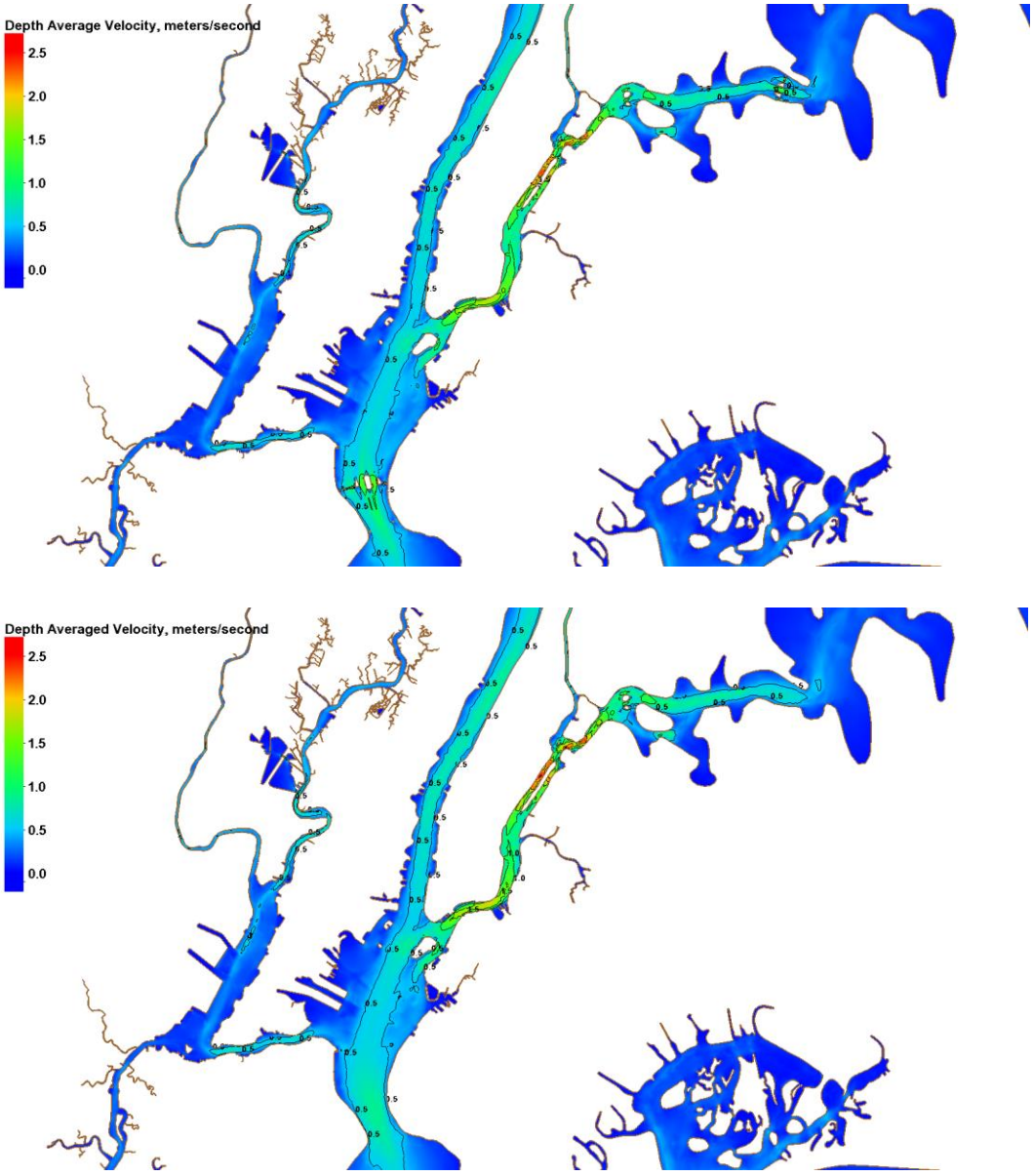


Figure 102: Set 1 Depth Averaged Velocity 50th percentile in Lower Bay (Alternative 3b top, Base Bottom)

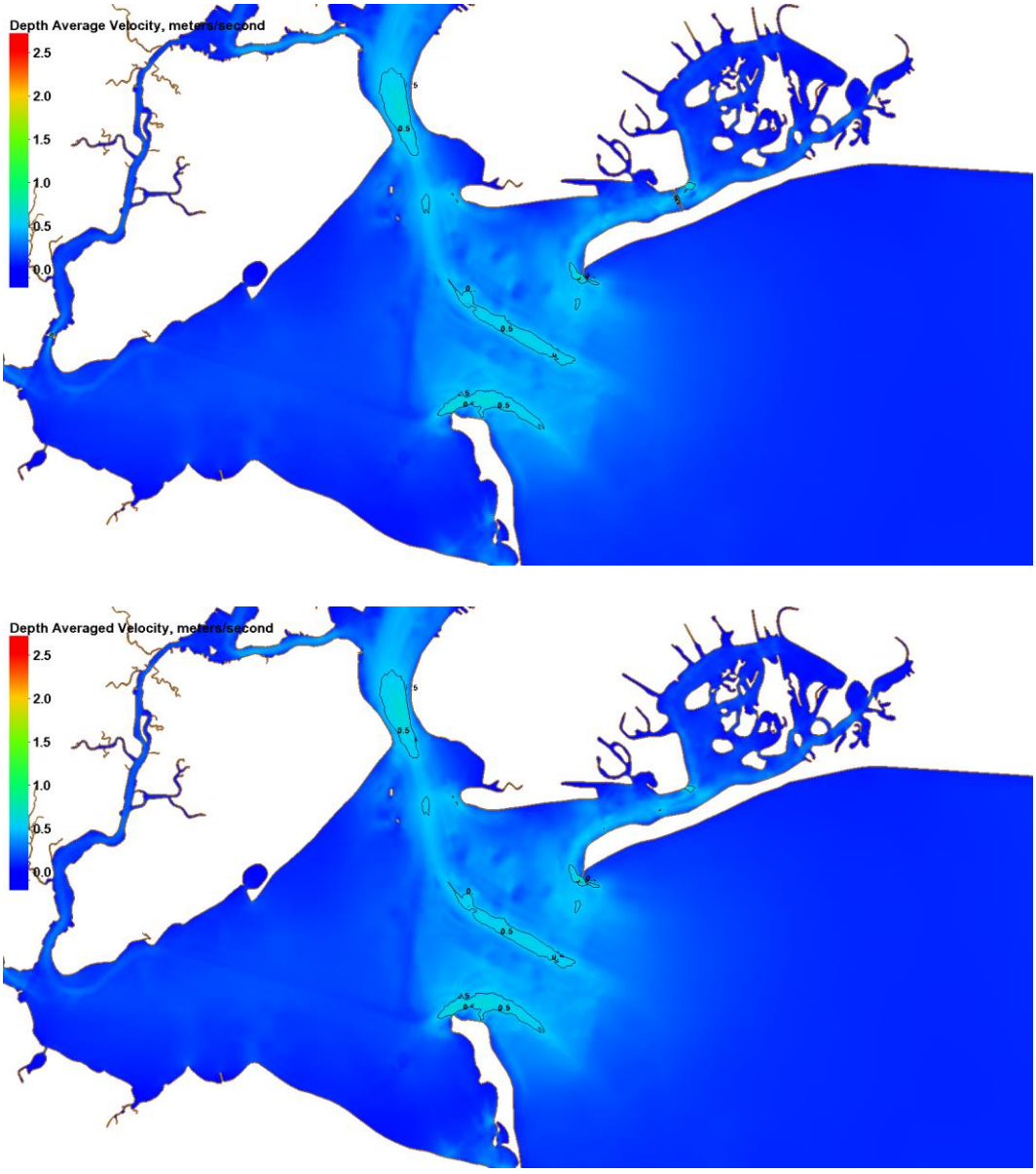


Figure 103: Set 1 Depth Averaged Velocity 75th percentile in Lower Bay (Alternative 3b top, Base Bottom)

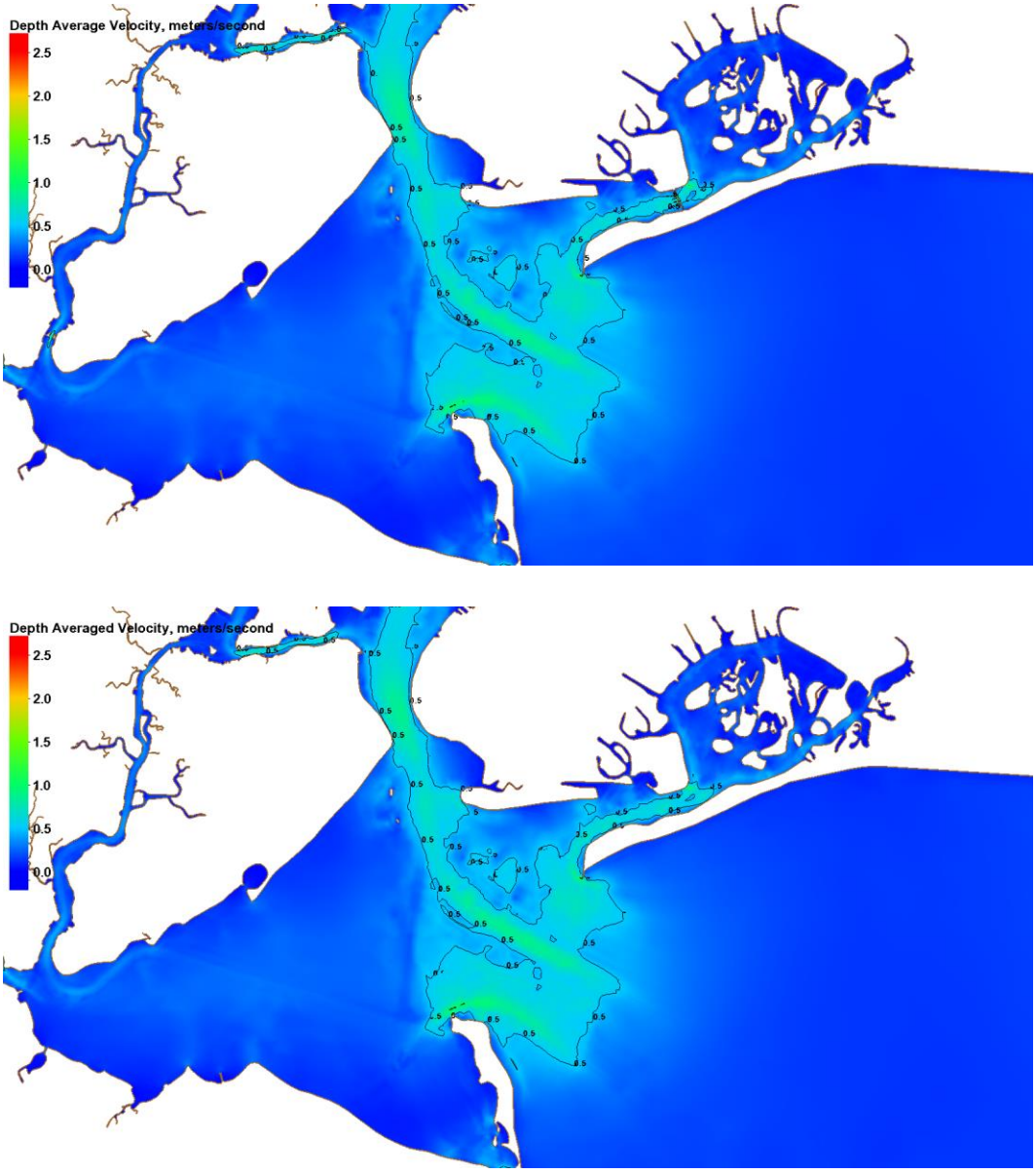


Figure 104: Set 1 Depth Averaged Velocity 50th percentile in Upper Bay (Alternative 3b top, Base Bottom)

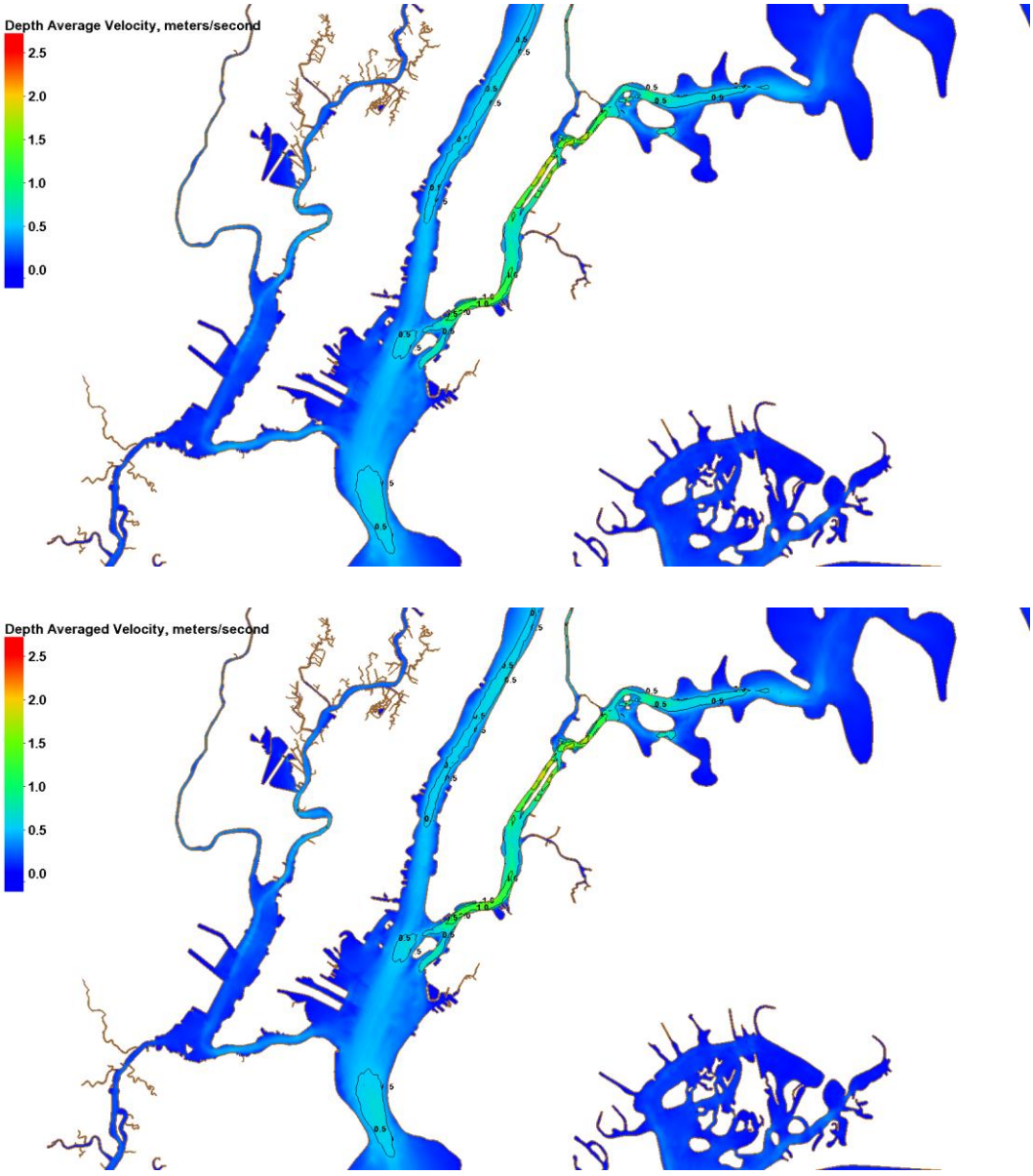


Figure 105: Set 1 Depth Averaged Velocity 75th percentile in Upper Bay (Alternative 3b top, Base Bottom)

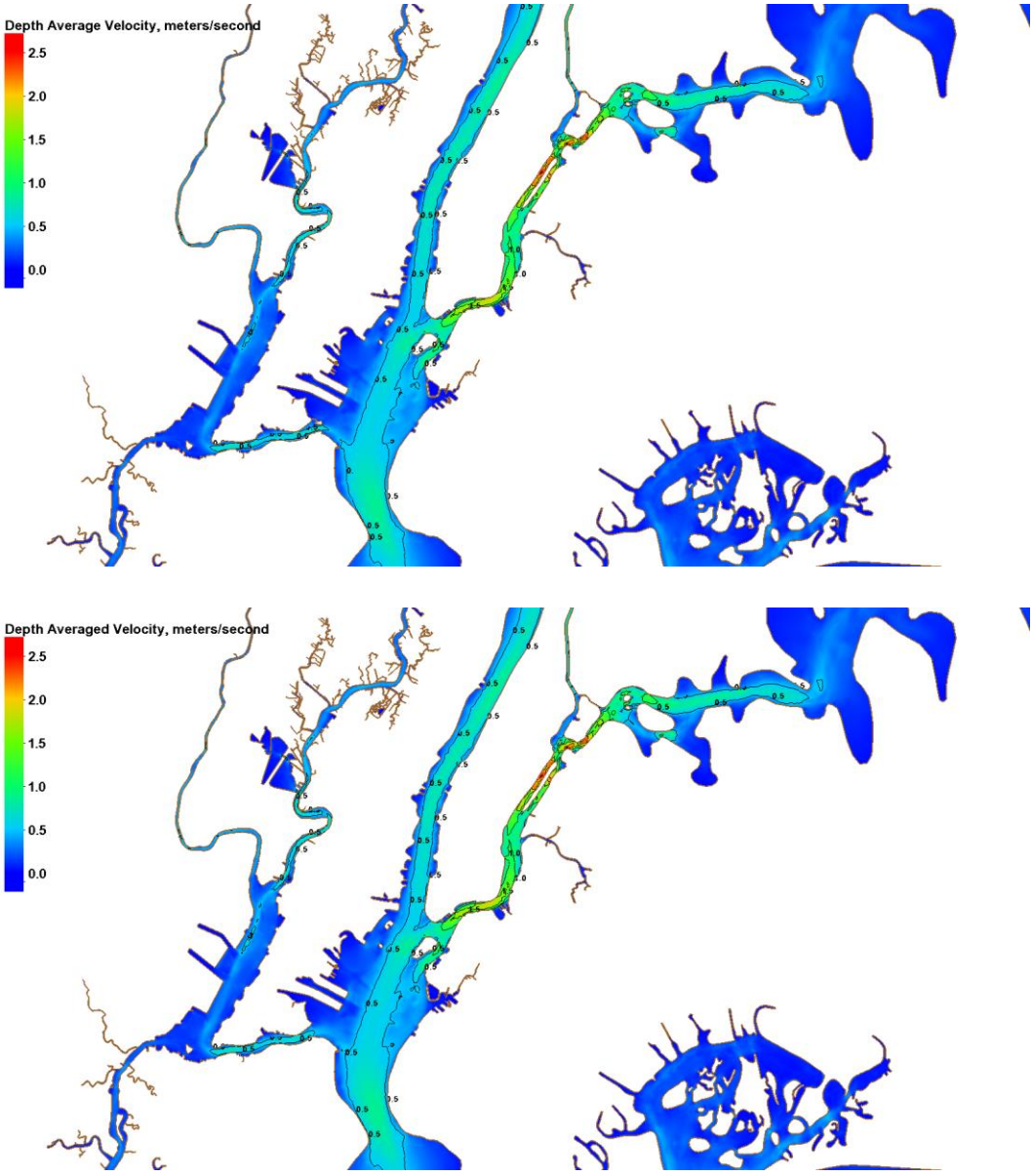


Figure 106: : Set 1 Depth Averaged Velocity 50th percentile in Lower Bay (Alternative 4 top, Base Bottom)

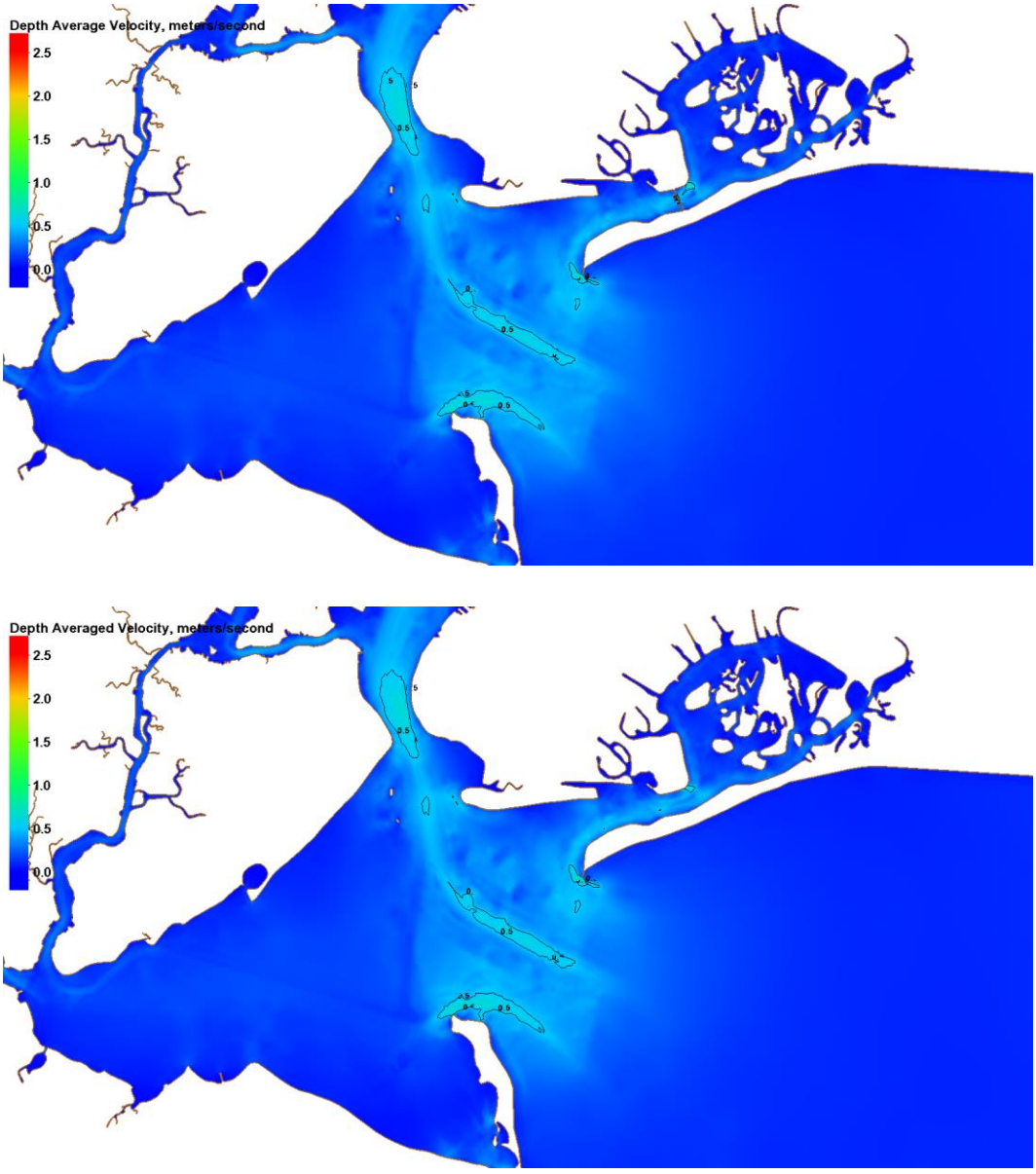


Figure 107: Set 1 Depth Averaged Velocity 75th percentile in Lower Bay (Alternative 4 top, Base Bottom)

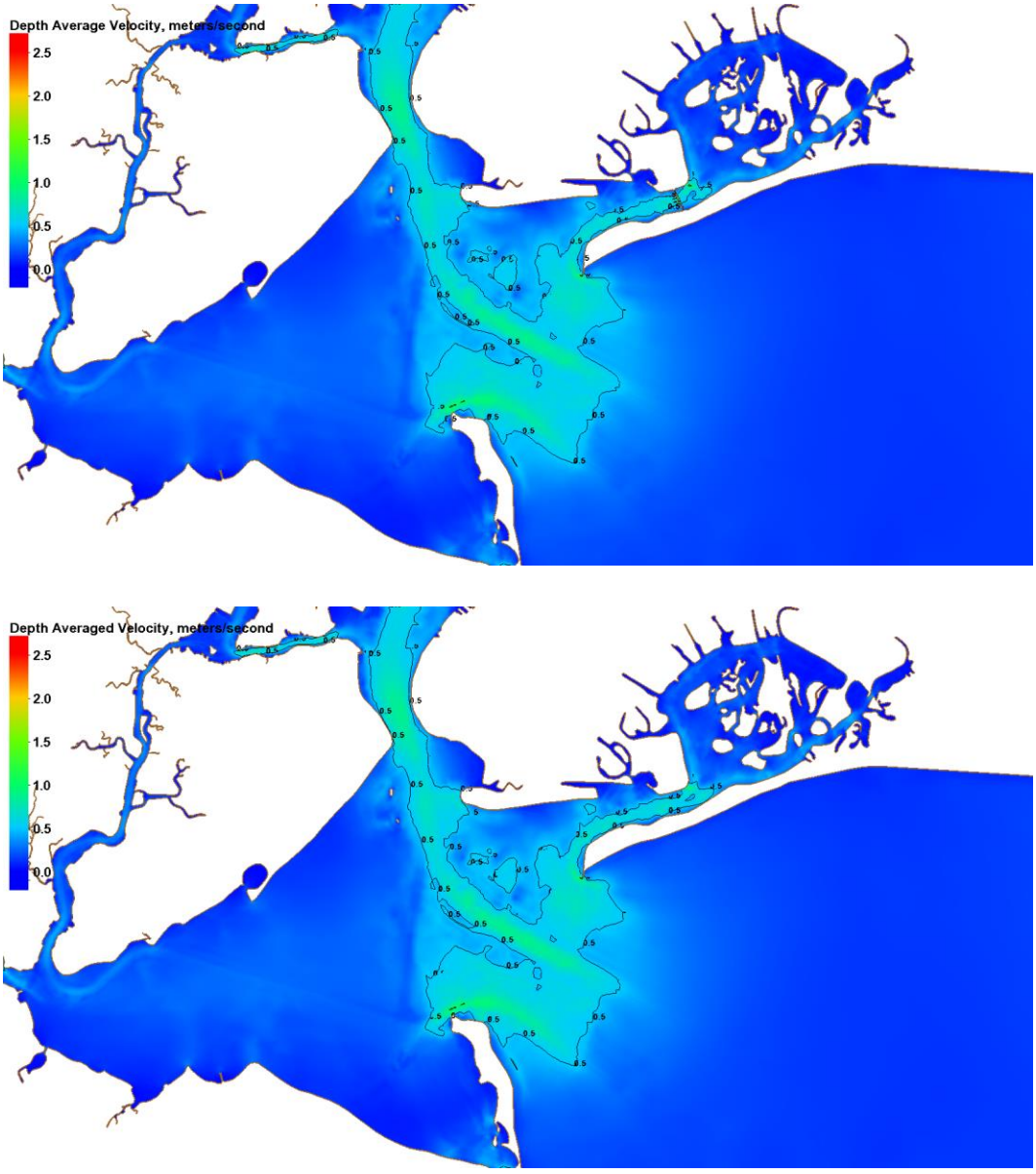


Figure 108: Set 1 Depth Averaged Velocity 50th percentile in Upper Bay (Alternative 4 top, Base Bottom)

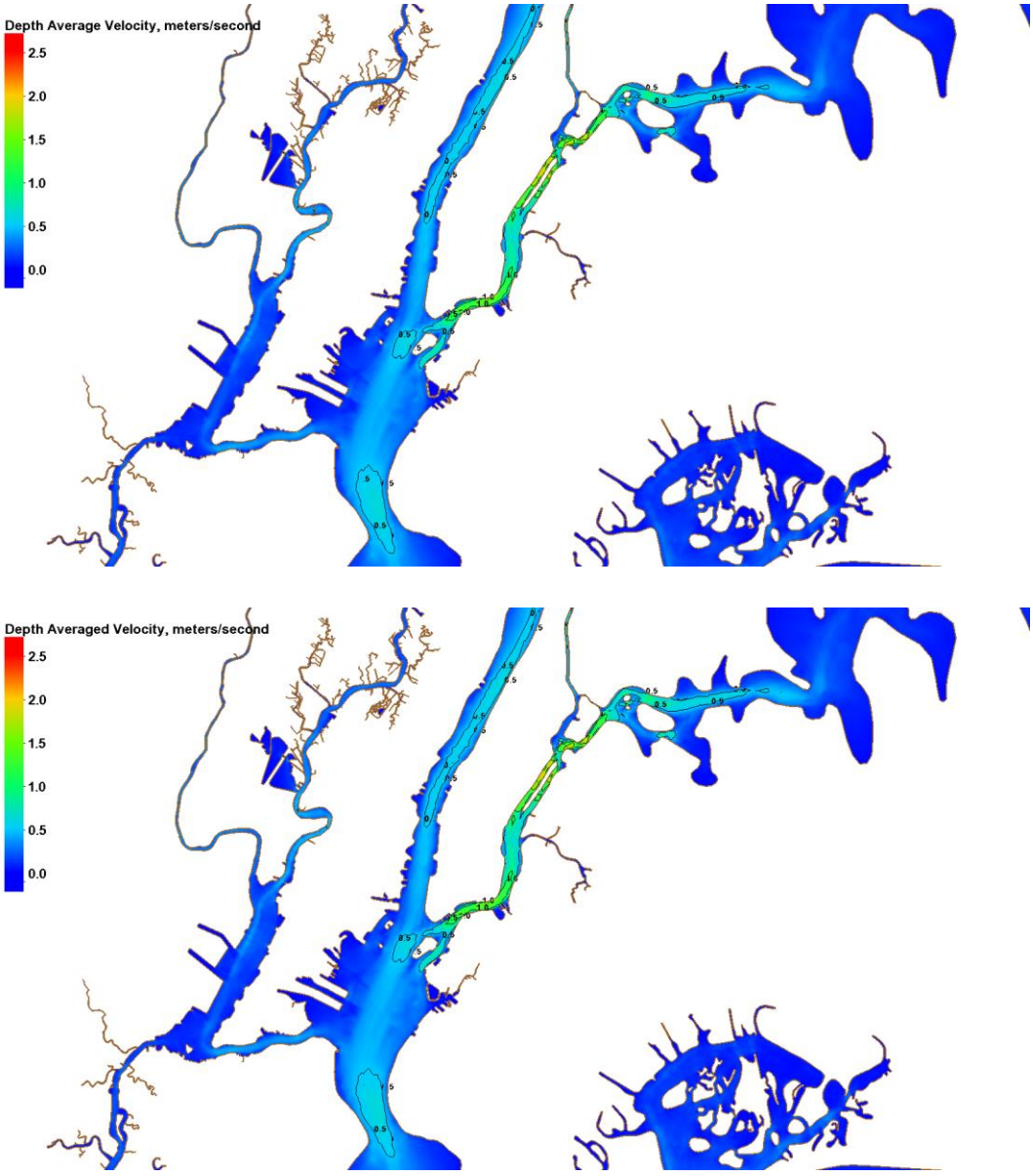
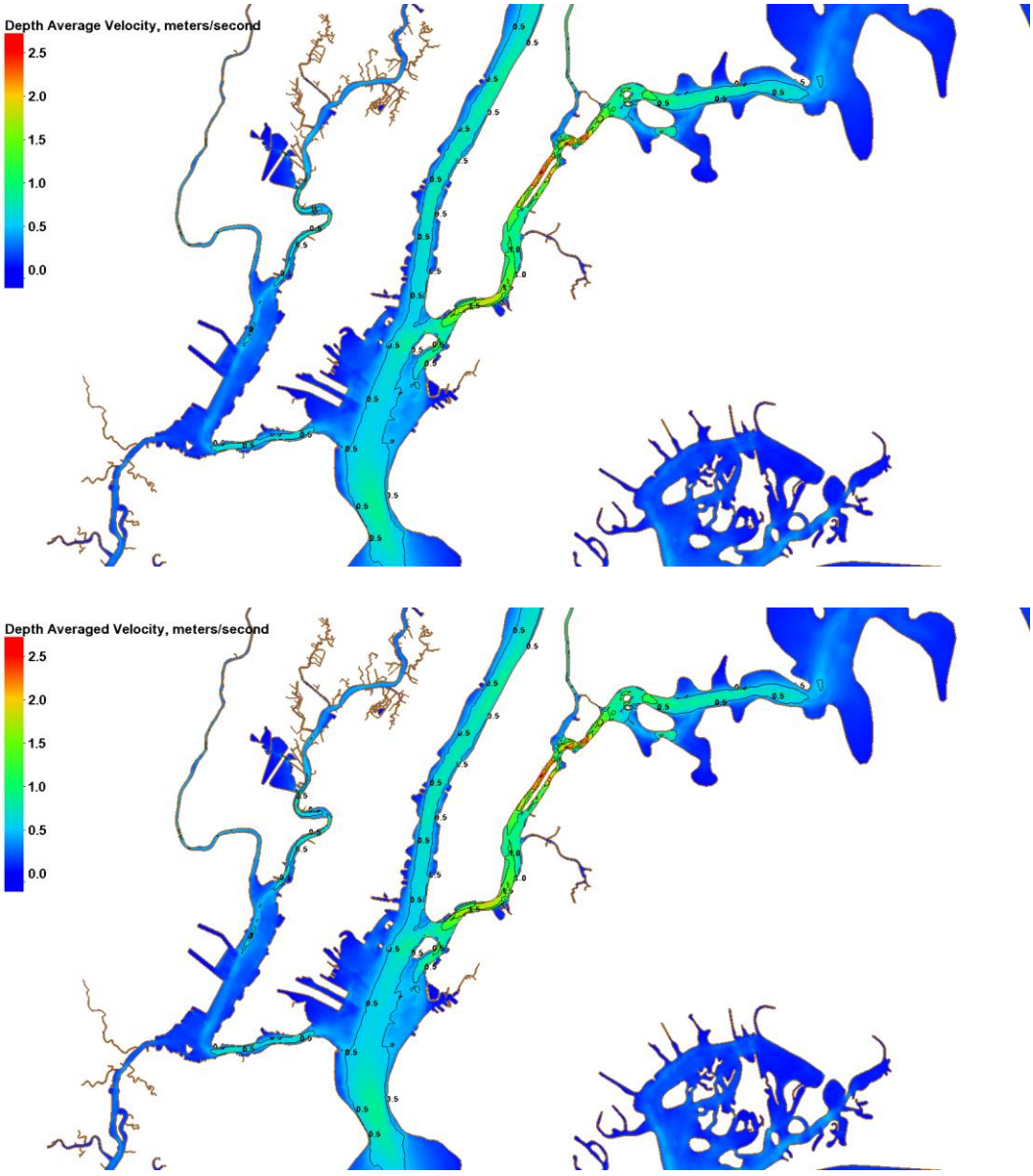


Figure 109: Set 1 Depth Averaged Velocity 75th percentile in Upper Bay (Alternative 4 top, Base Bottom)



2.2.2.2 Spatial Salinity Percentiles

The spatial depth average velocity percentile figures are depicted in Figure 110 through Figure 125.

Figure 110: Set 1 Bottom Salinity 50th percentile in Lower Bay (Alternative 2 top, Base Bottom)

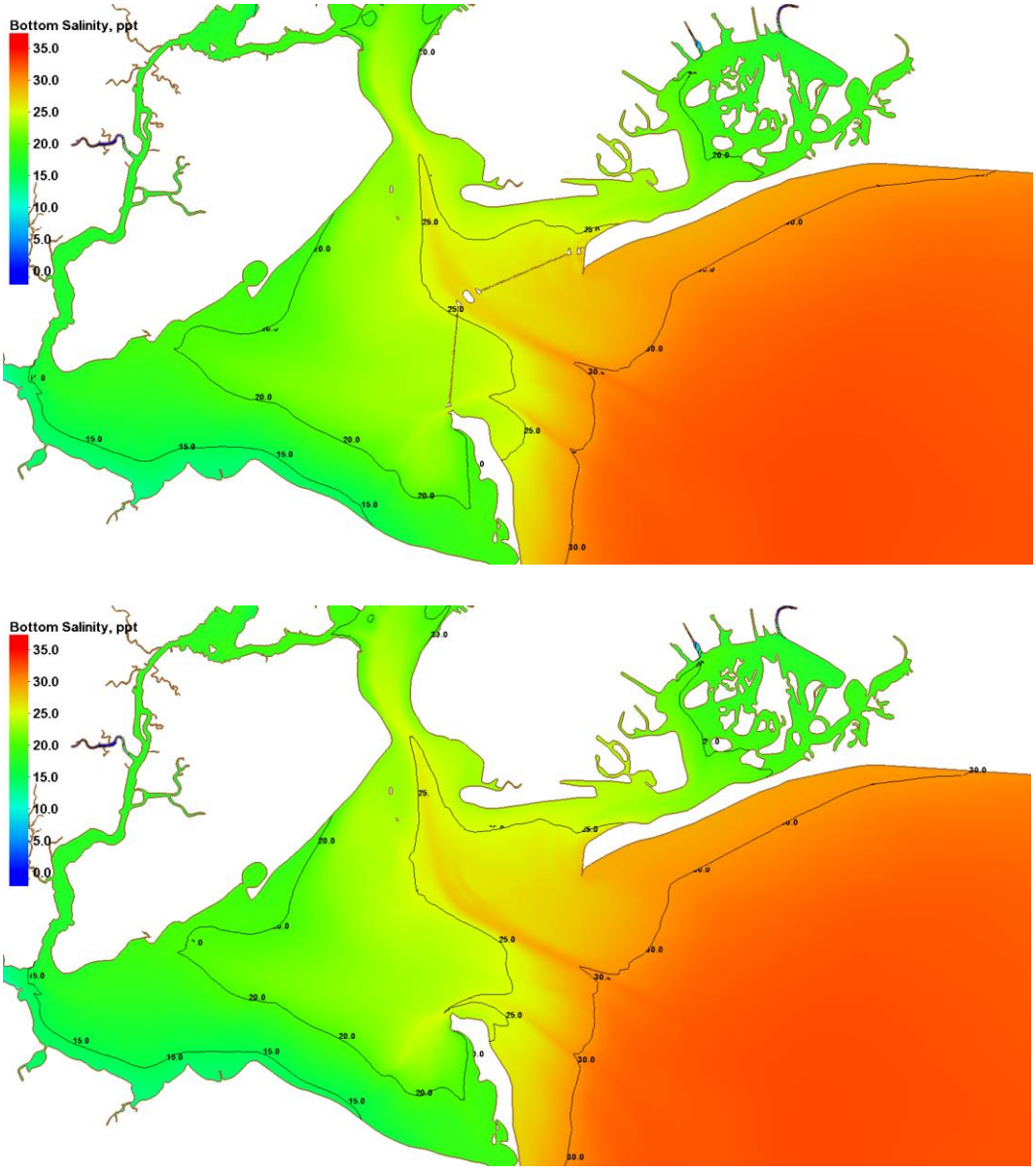


Figure 111: Set 1 Bottom Salinity 75th percentile in Lower Bay (Alternative 2 top, Base Bottom)

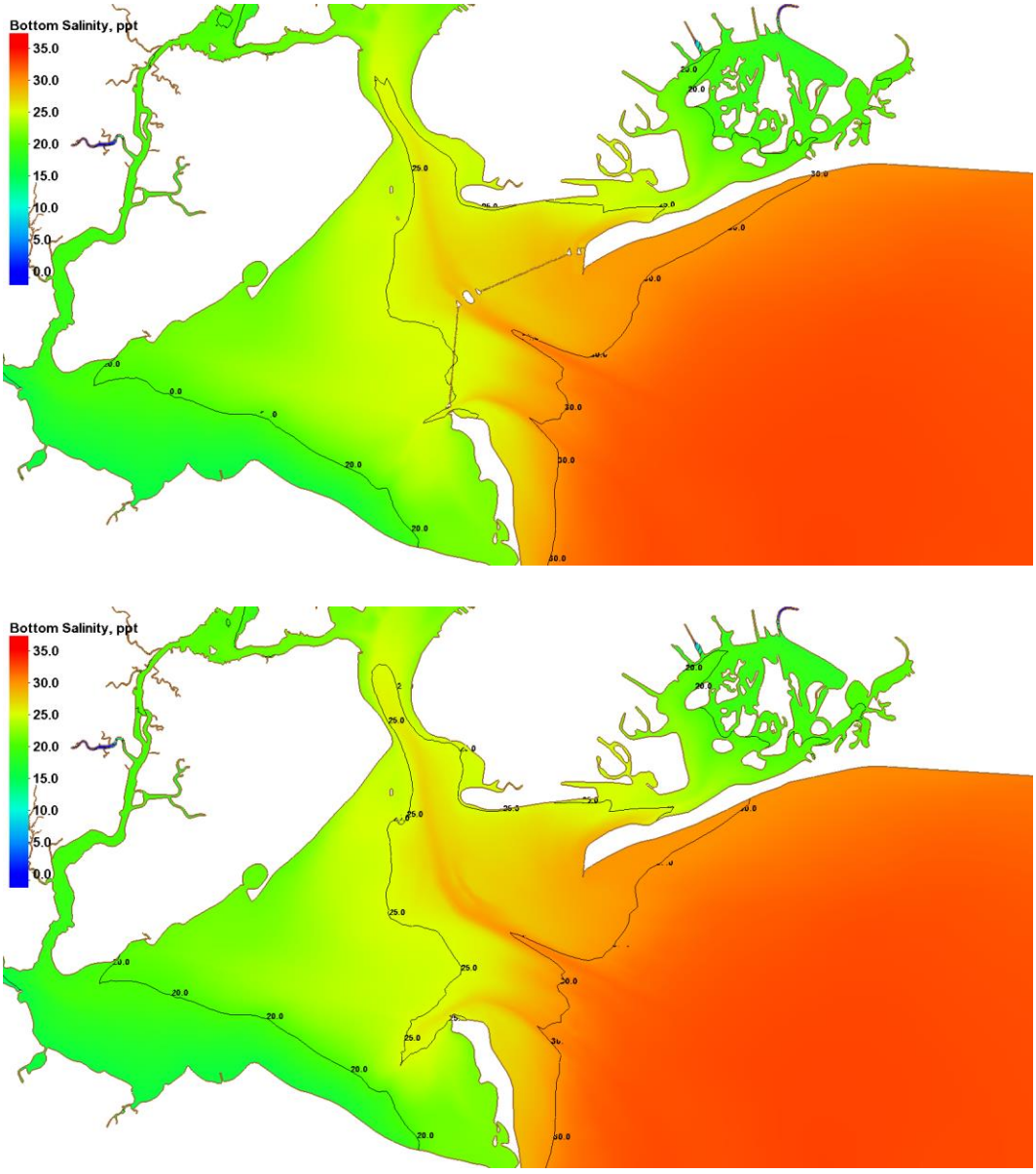


Figure 112: Set 1 Bottom Salinity 50th percentile in Upper Bay (Alternative 2 top, Base Bottom)

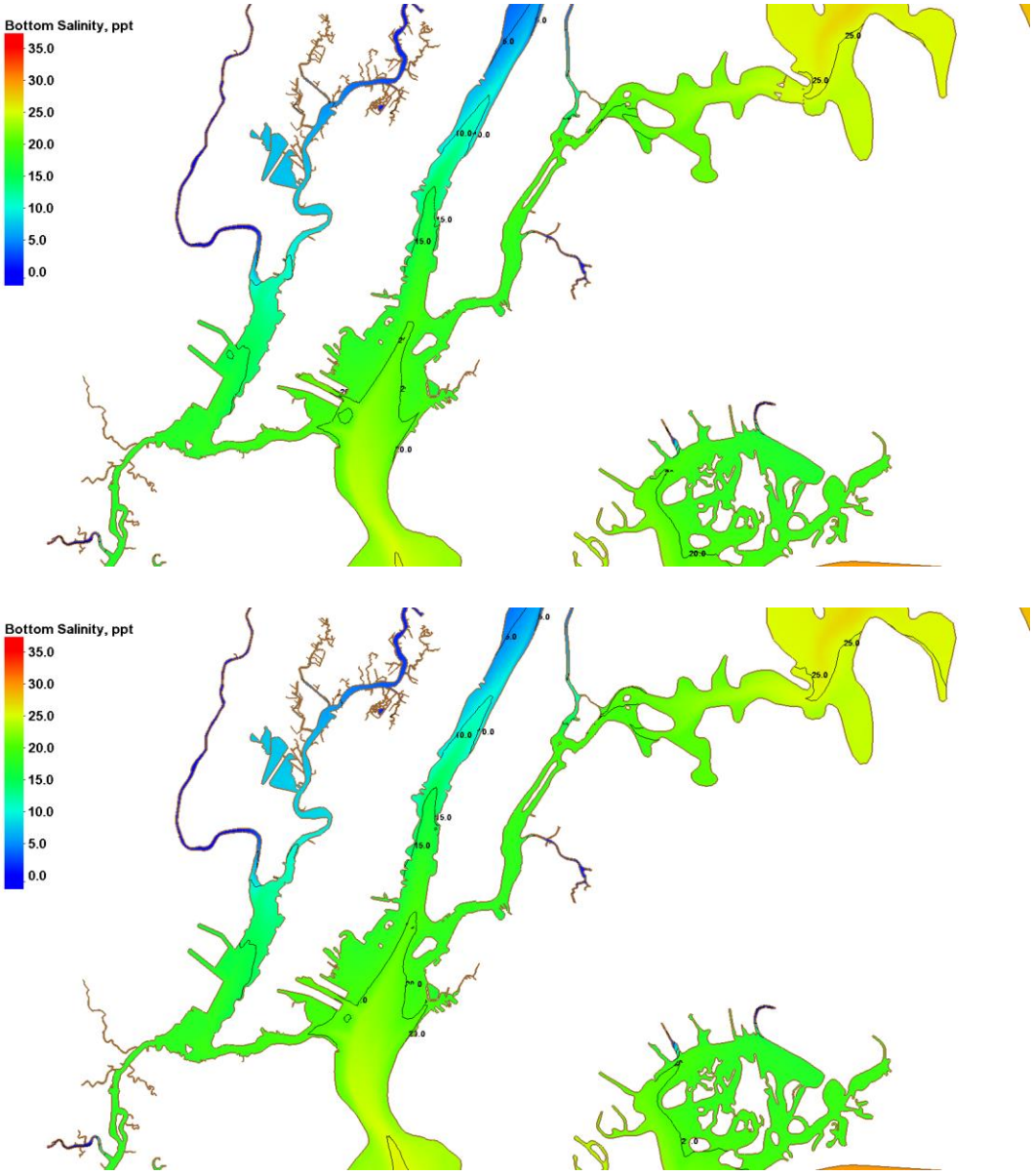


Figure 113: Set 1 Bottom Salinity 75th percentile in Upper Bay (Alternative 2 top, Base Bottom)

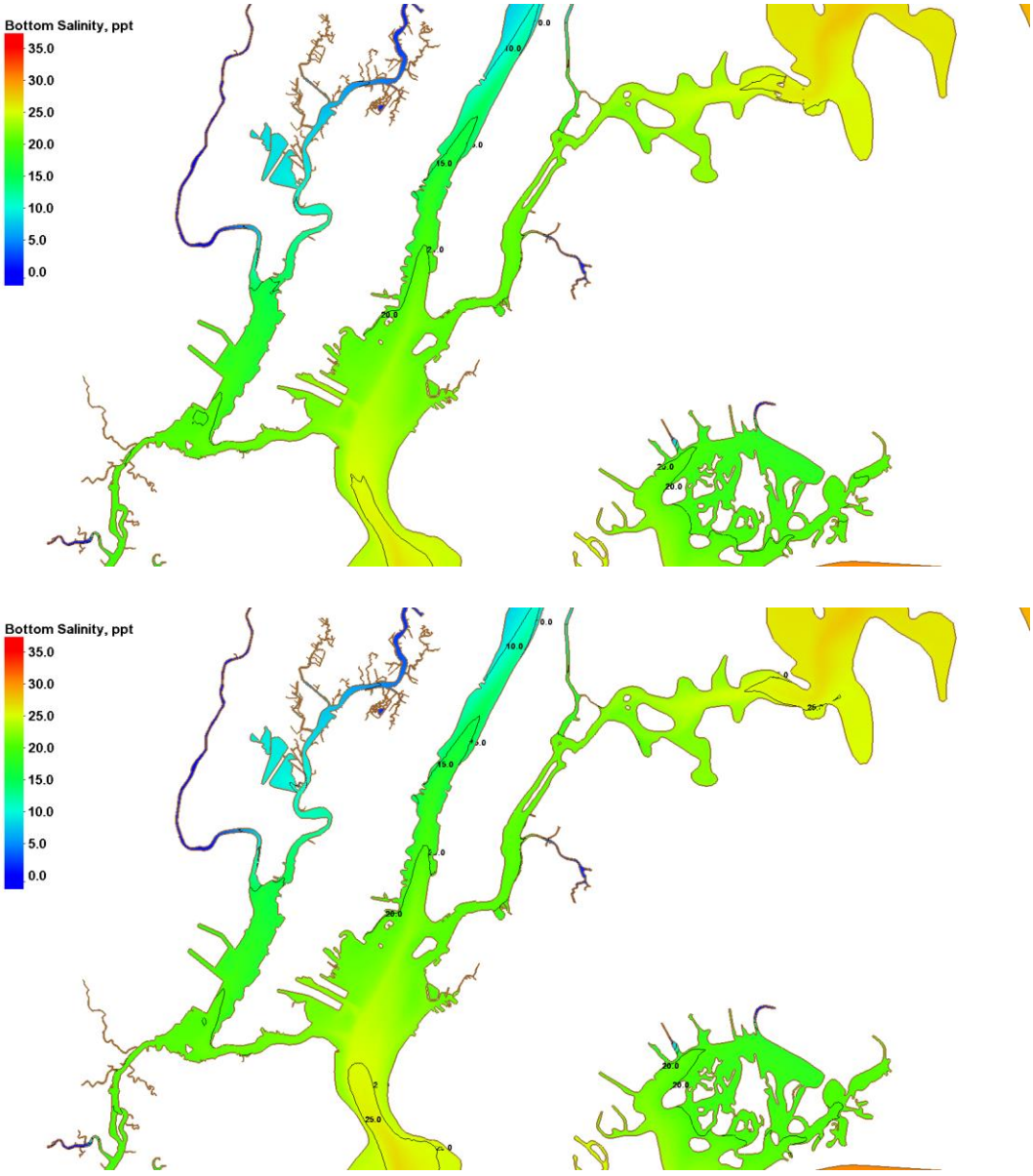


Figure 114: Set 1 Bottom Salinity 50th percentile in Lower Bay (Alternative 3a top, Base Bottom)

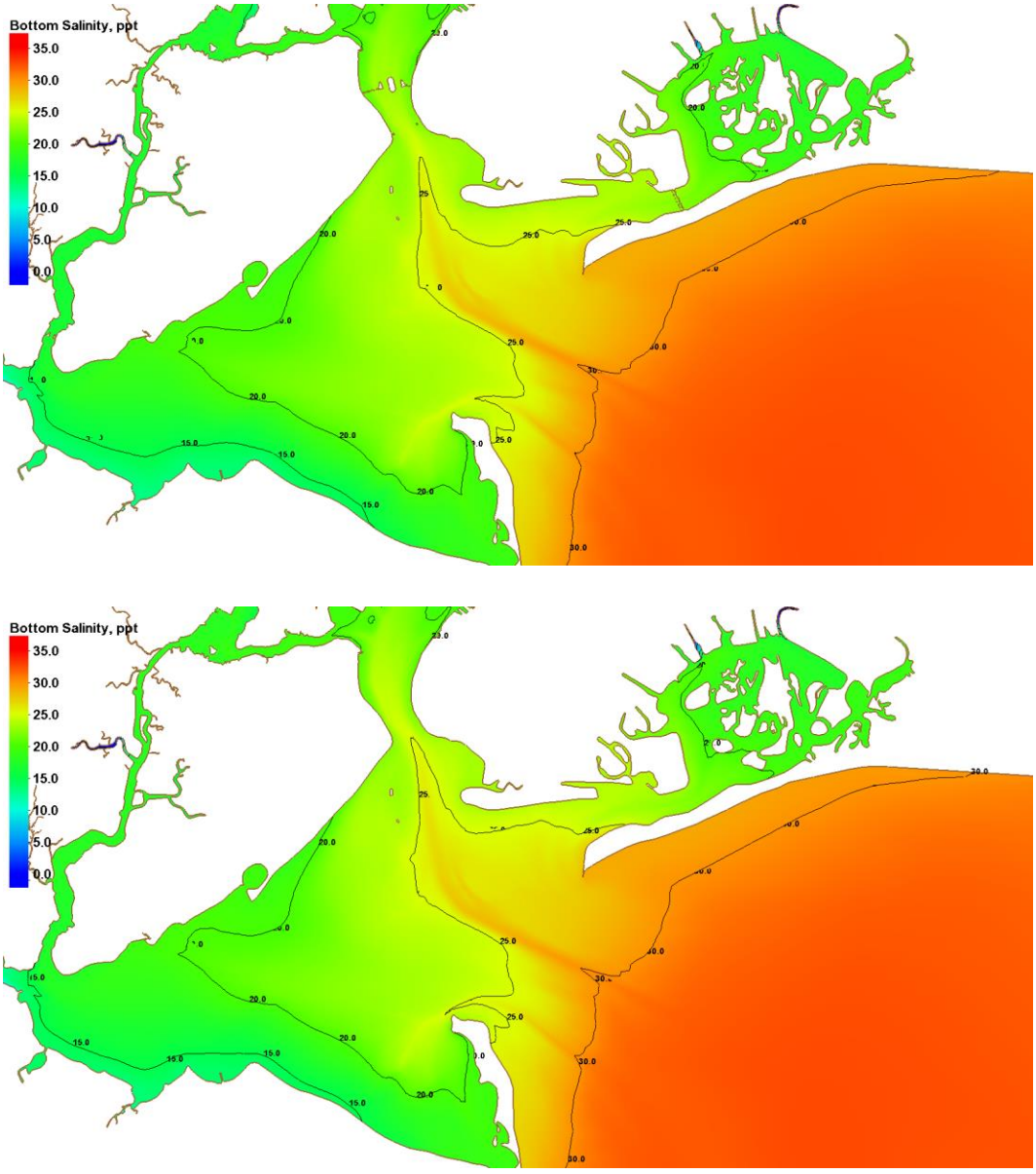


Figure 115: Set 1 Bottom Salinity 75th percentile in Lower Bay (Alternative 3a top, Base Bottom)

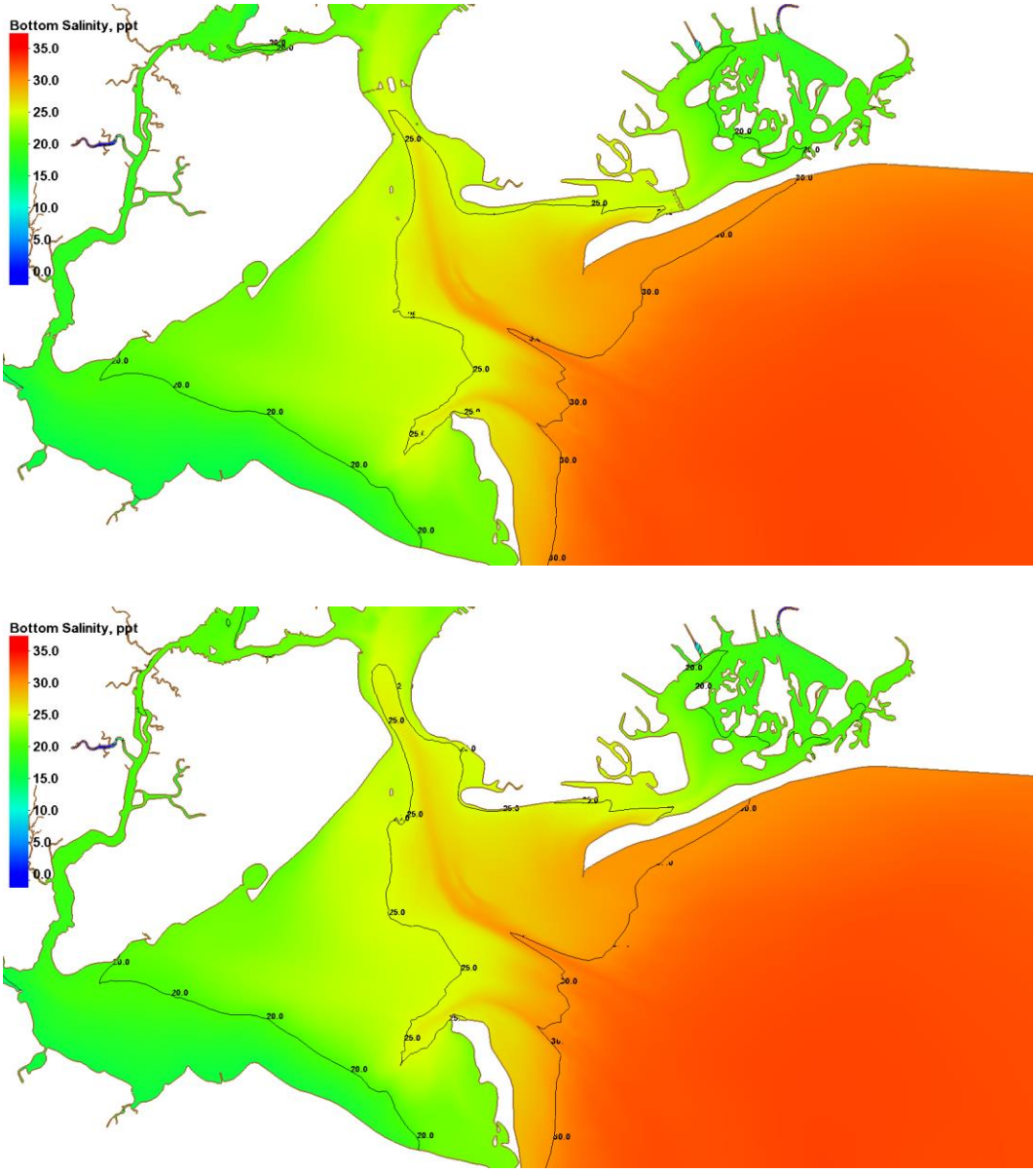


Figure 116: Set 1 Bottom Salinity 50th percentile in Upper Bay (Alternative 3a top, Base Bottom)

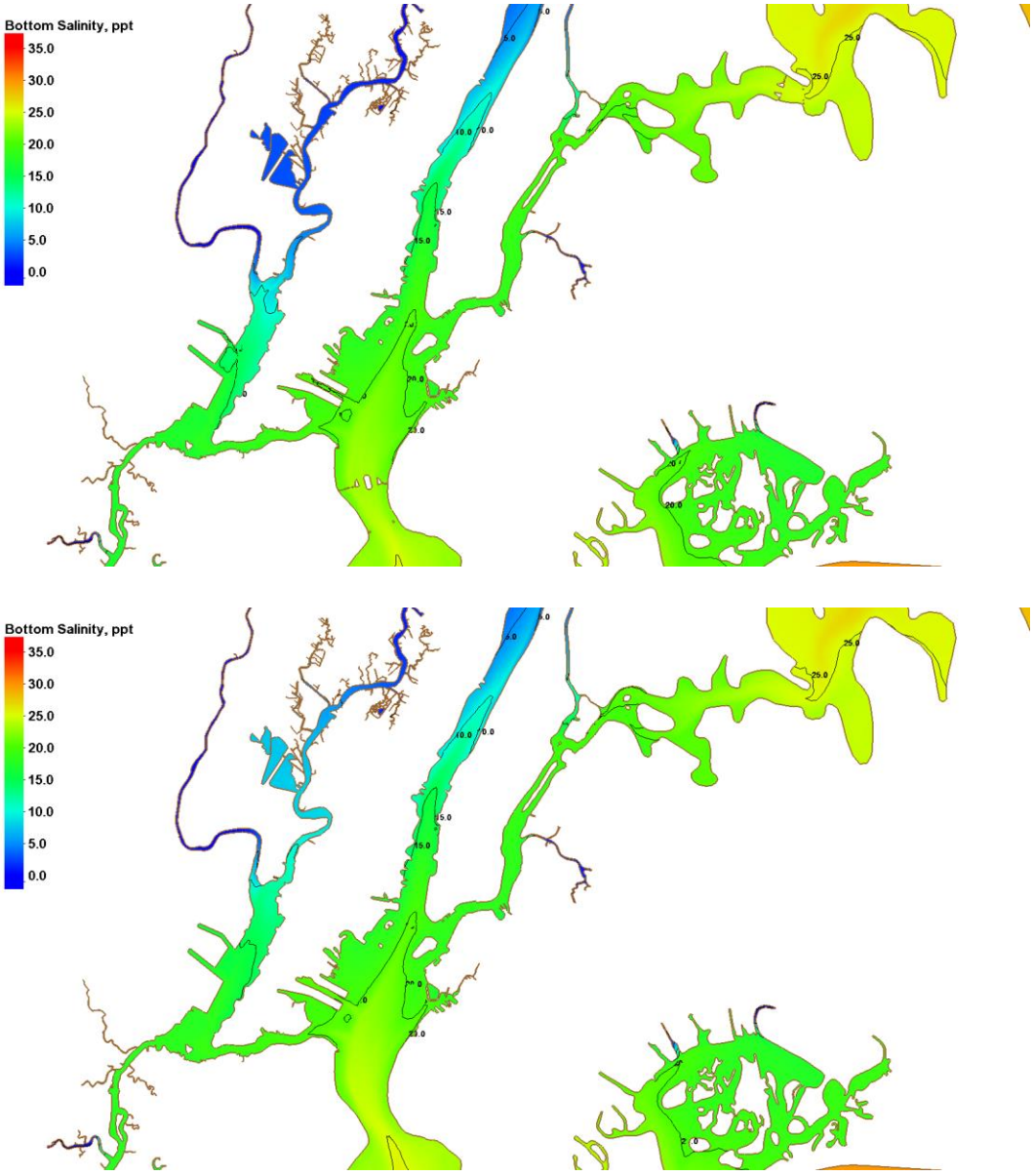


Figure 117: Set 1 Bottom Salinity 75th percentile in Upper Bay (Alternative 3a top, Base Bottom)

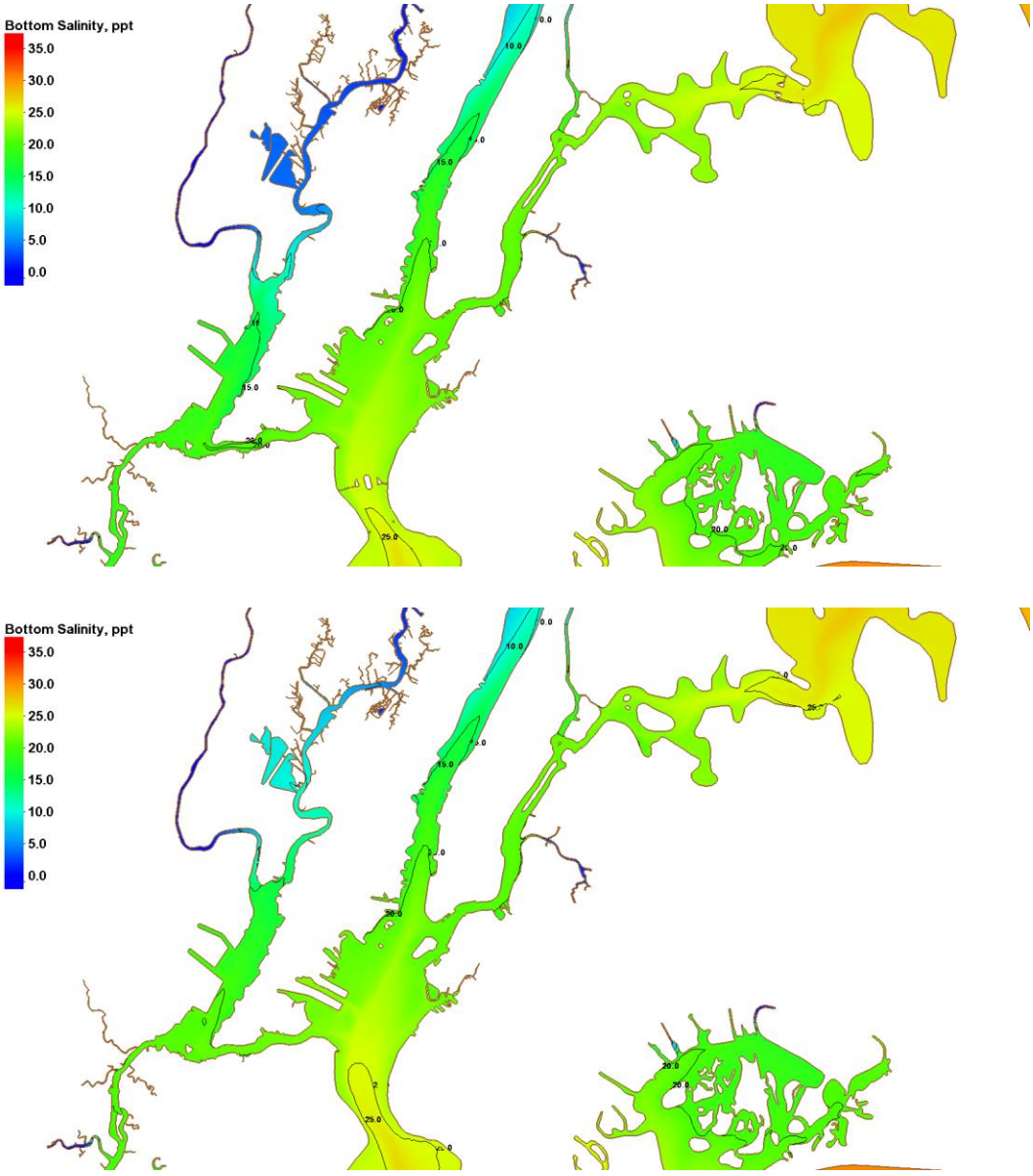


Figure 118: Set 1 Bottom Salinity 50th percentile in Lower Bay (Alternative 3b top, Base Bottom)

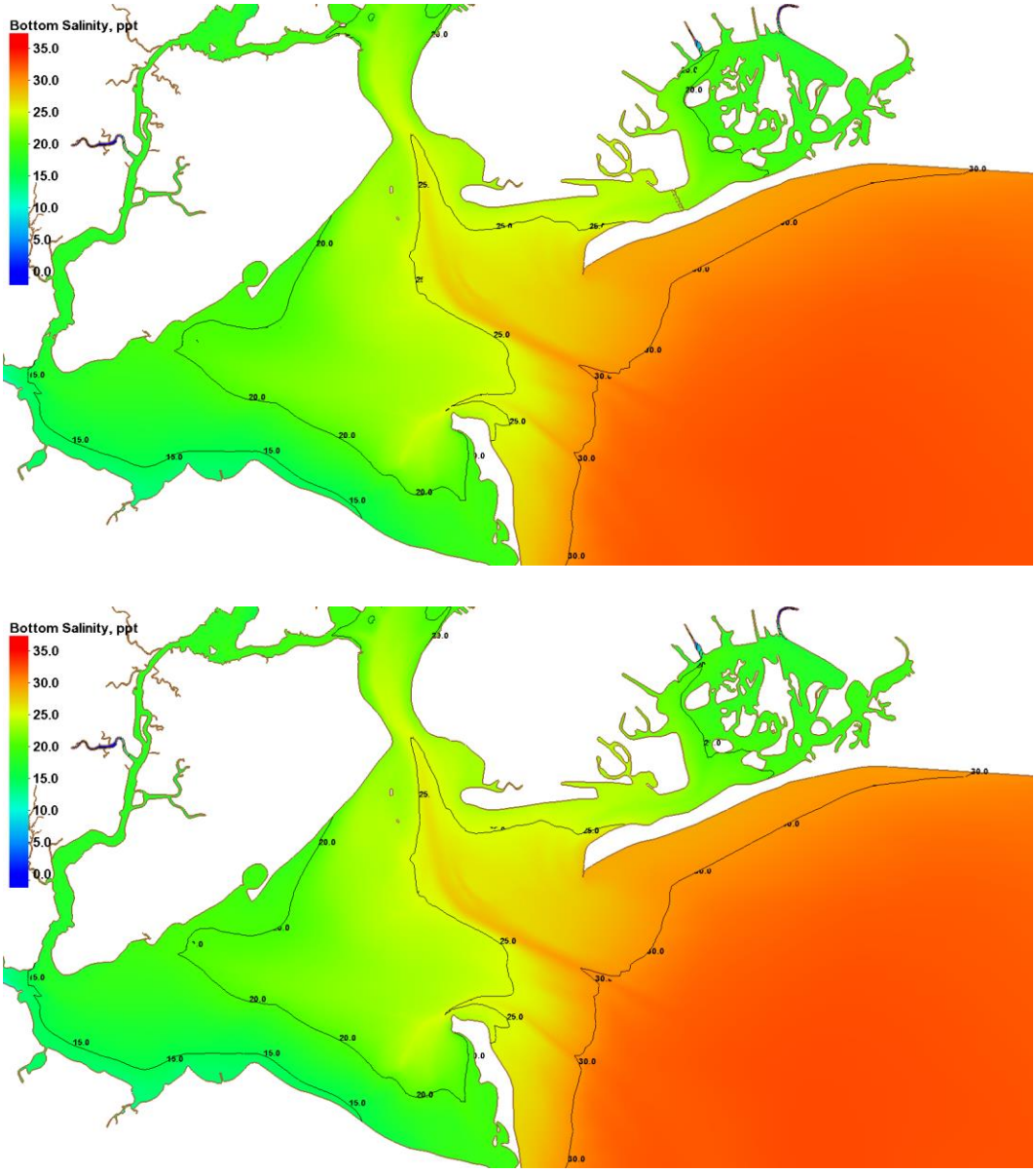


Figure 119: Set 1 Bottom Salinity 75th percentile in Lower Bay (Alternative 3b top, Base Bottom)

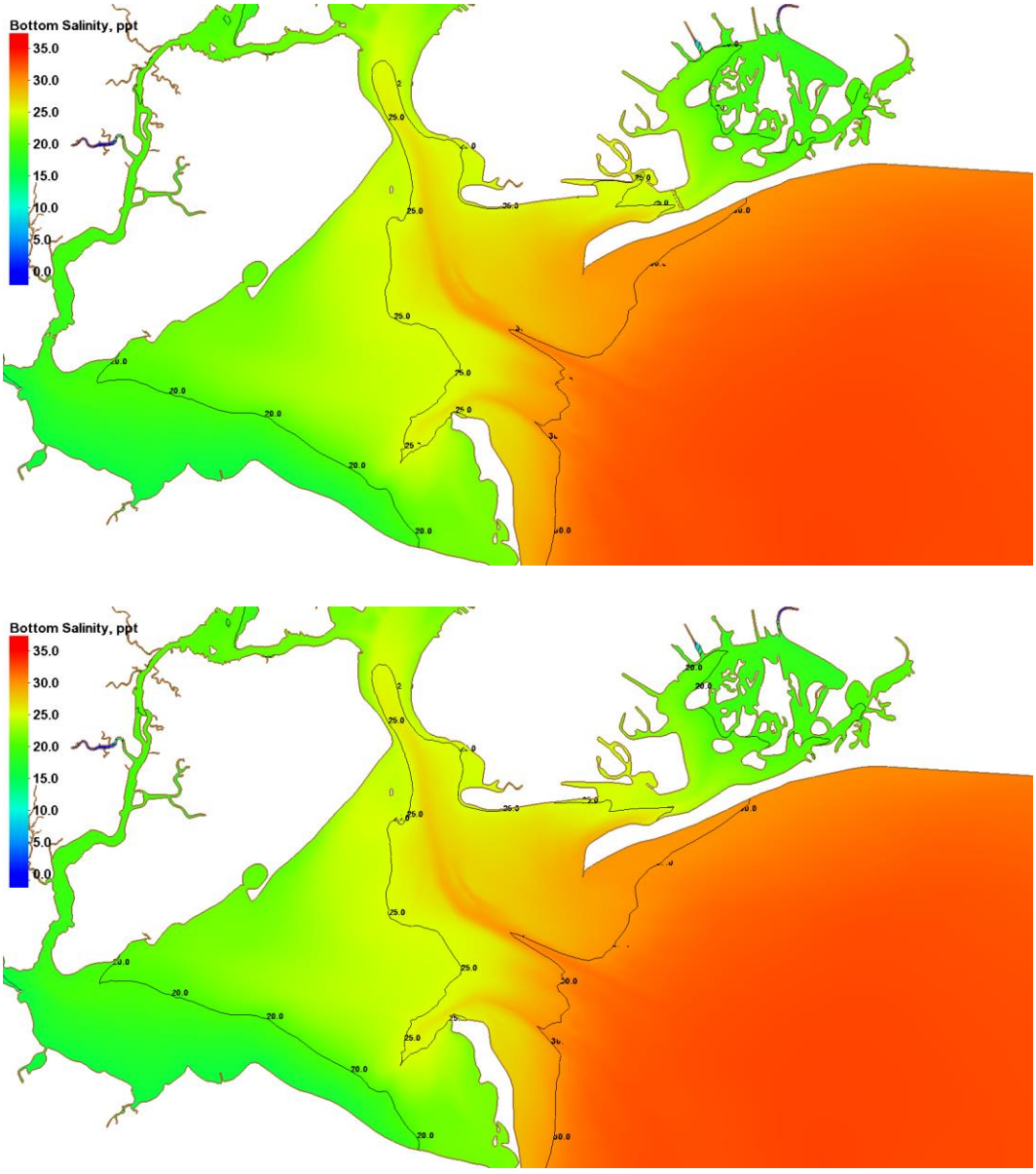


Figure 120: Set 1 Bottom Salinity 50th percentile in Upper Bay (Alternative 3b top, Base Bottom)

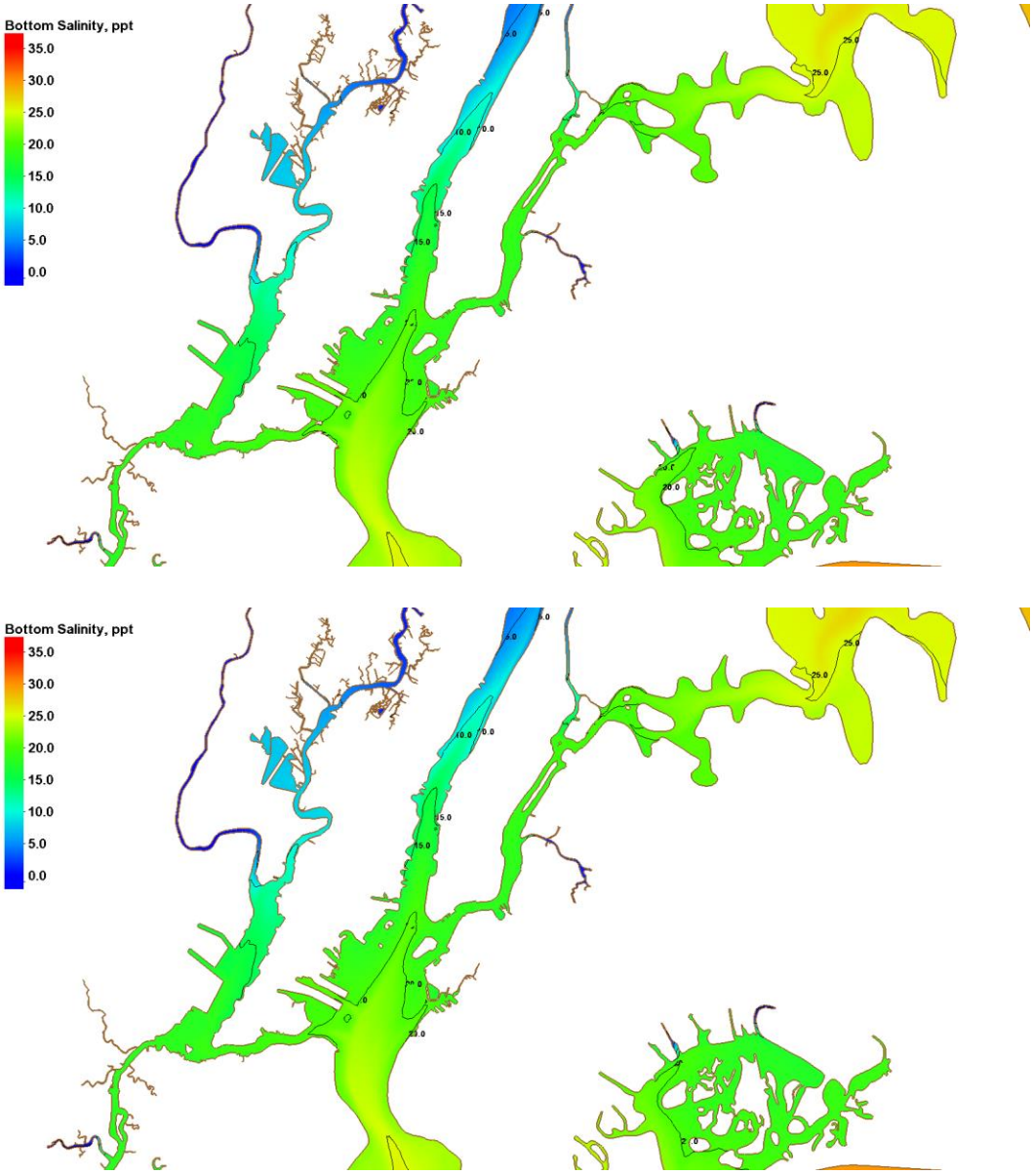


Figure 121: Set 1 Bottom Salinity 75th percentile in Upper Bay (Alternative 3b top, Base Bottom)

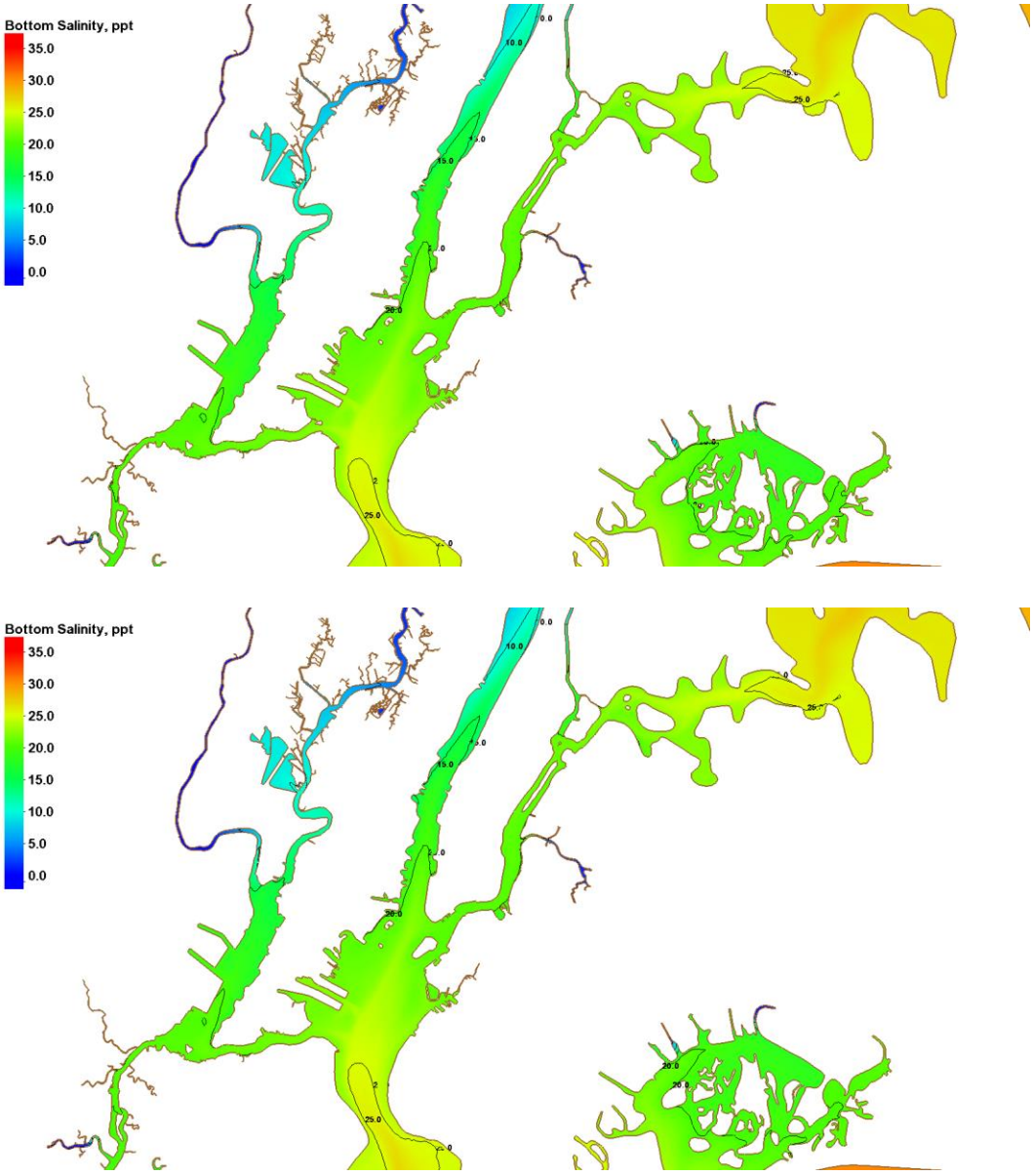


Figure 122: Set 1 Bottom Salinity 50th percentile in Lower Bay (Alternative 4 top, Base Bottom)

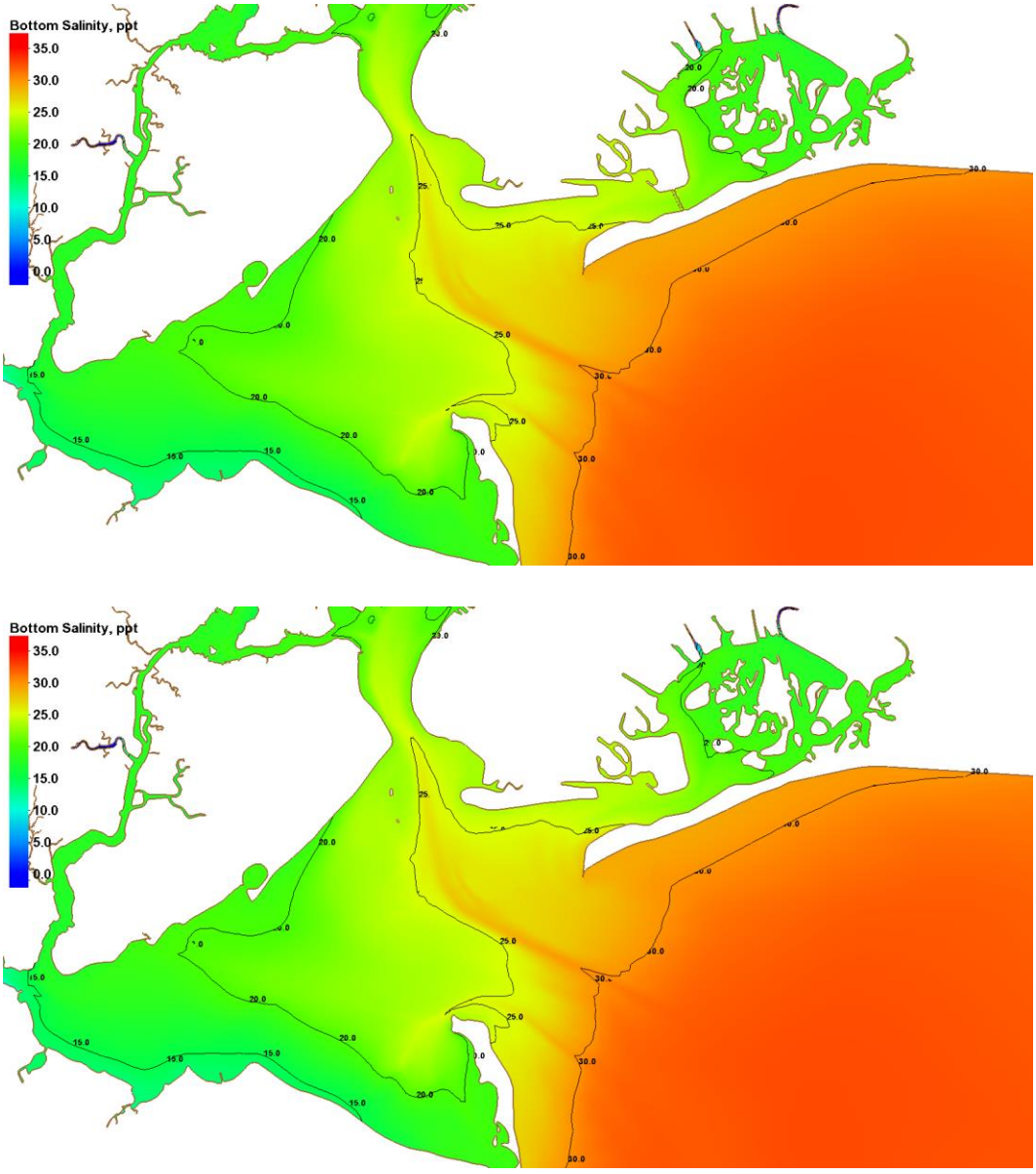


Figure 123: Set 1 Bottom Salinity 75th percentile in Lower Bay (Alternative 4 top, Base Bottom)

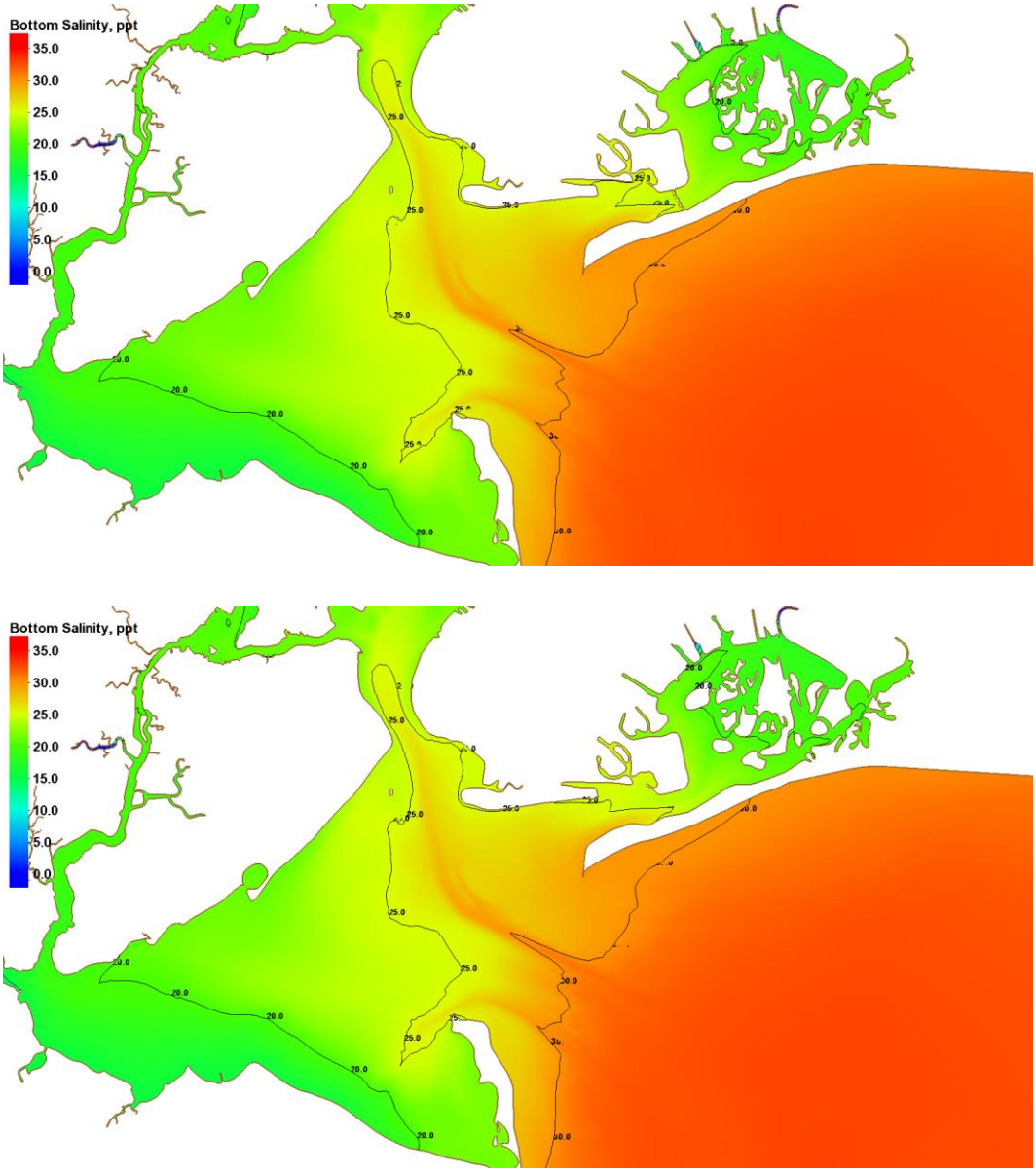


Figure 124: Set 1 Bottom Salinity 50th percentile in Upper Bay (Alternative 4 top, Base Bottom)

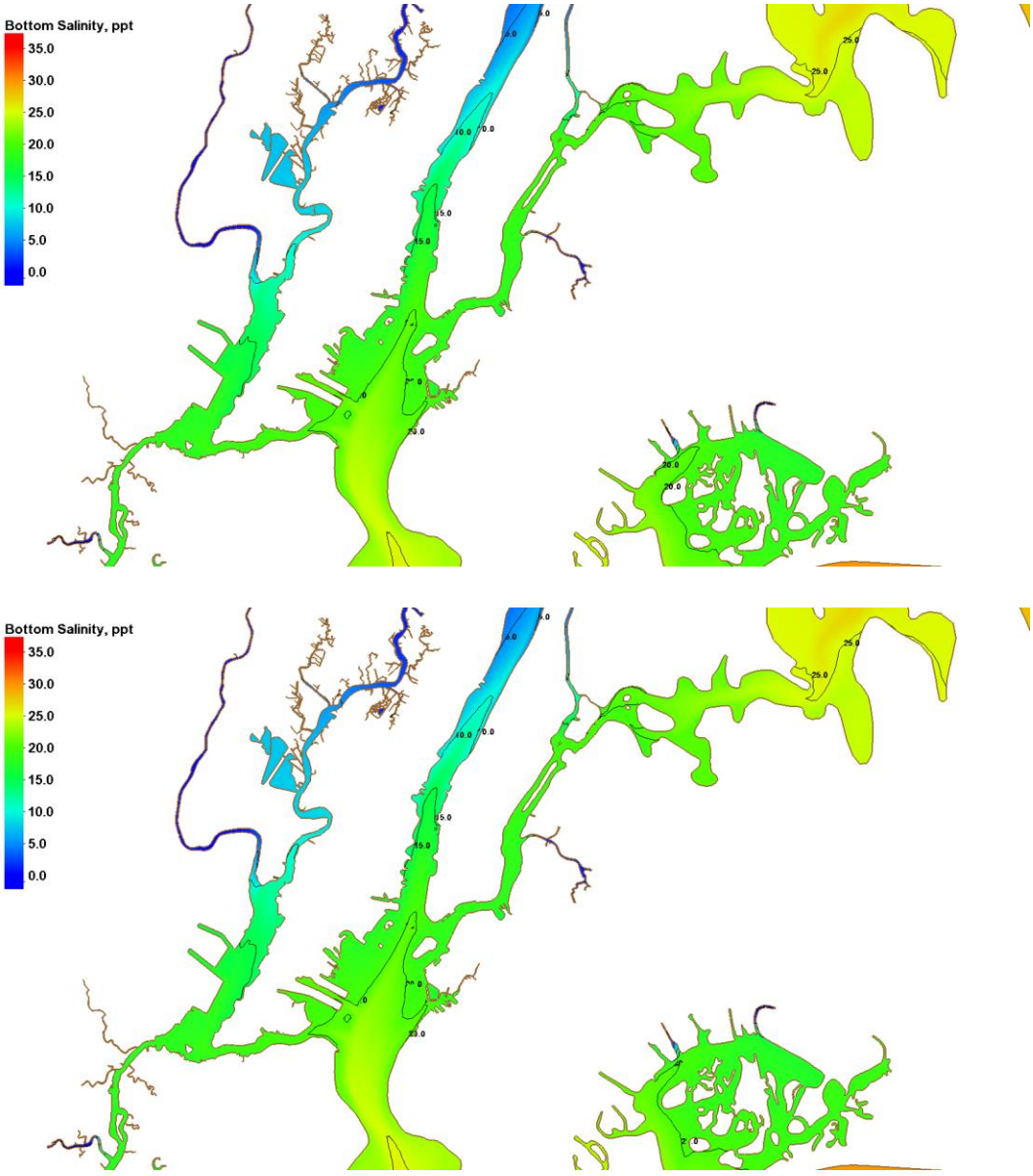
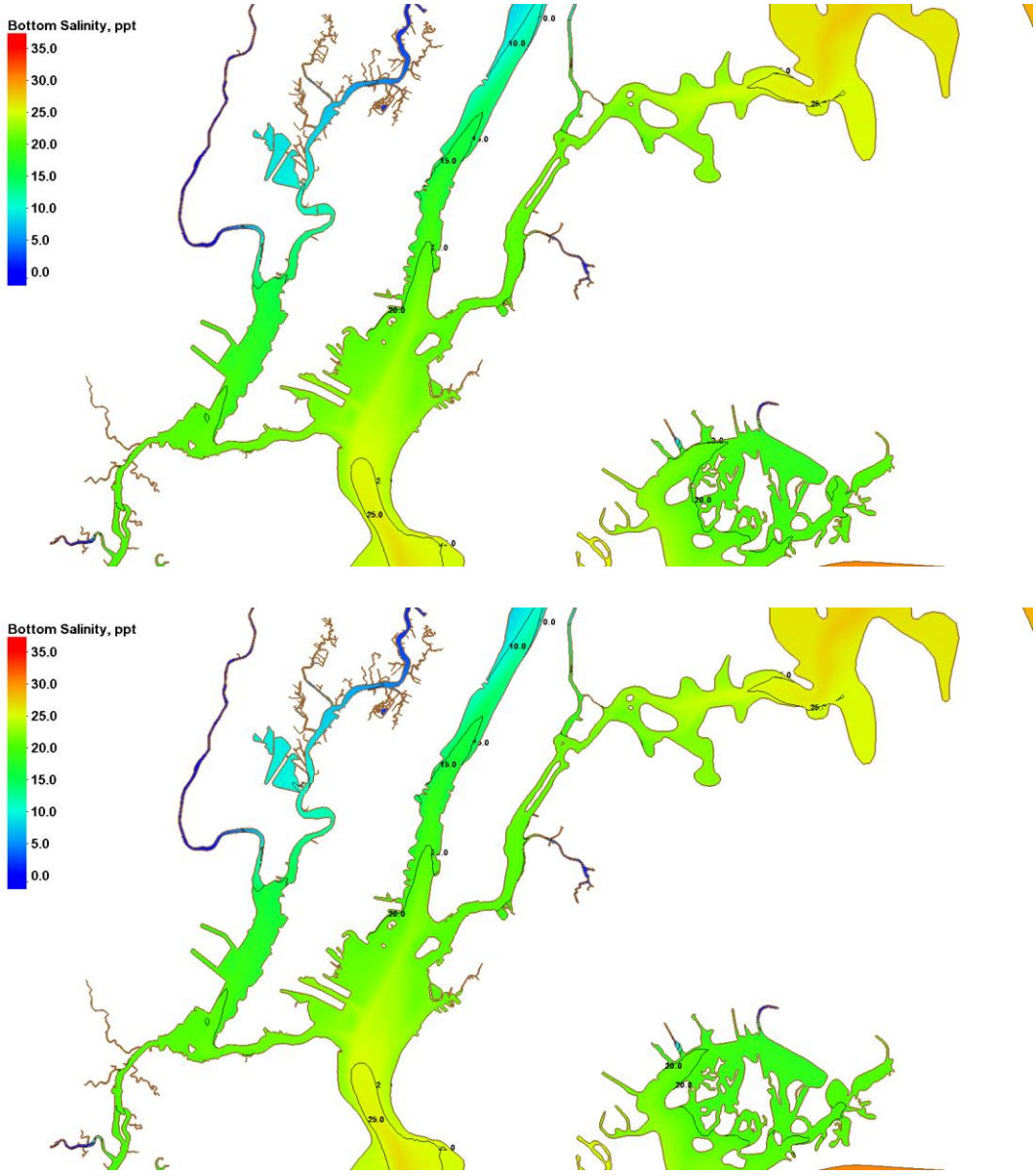


Figure 125: Set 1 Bottom Salinity 75th percentile in Upper Bay (Alternative 4 top, Base Bottom)



2.2.3 Tidal Prism Analysis

A tidal prism percentile analysis was performed for the areas in the mesh that are impounded by the barriers and then compared to the tidal prism for the same areas in the without project.

2.2.3.1 Alternative 2

Figure 126 is a schematic of the area used to calculate the tidal prism, and Figure 127 is the percentile analysis.

Figure 126: Set 1 Alternative 2 Tidal Prism Material Schematic

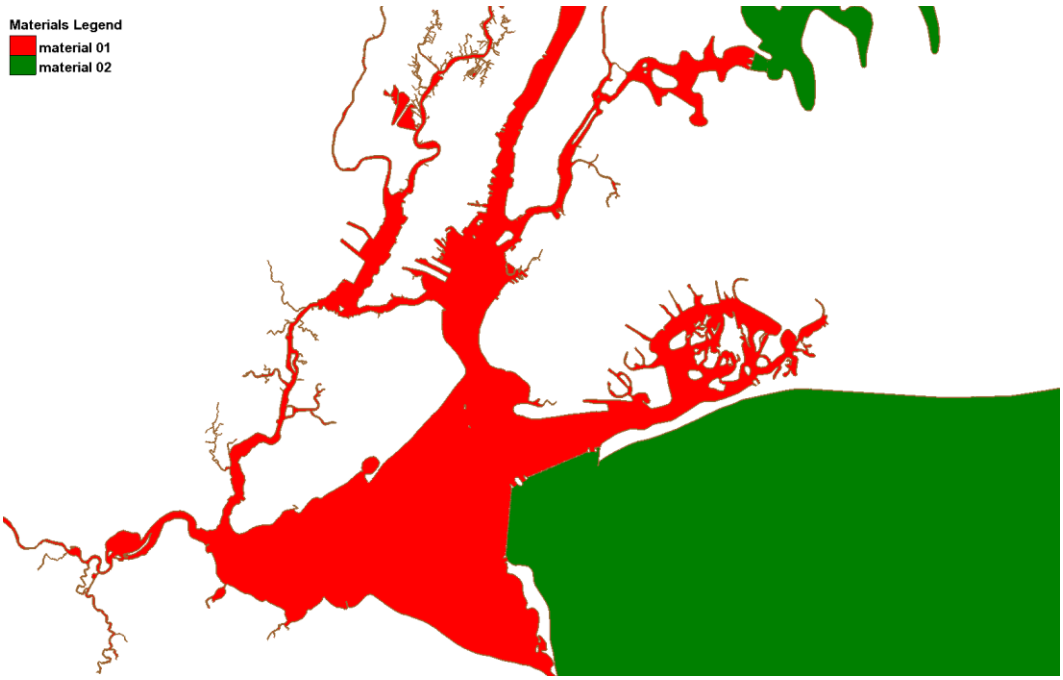
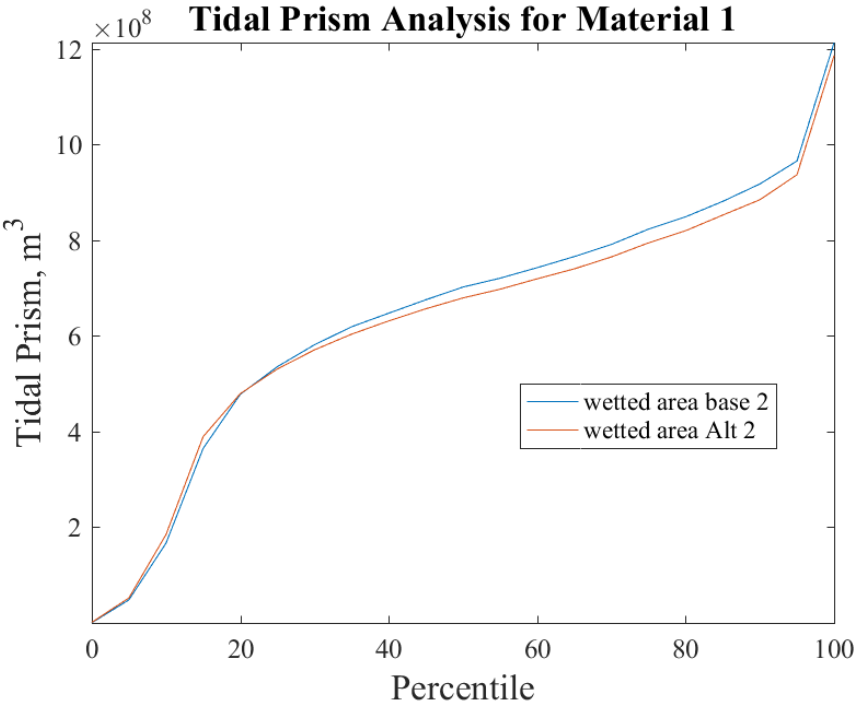


Figure 127: Set 1 Alternative 2 Tidal Prism Material 1



2.2.3.2 Alternative 3a

Figure 128 is a schematic of the area used to calculate the tidal prism, and Figure 129 and Figure 130 is the percentile analysis.

Figure 128: Set 1 Alternative 3a Tidal Prism Material Schematic

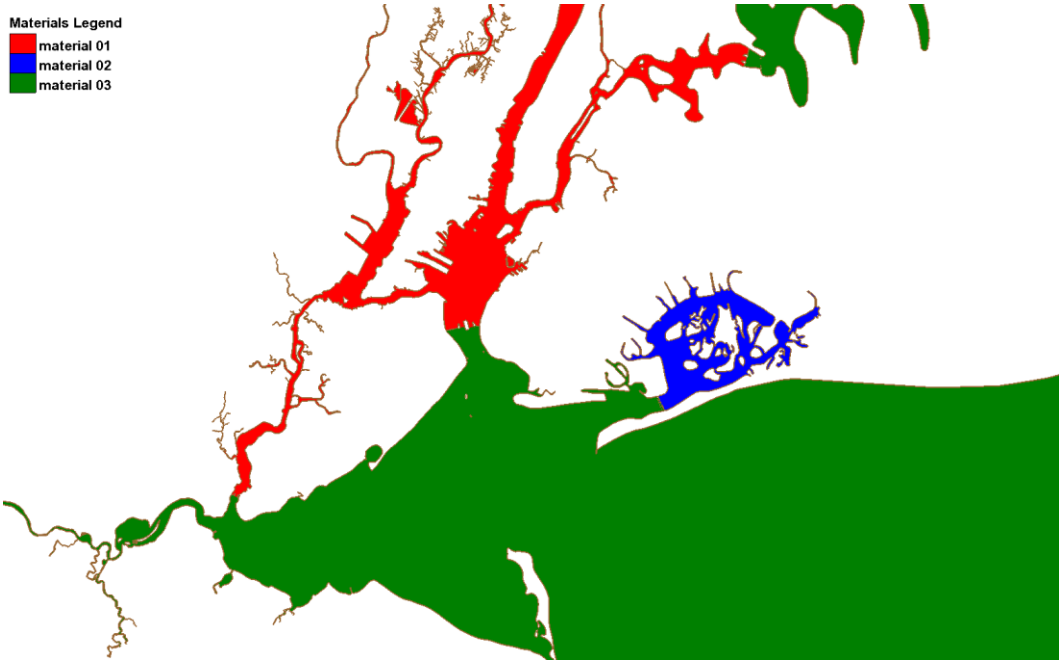


Figure 129: Set 1 Alternative 3a Tidal Prism Material 1

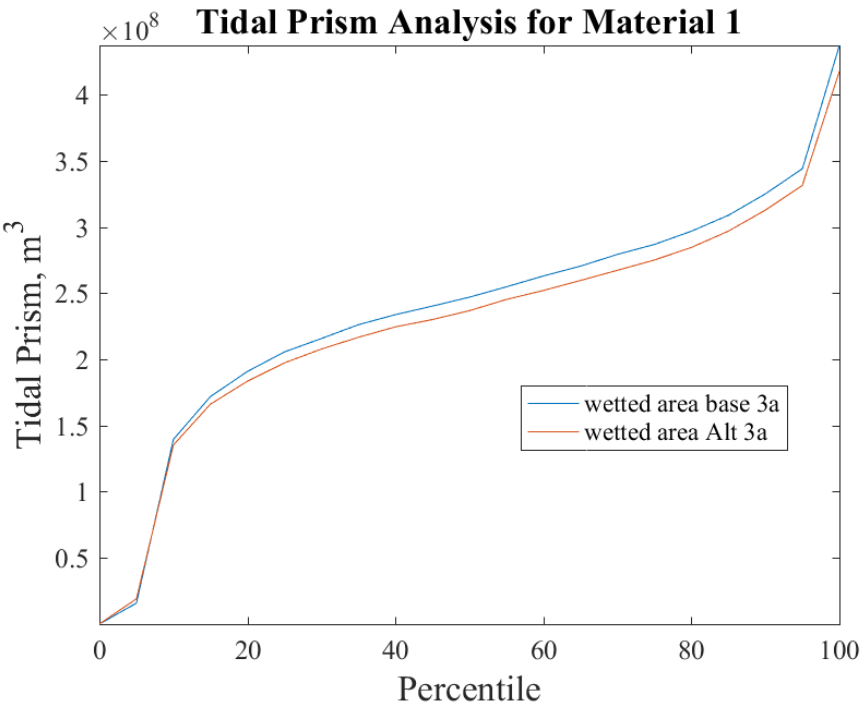
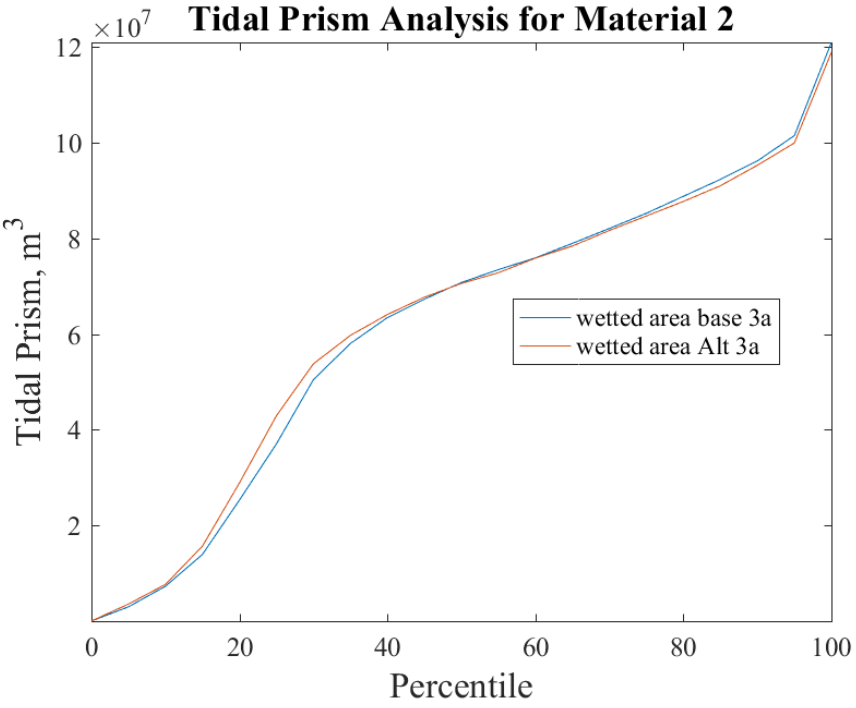


Figure 130: Set 1 Alternative 3a Tidal Prism Material 2



2.2.3.3 Alternative 3b

Figure 131 is a schematic of the area used to calculate the tidal prism, and Figure 132 and Figure 133 is the percentile analysis.

Figure 131: Set 1 Alternative 3b Tidal Prism Material Schematic

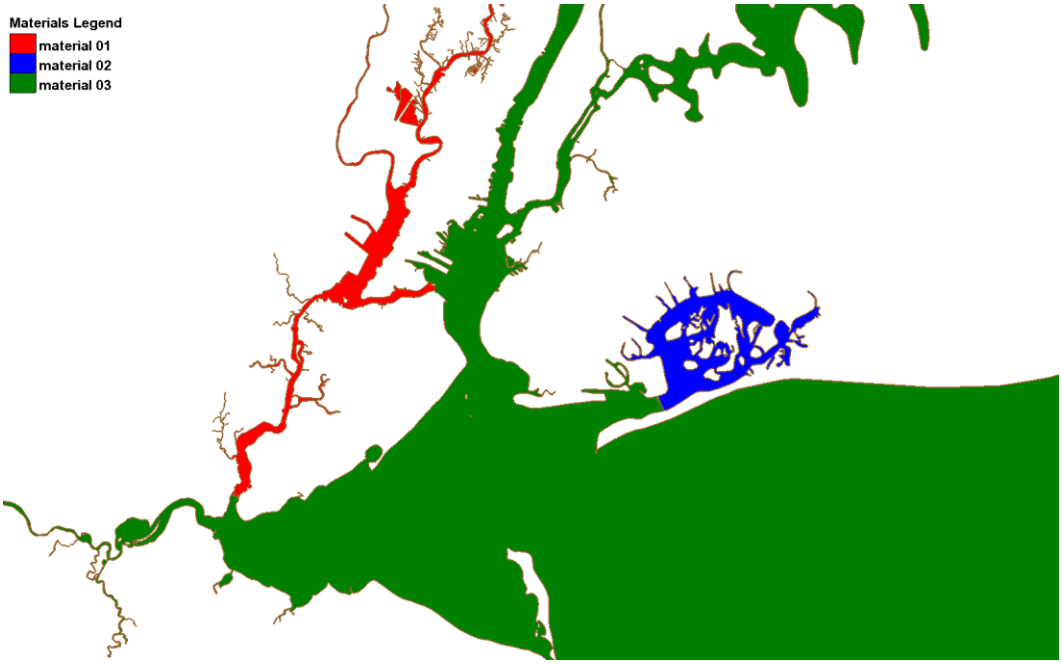


Figure 132: Set 1 Alternative 3b Tidal Prism Material 1

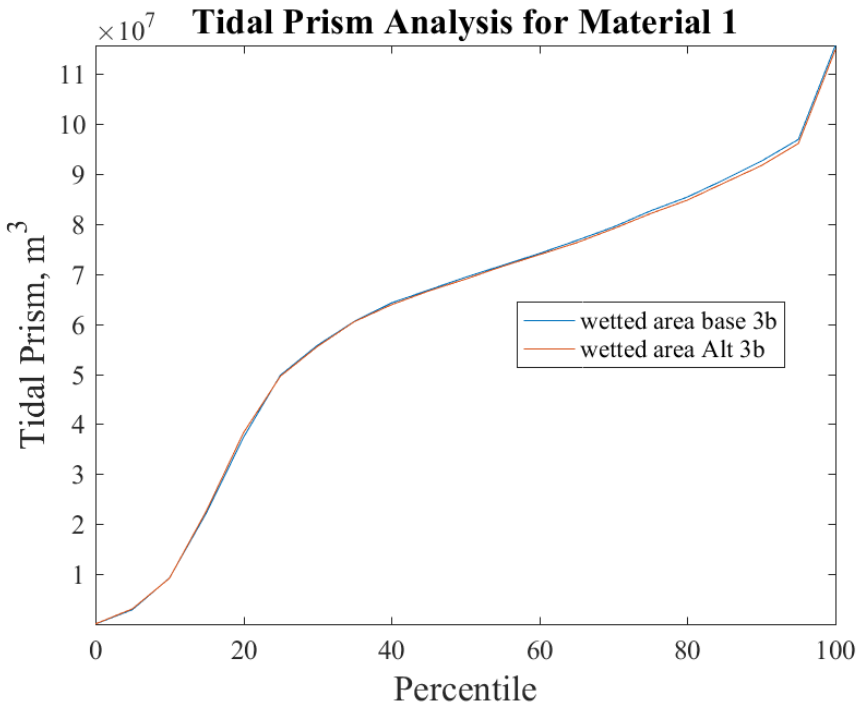
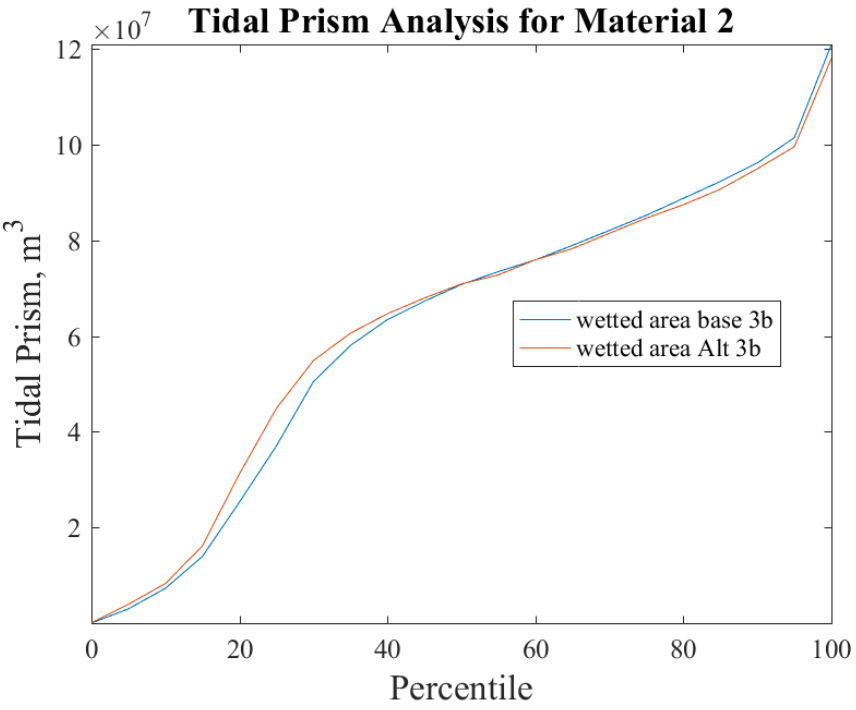


Figure 133: Set 1 Alternative 3b Tidal Prism Material 2



2.2.3.4 Alternative 4

Figure 134 is a schematic of the area used to calculate the tidal prism, and Figure 135 and Figure 136 is the percentile analysis.

Figure 134: Set 1 Alternative 4 Tidal Prism Material Schematic

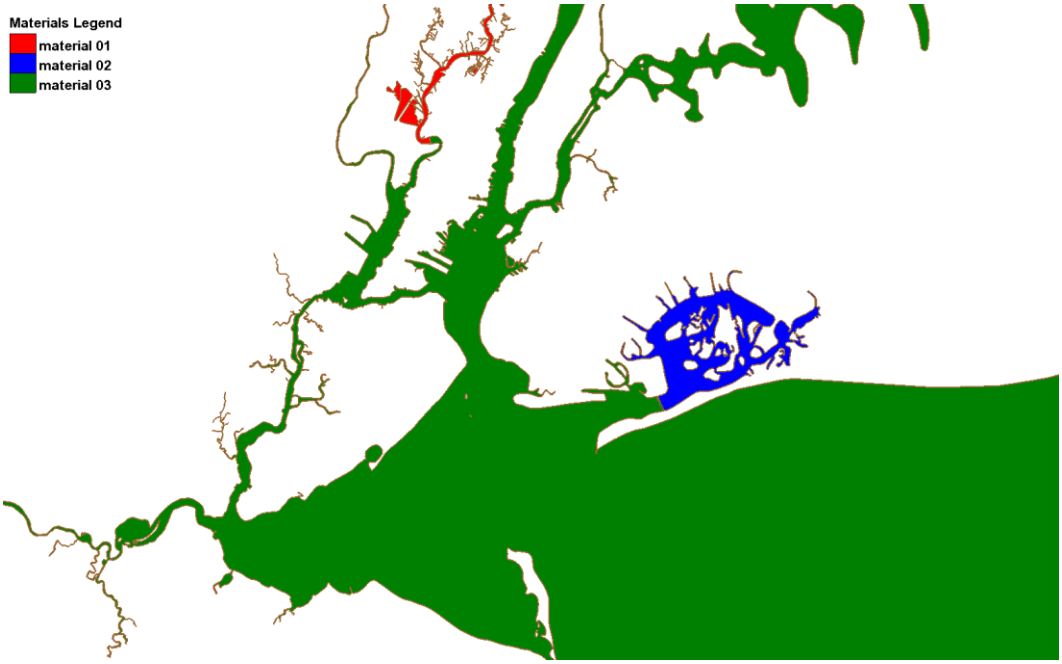


Figure 135: Set 1 Alternative 4 Tidal Prism Material 1

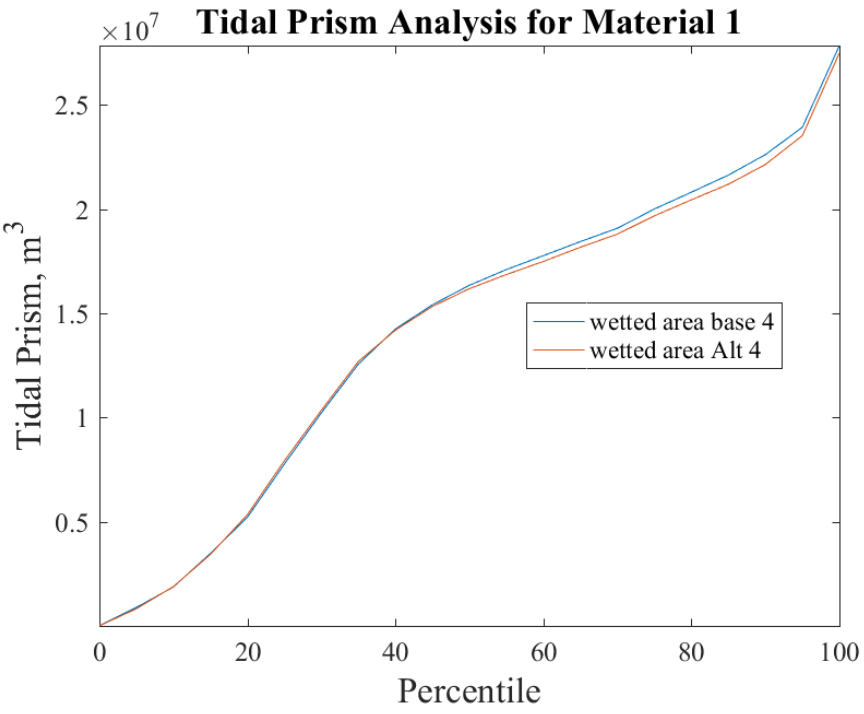
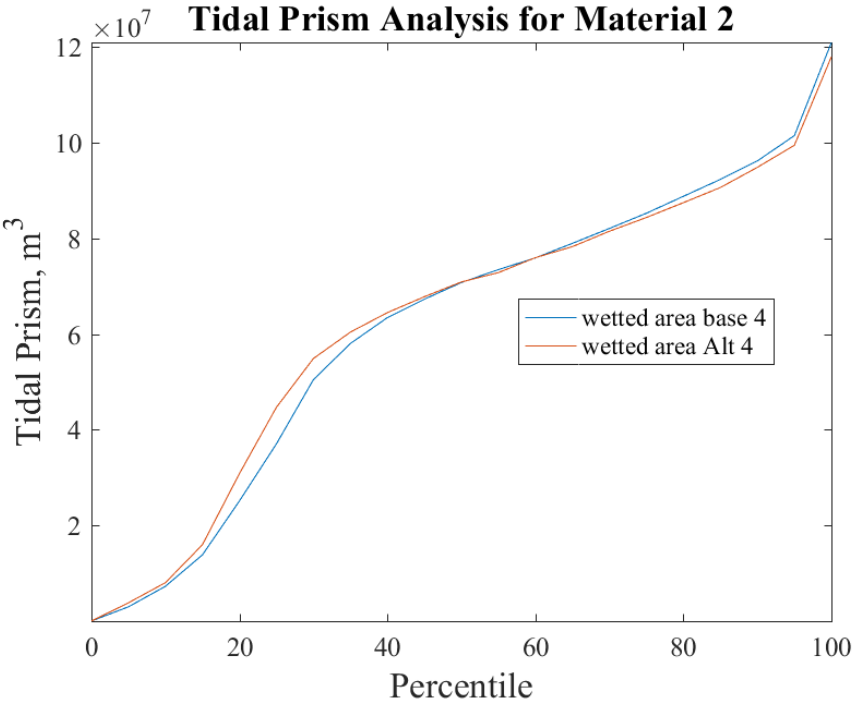


Figure 136: Set 1 Alternative 4 Tidal Prism Material 2



2.2.4 Summary of Results

The biggest impacts on the water surface elevation occur in Alternatives 2 and 3a. For Alternative 2, the tidal range decreases slightly at almost all of the analysis points which are within the impounded area. In Alternative 3a, the tidal range has a slightly higher decrease in the impounded areas. There are, however, negligible effects in the areas not impounded by the proposed structures in Alternative 3a. Alternatives 3b and 4, on the other hand, have negligible effects on the water surface elevations at the analyzed locations. The velocities, on the other hand, do follow a general trend. The velocities do increase at the points that are at a proposed structure. For example, point V2 is in-between one of the sector gates for Alternative 2, and the velocity for Alternative 2 is much higher compared to the without project velocities. This behavior is expected since the cross sectional area of flow decreases at the barriers. At the same point (V2) the velocities for Alternatives 3a, 3b, and 4 does not deviate much from the without project velocity since there is no structure at that point for these alternatives. The velocity percentile analysis plots are another representation of this same behavior. This is also noticeable in the spatial velocity figures. In Figure 95, for example, looking closely at the sector gate on the Ambrose Channel, the velocities are higher at the barrier than the without

project. Then moving away from the barriers spatially, the velocity starts to match the without project velocities. The tidal percentile analysis represents how much water is moving into or out of a certain area. In theory, it is expected that the areas impounded by the barriers would have a slightly lower tidal prism than the without project assuming a reduction in cross-sectional area due to the proposed structure. In general, this trend does seem to be followed.

3 Alternative Set 2 and Results

The second set of simulations evaluated four alternative alignments similar to the previous set of alternatives but were modified based on the previous results and additional analysis of ship sizes, frequency and historical pathways.

3.1 Alternatives

The four alternatives included alignments that ranged from large scale bay-wide protection (Sandy Hook to Rockaway structures in alignment 2) to smaller scale more localized protections. A profile view of all the structures for set 2 are provided in Appendix B. It should be noted that structures present in multiple alternatives are equivalent in all of the alternatives. An example would be Jamaica Bay. This structure is consistent in Alternative 3a, 3b, and 4.

3.1.1 Alternative 2

Alternative 2 consisted of 2 structures with the alignment shown in Figure 137. The Sandy Hook to Rockaway structure is shown in Figure 138 and the Throgs Neck structure is shown in Figure 139.

Figure 137: Set 2 Alternative 2

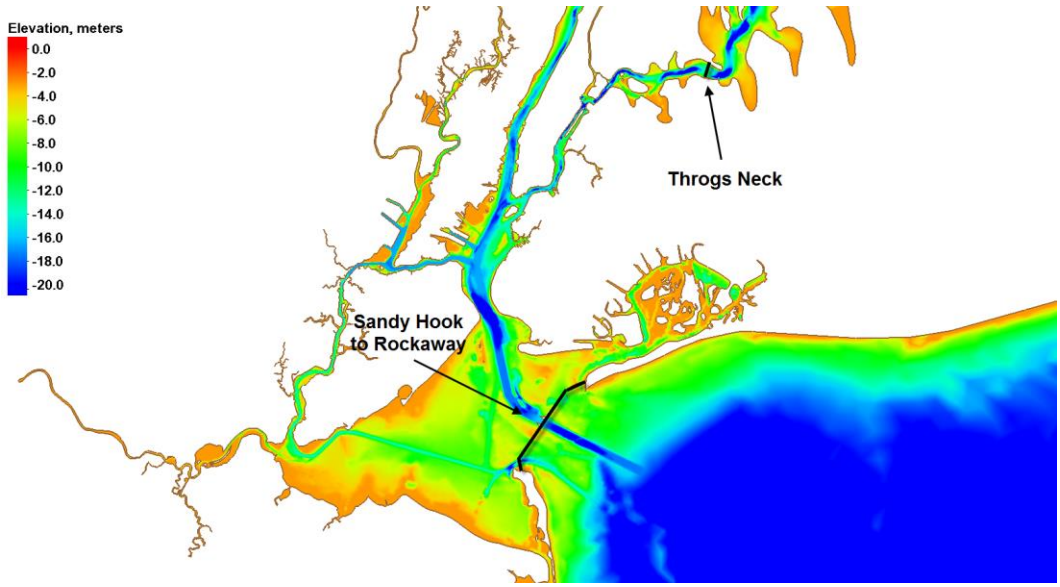


Figure 138: Set 2 Alternative 2 Sandy Hook to Rockaway Structure

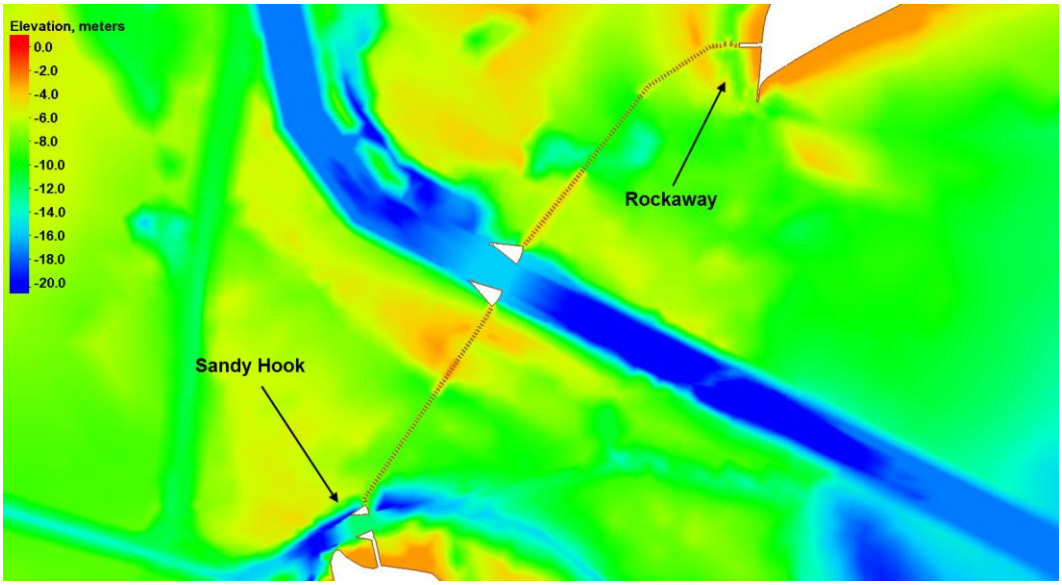
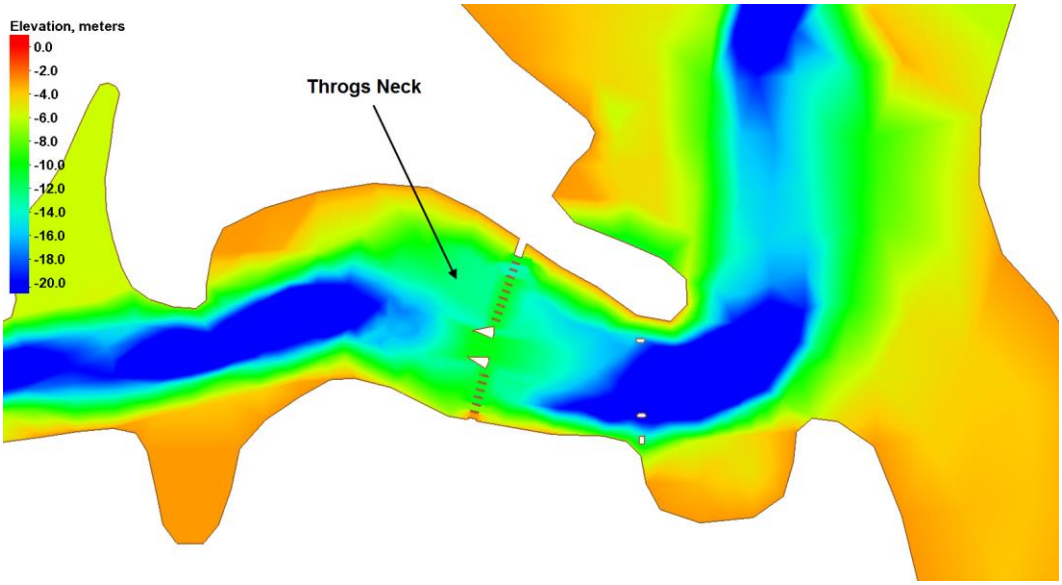


Figure 139: Set 2 Alternative 2 Throg's Neck Structure



3.1.2 Alternative 3a

Alternative 3a consisted of 4 structures with the alignment shown in Figure 140. The Throg's Neck Structure in Alternative 3a is aligned the same as Alternative 2's Throg's Neck Structure depicted in Figure 139.

Figure 140: Set 2 Alternative 3a

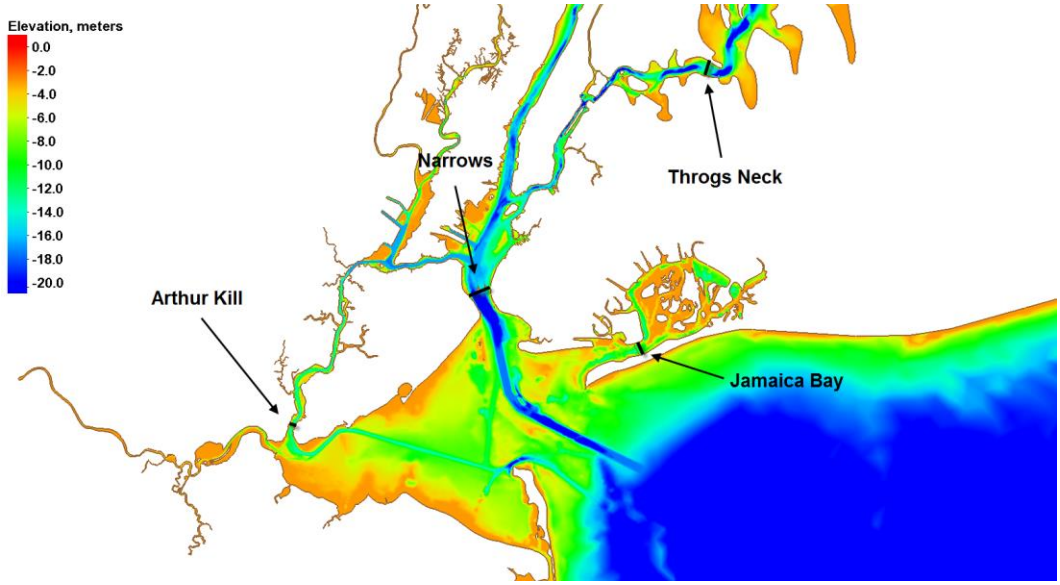


Figure 141: Set 2 Alternative 3a Arthur Kill Structure

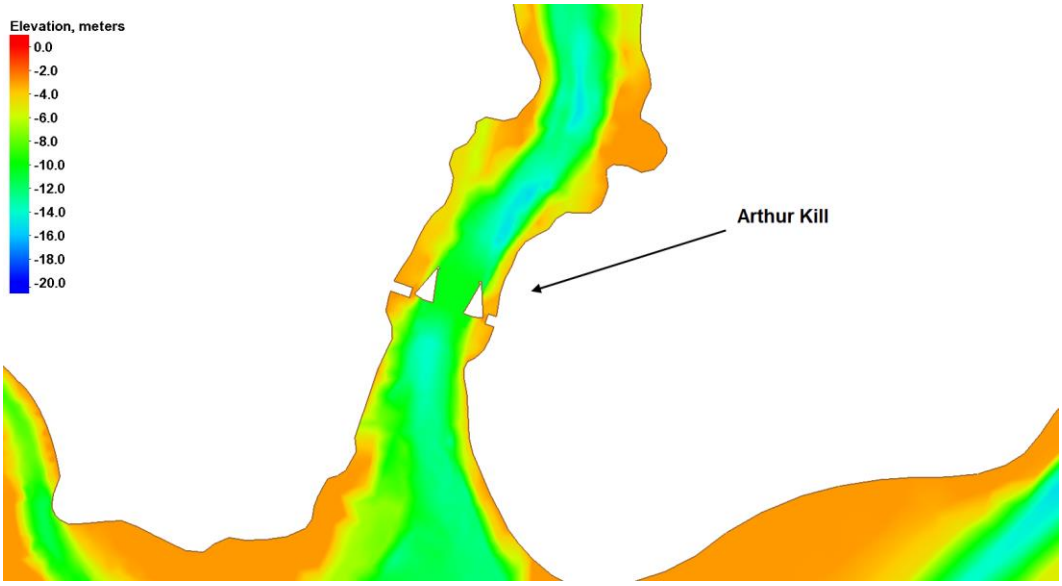


Figure 142: Set 2 Alternative 3a Narrows Structure

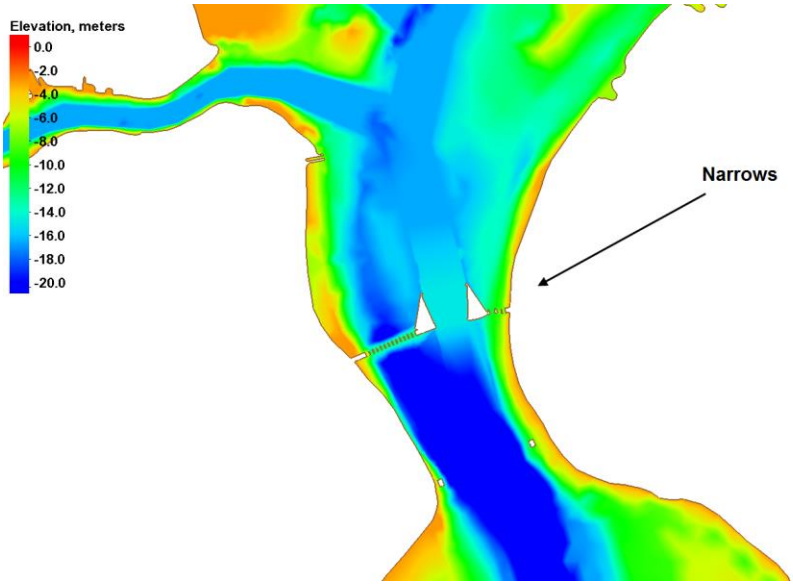
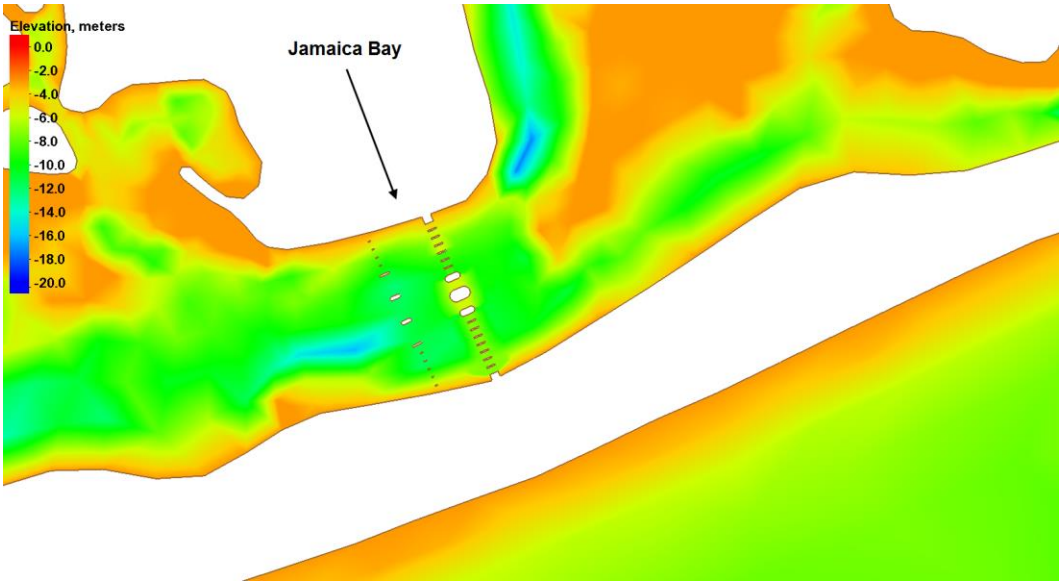


Figure 143: Set 2 Alternative 3a Jamaica Bay Structure



3.1.3 Alternative 3b

Alternative 3b consisted of 3 structures with the alignment shown in Figure 144. The Arthur Kill and Jamaica Bay Structures in Alternative 3b are aligned the same as Alternative 3a's Arthur Kill and Jamaica Bay structures which are depicted in Figure 141 and Figure 143 respectively.

Figure 144: Set 2 Alternative 3b

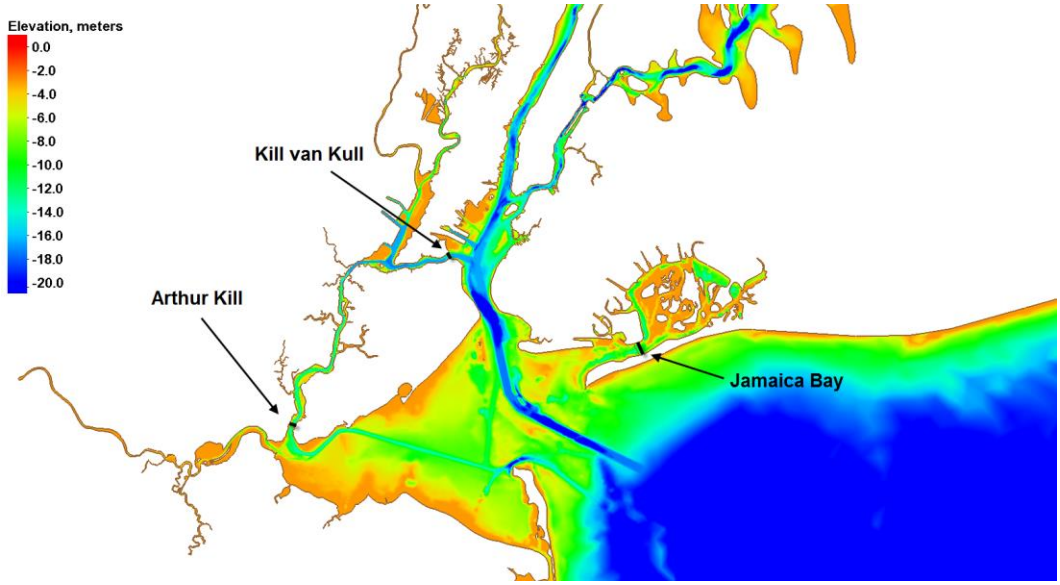
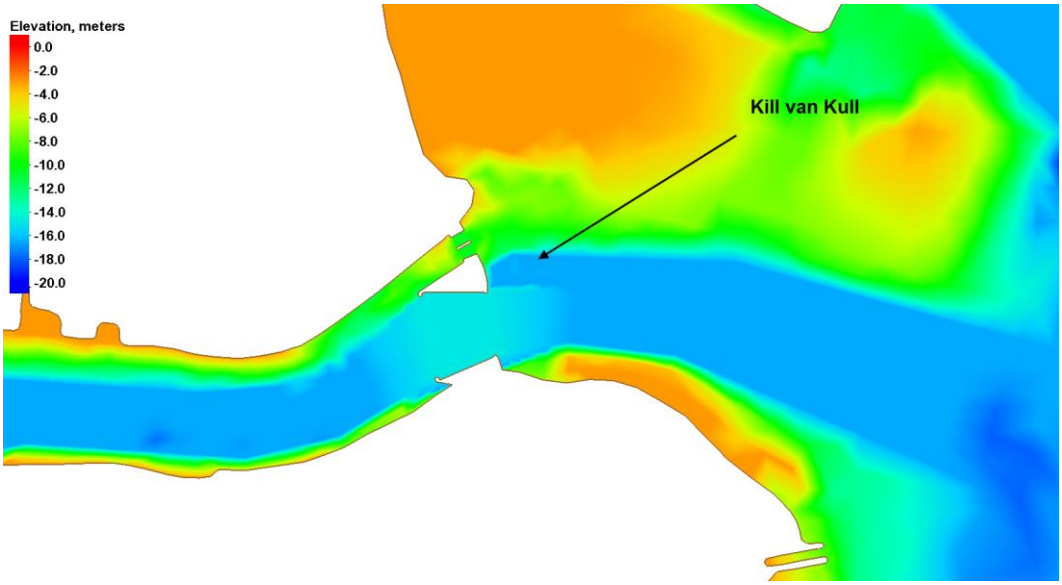


Figure 145: Set 2 Alternative 3b Kill van Kull Structure



3.1.4 Alternative 4

Alternative 4 consisted of 2 structures with the alignment shown in Figure 146. The Jamaica Bay Structure in Alternative 4 is aligned the same as Alternative 3a's Jamaica Bay Structure depicted in Figure 143.

Figure 146: Set 2 Alternative 4

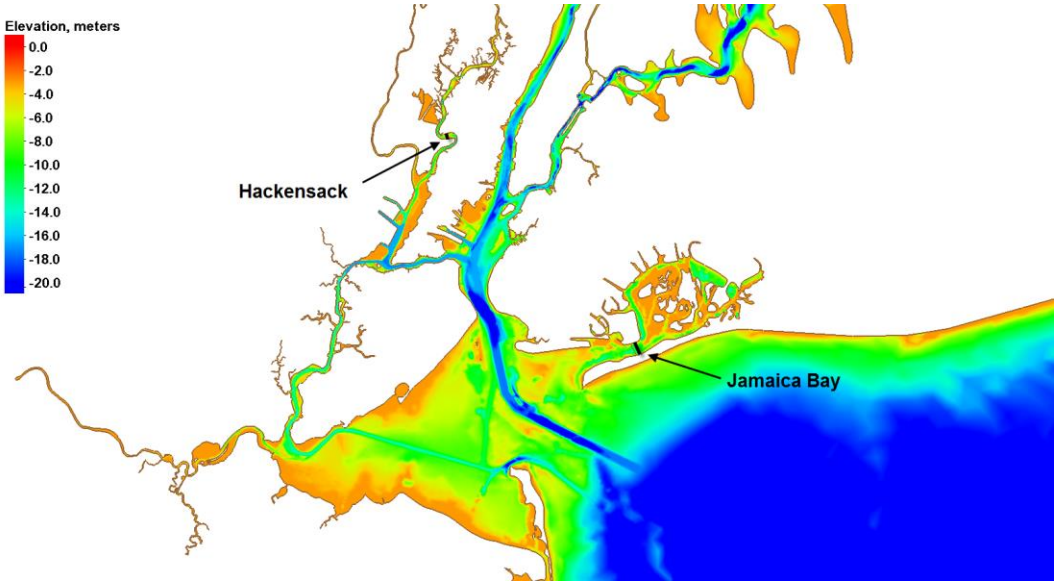
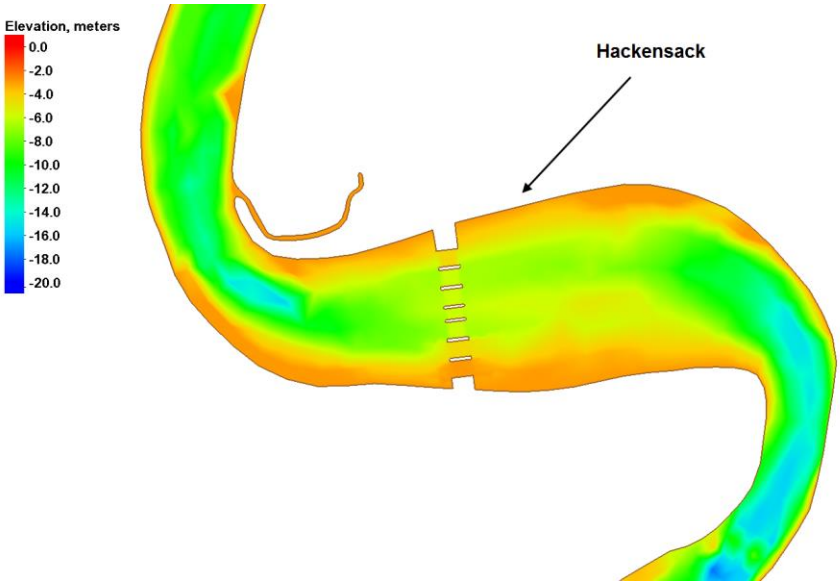


Figure 147: Set 2 Alternative 4 Hackensack Structure



3.2 Model Results and Analysis

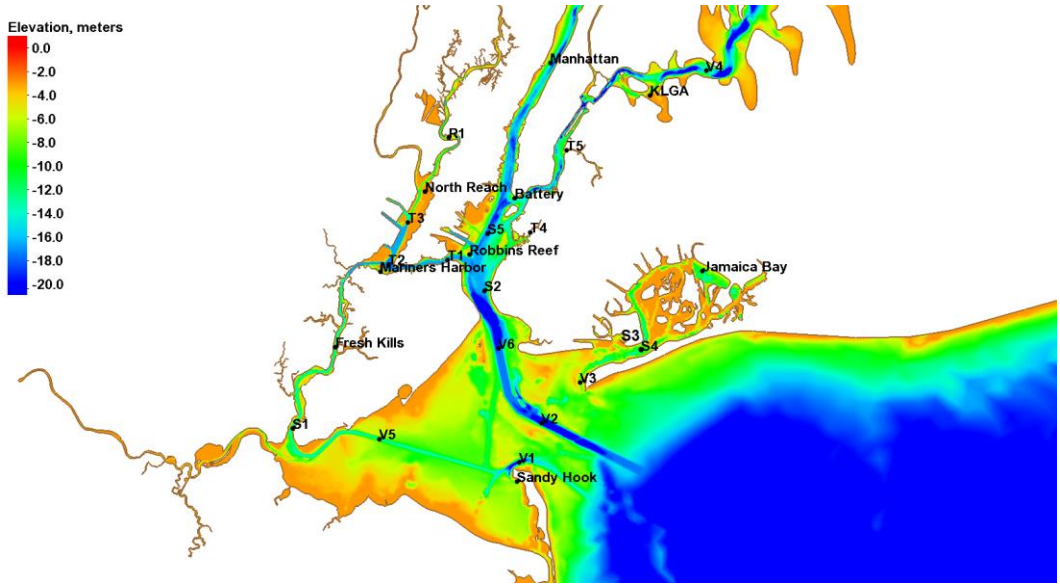
The four alternatives were compared to the without project simulation to determine the impact of the alternatives on water levels, velocities, and the tidal prism. The analysis for the second set of alternatives are provided in the following sections and consist of point velocity and water surface elevation model comparisons, velocity percentiles, spatial figures of the 50th

and 75th percentile velocities and salinity, and tidal prism percentiles for the areas affected by the structures.

3.2.1 Point Comparisons

Point comparisons were extracted from the model results at the locations shown in Figure 148. These comparisons are shown in Figure 149 to Figure 226. Even though the AdH simulations are a yearlong, all of the time series plots are zoomed in to the first two weeks of the month of July so the impact of each alternative is more easily visible.

Figure 148: Set 2 Analysis Points



3.2.1.1 Water Surface Elevation Point Comparisons

In Figure 149 through Figure 174, the water surface elevation time series values are plotted for each alternative and the without project at the locations shown in Figure 148.

Figure 149: Set 2 Point V1 Water Surface Elevation

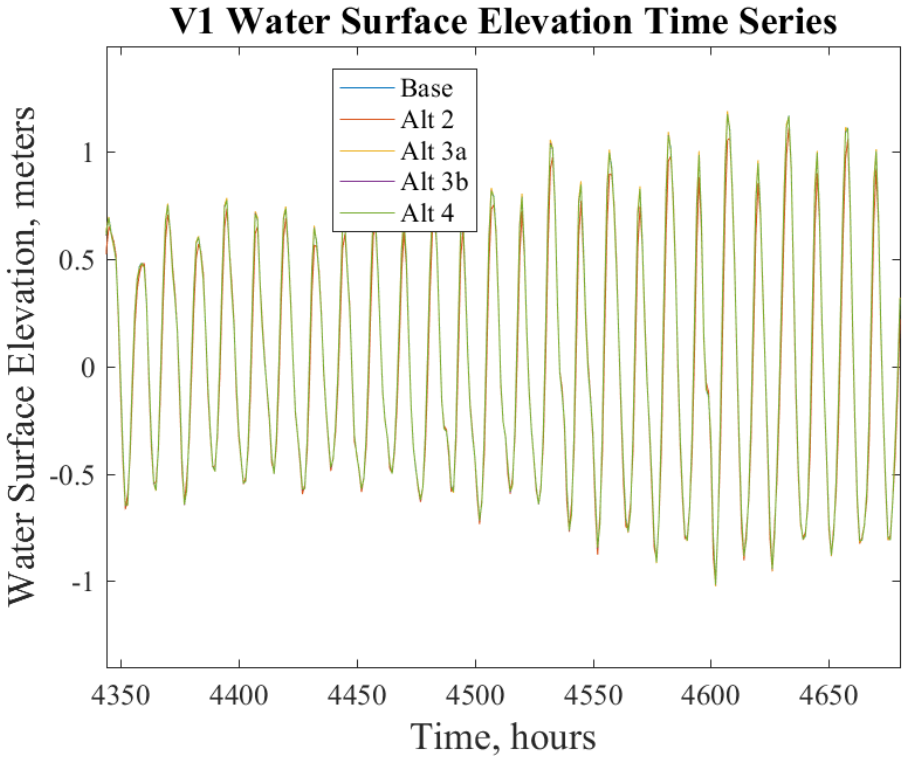


Figure 150: Set 2 Point V2 Water Surface Elevation

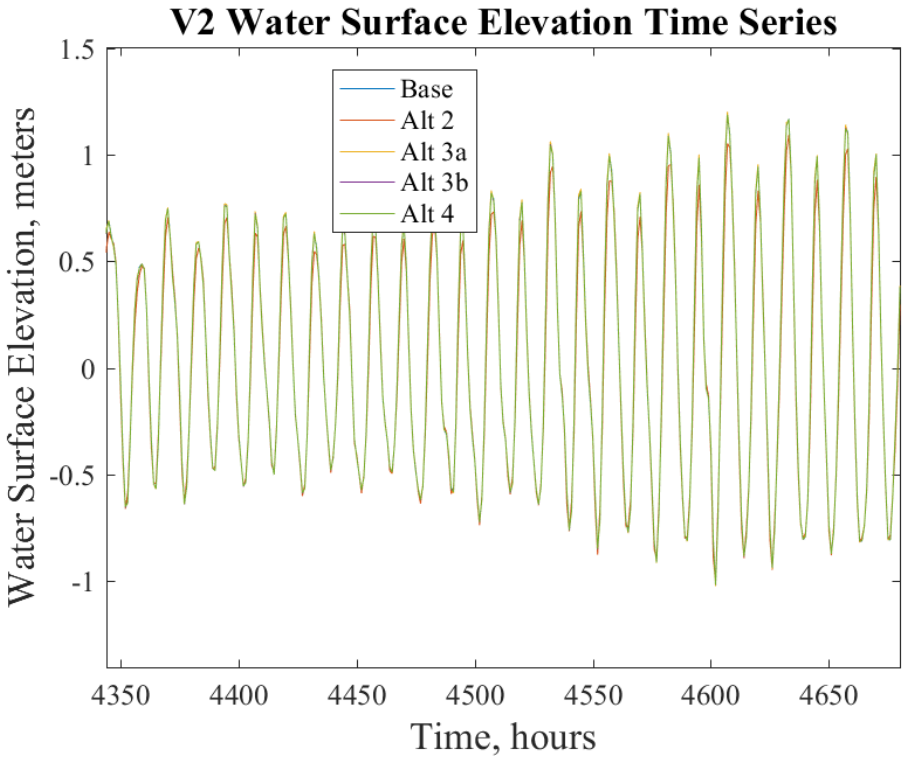


Figure 151: Set 2 Point V3 Water Surface Elevation

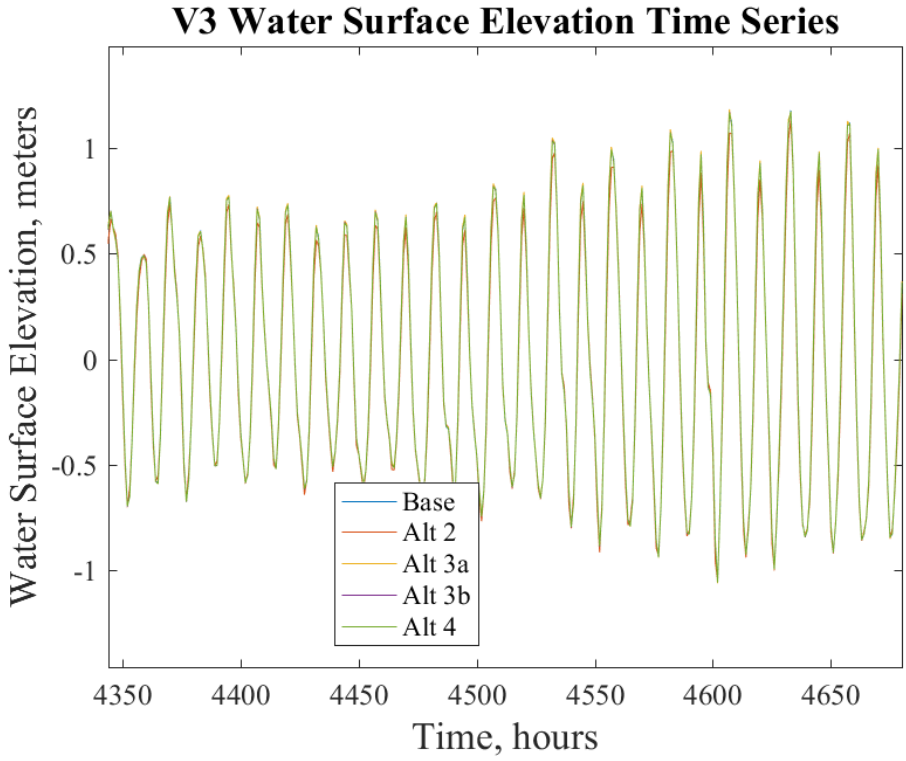


Figure 152: Set 2 Point V4 Water Surface Elevation

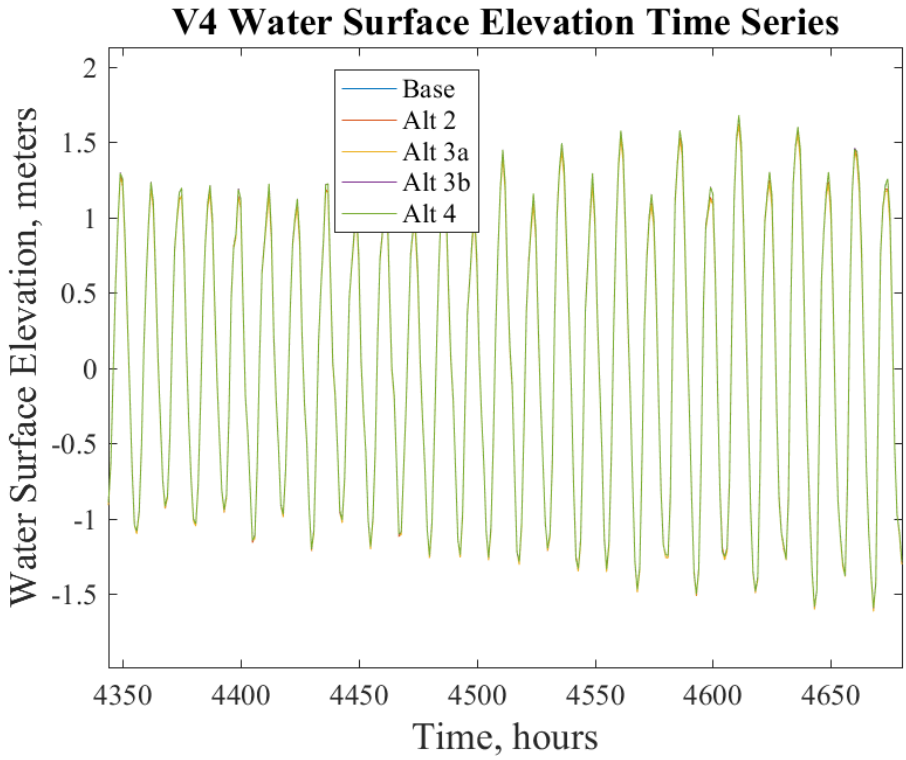


Figure 153: Set 2 Point V5 Water Surface Elevation

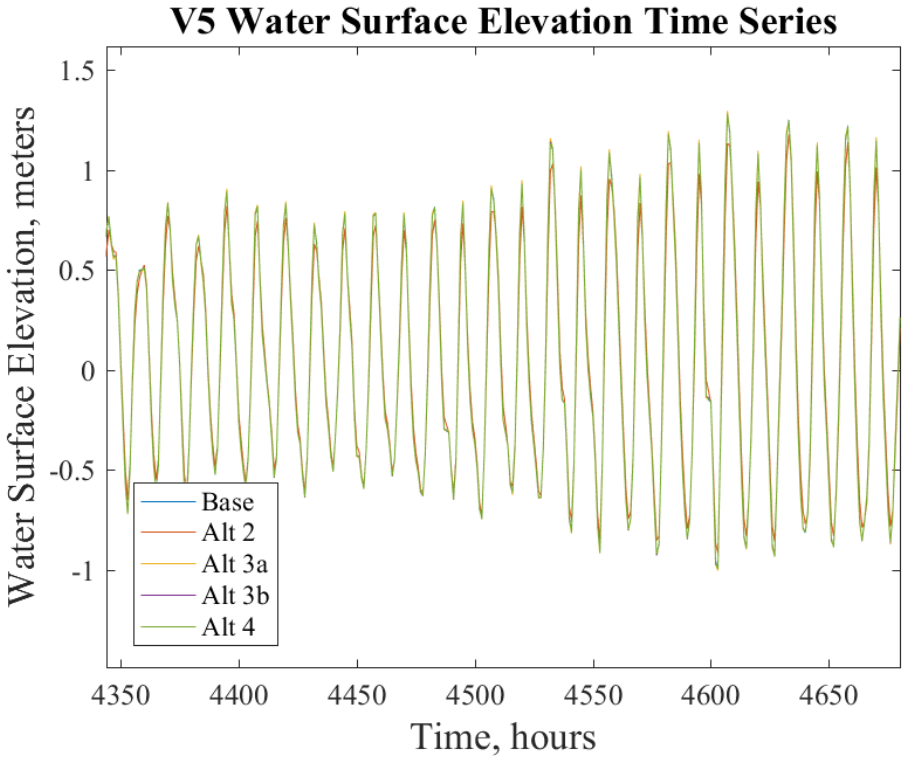


Figure 154: Set 2 Point V6 Water Surface Elevation

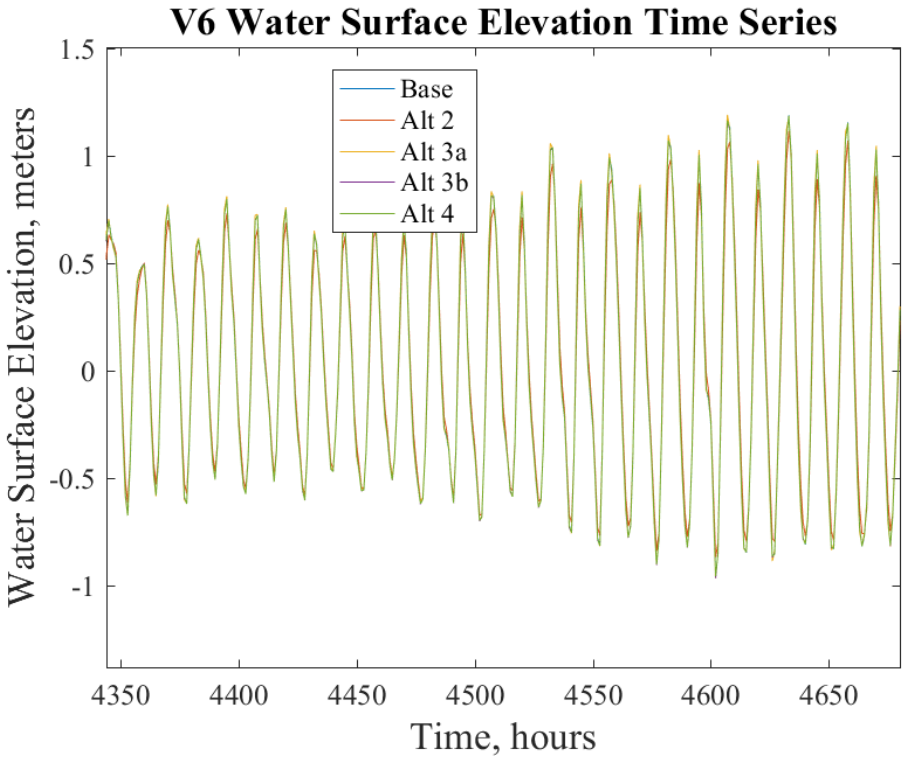


Figure 155: Set 2 Point S1 Water Surface Elevation

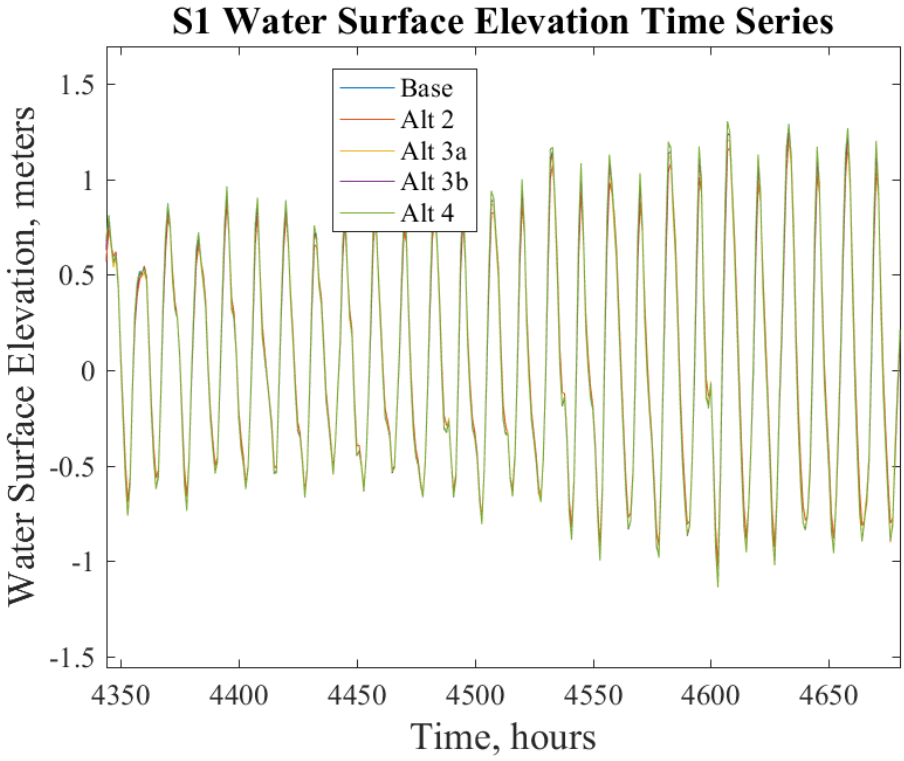


Figure 156: Set 2 Point S2 Water Surface Elevation

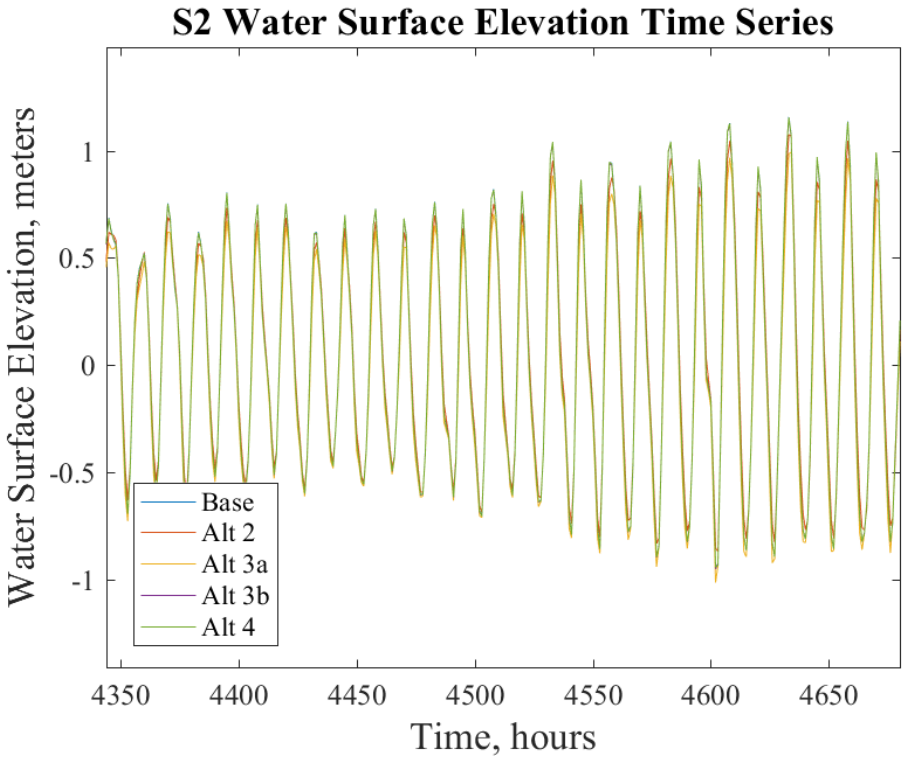


Figure 157: Set 2 Point S3 Water Surface Elevation

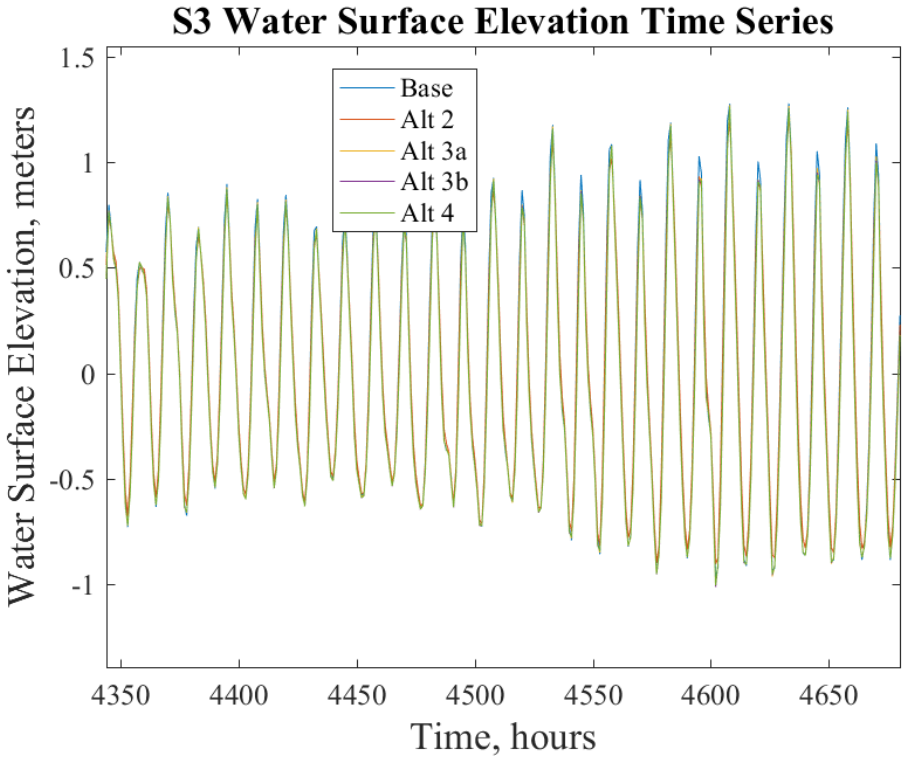


Figure 158: Set 2 Point S4 Water Surface Elevation

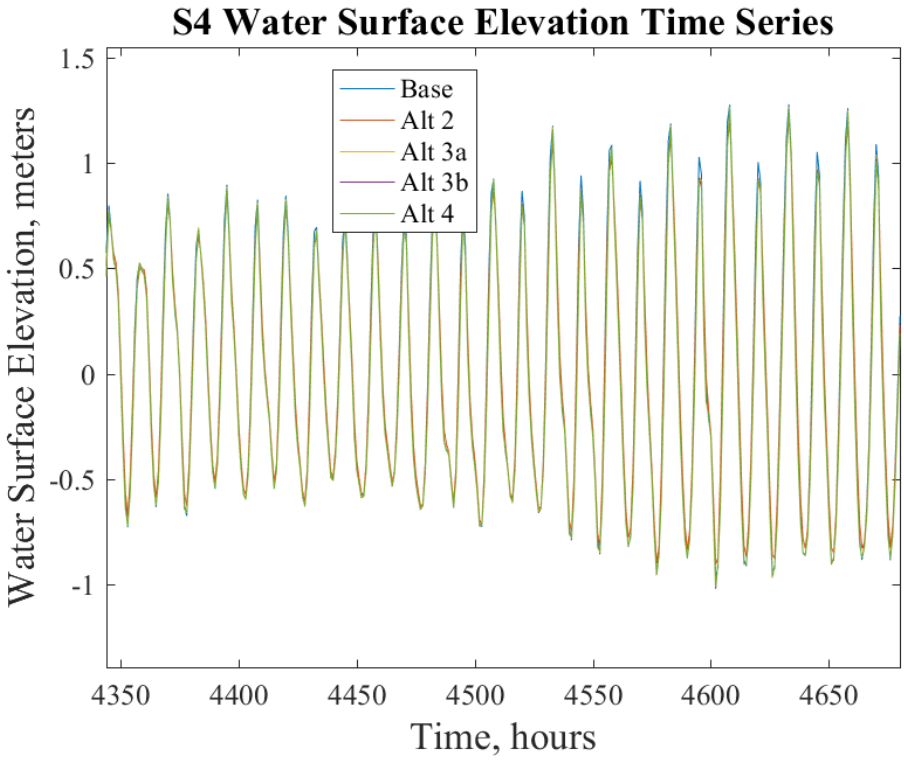


Figure 159: Set 2 Point S5 Water Surface Elevation

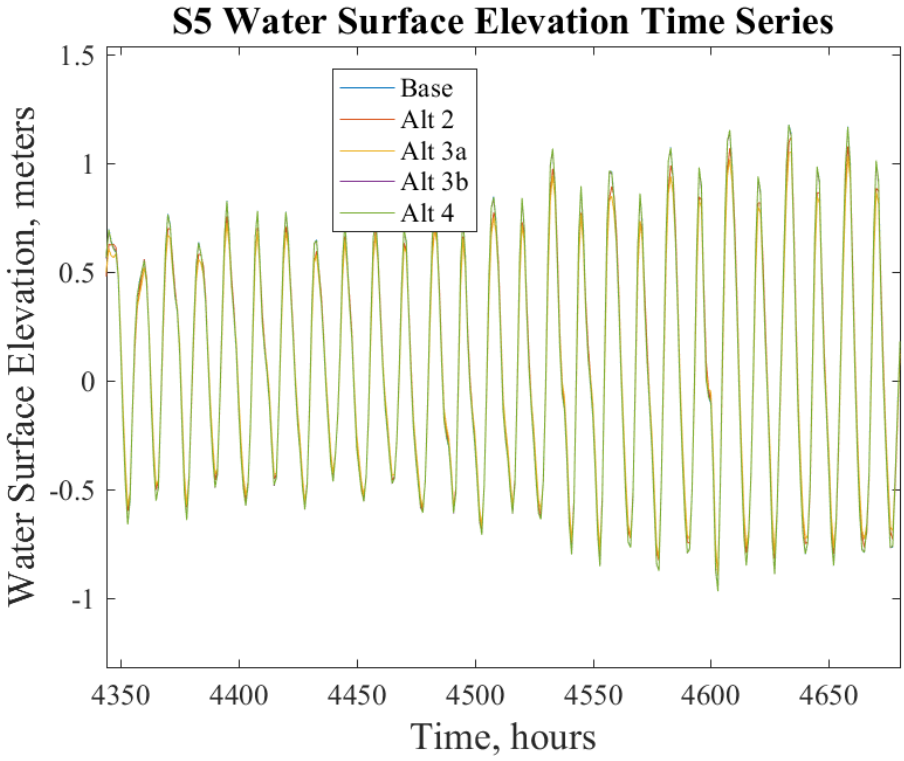


Figure 160: Set 2 Point T1 Water Surface Elevation

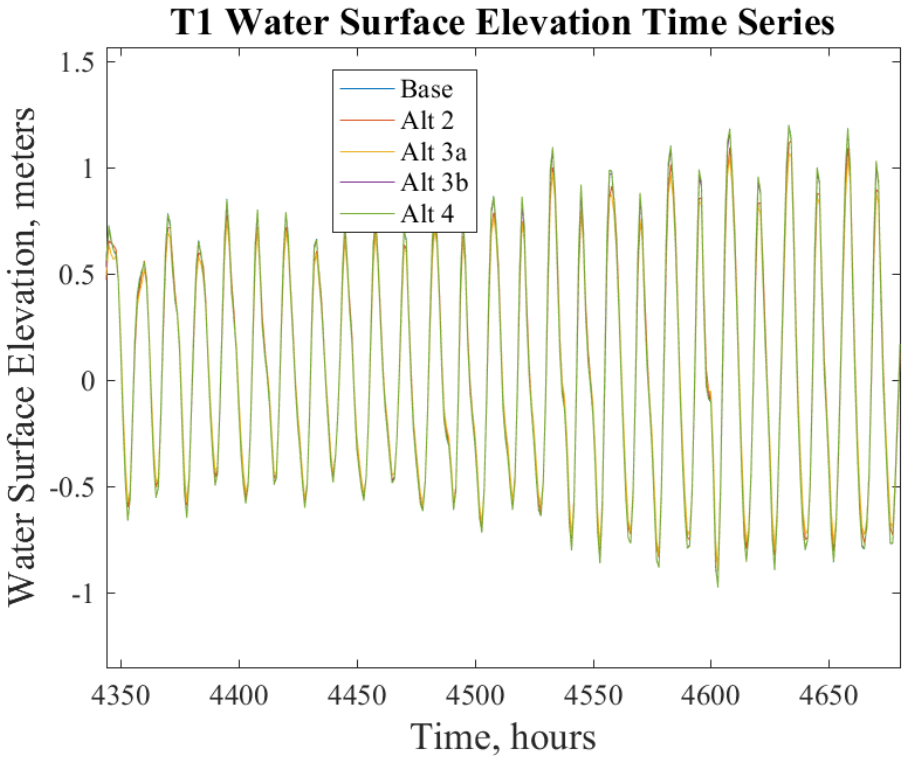


Figure 161: Set 2 Point T2 Water Surface Elevation

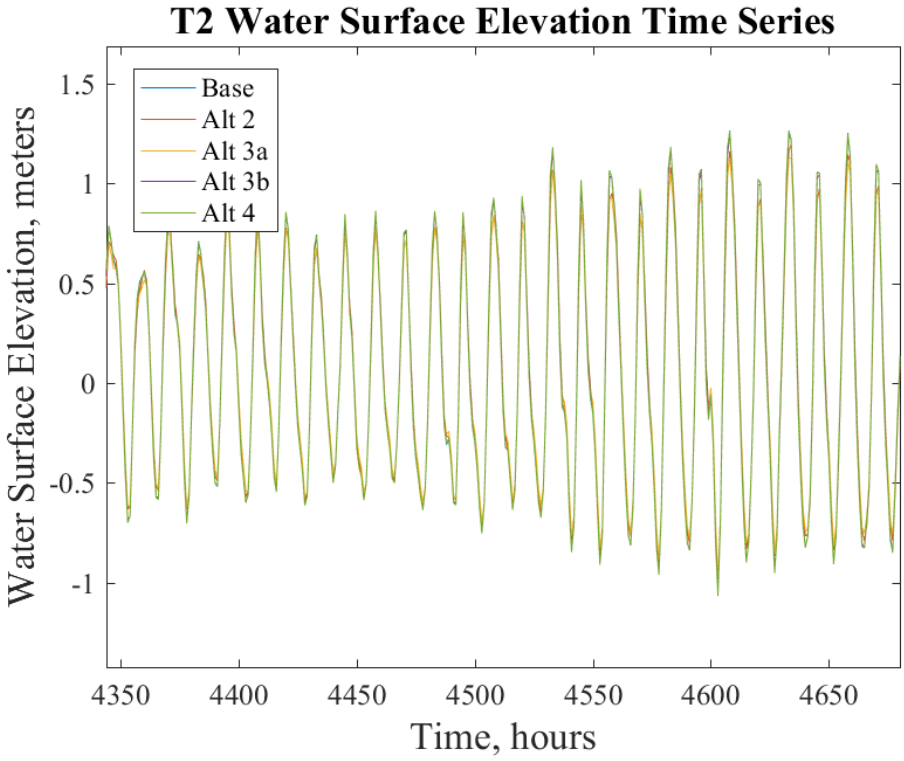


Figure 162: Set 2 Point T3 Water Surface Elevation

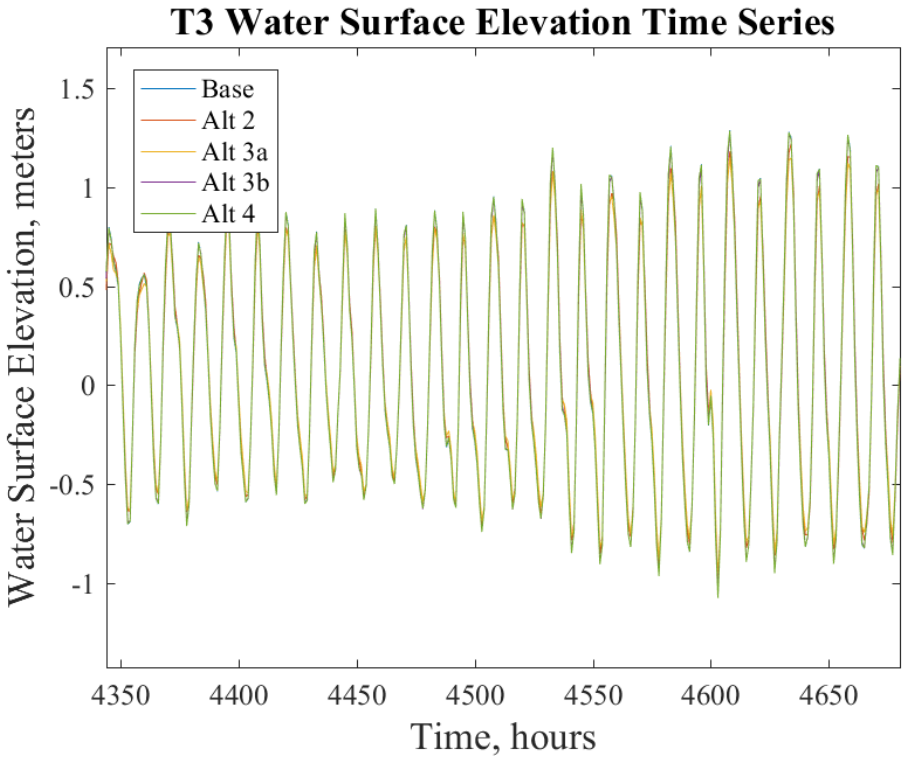


Figure 163: Set 2 Point T4 Water Surface Elevation

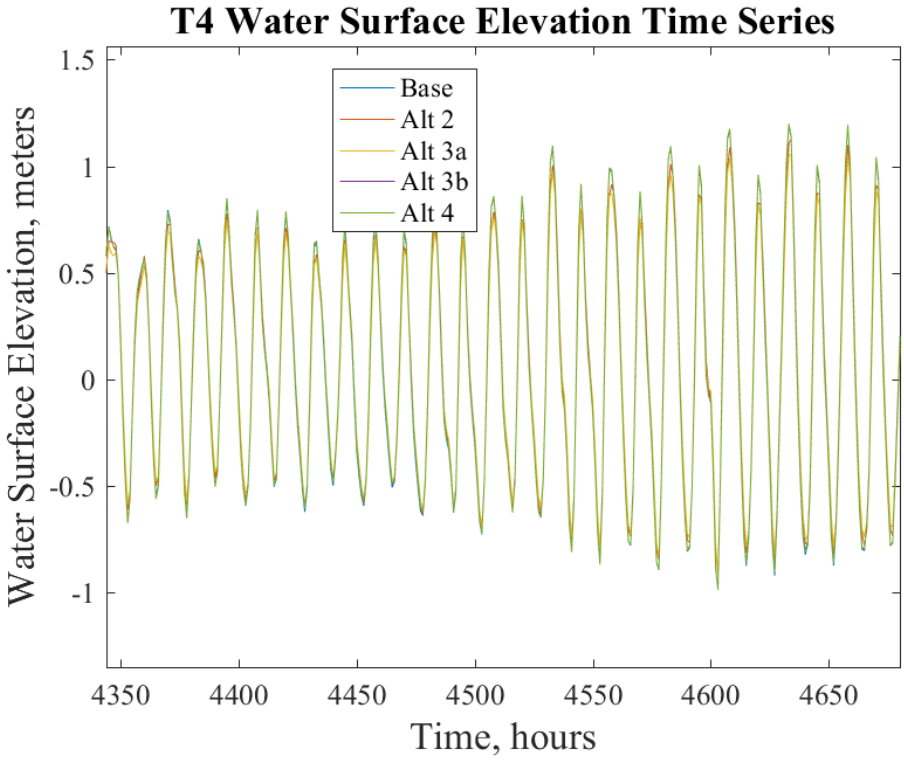


Figure 164: Set 2 Point T5 Water Surface Elevation

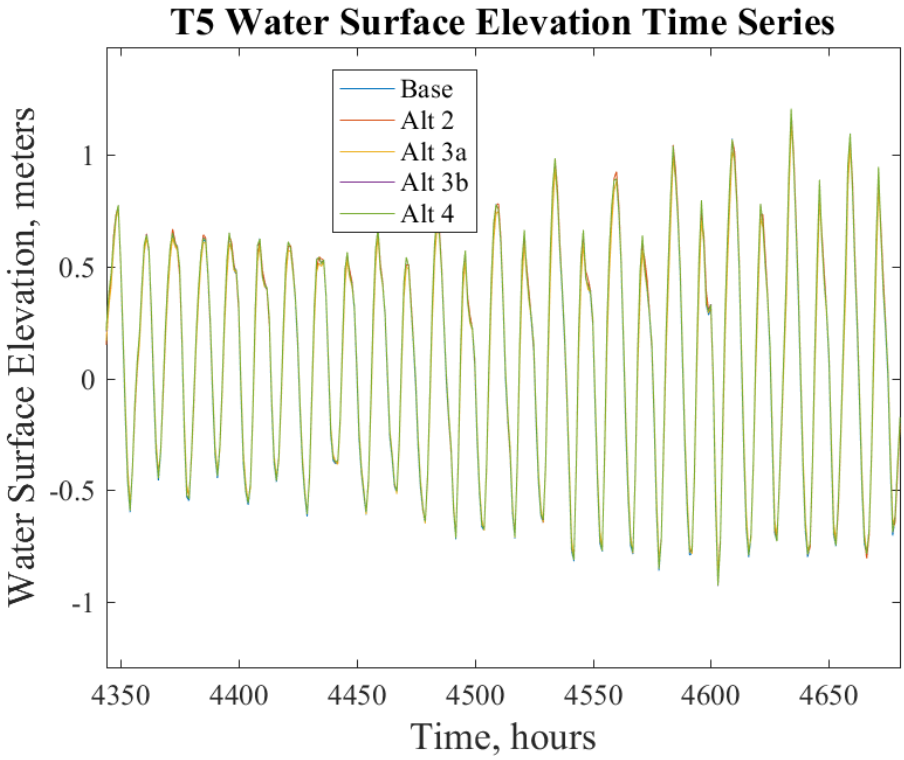


Figure 165: Set 2 Point R1 Water Surface Elevation

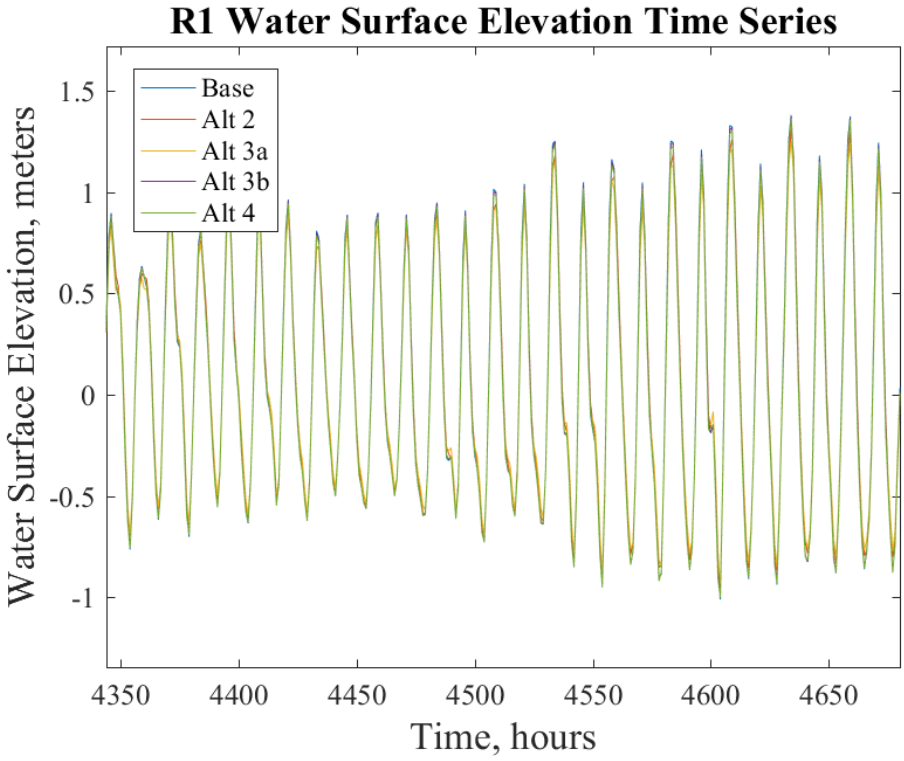


Figure 166: Set 2 Point Fresh Kills Water Surface Elevation

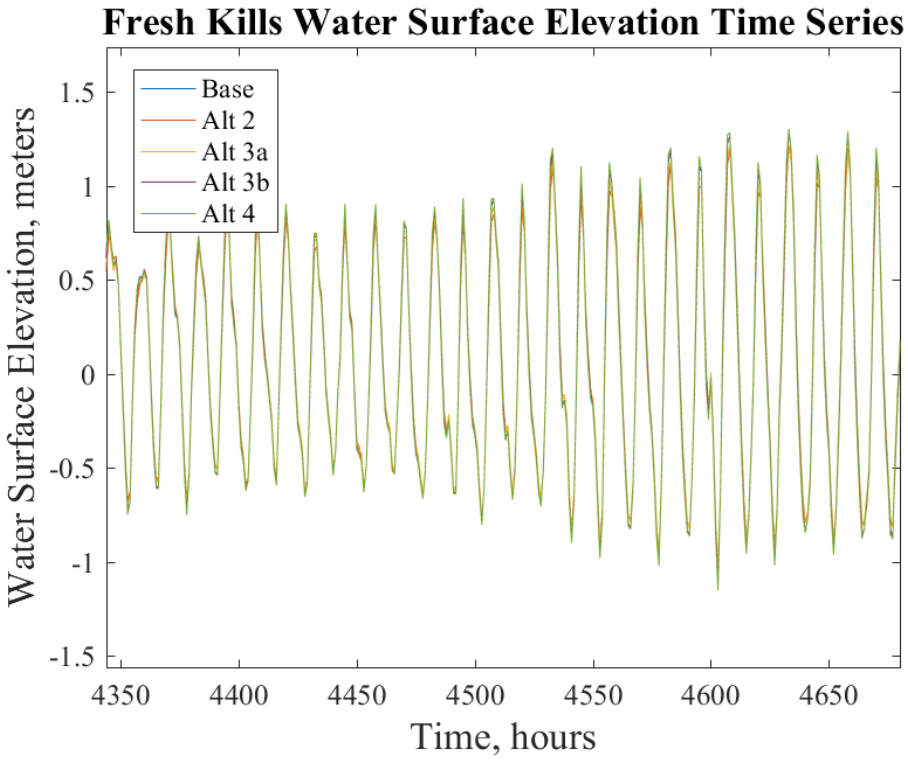


Figure 167: Set 2 Point Sandy Hook Water Surface Elevation

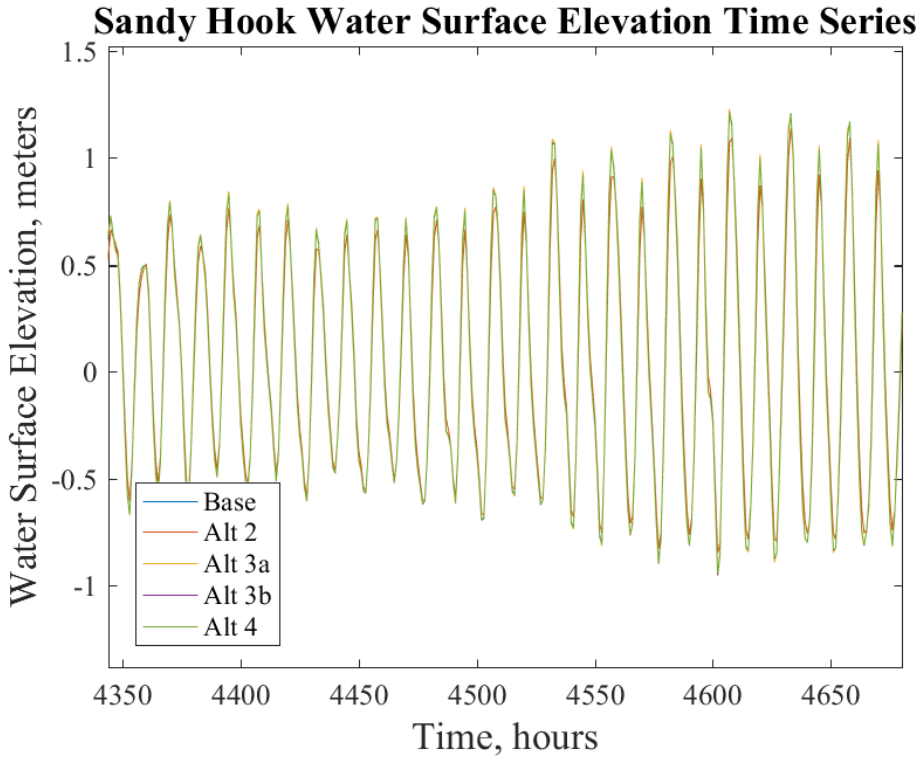


Figure 168: Set 2 Point Jamaica Bay Water Surface Elevation

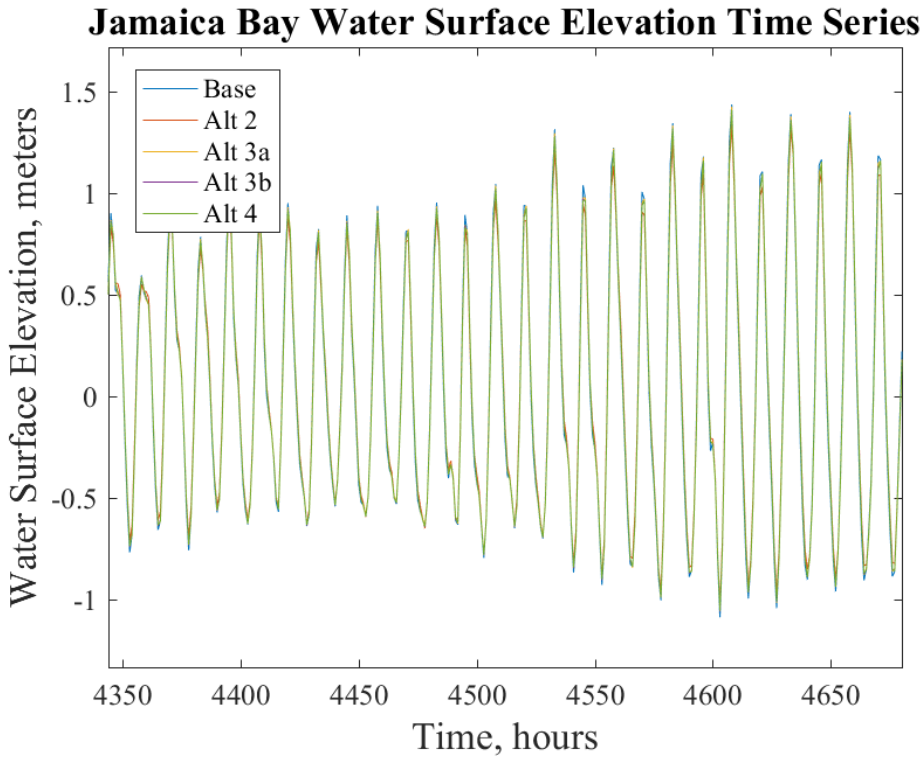


Figure 169: Set 2 Point Robbins Reef Water Surface Elevation

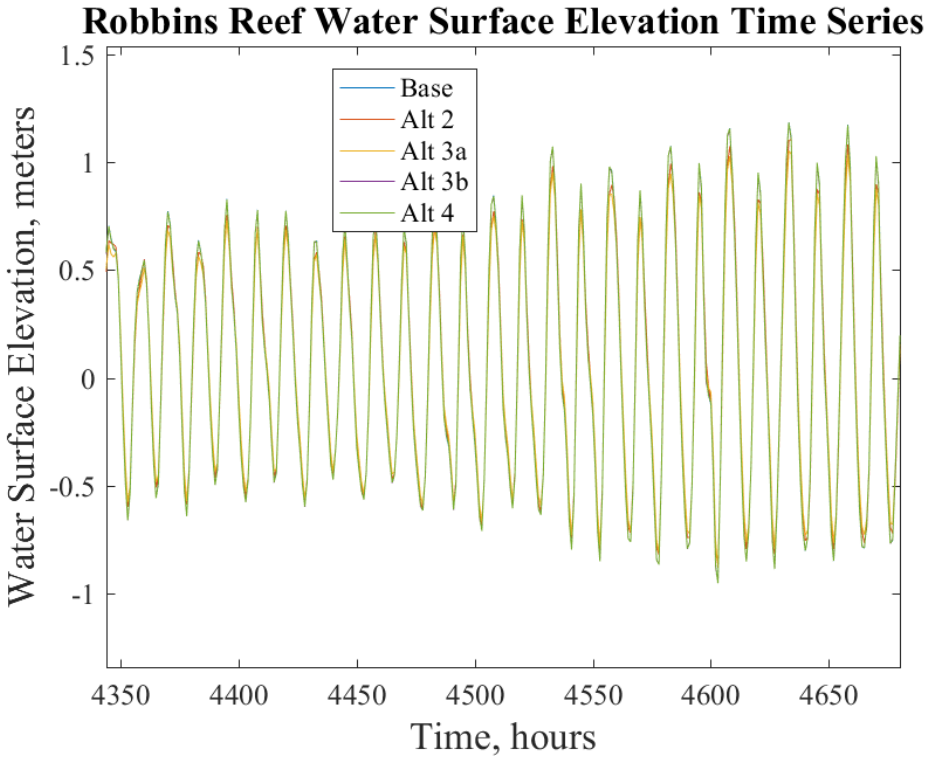


Figure 170: Set 2 Point Battery Water Surface Elevation

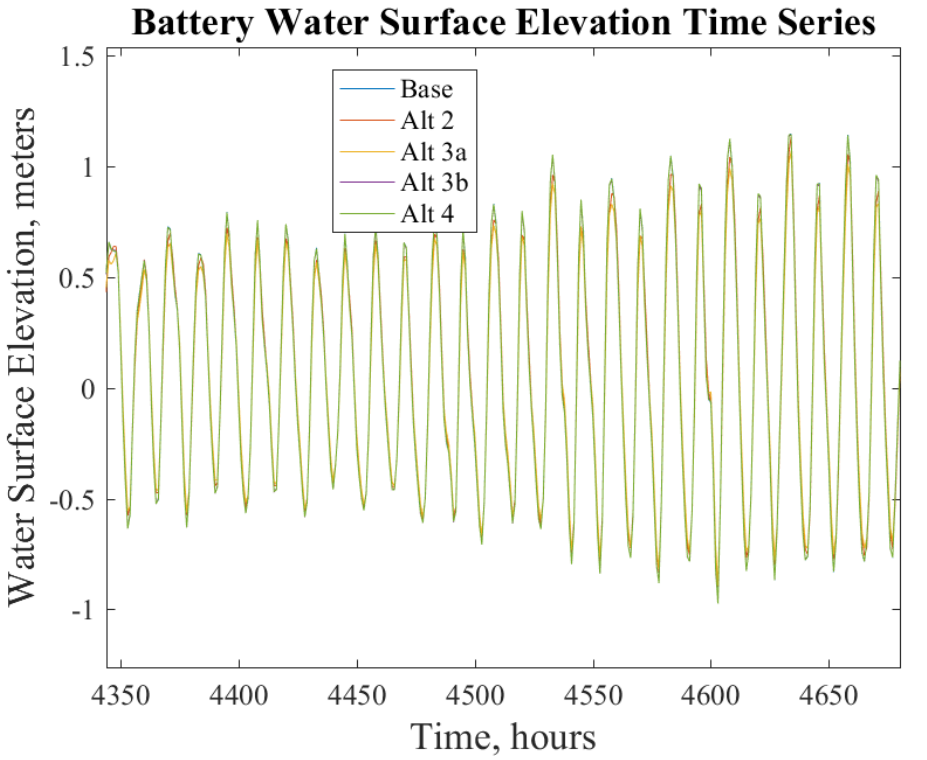


Figure 171: Set 2 Point KLGA Water Surface Elevation

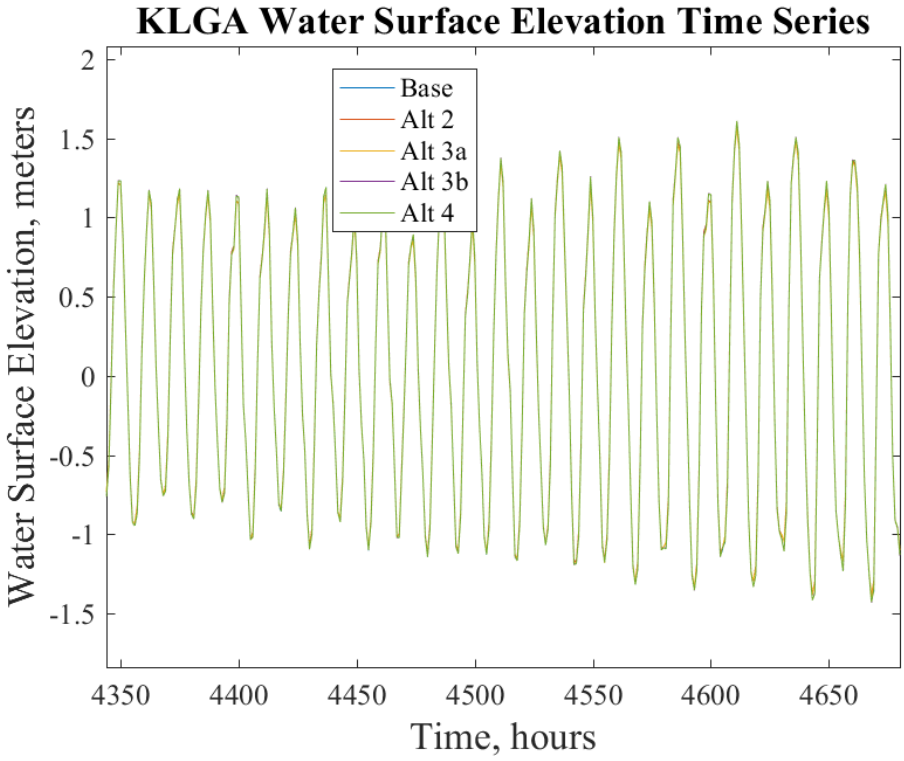


Figure 172: Set 2 Point Mariners Harbor Water Surface Elevation

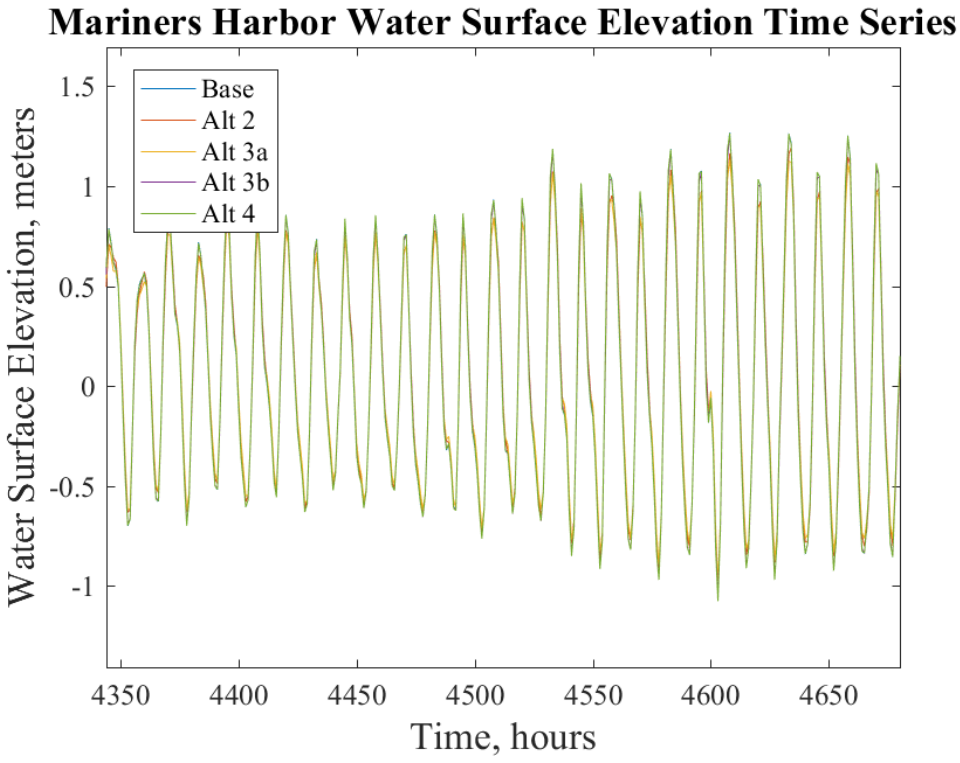


Figure 173: Set 2 Point North Reach Water Surface Elevation

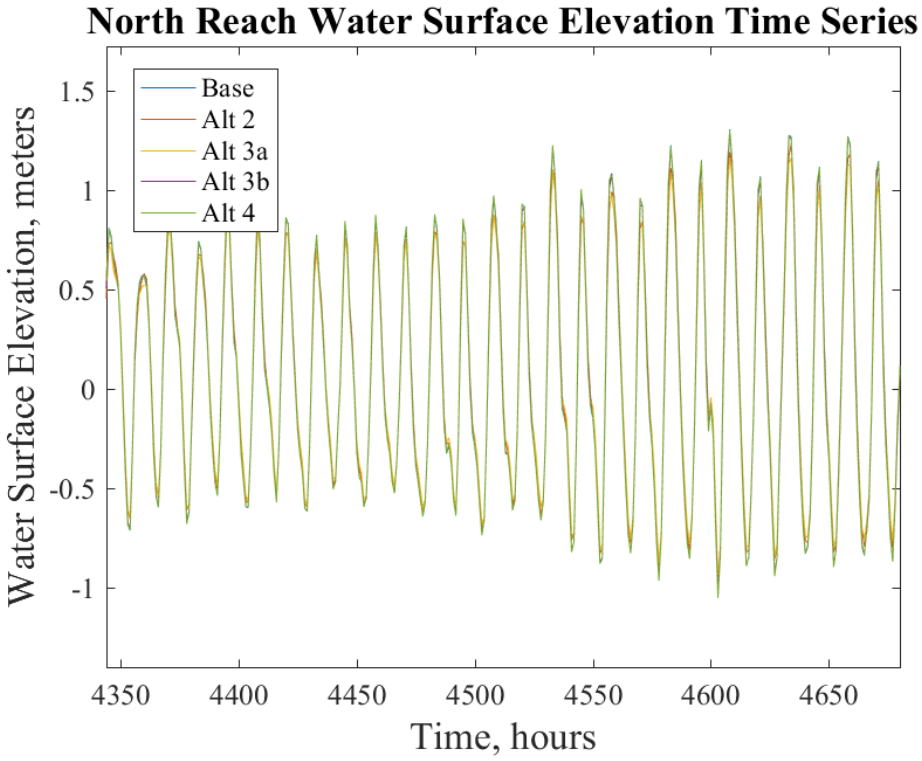
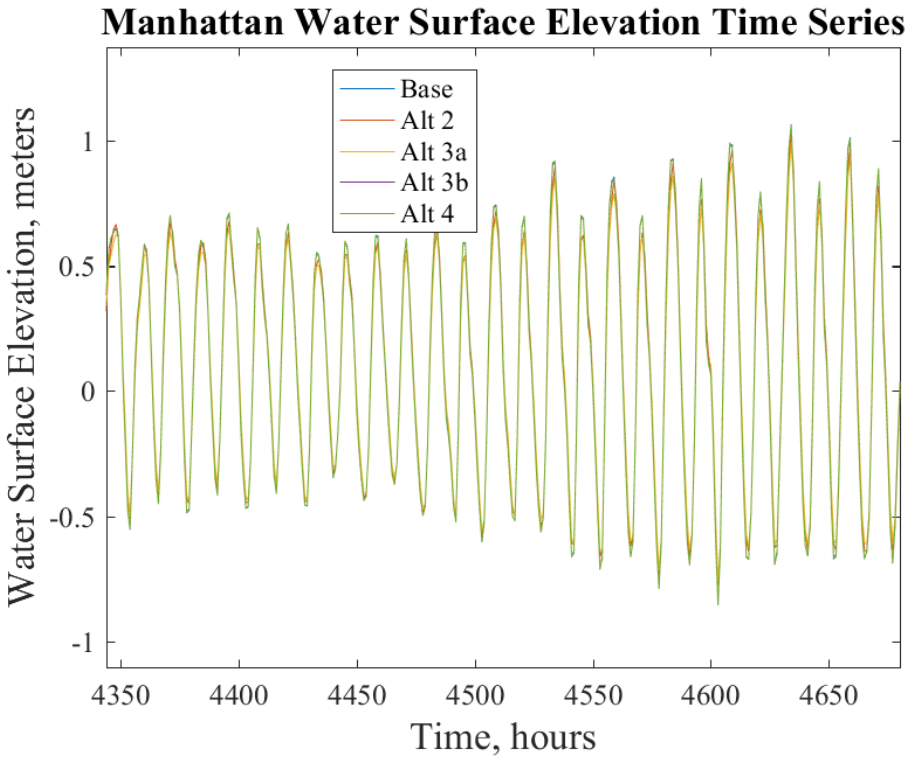


Figure 174: Set 2 Point Manhattan Water Surface Elevation



3.2.1.2 Velocity Point Comparisons

In Figure 175 through Figure 200, the depth average velocity magnitude time series values are plotted for each alternative and the without project for the locations shown in Figure 148.

Figure 175: Set 2 Point V1 Depth Averaged Velocity Magnitudes

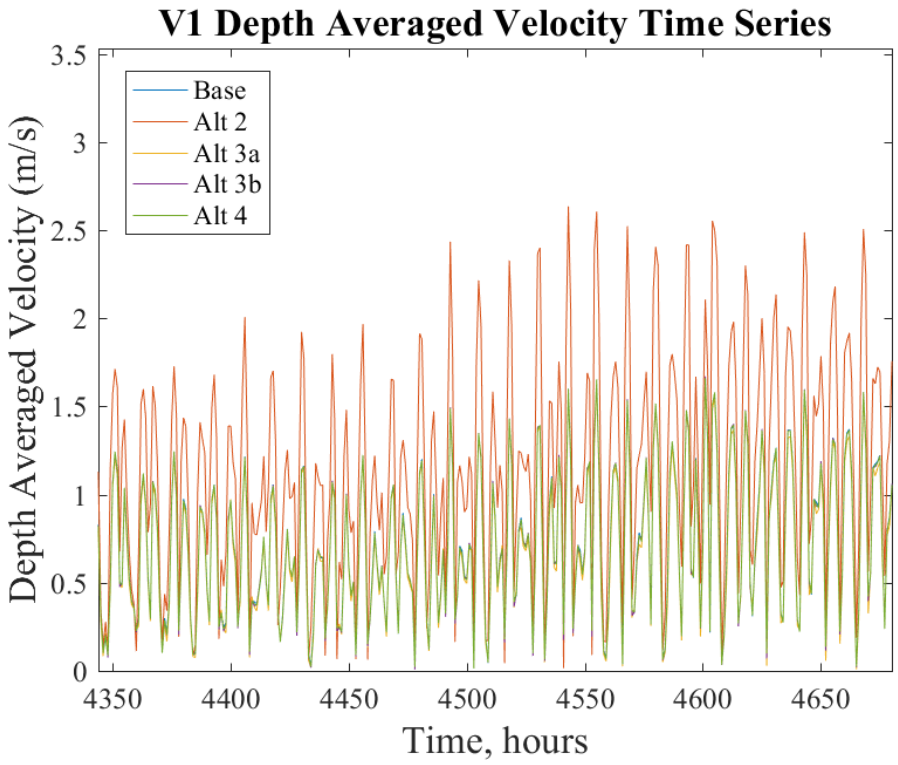


Figure 176: Set 2 Point V2 Depth Averaged Velocity Magnitudes

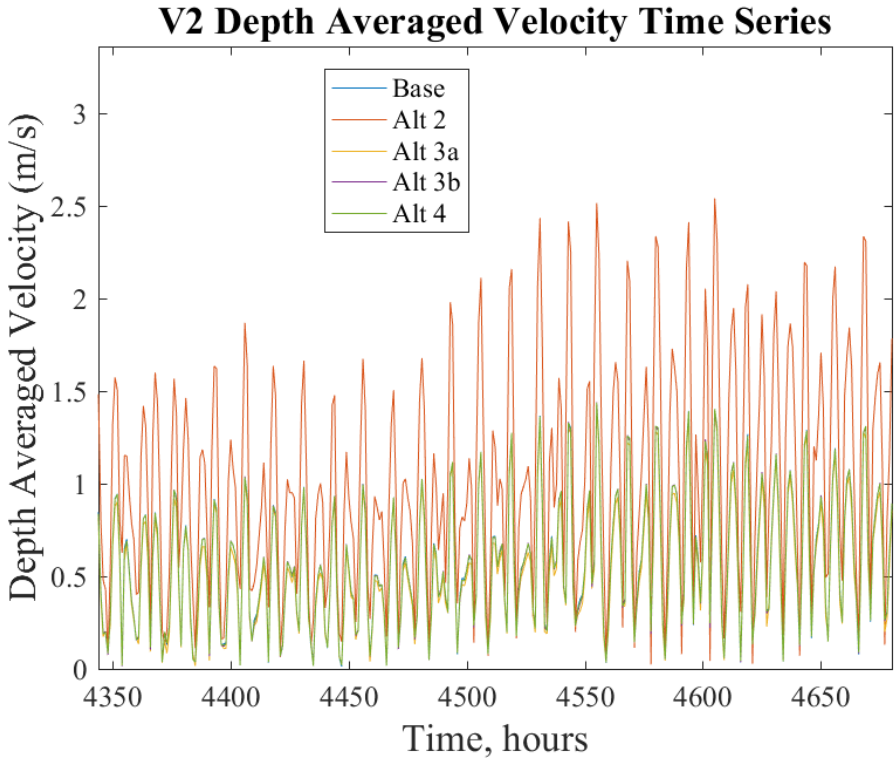


Figure 177: Set 2 Point V3 Depth Averaged Velocity Magnitudes

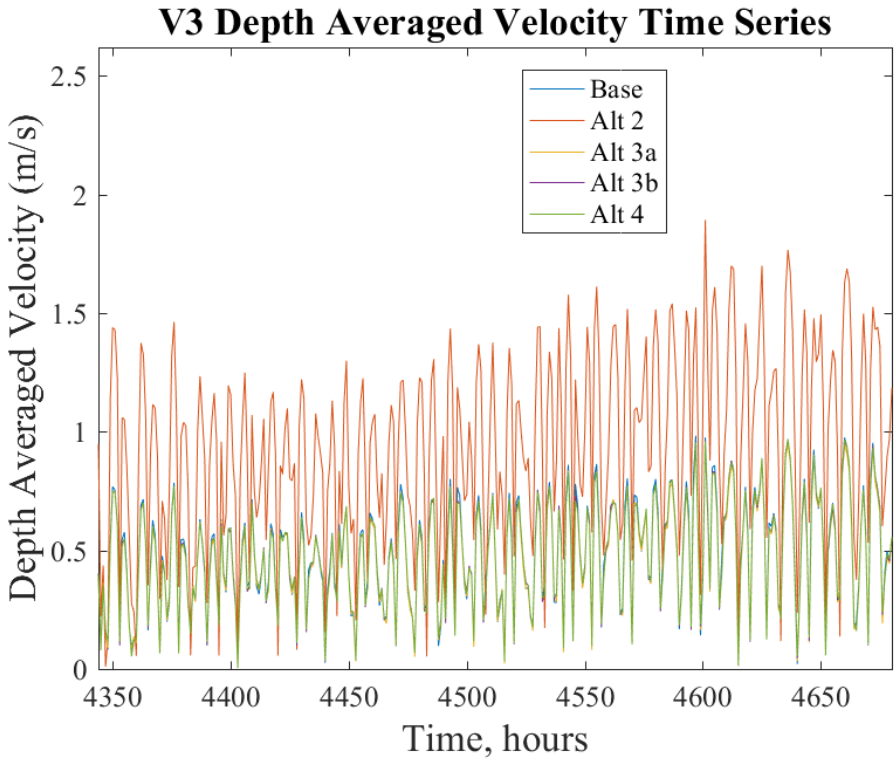


Figure 178: Set 2 Point V4 Depth Averaged Velocity Magnitudes

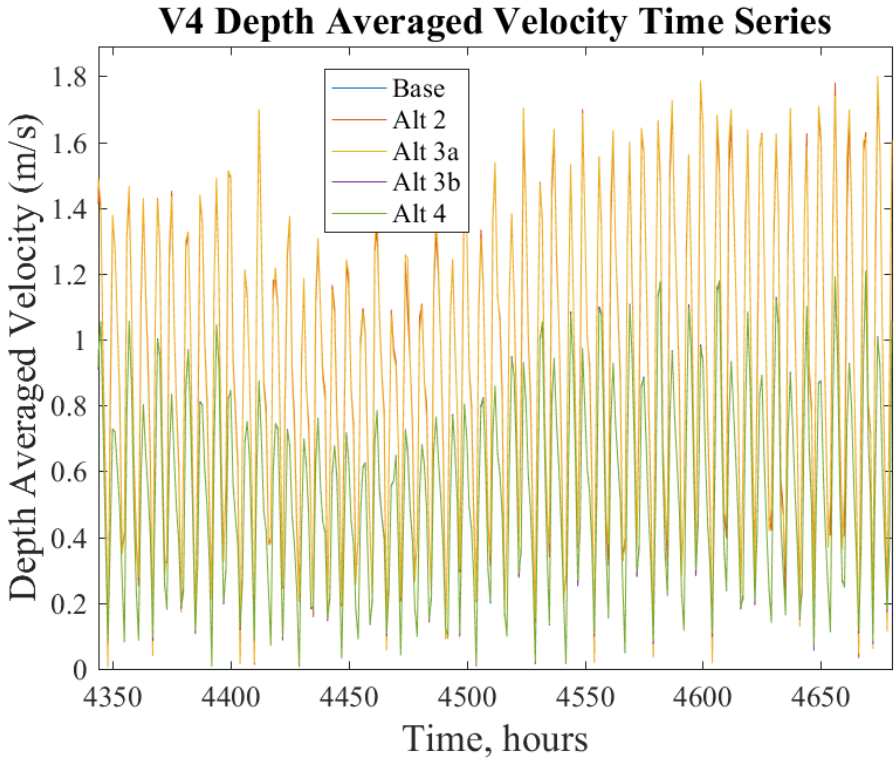


Figure 179: Set 2 Point V5 Depth Averaged Velocity Magnitudes

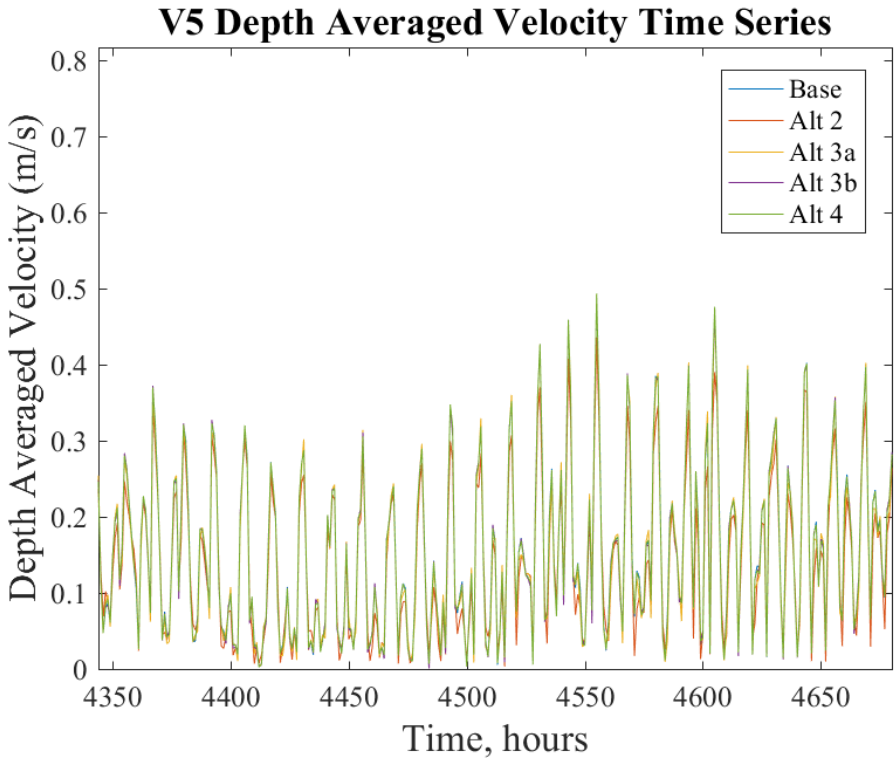


Figure 180: Set 2 Point V6 Depth Averaged Velocity Magnitudes

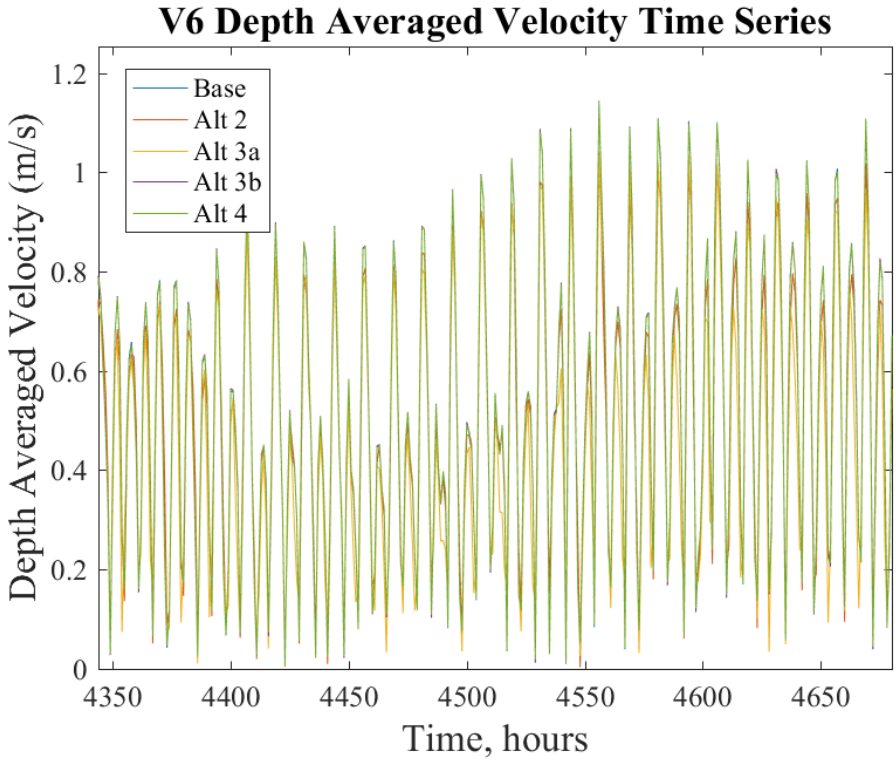


Figure 181: Set 2 Point S1 Depth Averaged Velocity Magnitudes

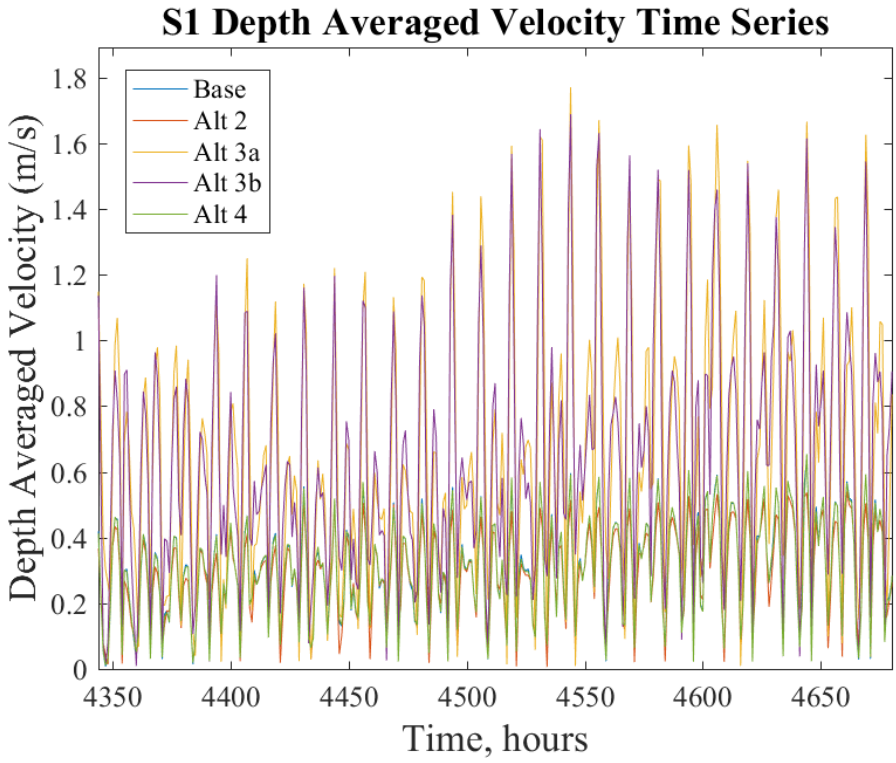


Figure 182: Set 2 Point S2 Depth Averaged Velocity Magnitudes

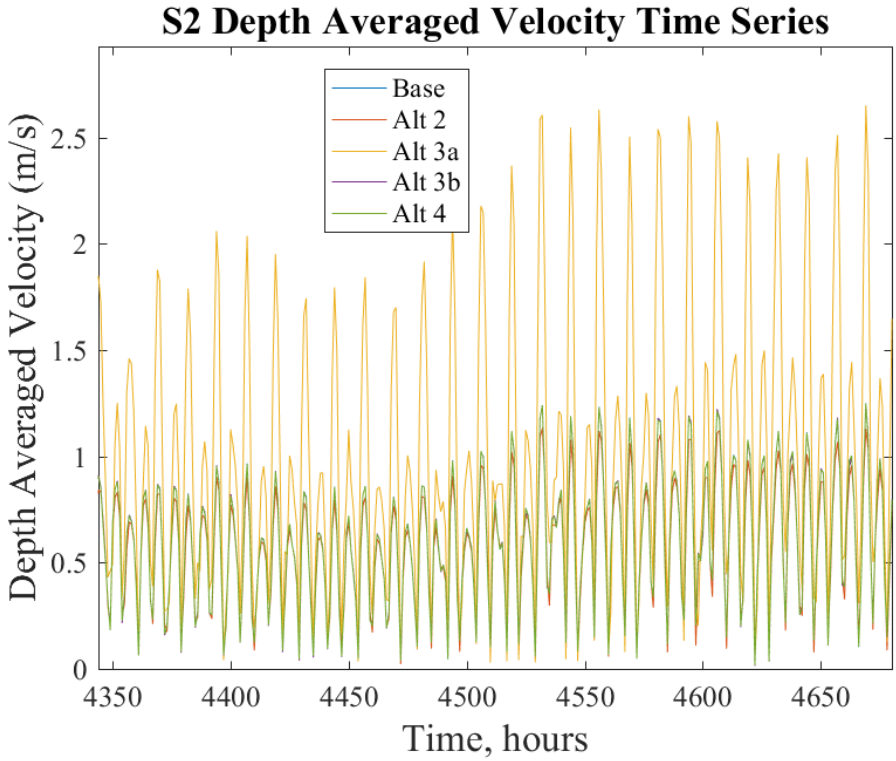


Figure 183: Set 2 Point S3 Depth Averaged Velocity Magnitudes

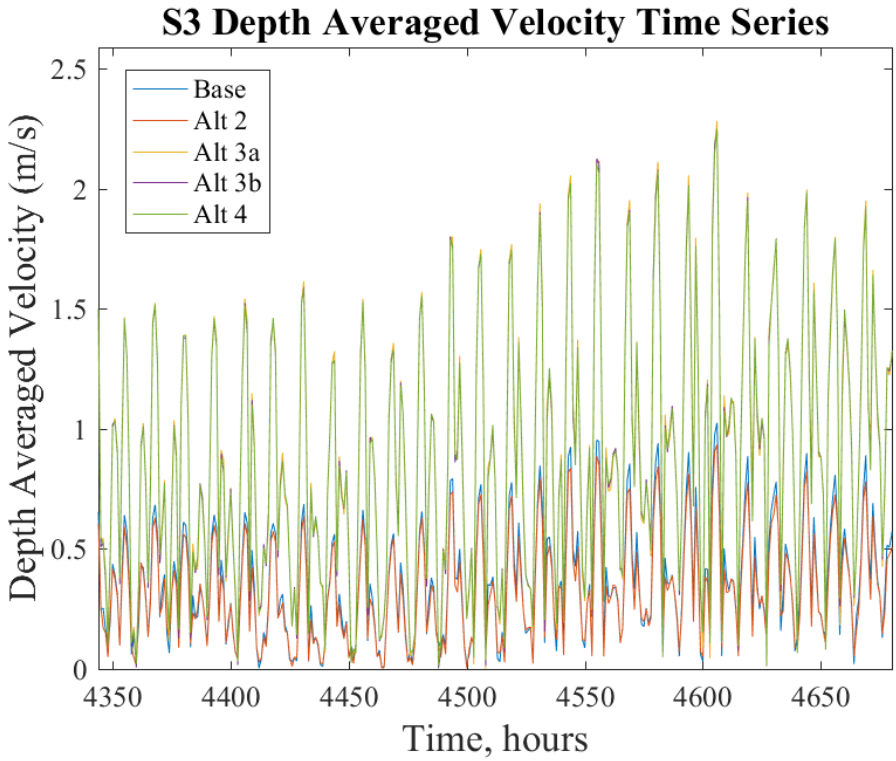


Figure 184: Set 2 Point S4 Depth Averaged Velocity Magnitudes

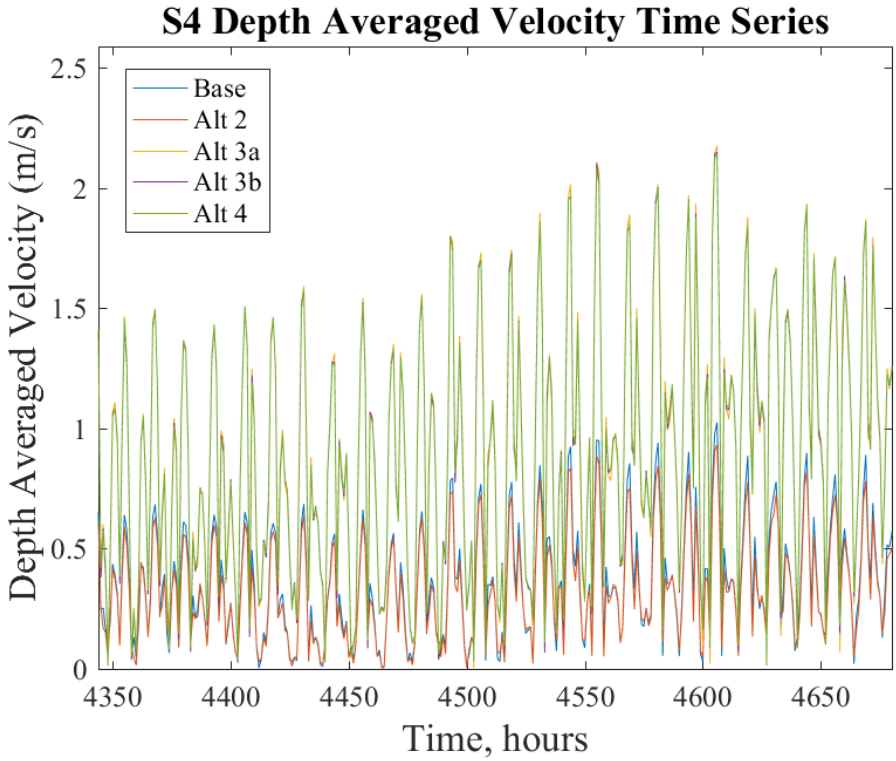


Figure 185: Set 2 Point S5 Depth Averaged Velocity Magnitudes

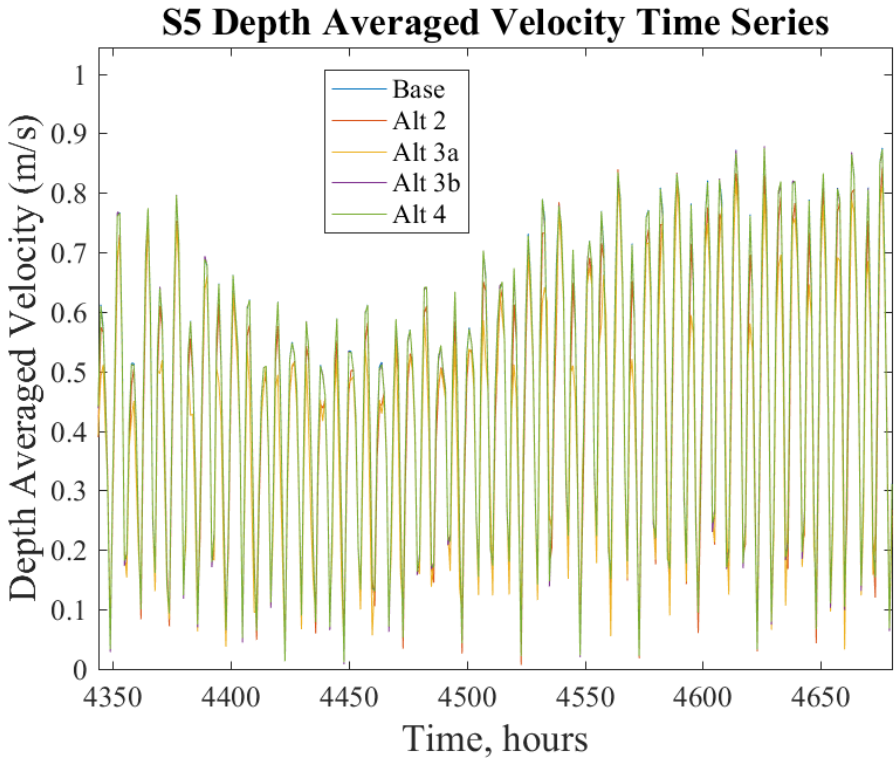


Figure 186: Set 2 Point T1 Depth Averaged Velocity Magnitudes

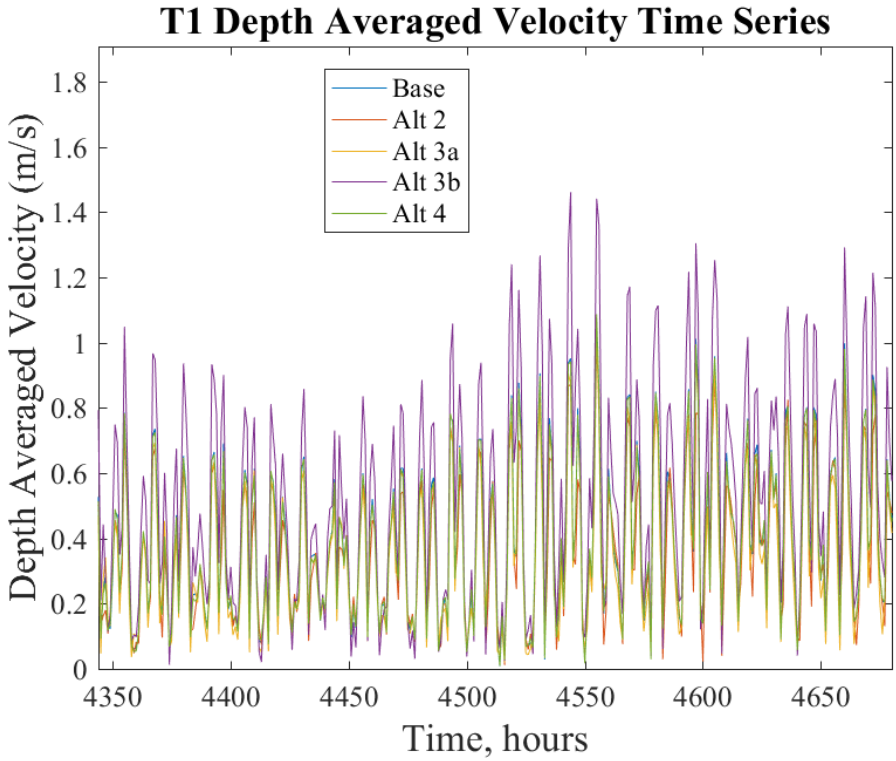


Figure 187: Set 2 Point T2 Depth Averaged Velocity Magnitudes

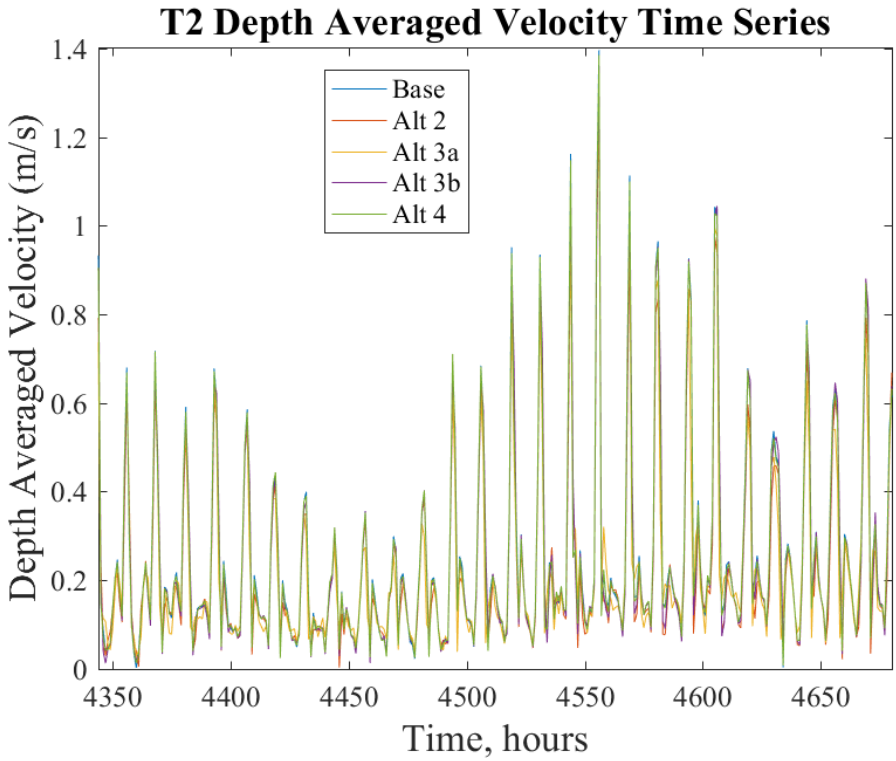


Figure 188: Set 2 Point T3 Depth Averaged Velocity Magnitudes

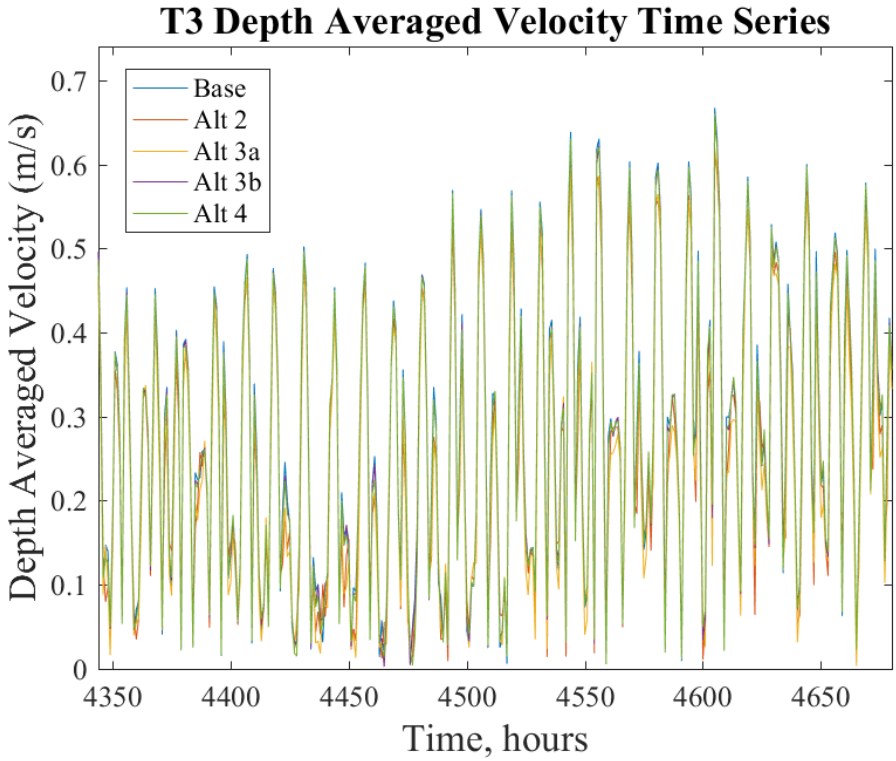


Figure 189: Set 2 Point T4 Depth Averaged Velocity Magnitudes

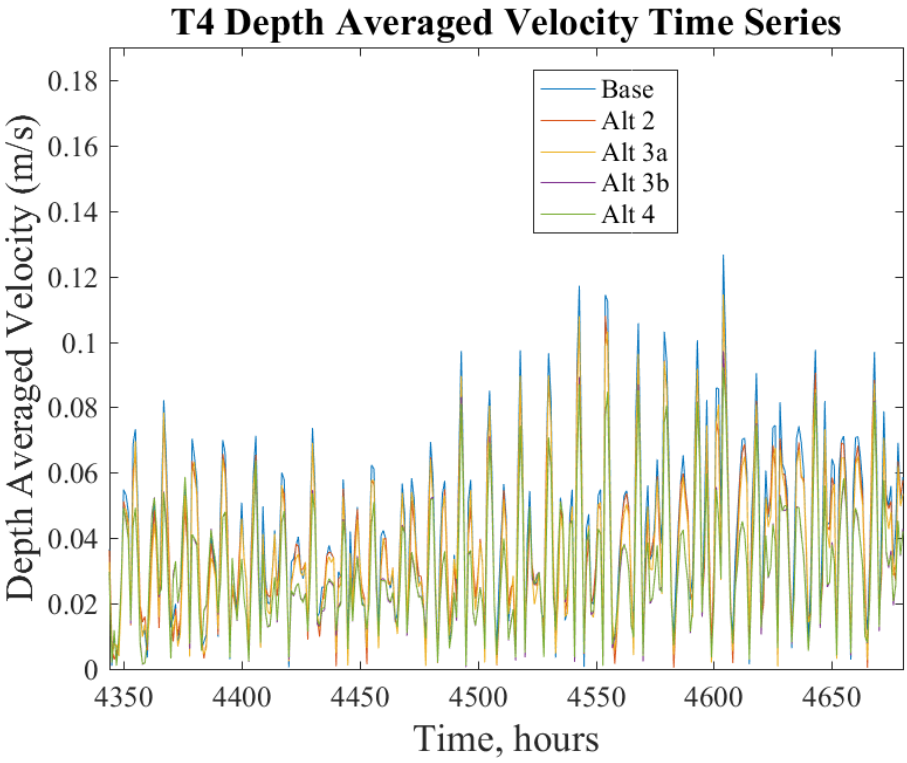


Figure 190: Set 2 Point T5 Depth Averaged Velocity Magnitudes

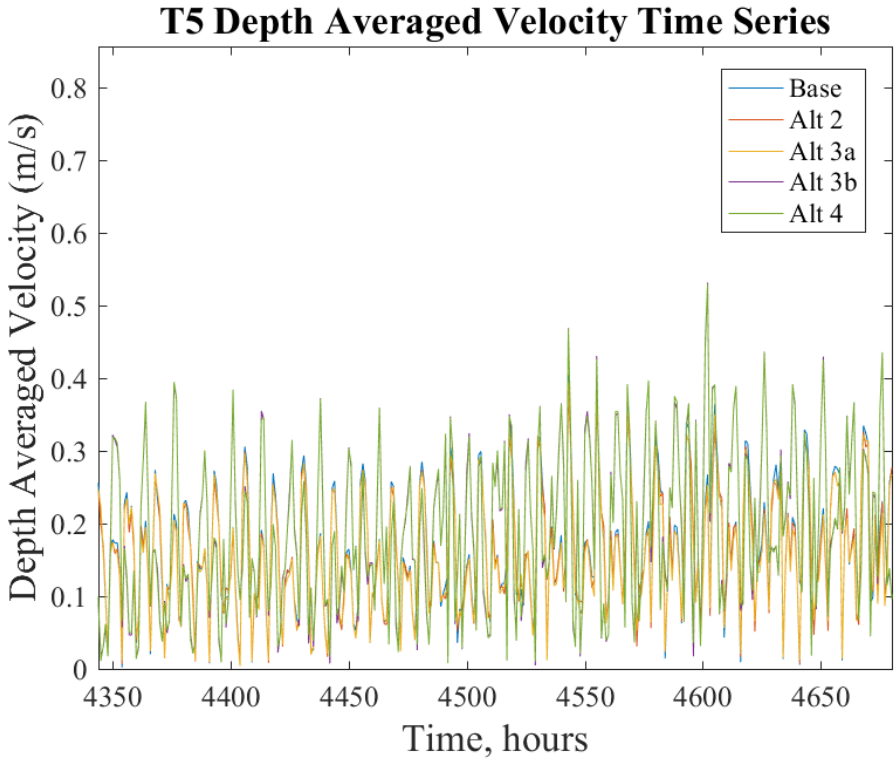


Figure 191: Set 2 Point R1 Depth Averaged Velocity Magnitudes

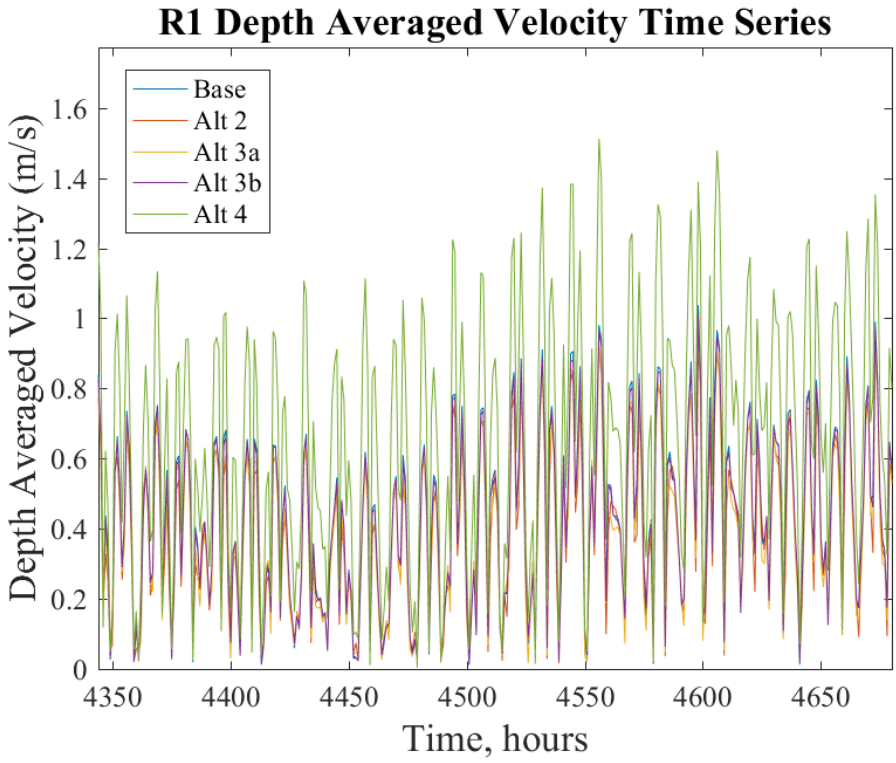


Figure 192: Set 2 Point Fresh Kills Depth Averaged Velocity Magnitudes

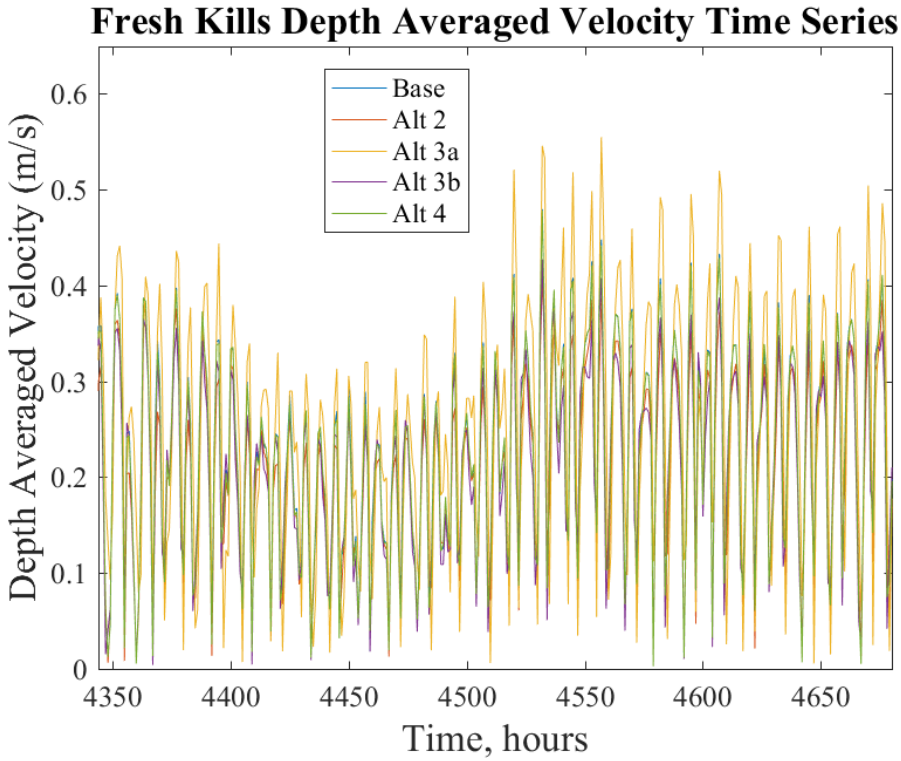


Figure 193: Set 2 Point Sandy Hook Depth Averaged Velocity Magnitudes

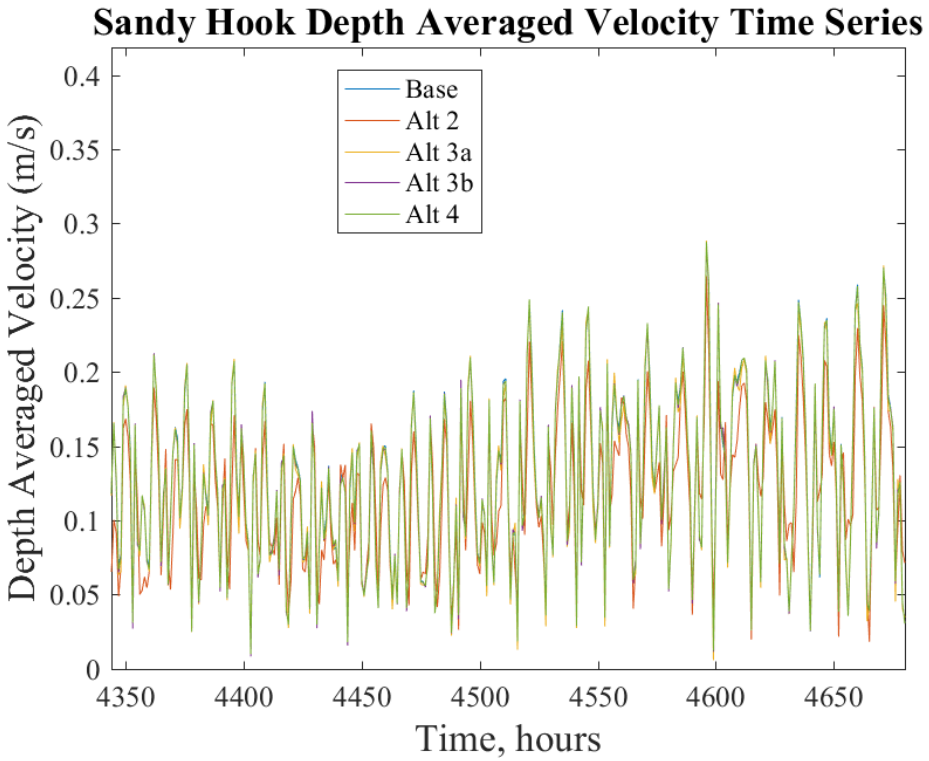


Figure 194: Set 2 Point Jamaica Bay Depth Averaged Velocity Magnitudes

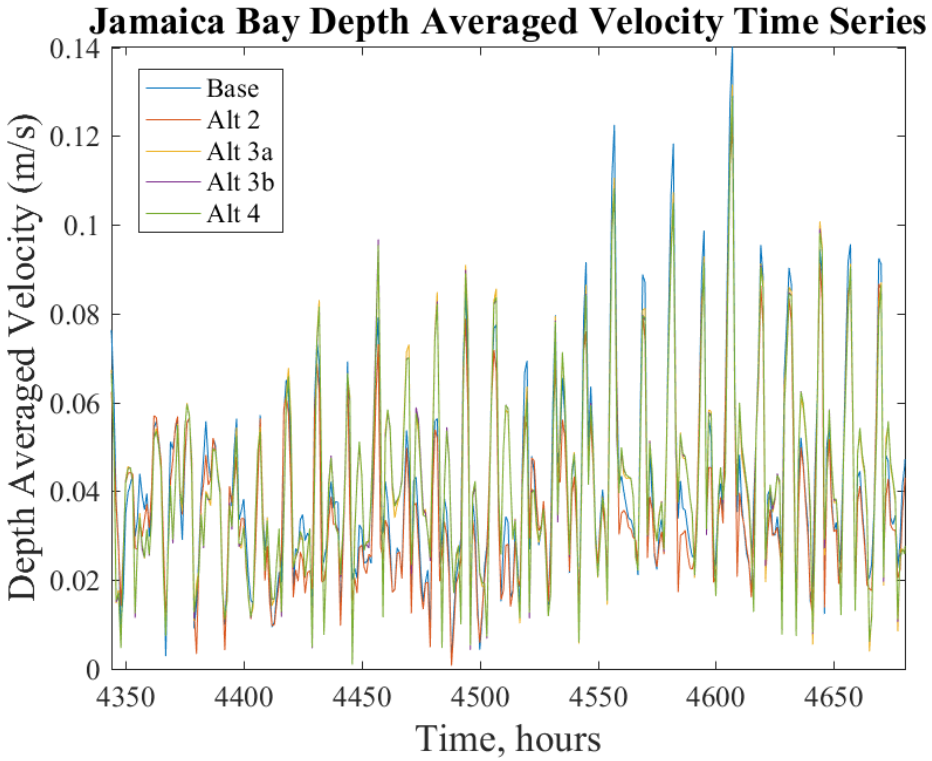


Figure 195: Set 2 Point Robbins Reef Depth Averaged Velocity Magnitudes

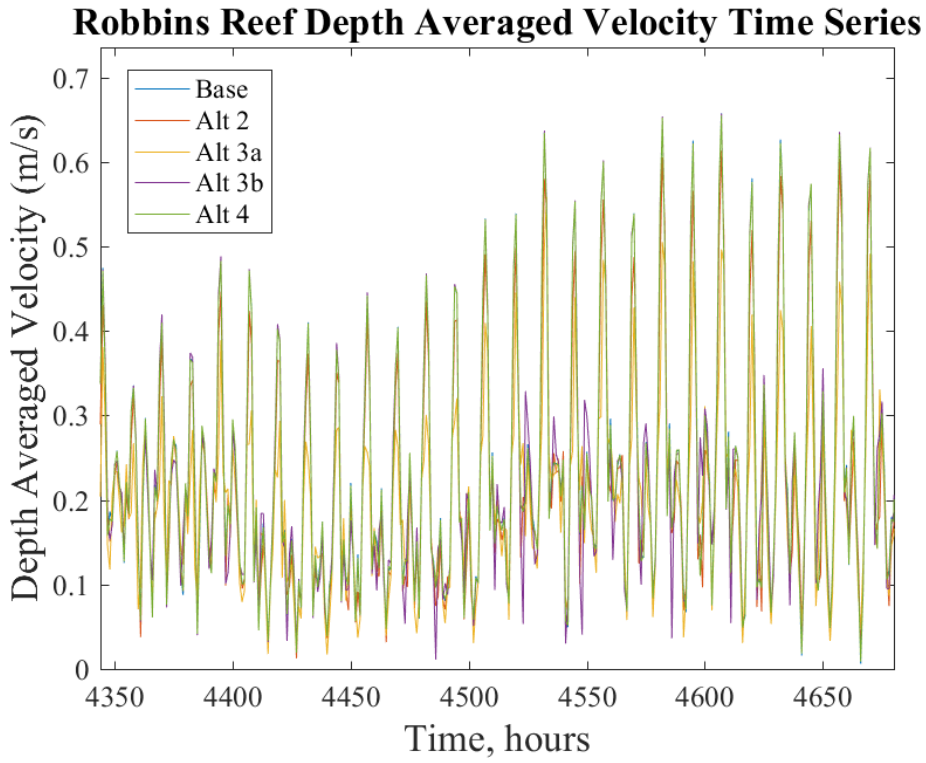


Figure 196: Set 2 Point Battery Depth Averaged Velocity Magnitudes

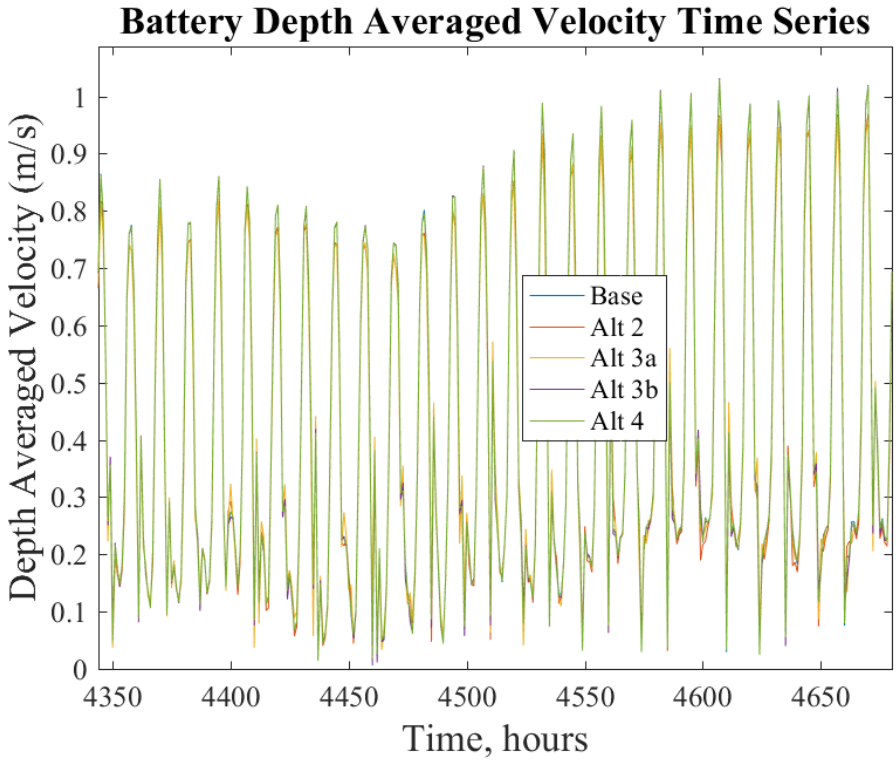


Figure 197: Set 2 Point KLGA Depth Averaged Velocity Magnitudes

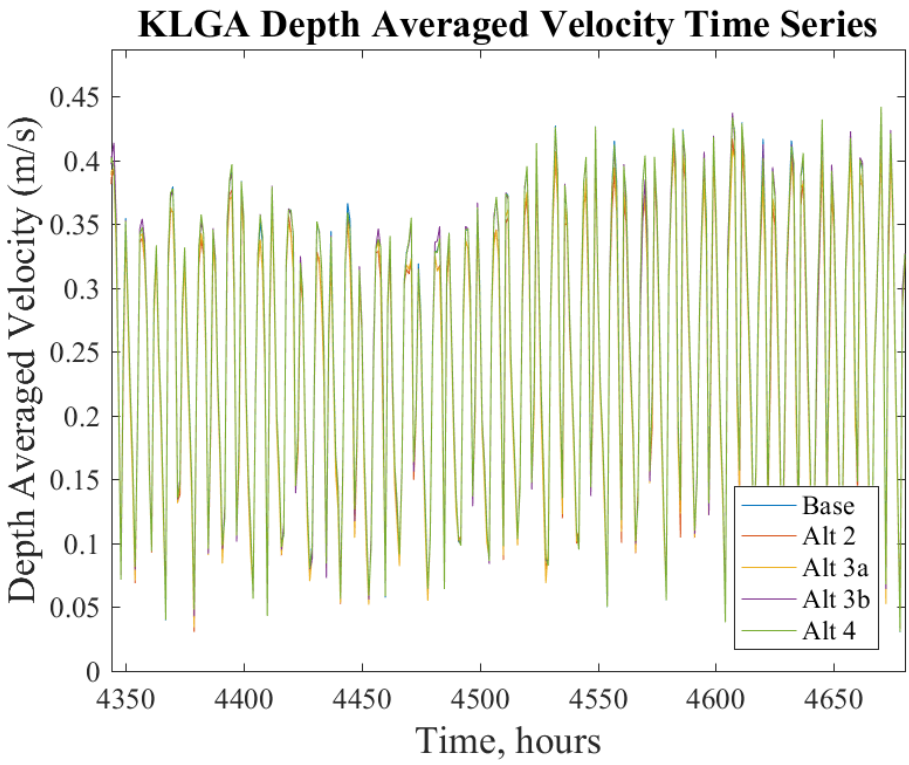


Figure 198: Set 2 Point Mariners Harbor Depth Averaged Velocity Magnitudes

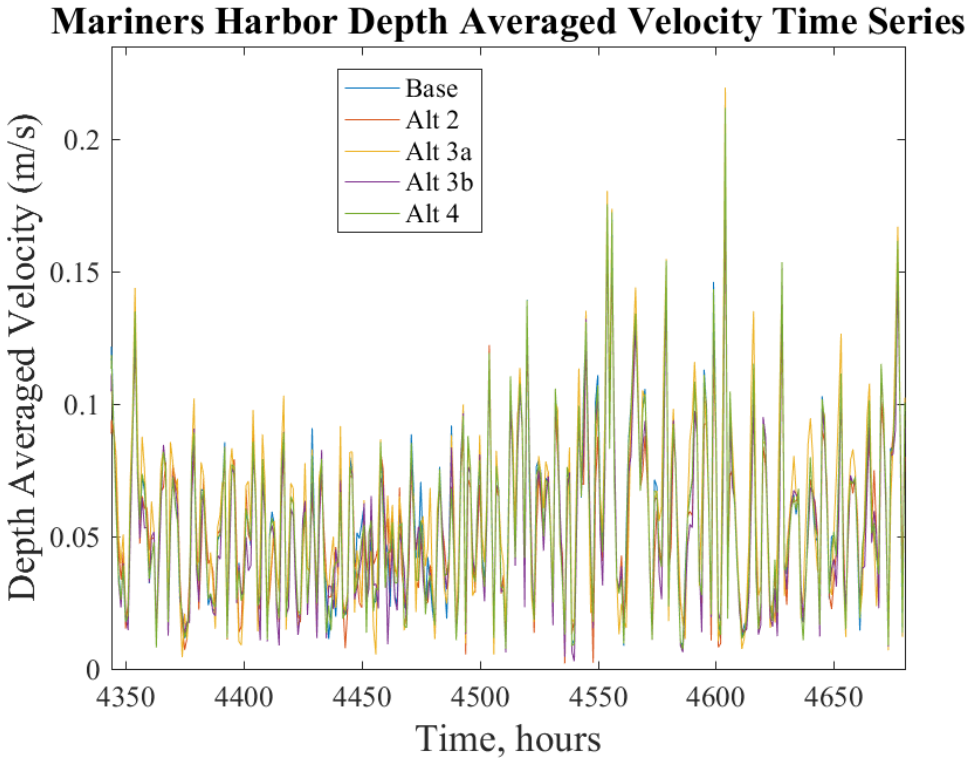


Figure 199: Set 2 Point North Reach Depth Averaged Velocity Magnitudes

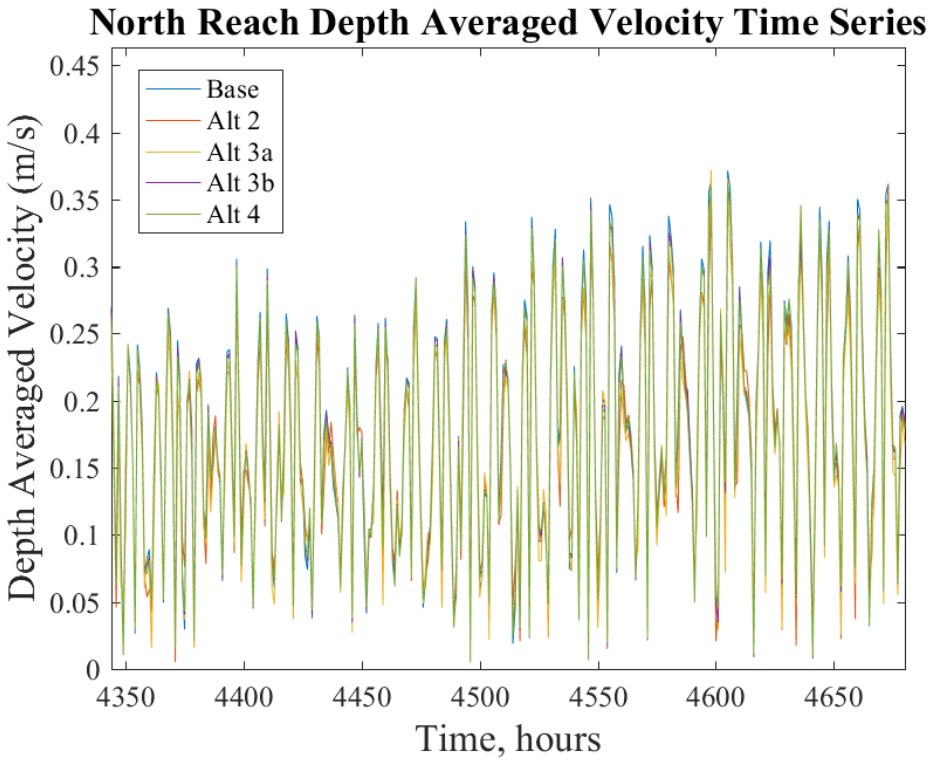
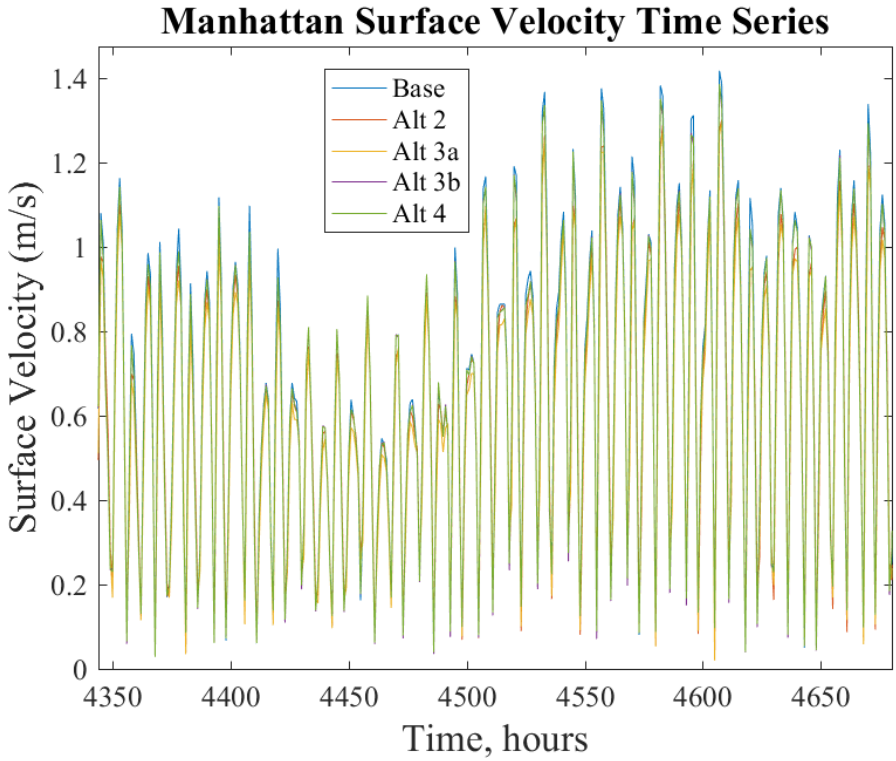


Figure 200: Set 2 Point Manhattan Depth Averaged Velocity Magnitudes



3.2.1.3 Velocity Percentiles

In Figure 201 through Figure 226, the depth average velocity percentiles are plotted for each alternative and the without project for the locations shown in Figure 148.

Figure 201: Set 2 Point V1 Depth Average Velocity Magnitude Percentile

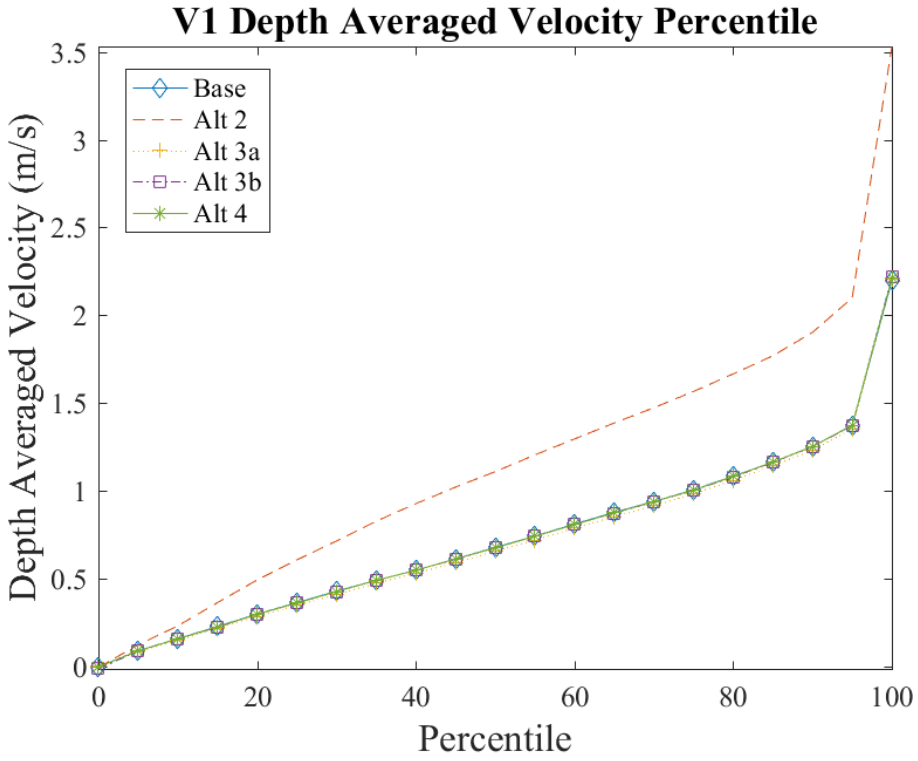


Figure 202: Set 2 Point V2 Depth Average Velocity Magnitude Percentile

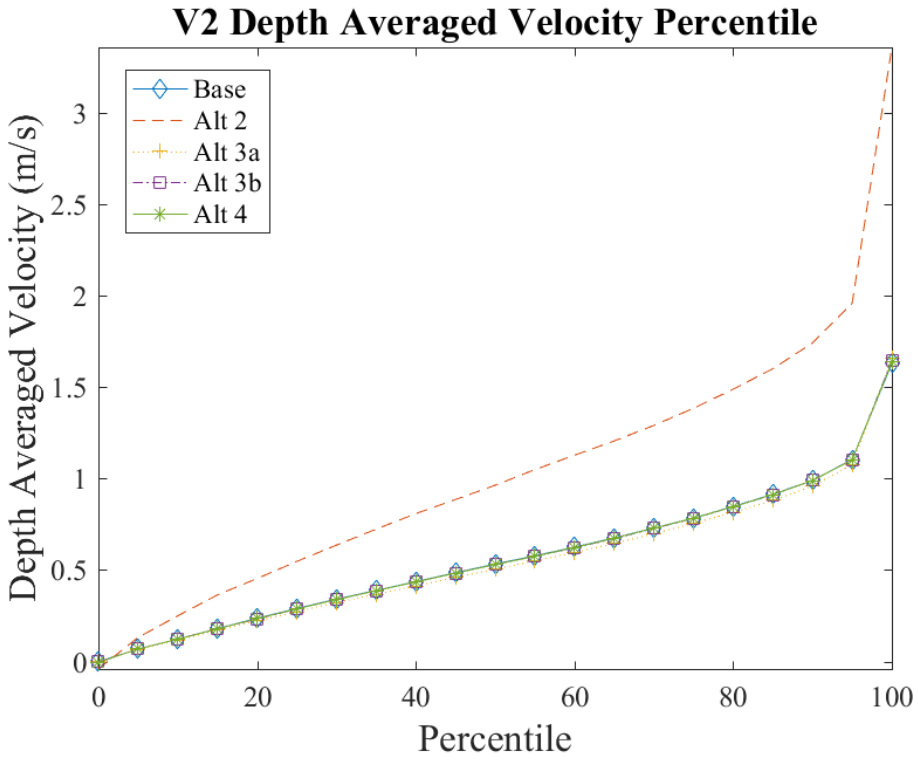


Figure 203: Set 2 Point V3 Depth Average Velocity Magnitude Percentile

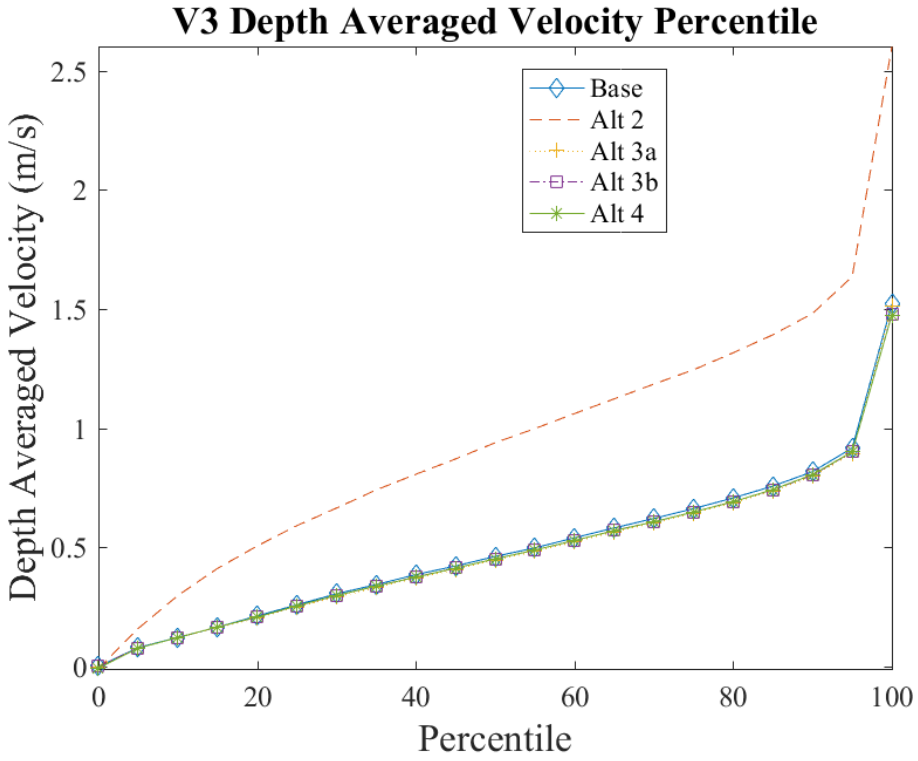


Figure 204: Set 2 Point V4 Depth Average Velocity Magnitude Percentile

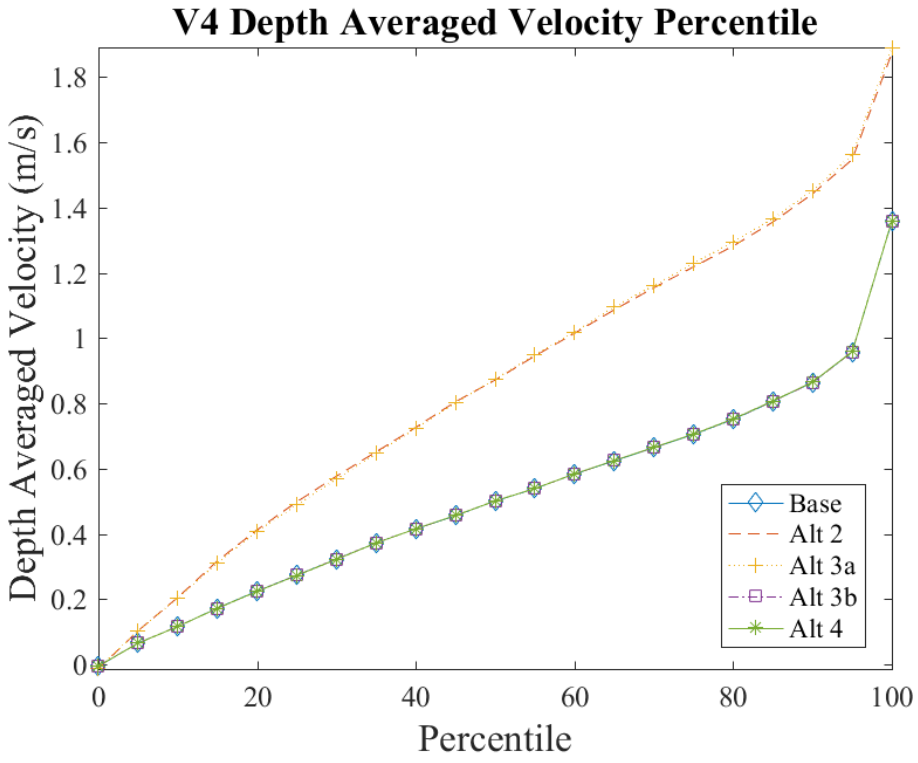


Figure 205: Set 2 Point V5 Depth Average Velocity Magnitude Percentile

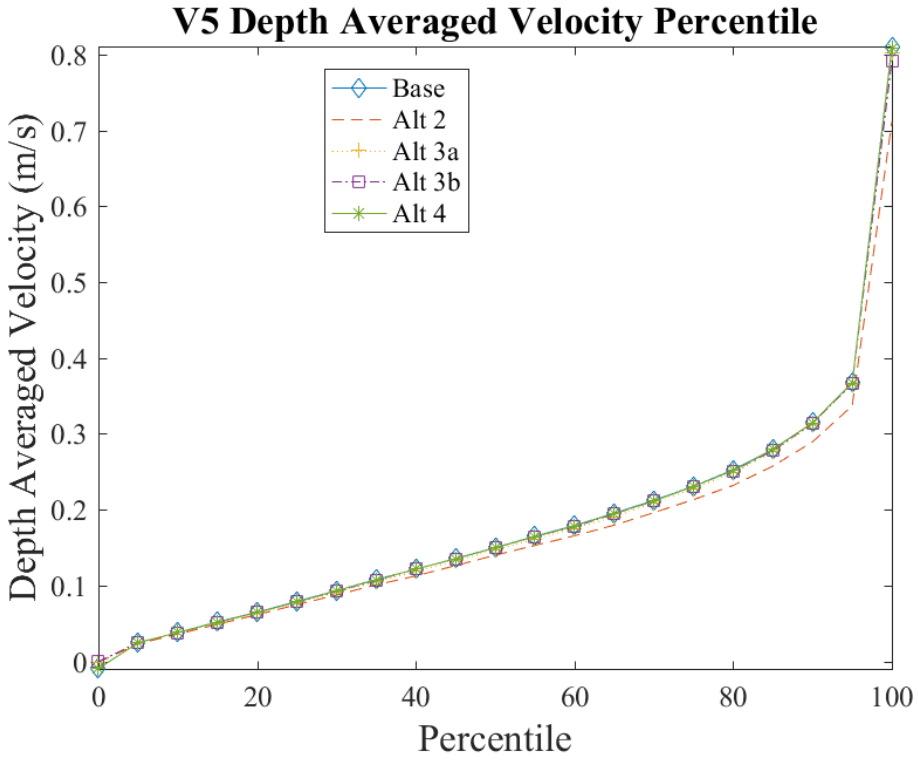


Figure 206: Set 2 Point V6 Depth Average Velocity Magnitude Percentile

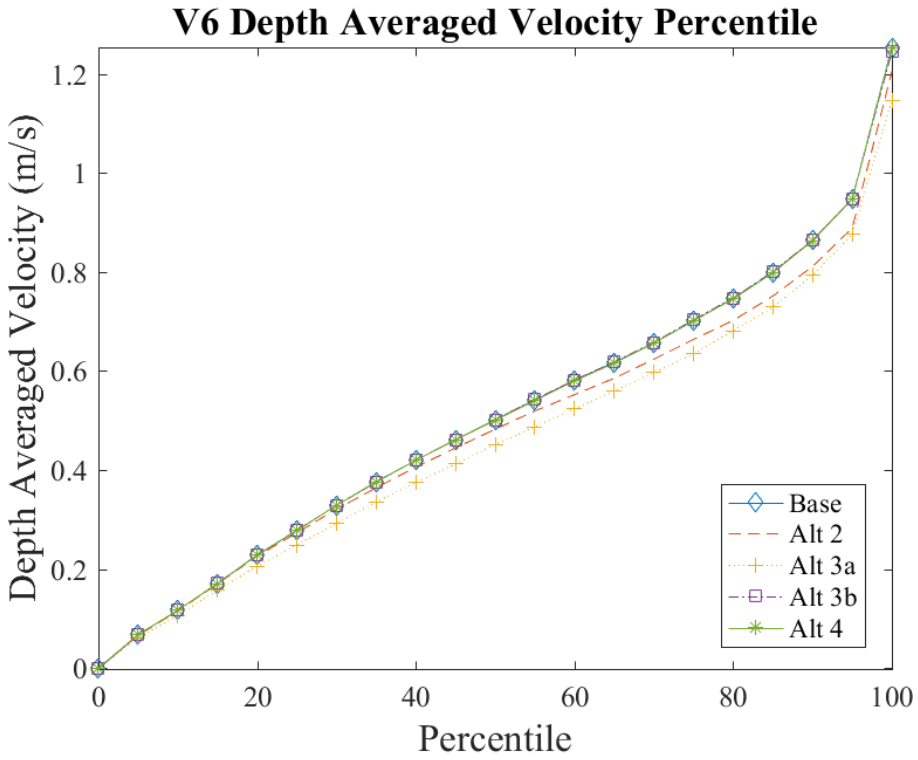


Figure 207: Set 2 Point S1 Depth Average Velocity Magnitude Percentile

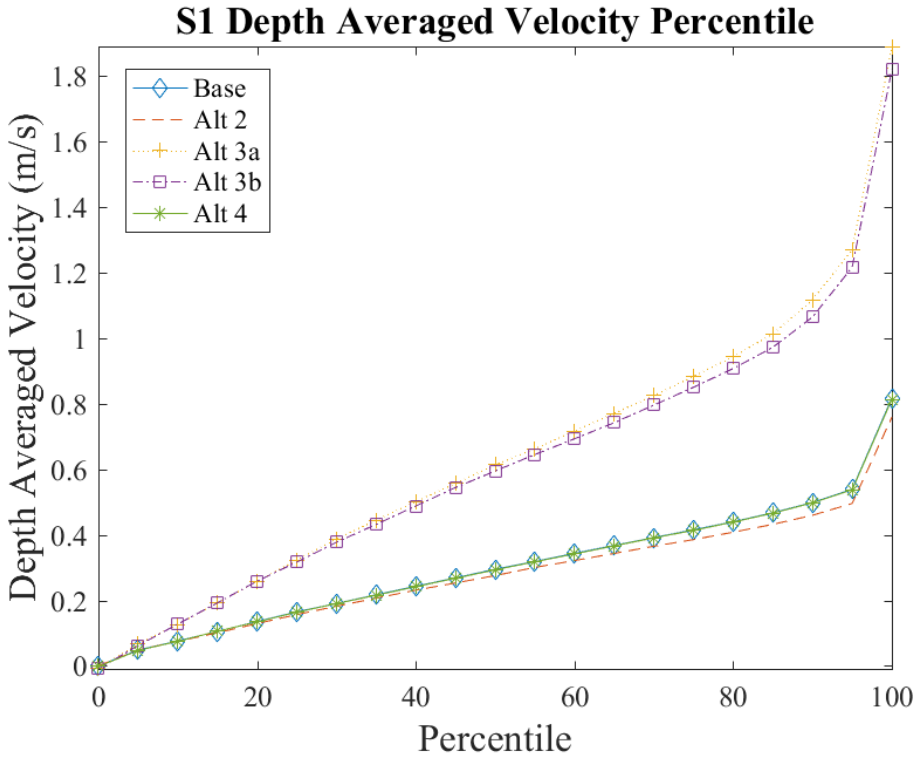


Figure 208: Set 2 Point S2 Depth Average Velocity Magnitude Percentile

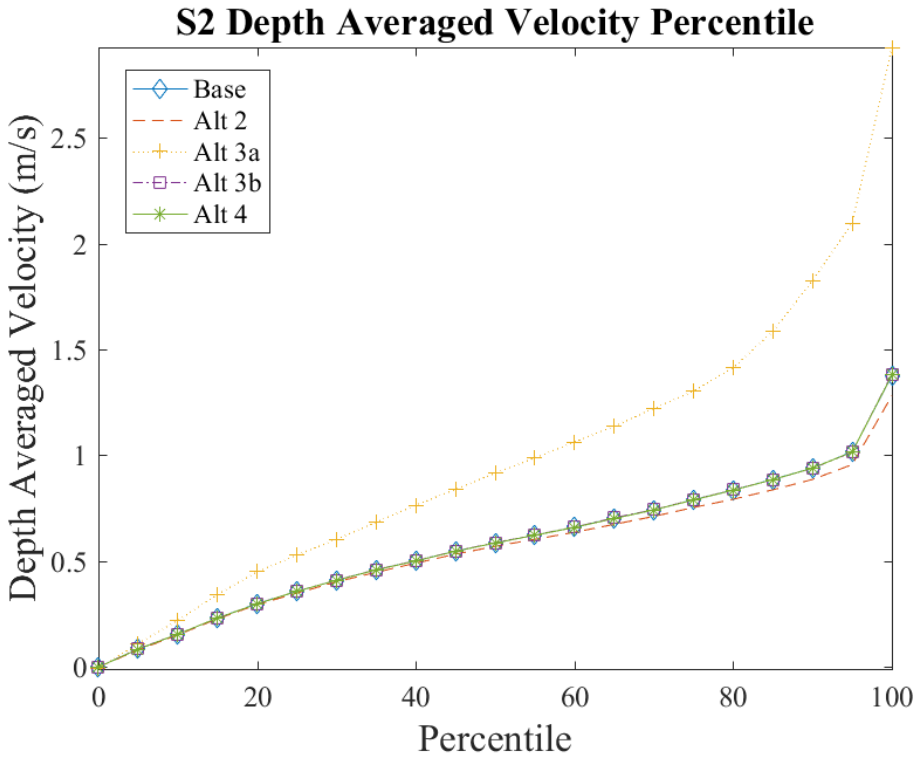


Figure 209: Set 2 Point S3 Depth Average Velocity Magnitude Percentile

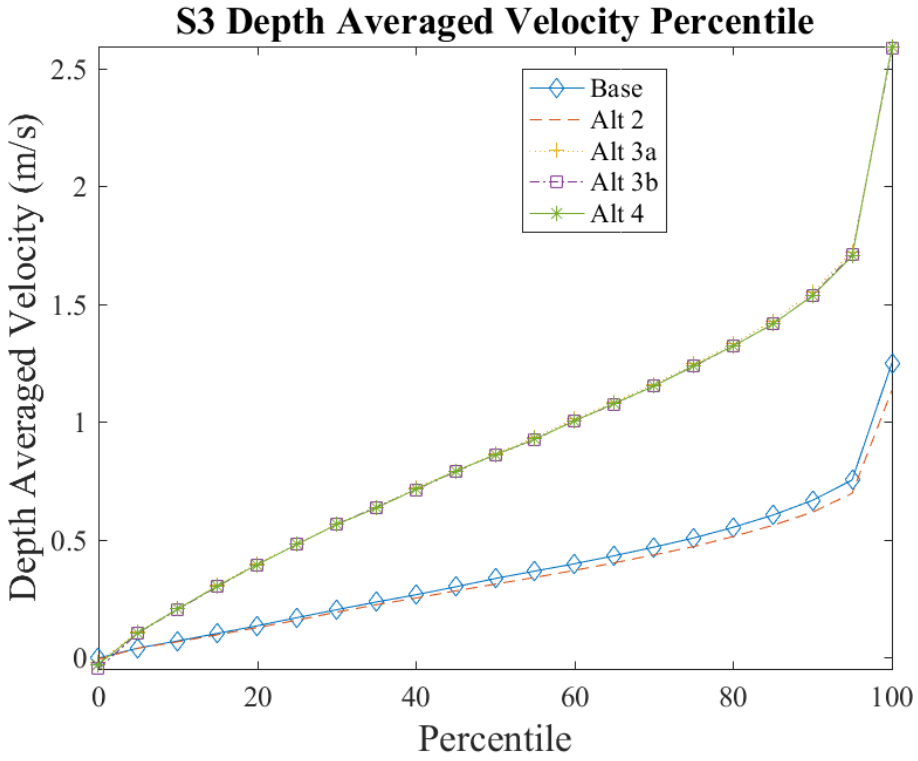


Figure 210: Set 2 Point S4 Depth Average Velocity Magnitude Percentile

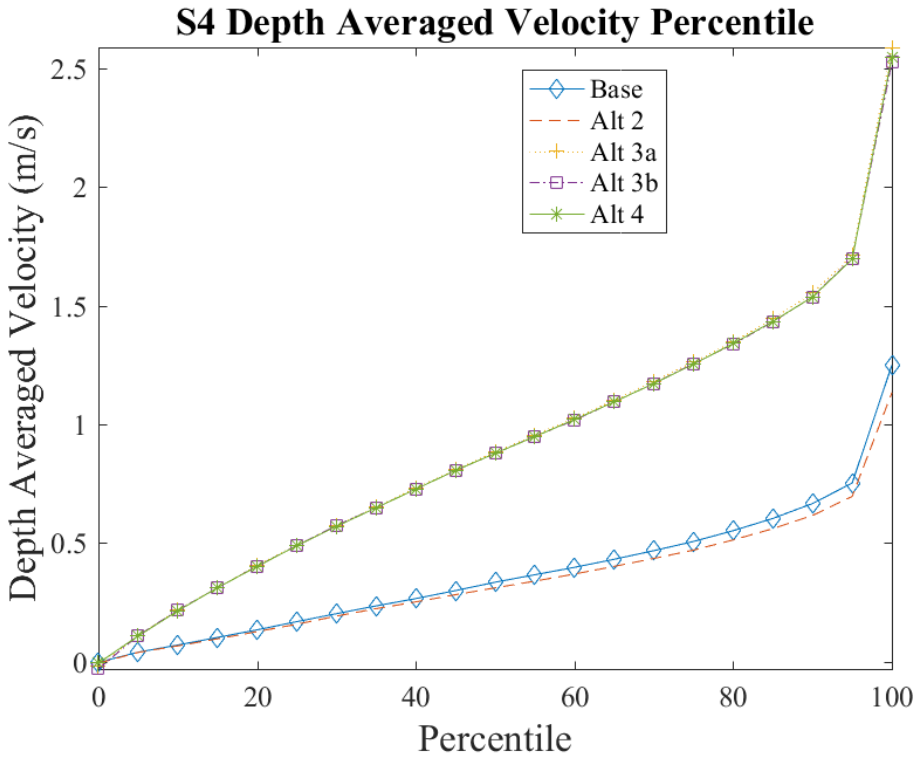


Figure 211: Set 2 Point S5 Depth Average Velocity Magnitude Percentile

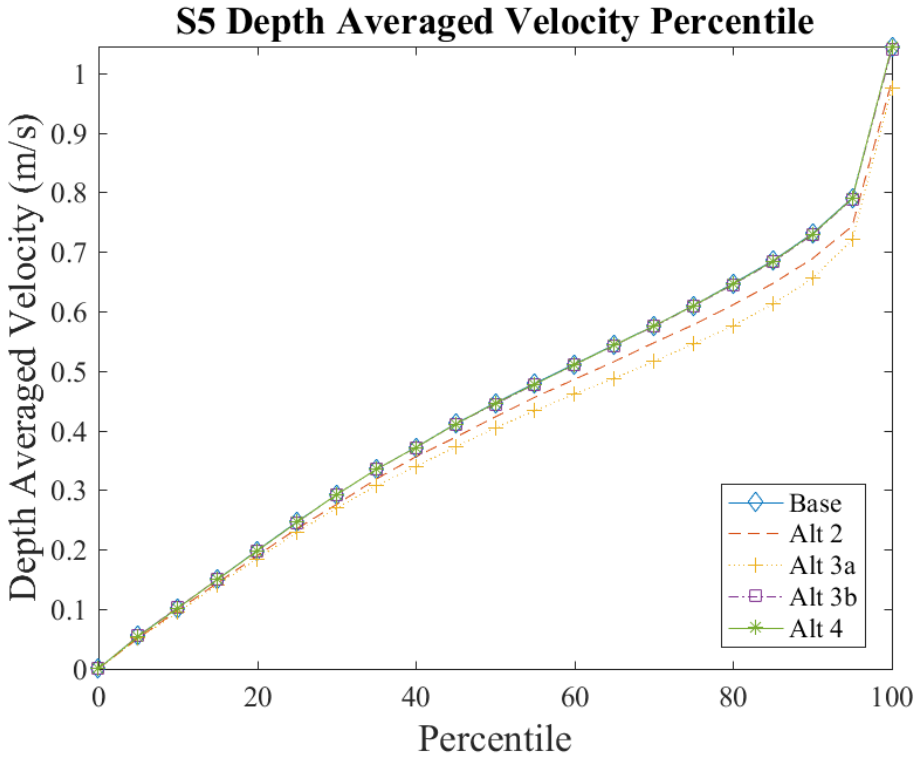


Figure 212: Set 2 Point T1 Depth Average Velocity Magnitude Percentile

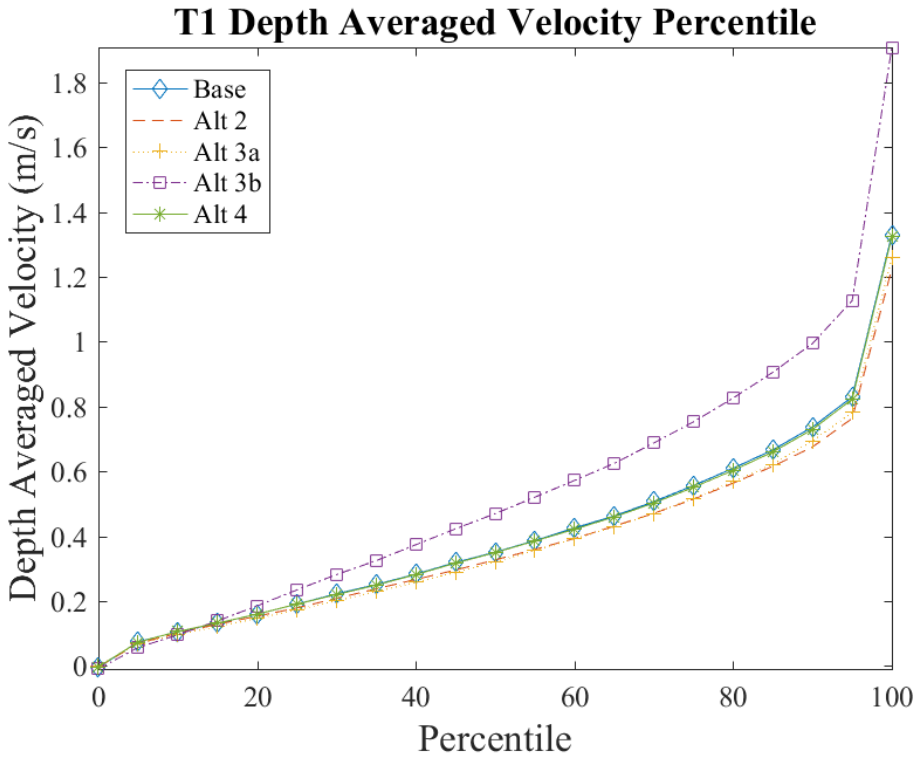


Figure 213: Set 2 Point T2 Depth Average Velocity Magnitude Percentile

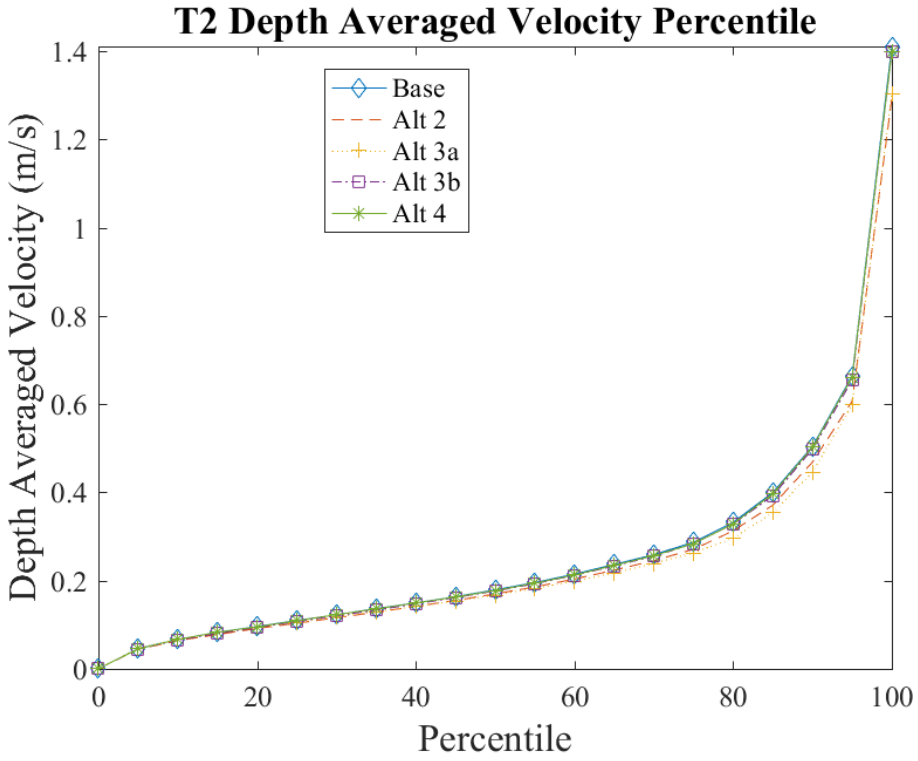


Figure 214: Set 2 Point T3 Depth Average Velocity Magnitude Percentile

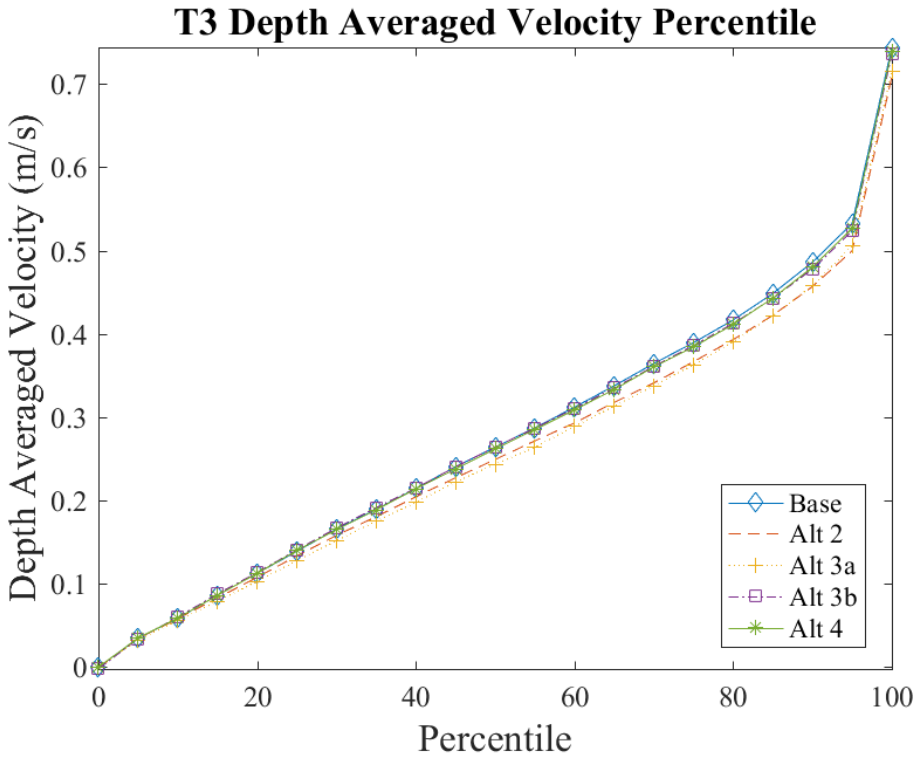


Figure 215: Set 2 Point T4 Depth Average Velocity Magnitude Percentile

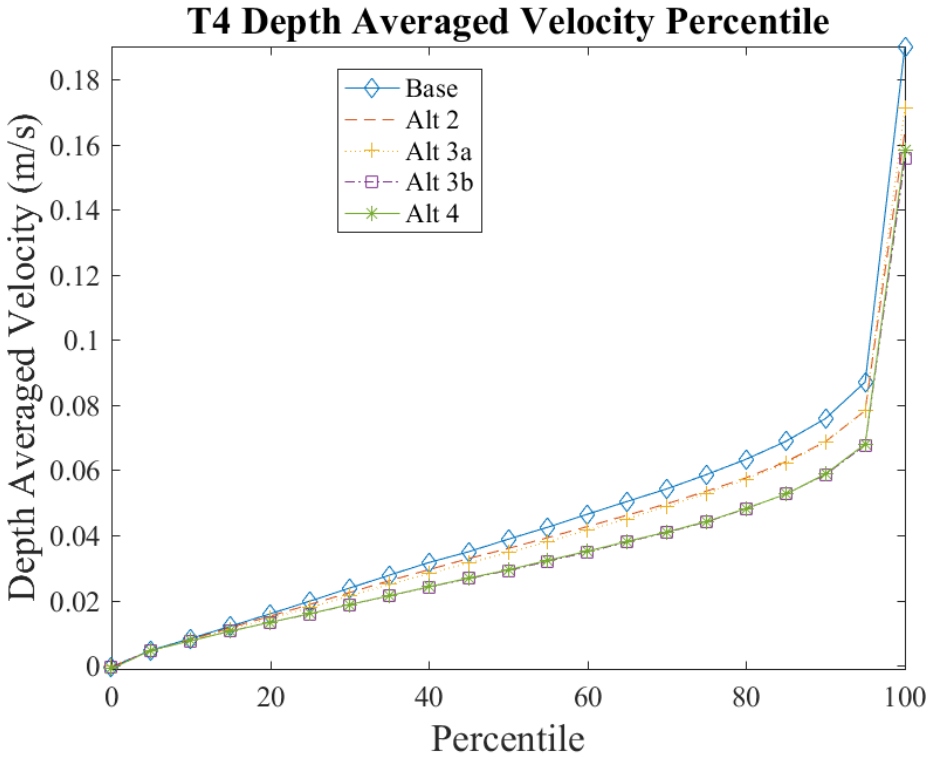


Figure 216: Set 2 Point T5 Depth Average Velocity Magnitude Percentile

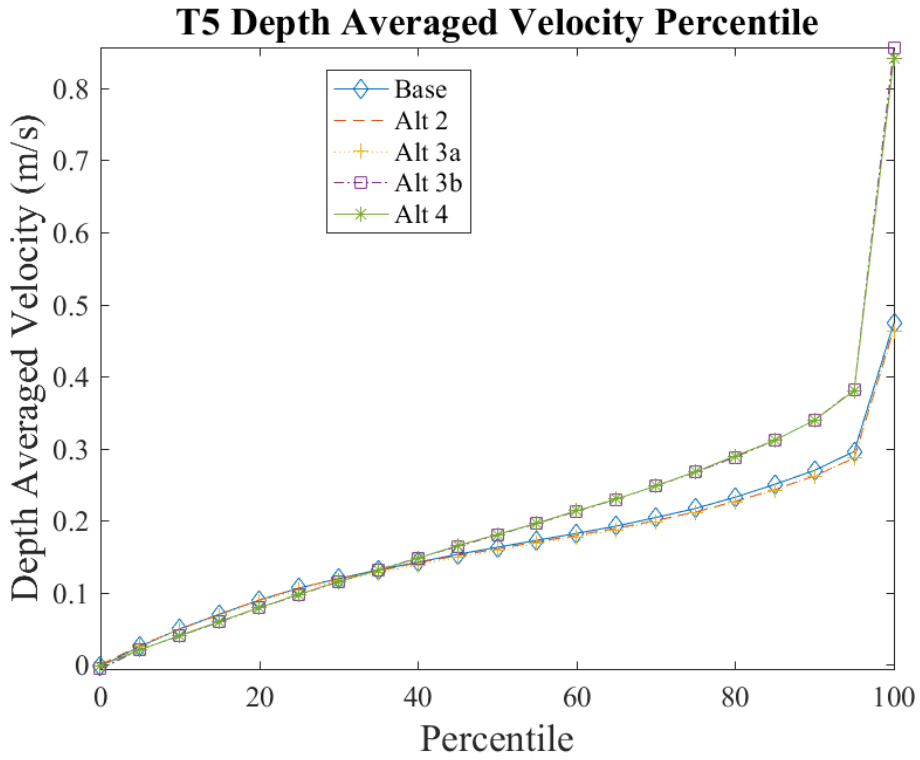


Figure 217: Set 2 Point R1 Depth Average Velocity Magnitude Percentile

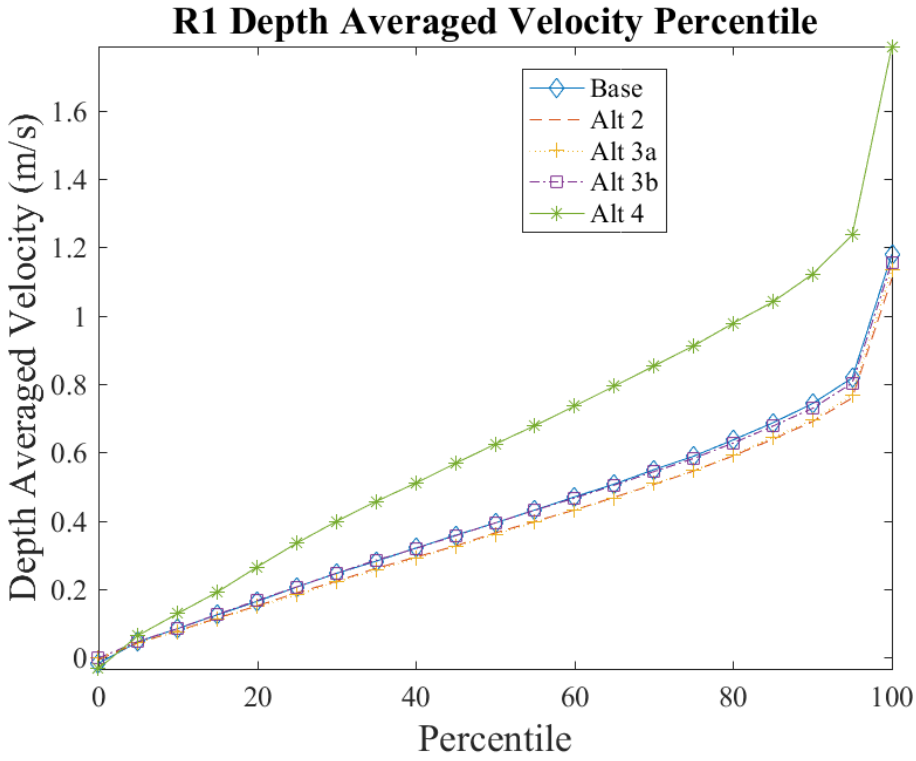


Figure 218: Set 2 Point Fresh Kills Depth Average Velocity Magnitude Percentile

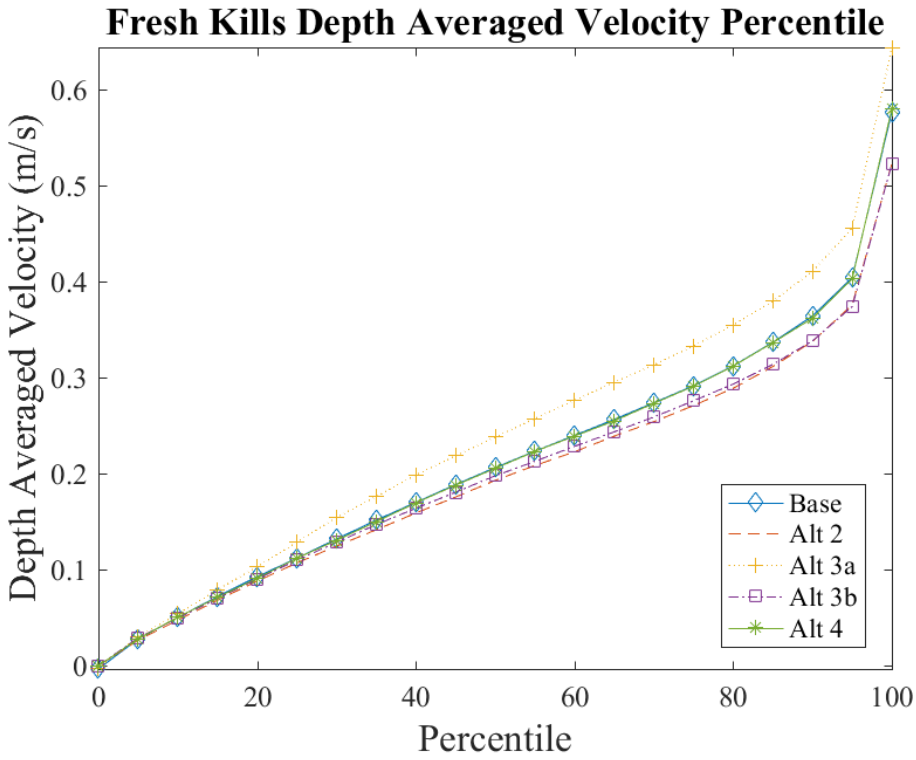


Figure 219: Set 2 Point Sandy Hook Depth Average Velocity Magnitude Percentile

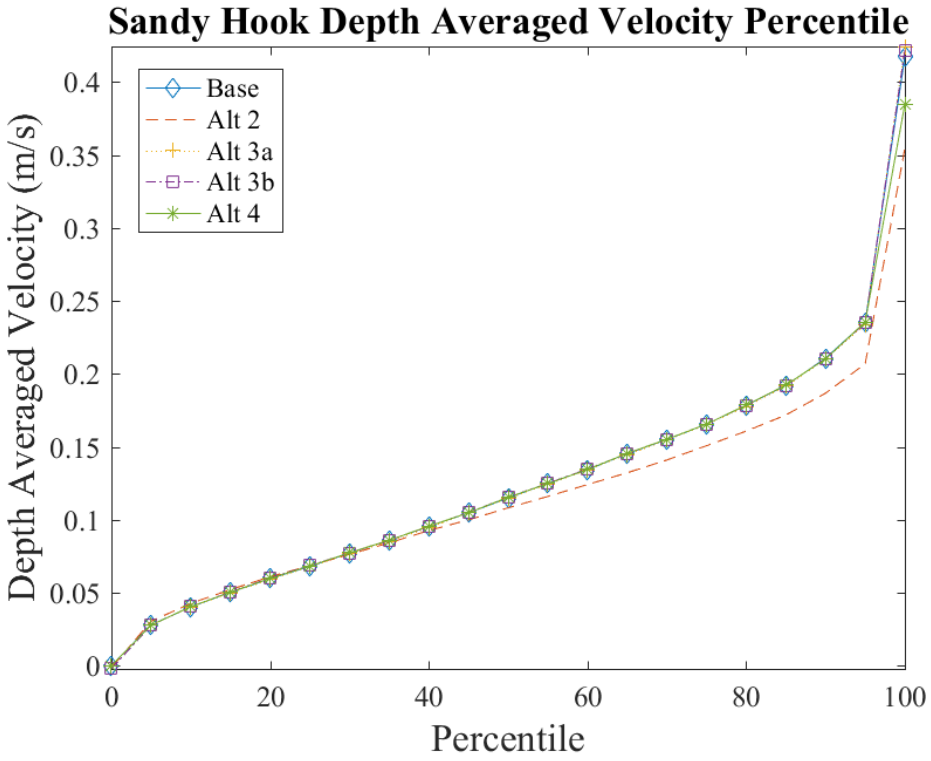


Figure 220: Set 2 Point Jamaica Bay Depth Average Velocity Magnitude Percentile

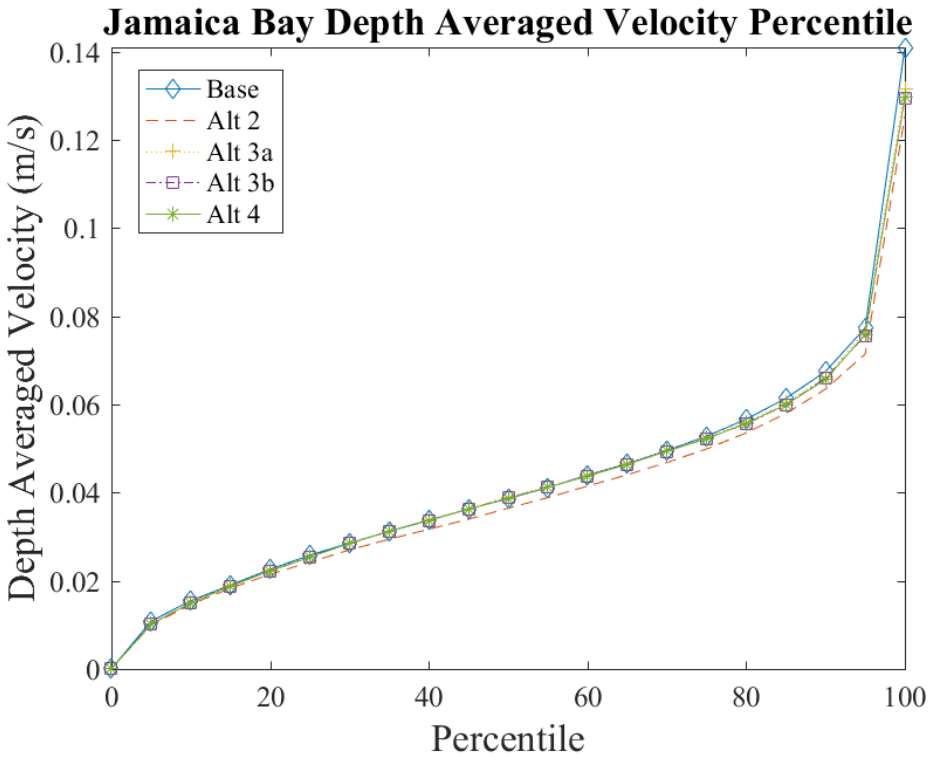


Figure 221: Set 2 Point Robbins Reef Depth Average Velocity Magnitude Percentile

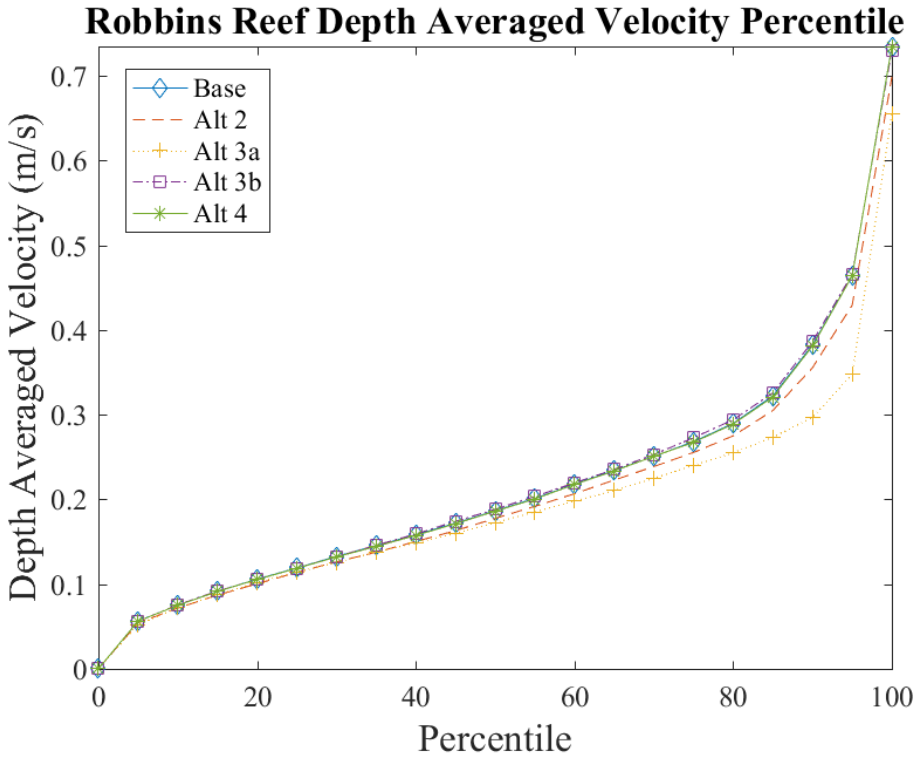


Figure 222: Set 2 Point Battery Depth Average Velocity Magnitude Percentile

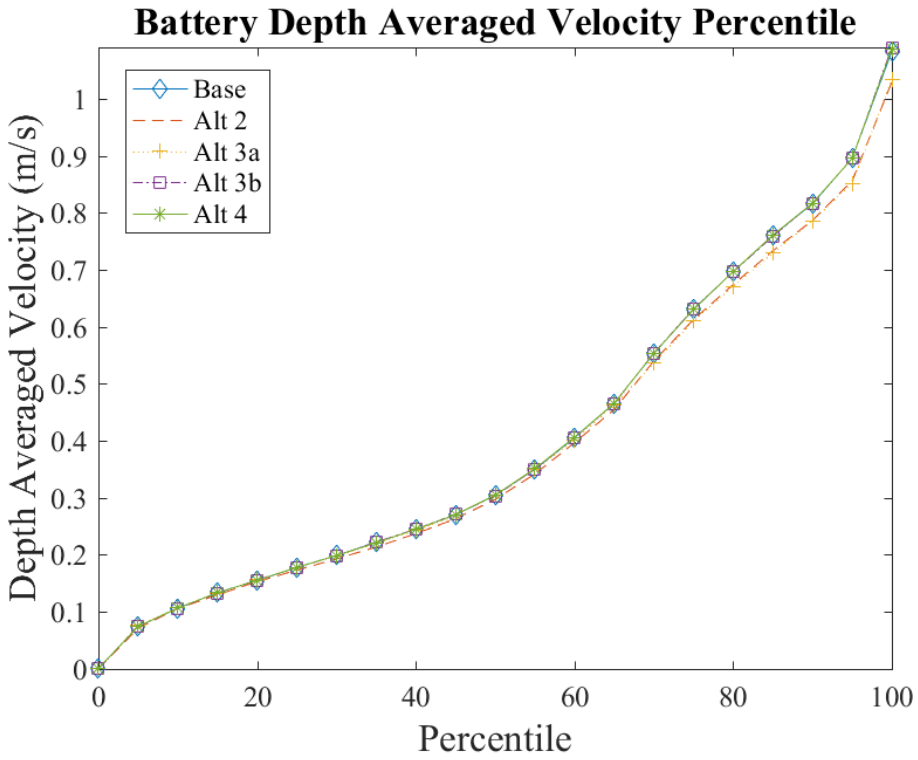


Figure 223: Set 2 Point KLGA Depth Average Velocity Magnitude Percentile

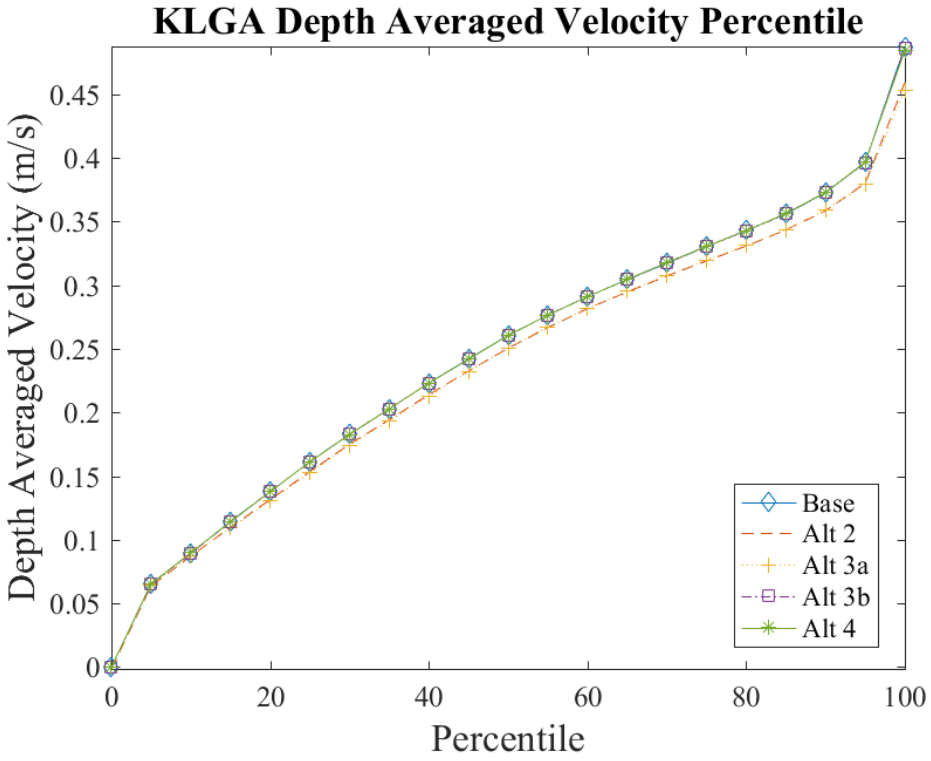


Figure 224: Set 2 Point Mariners Harbor Depth Average Velocity Magnitude Percentile

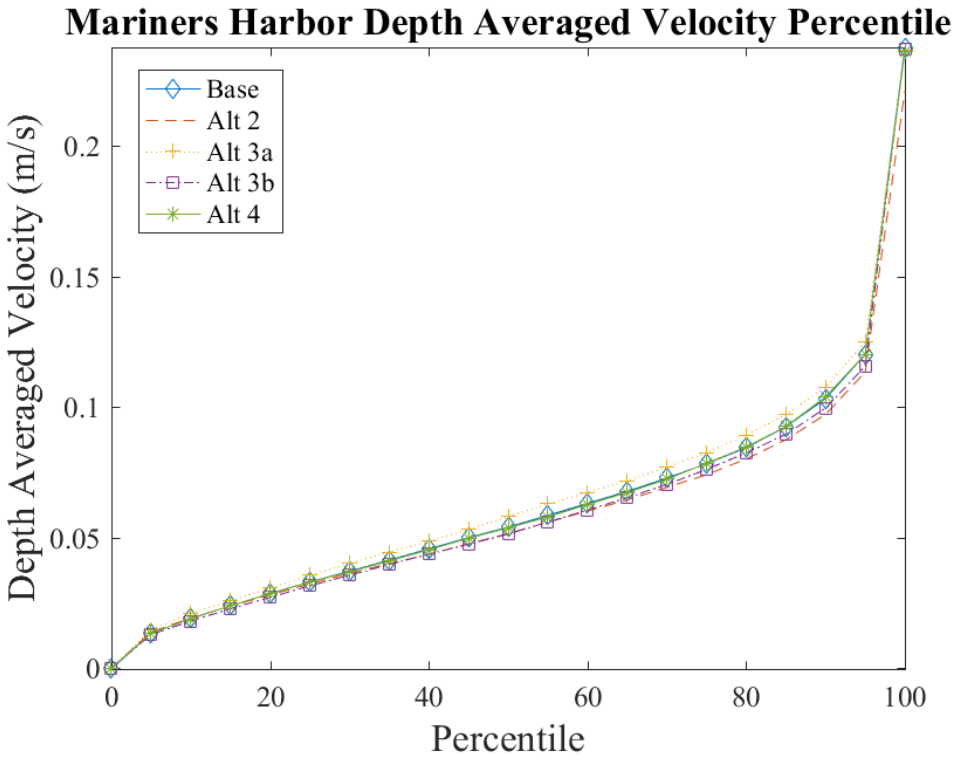


Figure 225: Set 2 Point North Reach Depth Average Velocity Magnitude Percentile

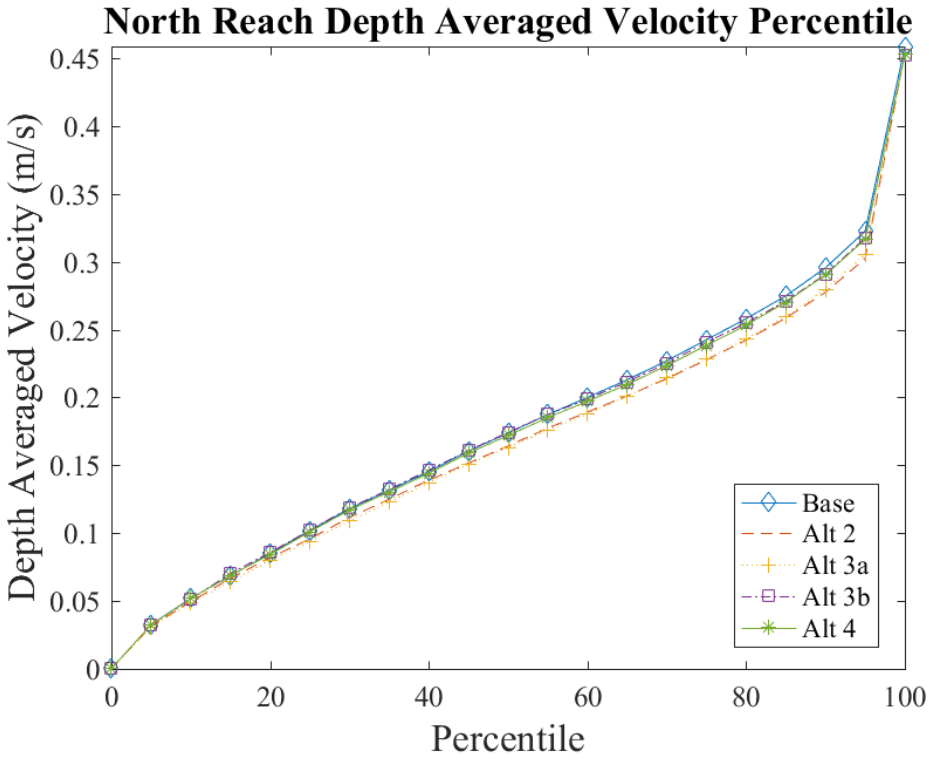
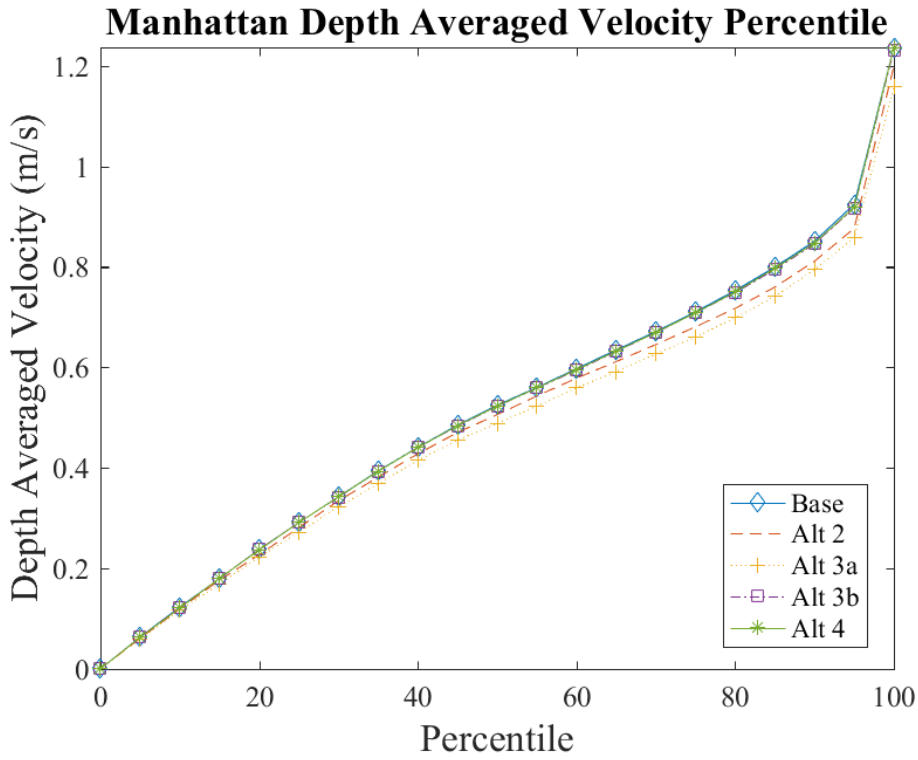


Figure 226: Set 2 Point Manhattan Depth Average Velocity Magnitude Percentile



3.2.2 Spatial Percentile Analysis

The 50th and 75th percentile depth average velocities and salinities throughout the entire mesh were calculated to visualize spatial distributions. This gives more insight and a better understanding into the barrier spatial impacts on the system.

3.2.2.1 Spatial Velocity Percentiles

The spatial depth average velocity percentile figures are depicted in Figure 227 through Figure 242.

Figure 227: Set 2 Depth Averaged Velocity 50th percentile in Lower Bay (Alternative 2 top, Base Bottom)

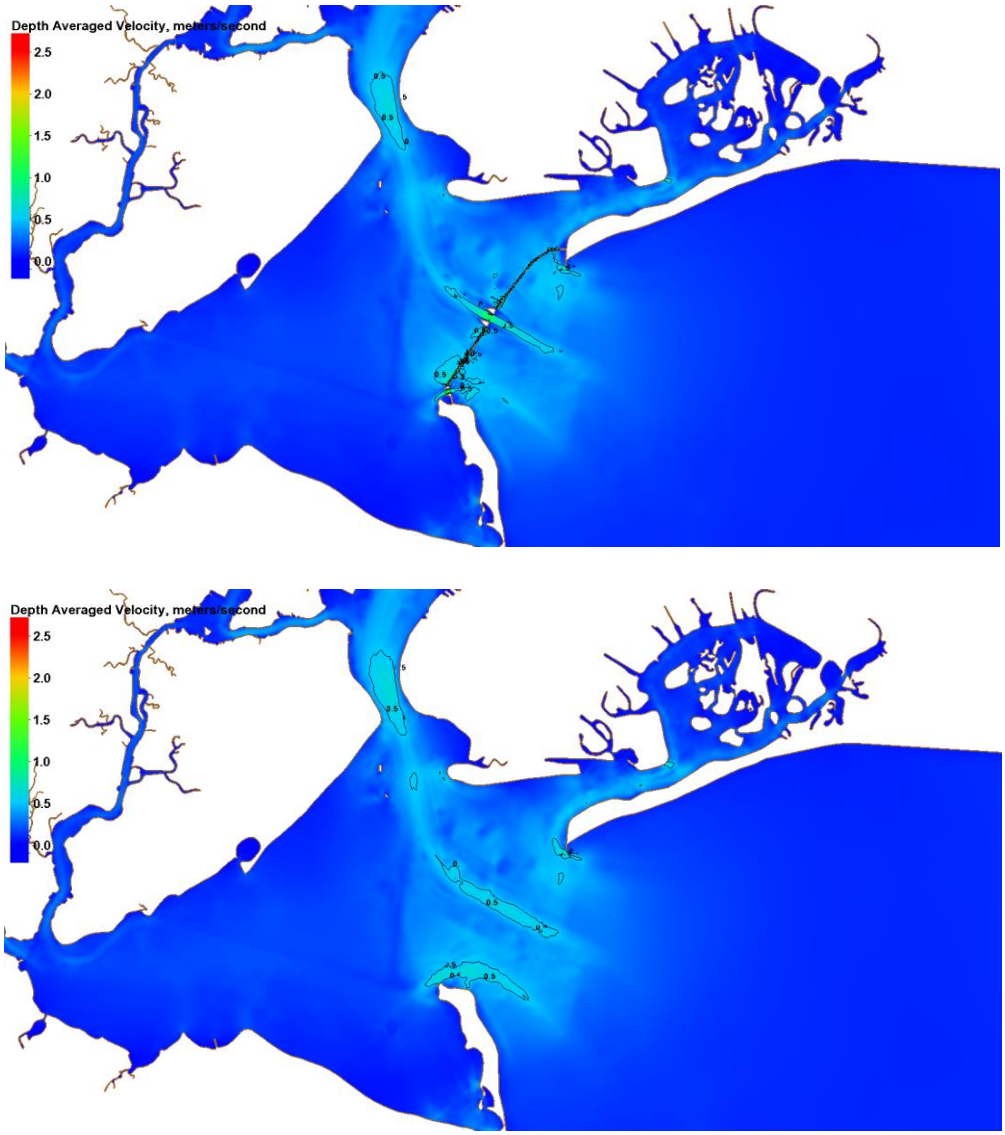


Figure 228: Set 2 Depth Averaged Velocity 75th percentile in Lower Bay (Alternative 2 top, Base Bottom)

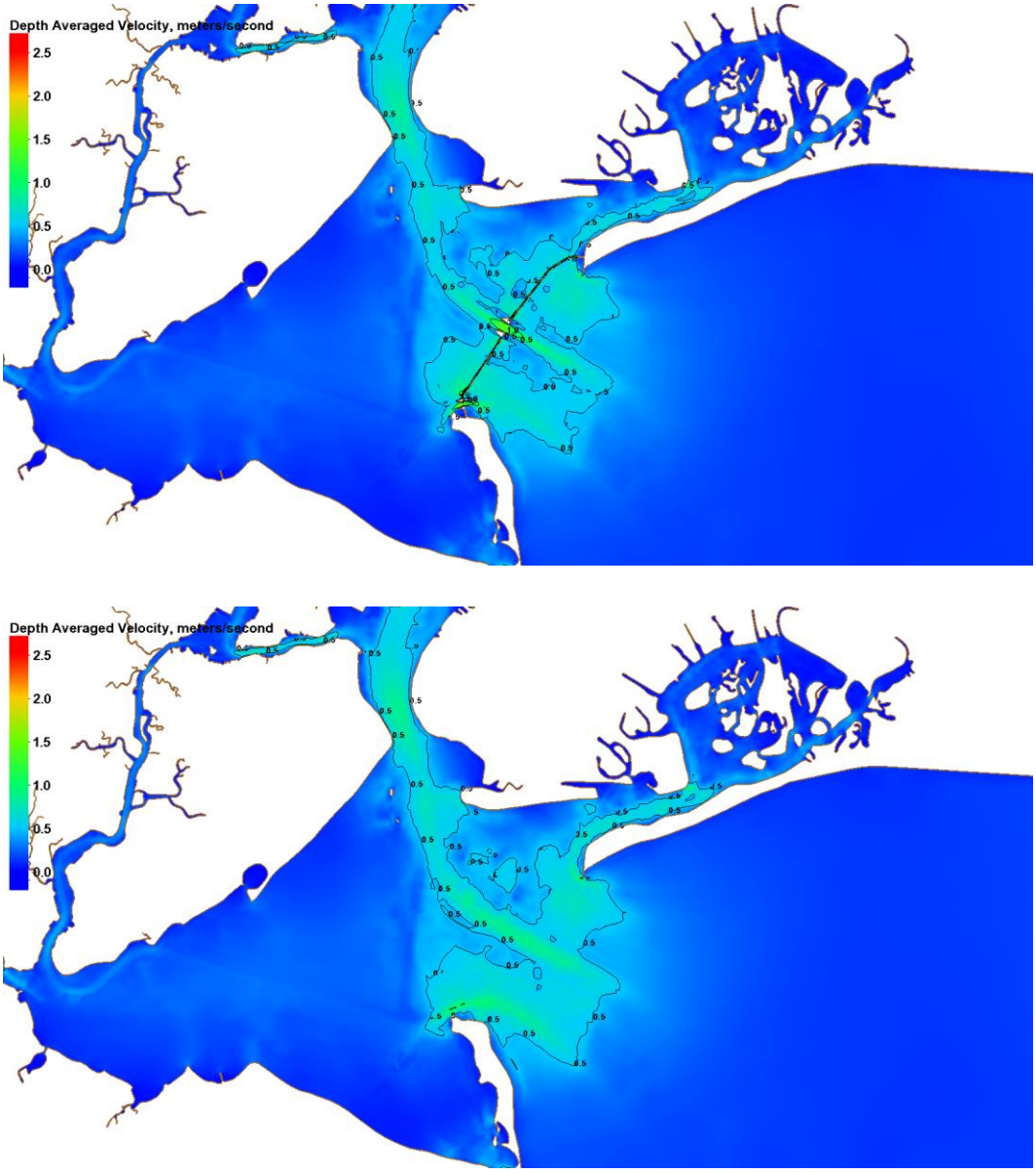


Figure 229: Set 2 Depth Averaged Velocity 50th percentile in Upper Bay (Alternative 2 top, Base Bottom)

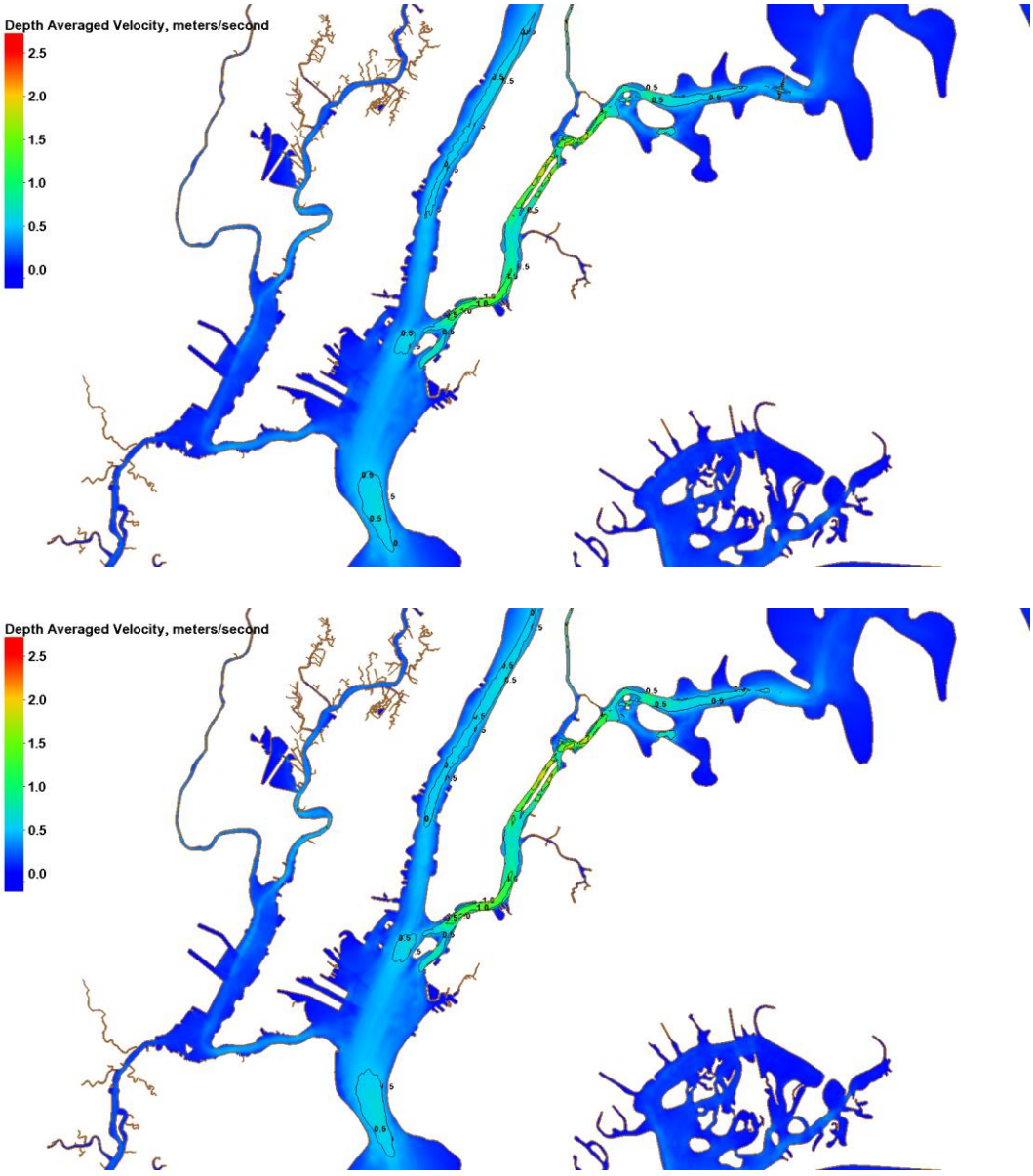


Figure 230: Set 2 Depth Averaged Velocity 75th percentile in Upper Bay (Alternative 2 top, Base Bottom)

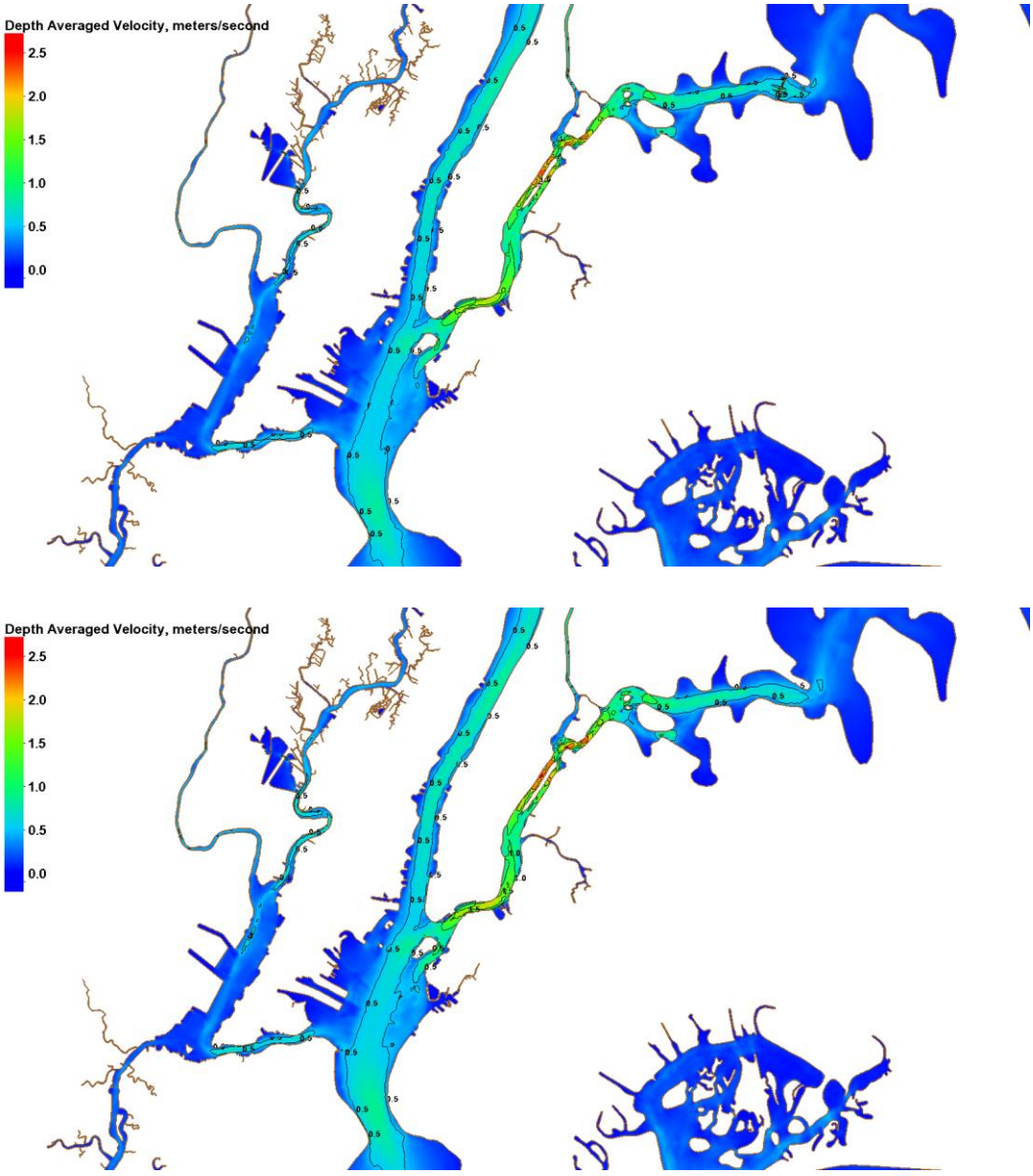


Figure 231: Set 2 Depth Averaged Velocity 50th percentile in Lower Bay (Alternative 3a top, Base Bottom)

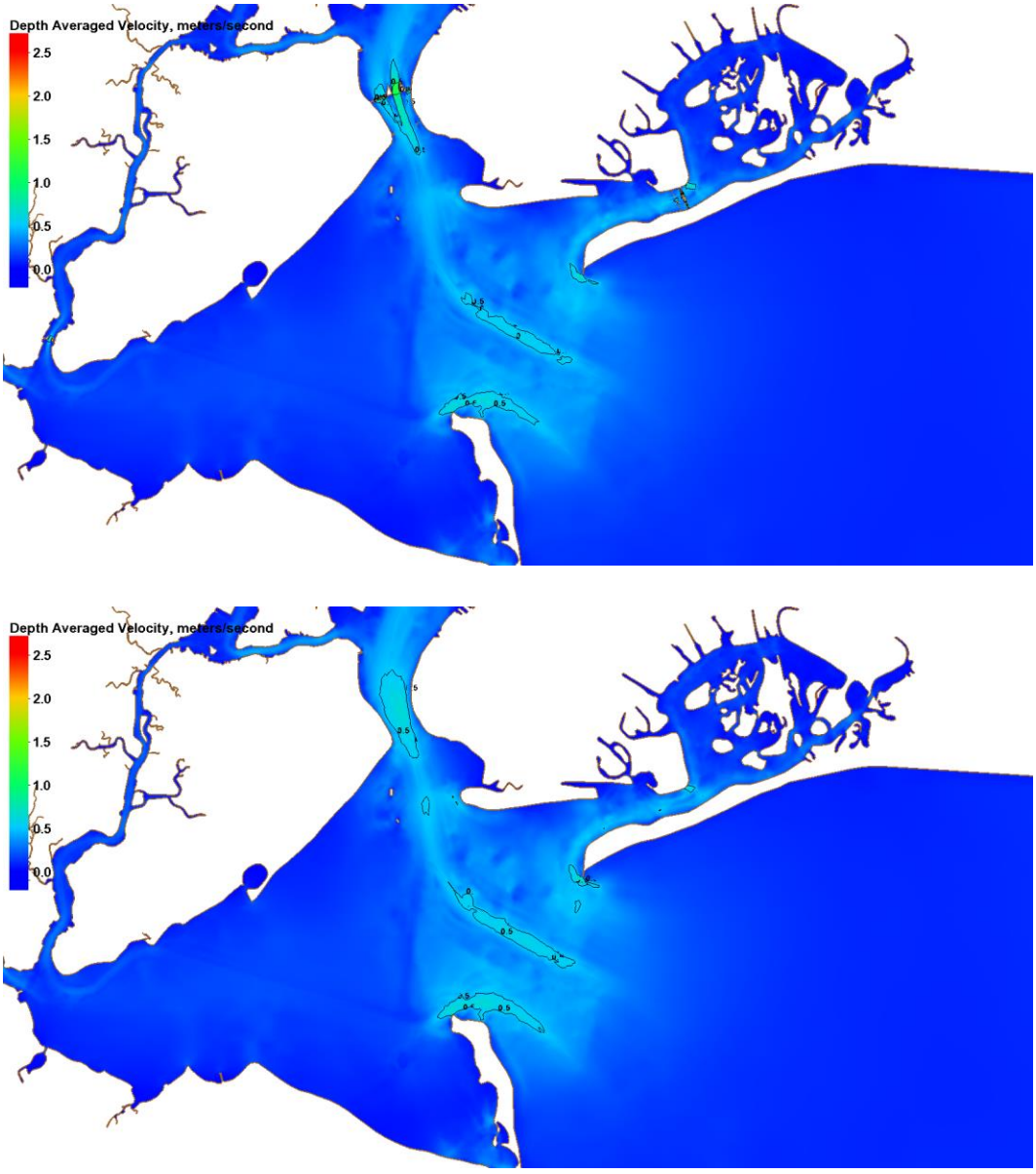


Figure 232: Set 2 Depth Averaged Velocity 75th percentile in Lower Bay (Alternative 3a top, Base Bottom)

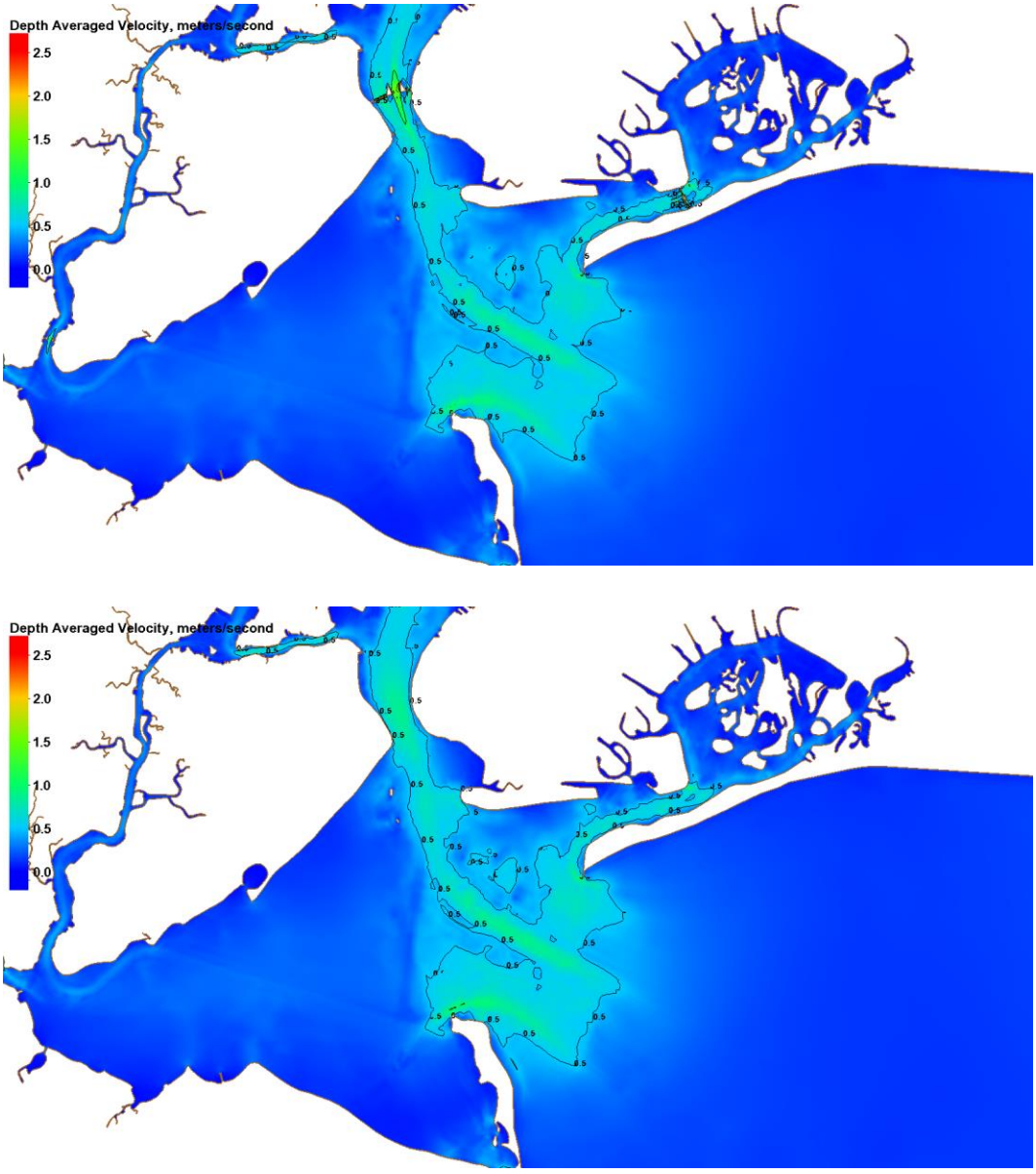


Figure 233: Set 2 Depth Averaged Velocity 50th percentile in Upper Bay (Alternative 3a top, Base Bottom)

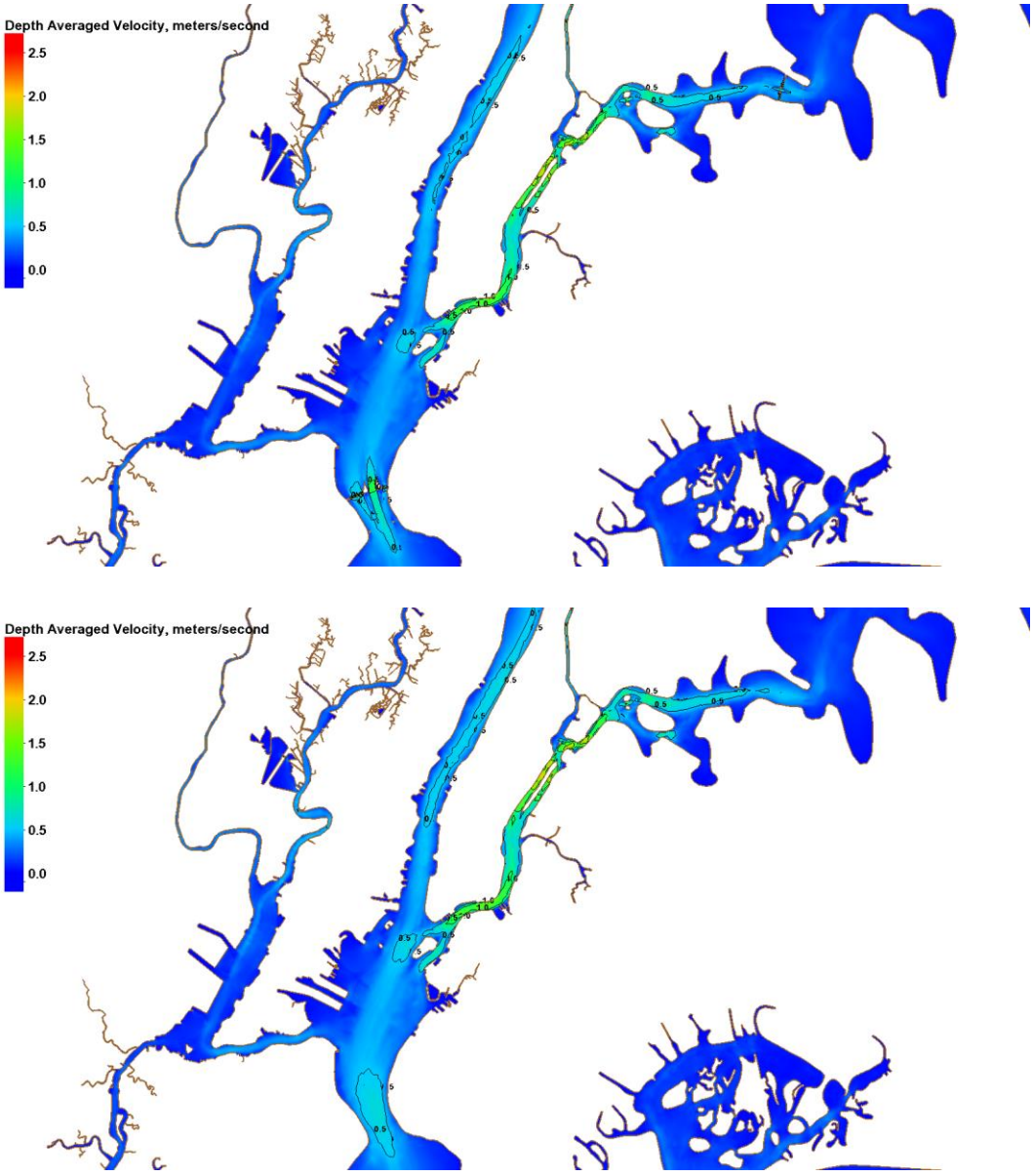


Figure 234: Set 2 Depth Averaged Velocity 75th percentile in Upper Bay (Alternative 3a top, Base Bottom)

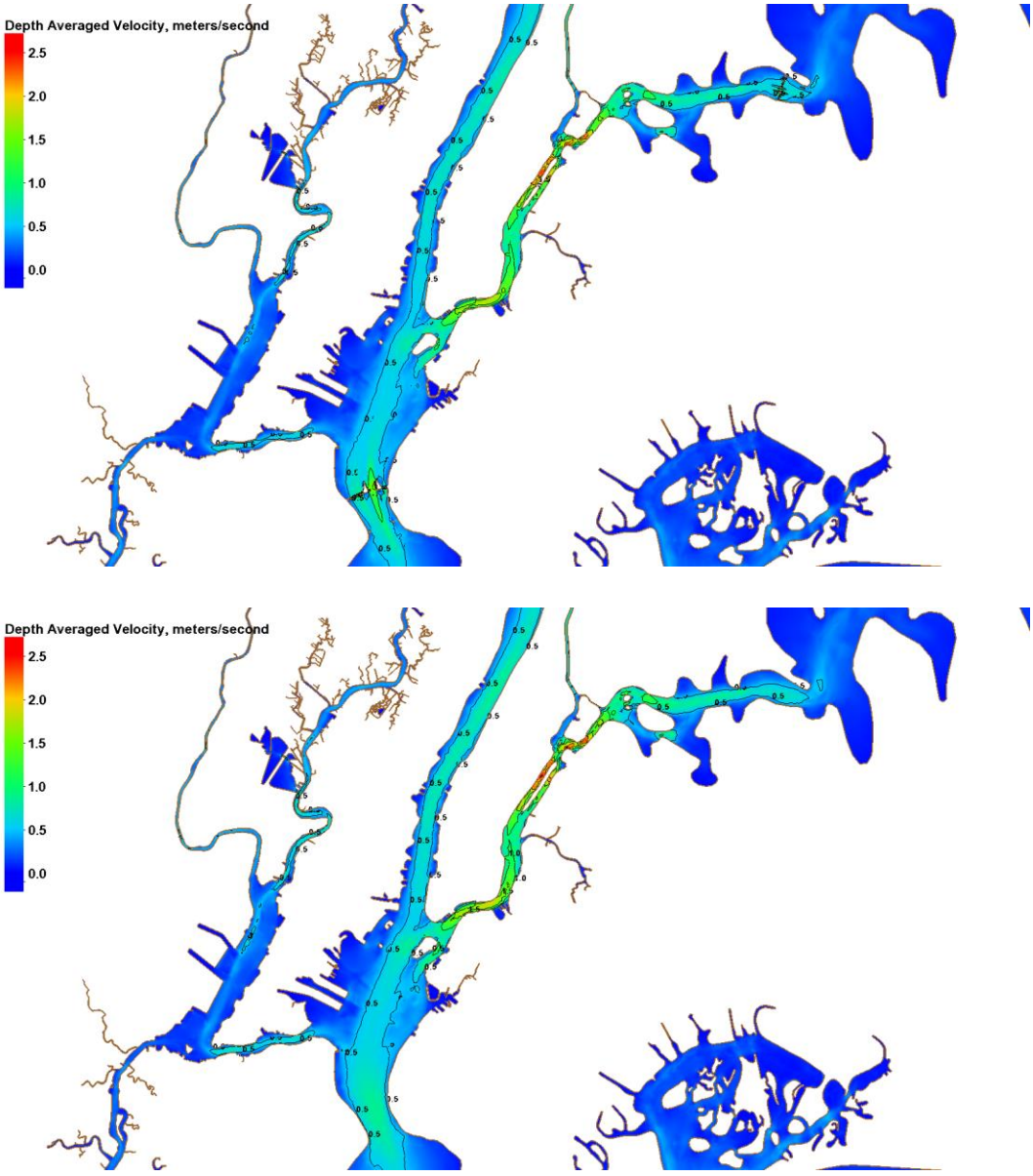


Figure 235: Set 2 Depth Averaged Velocity 50th percentile in Lower Bay (Alternative 3b top, Base Bottom)

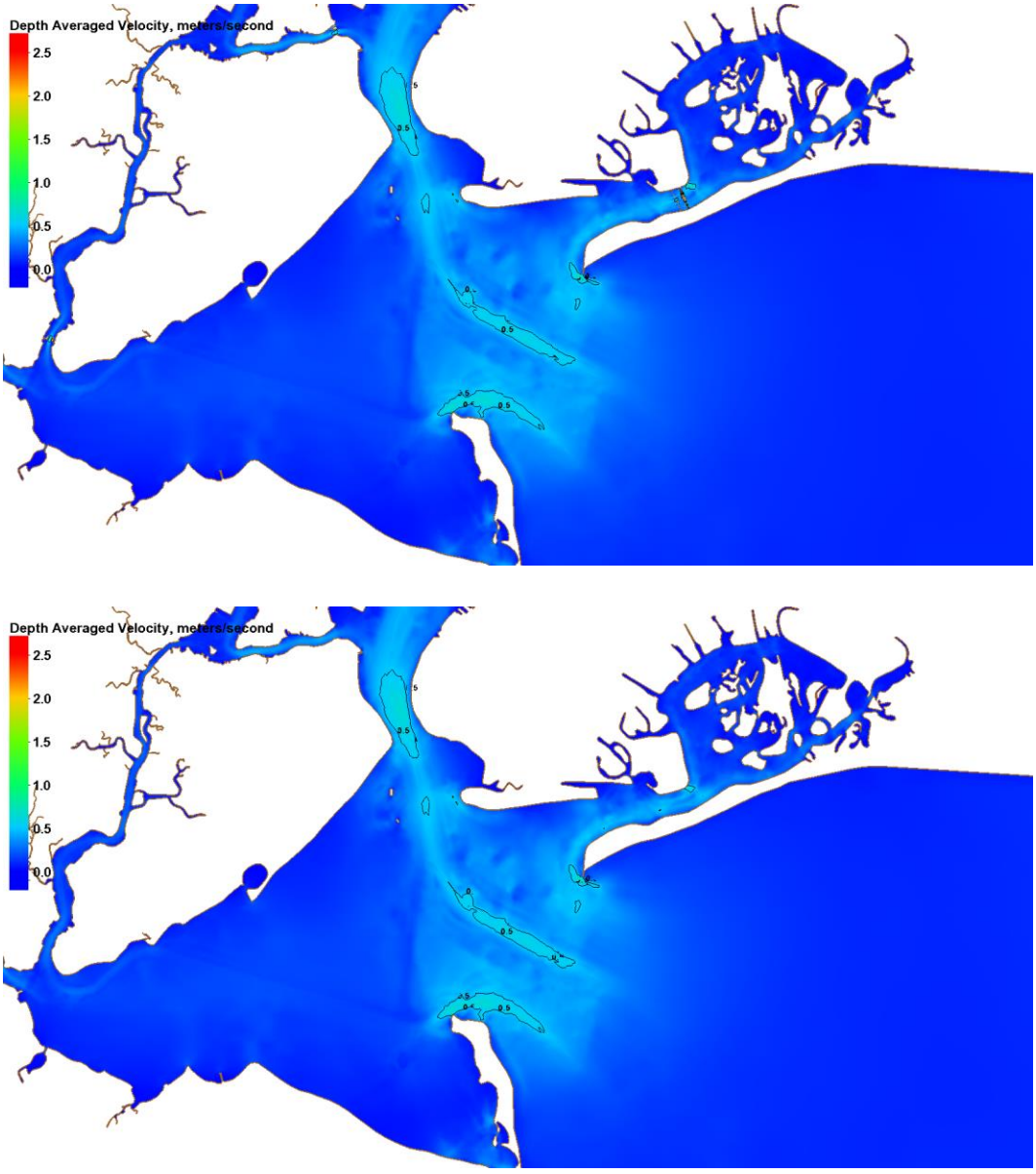


Figure 236: Set 2 Depth Averaged Velocity 75th percentile in Lower Bay (Alternative 3b top, Base Bottom)

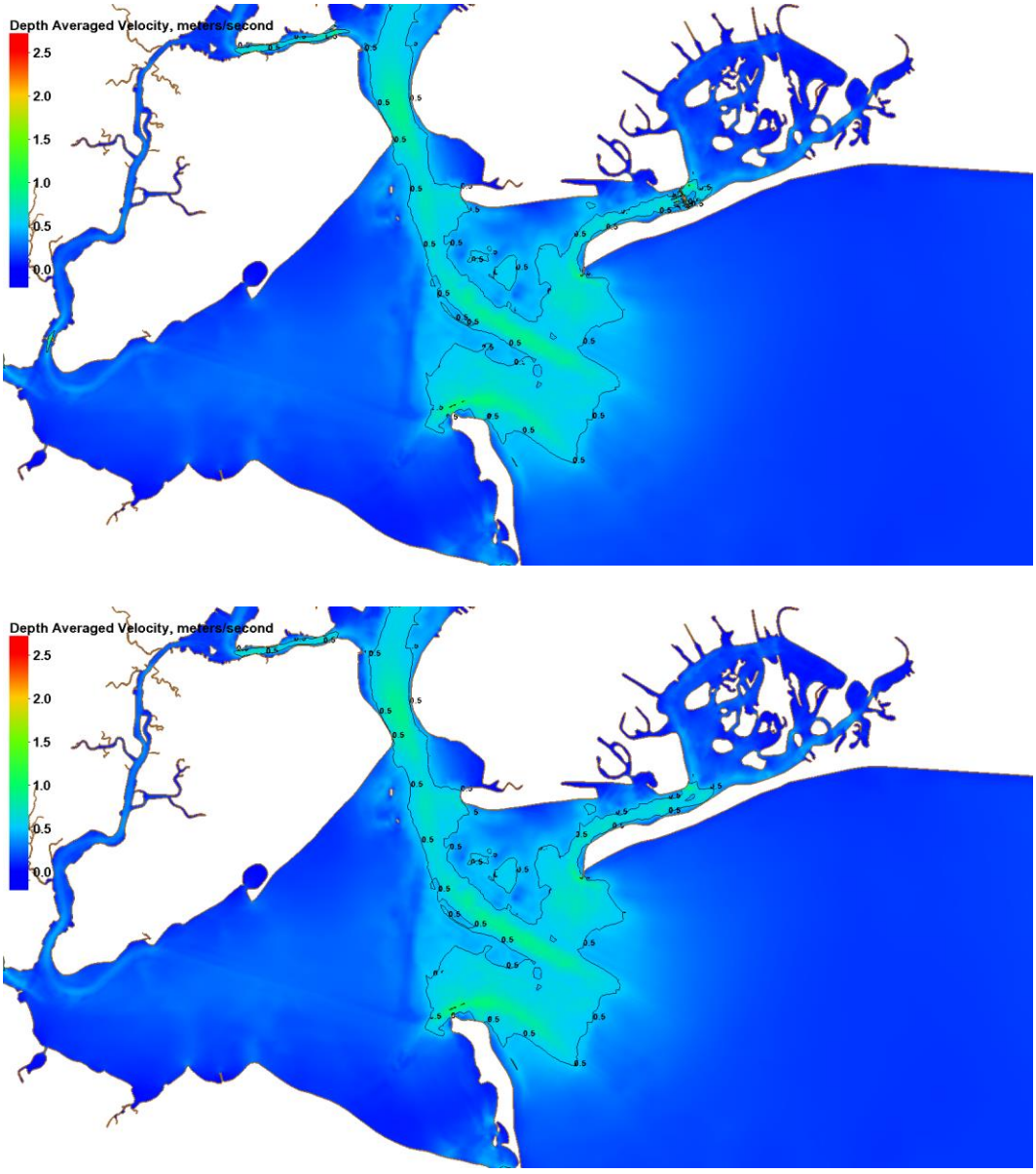


Figure 237: Set 2 Depth Averaged Velocity 50th percentile in Upper Bay (Alternative 3b top, Base Bottom)

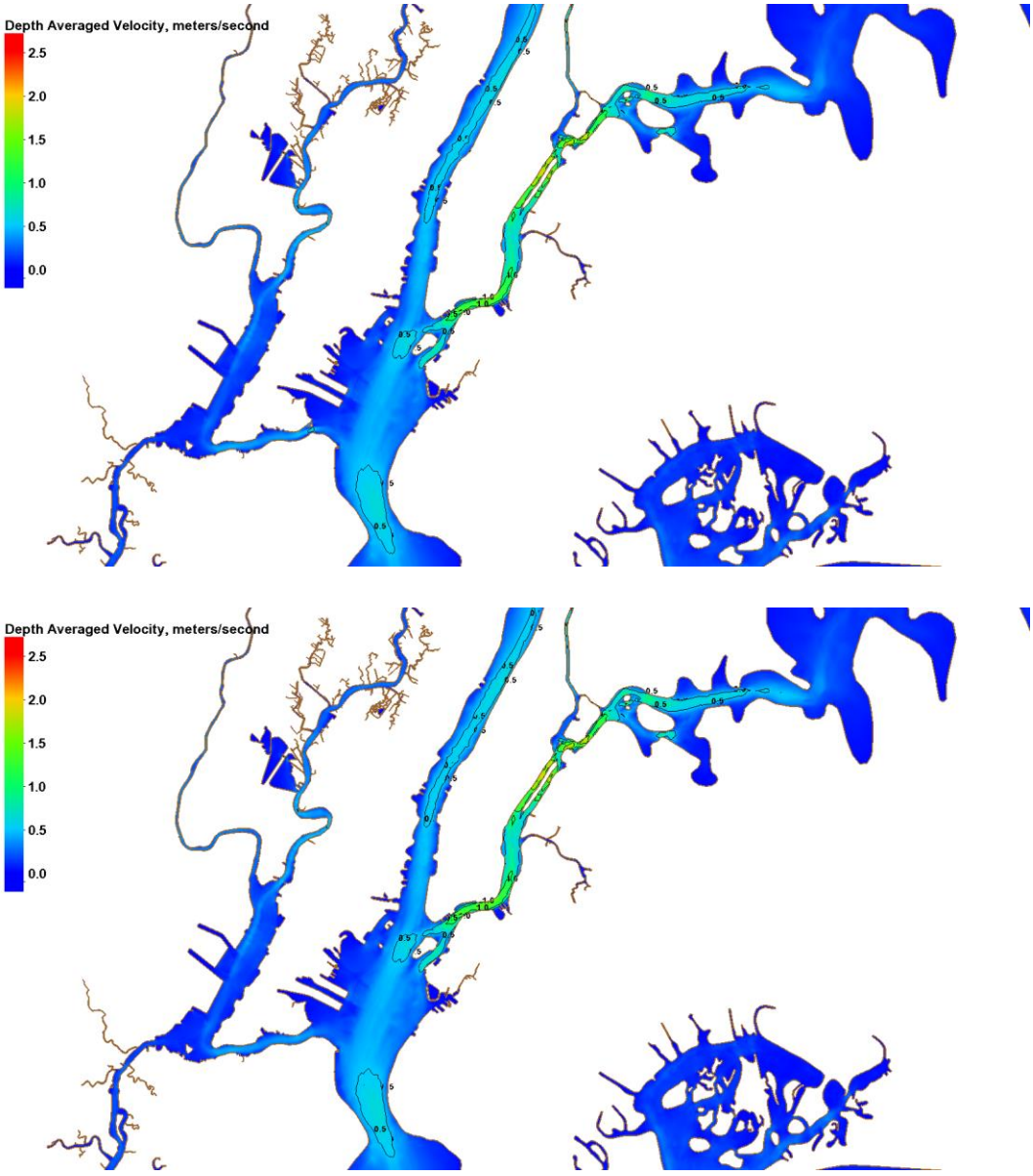


Figure 238: Set 2 Depth Averaged Velocity 75th percentile in Upper Bay (Alternative 3b top, Base Bottom)

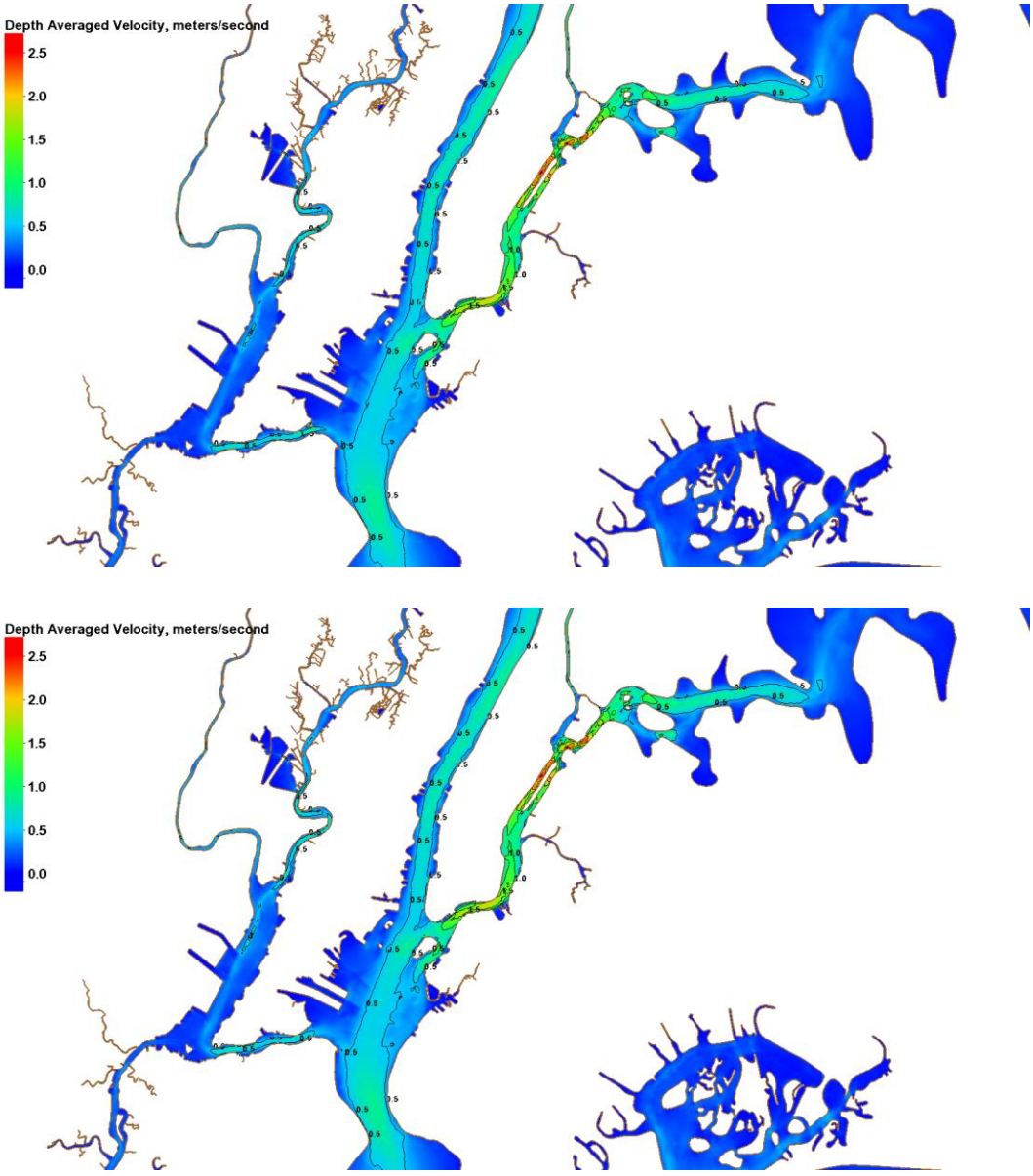


Figure 239: Set 2 Depth Averaged Velocity 50th percentile in Lower Bay (Alternative 4 top, Base Bottom)

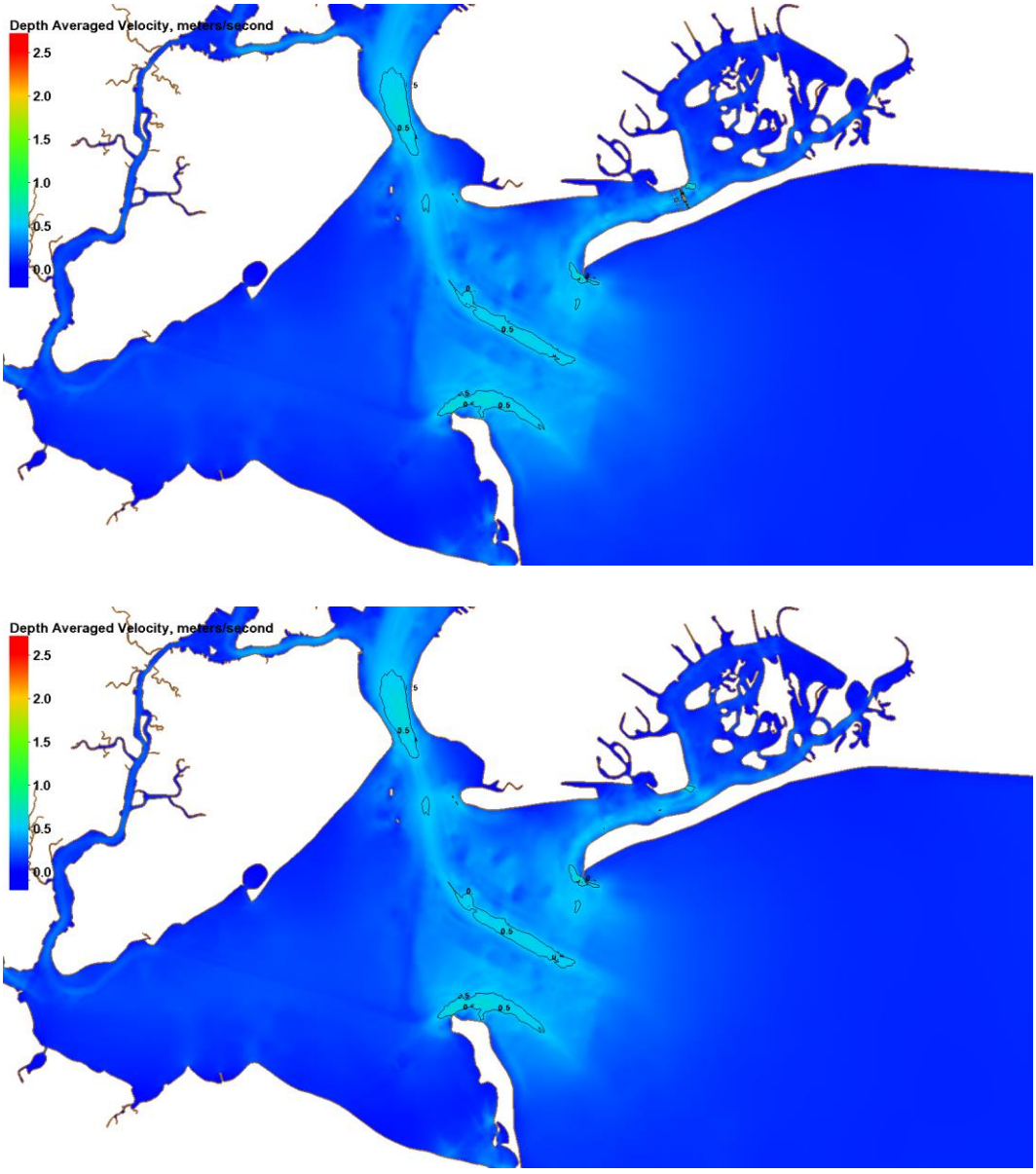


Figure 240: Set 2 Depth Averaged Velocity 75th percentile in Lower Bay (Alternative 4 top, Base Bottom)

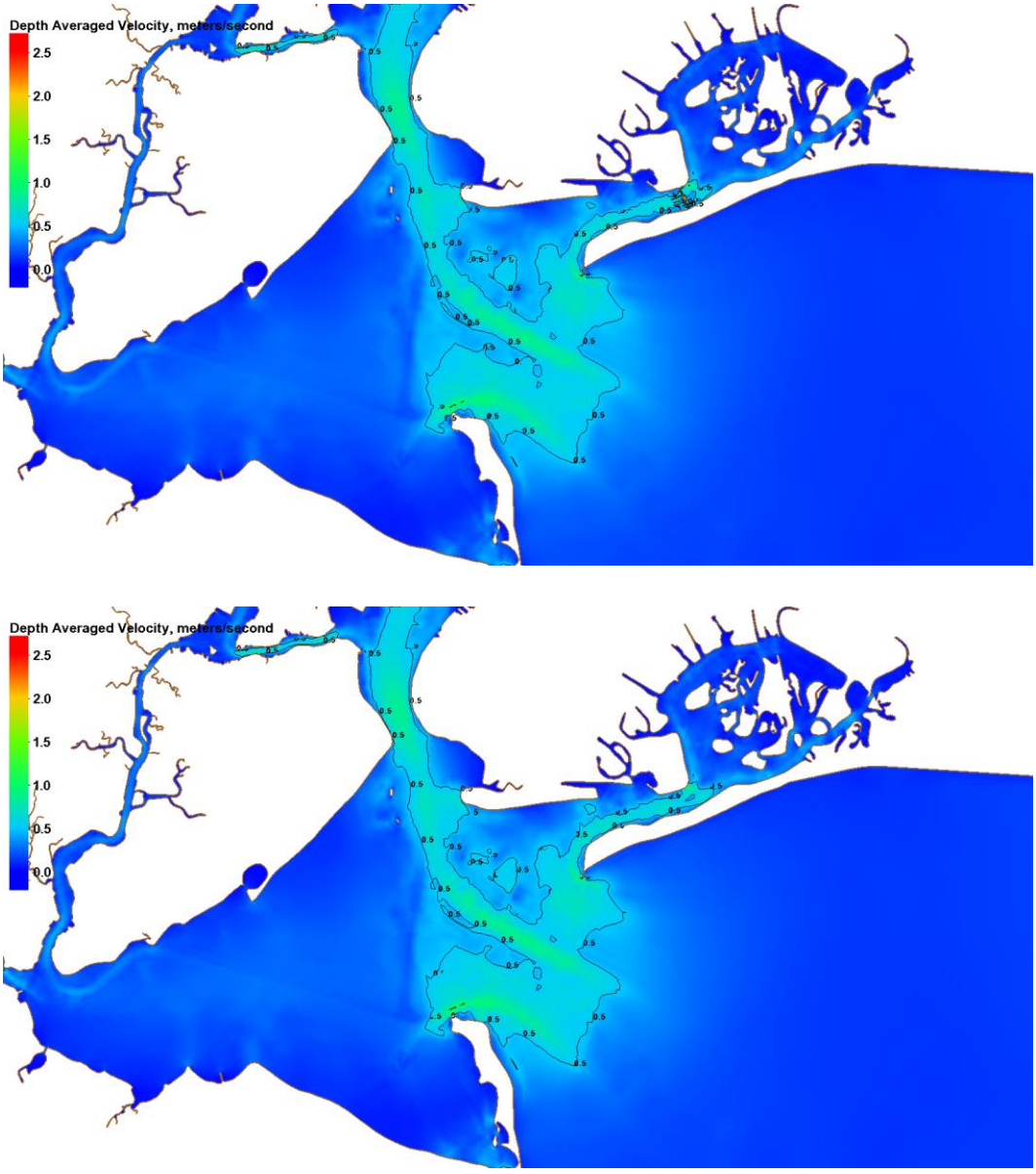


Figure 241: Set 2 Depth Averaged Velocity 50th percentile in Upper Bay (Alternative 4 top, Base Bottom)

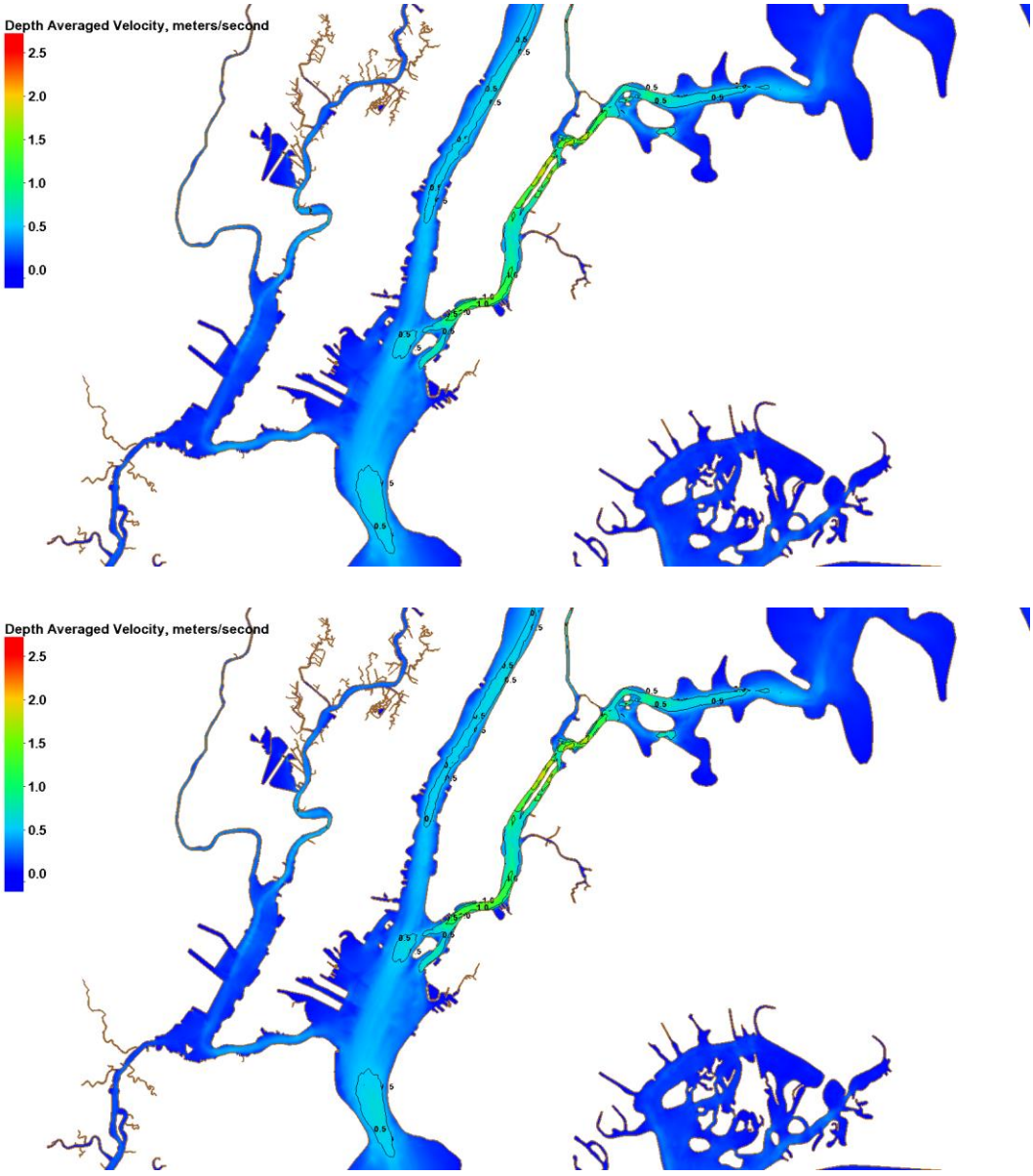
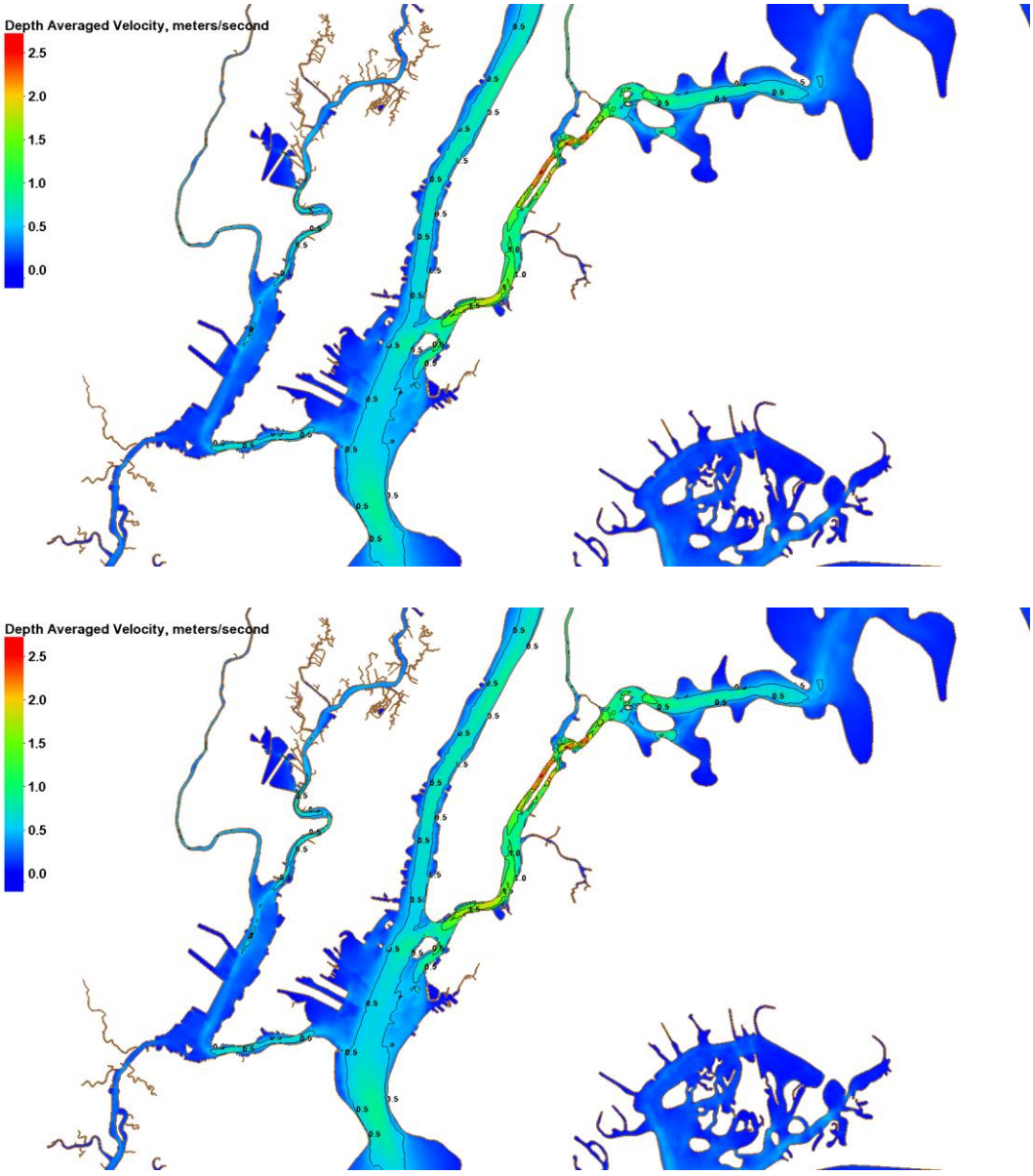


Figure 242: Set 2 Depth Averaged Velocity 75th percentile in Upper Bay (Alternative 4 top, Base Bottom)



3.2.2.2 Spatial Salinity Percentiles

The spatial depth average velocity percentile figures are depicted in Figure 243 through Figure 258.

Figure 243: Set 2 Bottom Salinity 50th percentile in Lower Bay (Alternative 2 top, Base Bottom)

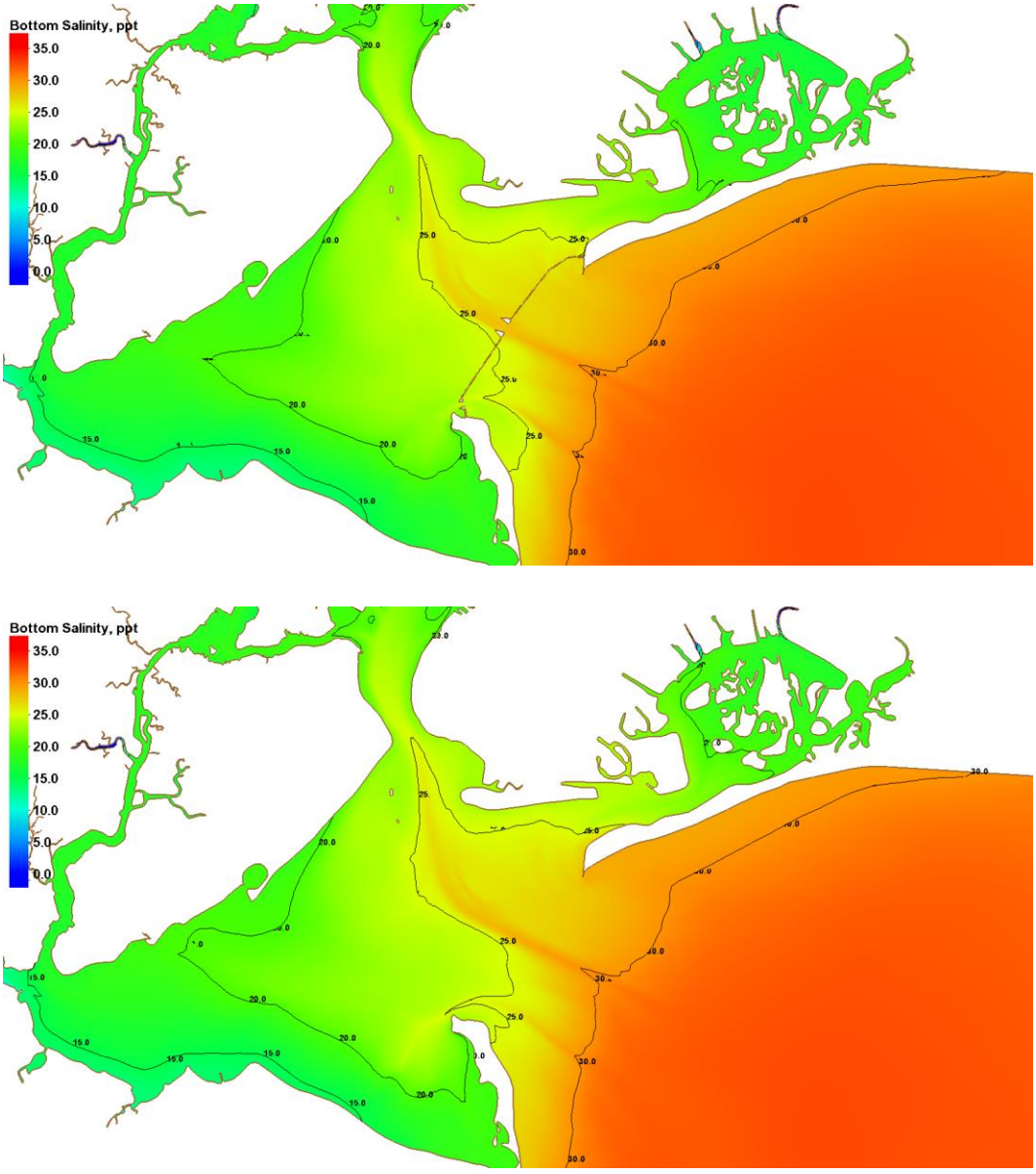


Figure 244: Set 2 Bottom Salinity 75th percentile in Lower Bay (Alternative 2 top, Base Bottom)

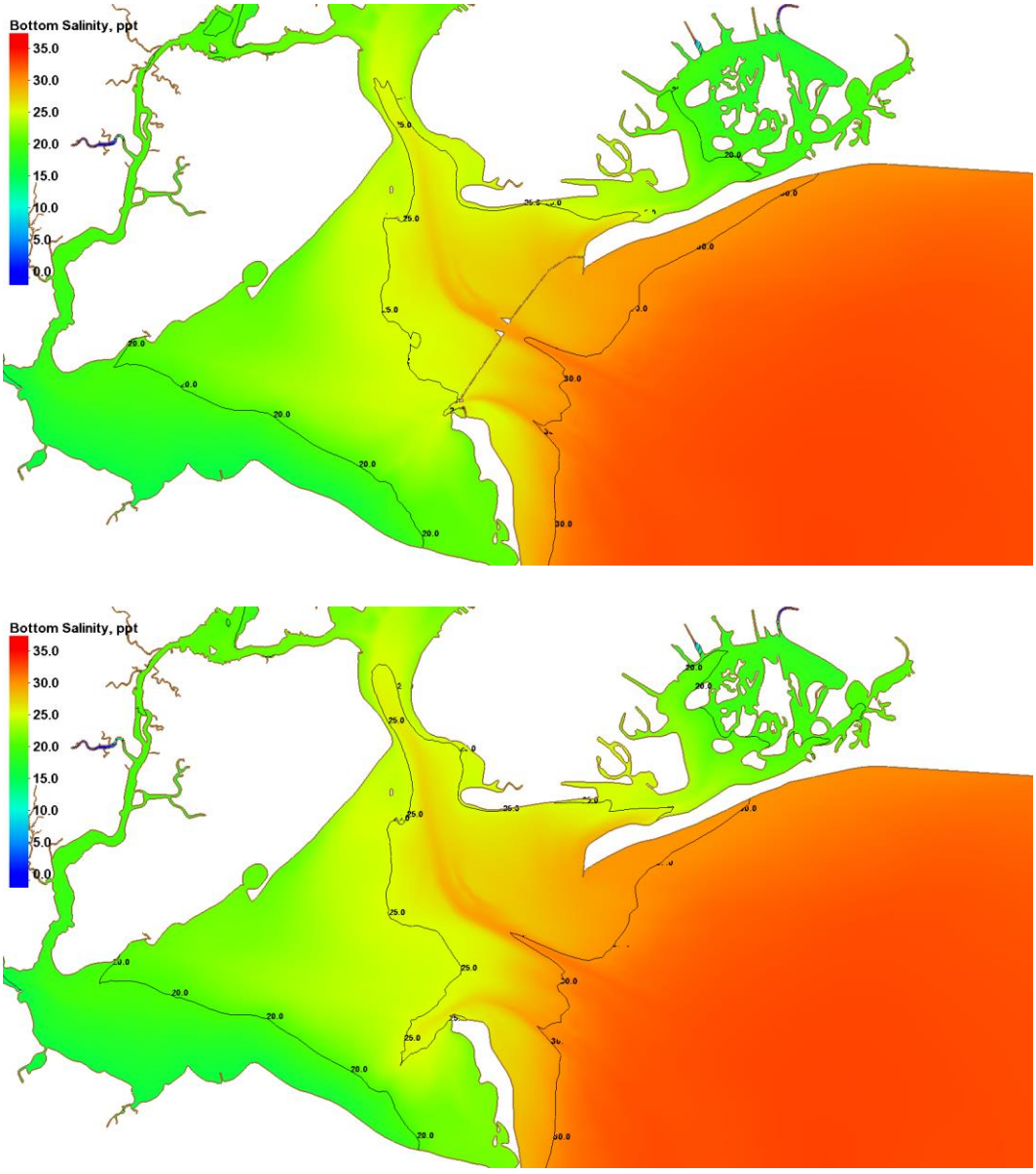


Figure 245: Set 2 Bottom Salinity 50th percentile in Upper Bay (Alternative 2 top, Base Bottom)

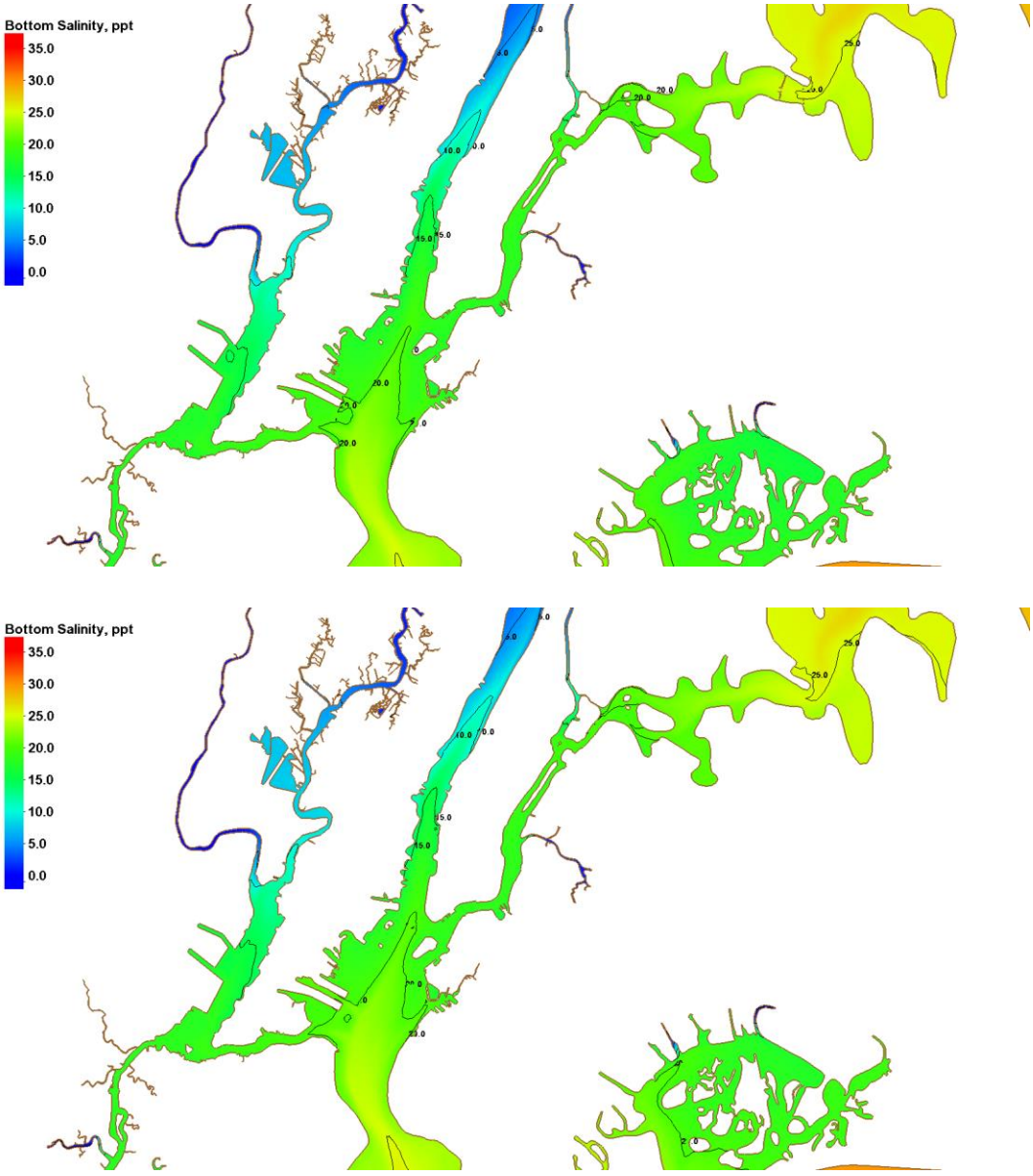


Figure 246: Set 2 Bottom Salinity 75th percentile in Upper Bay (Alternative 2 top, Base Bottom)

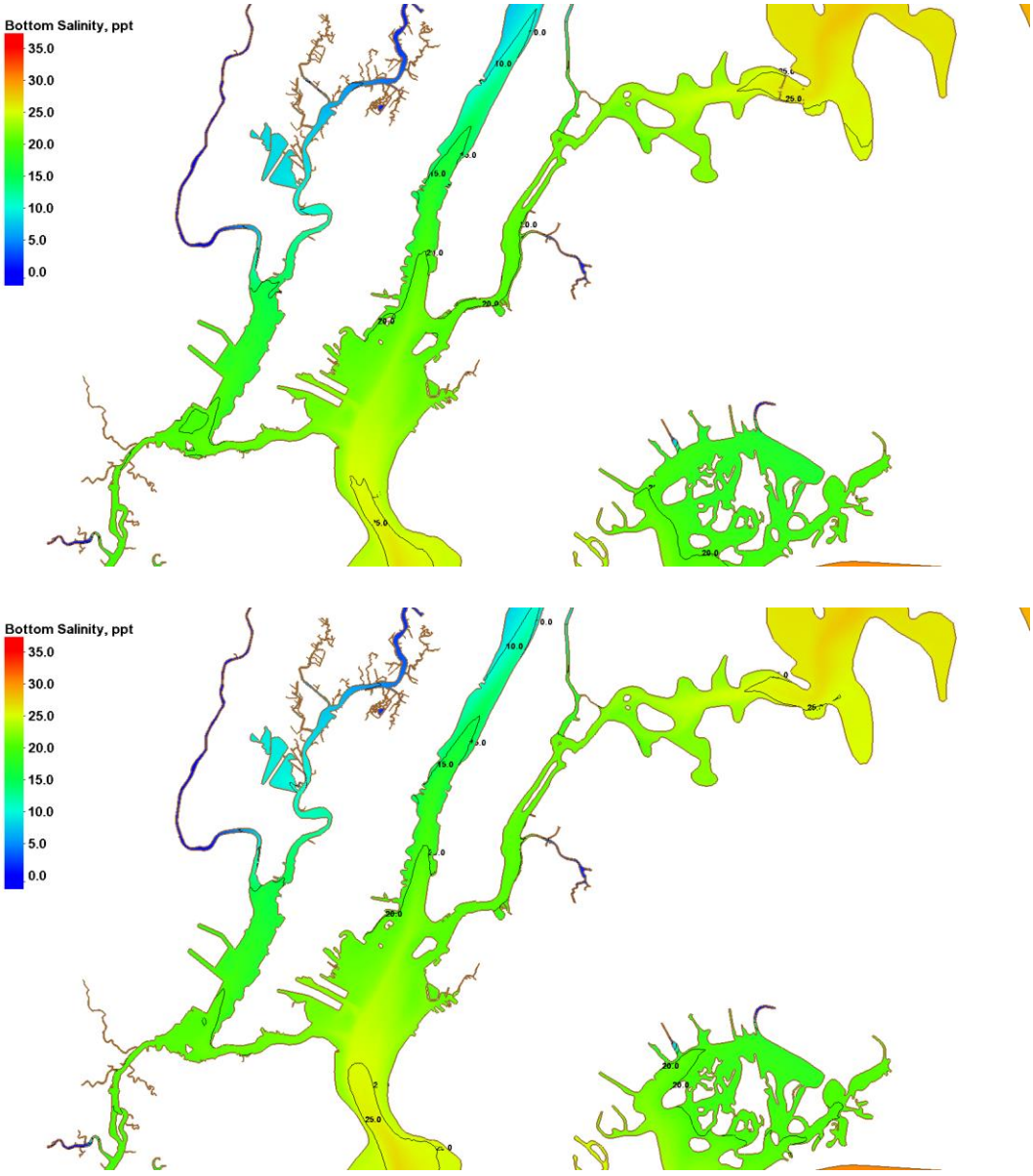


Figure 247: Set 2 Bottom Salinity 50th percentile in Lower Bay (Alternative 3a top, Base Bottom)

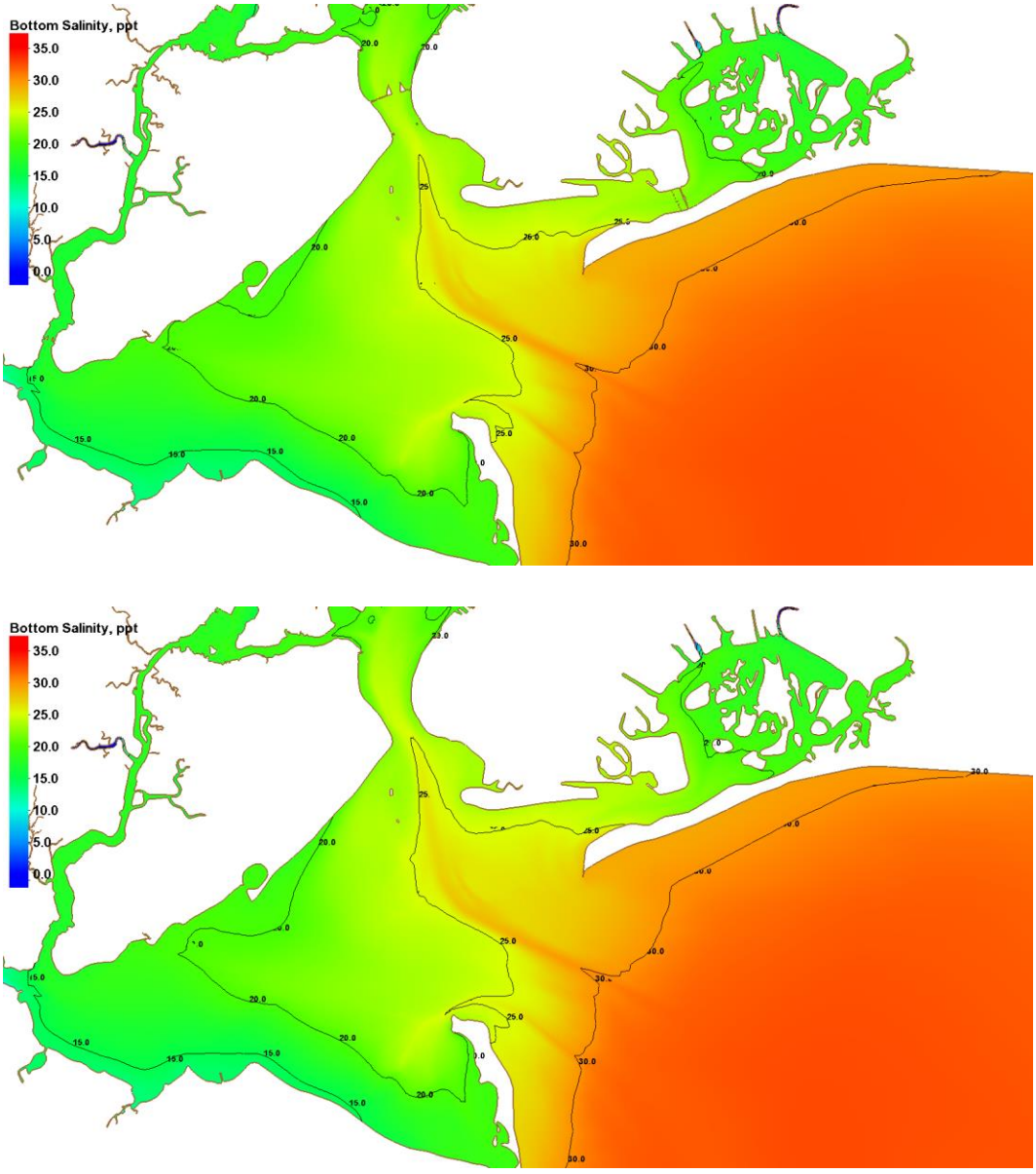


Figure 248: Set 2 Bottom Salinity 75th percentile in Lower Bay (Alternative 3a top, Base Bottom)

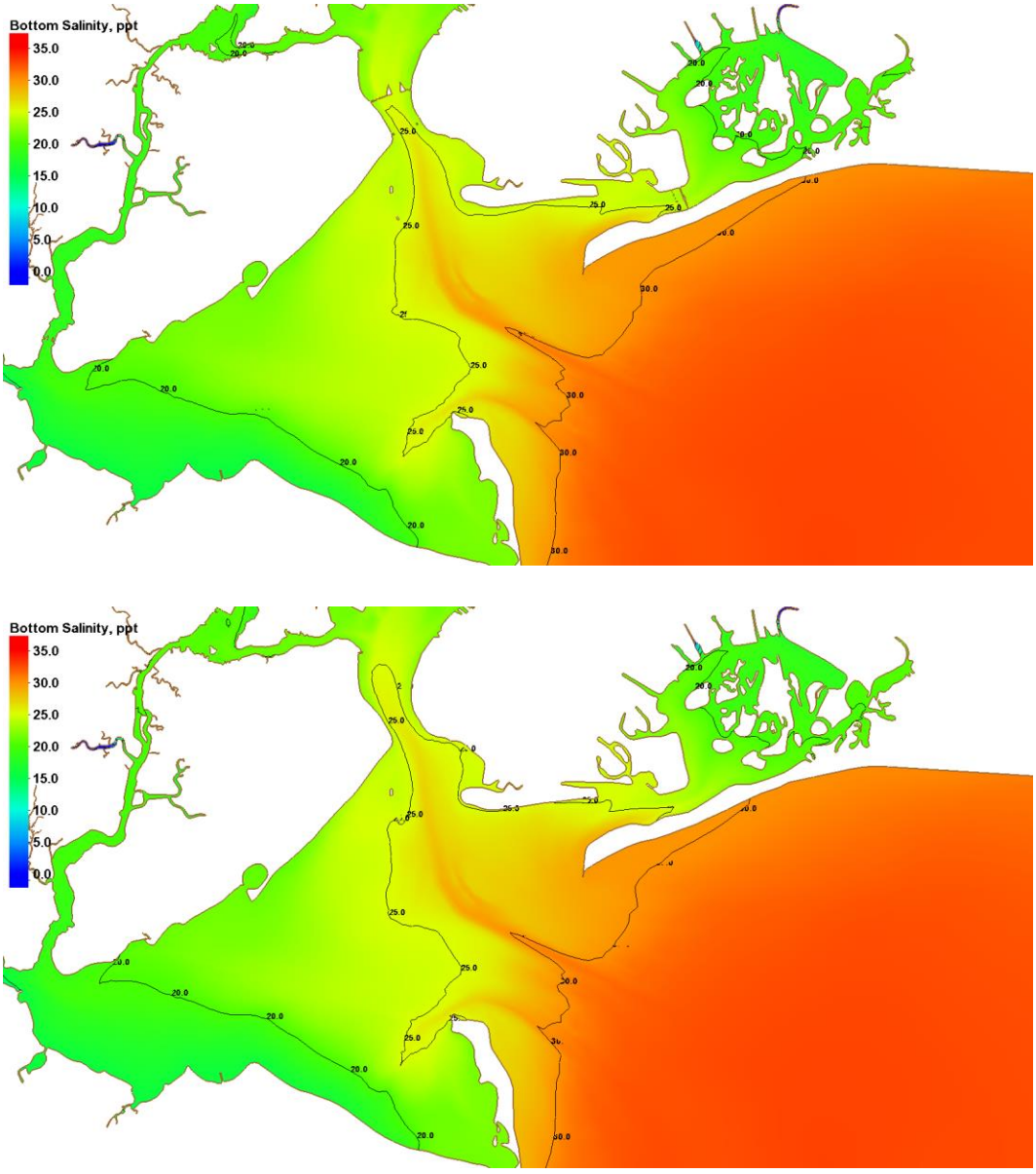


Figure 249: Set 2 Bottom Salinity 50th percentile in Upper Bay (Alternative 3a top, Base Bottom)

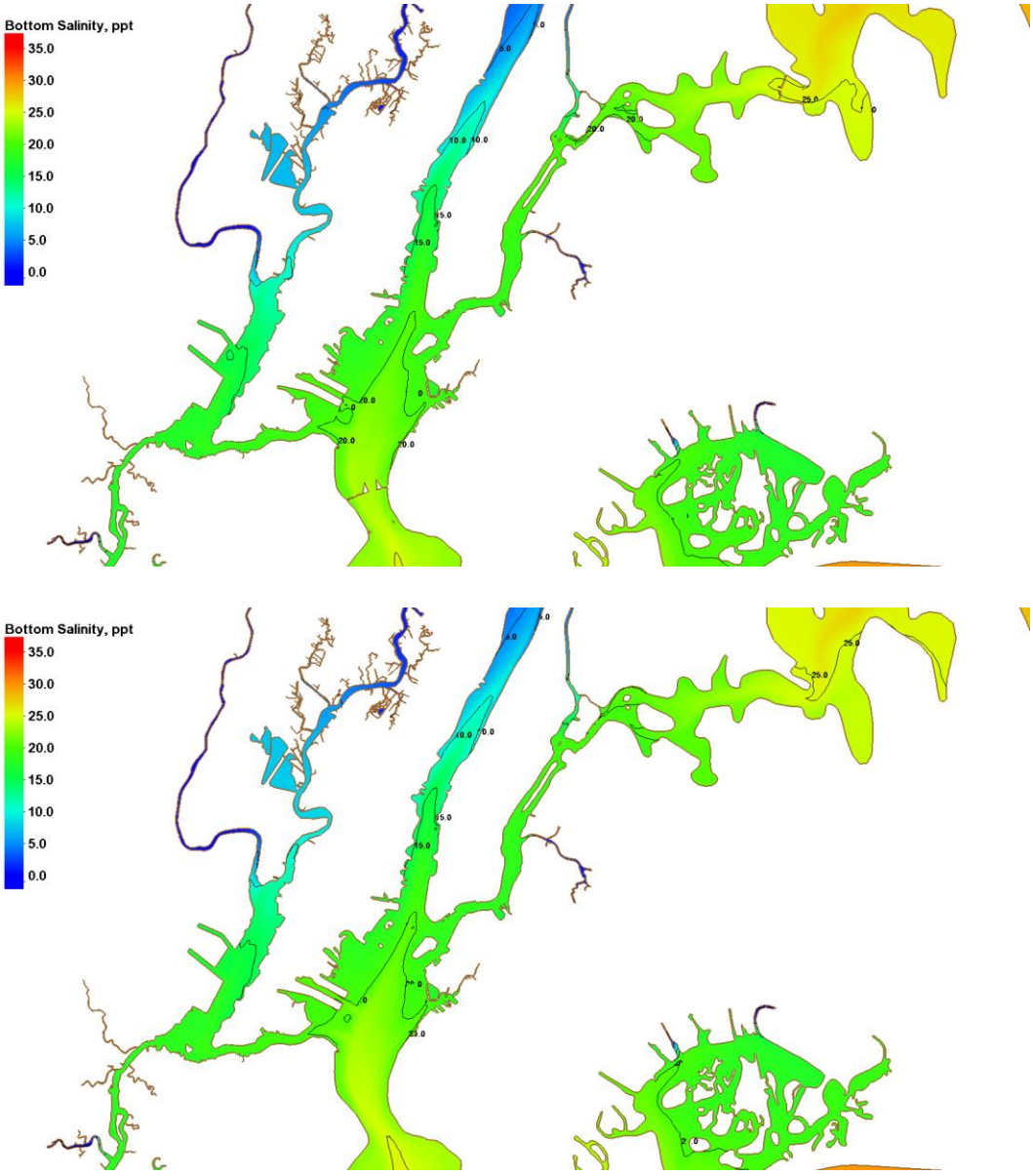


Figure 250: Set 2 Bottom Salinity 50th percentile in Upper Bay (Alternative 3a top, Base Bottom)

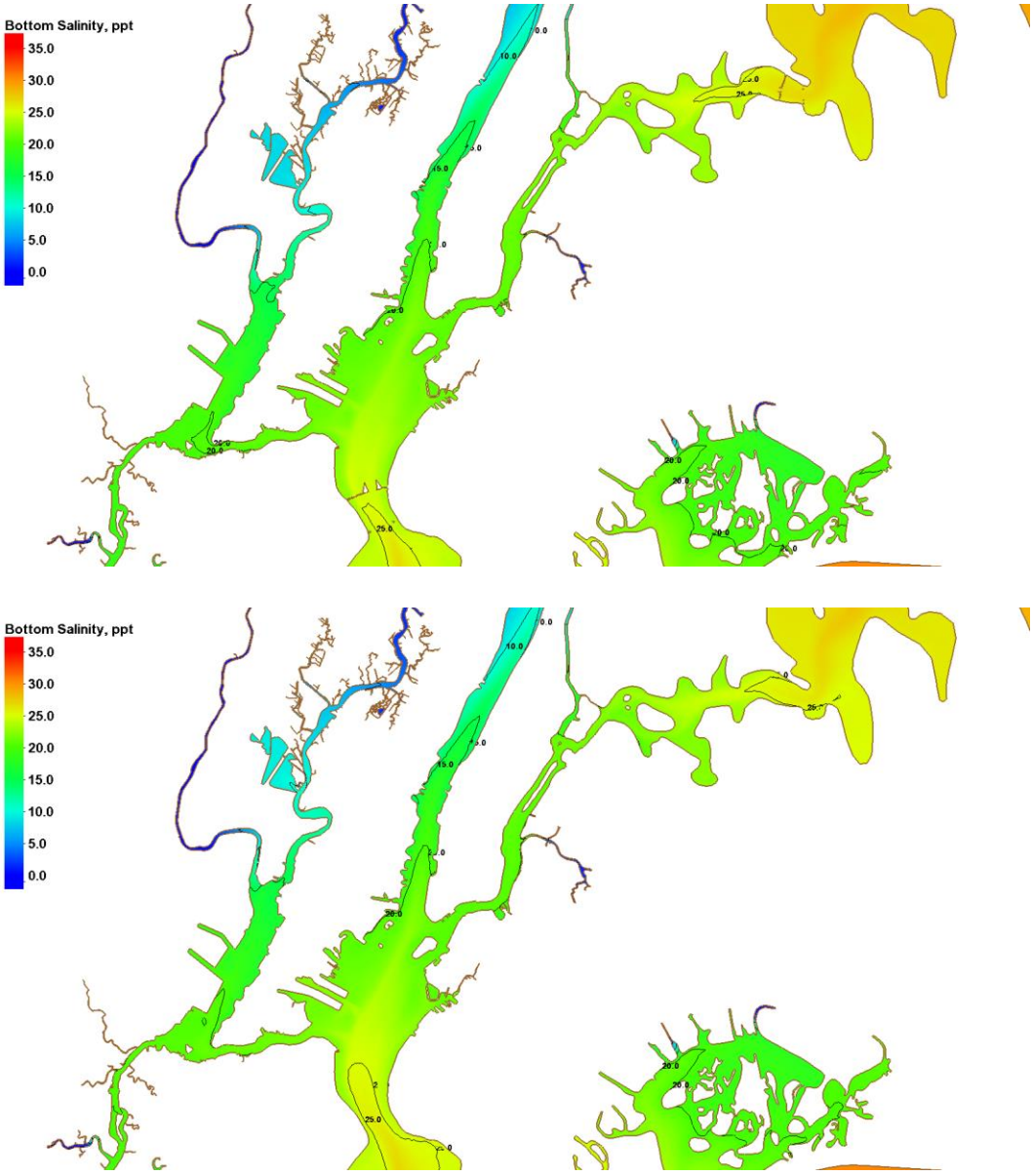


Figure 251: Set 2 Bottom Salinity 50th percentile in Lower Bay (Alternative 3b top, Base Bottom)

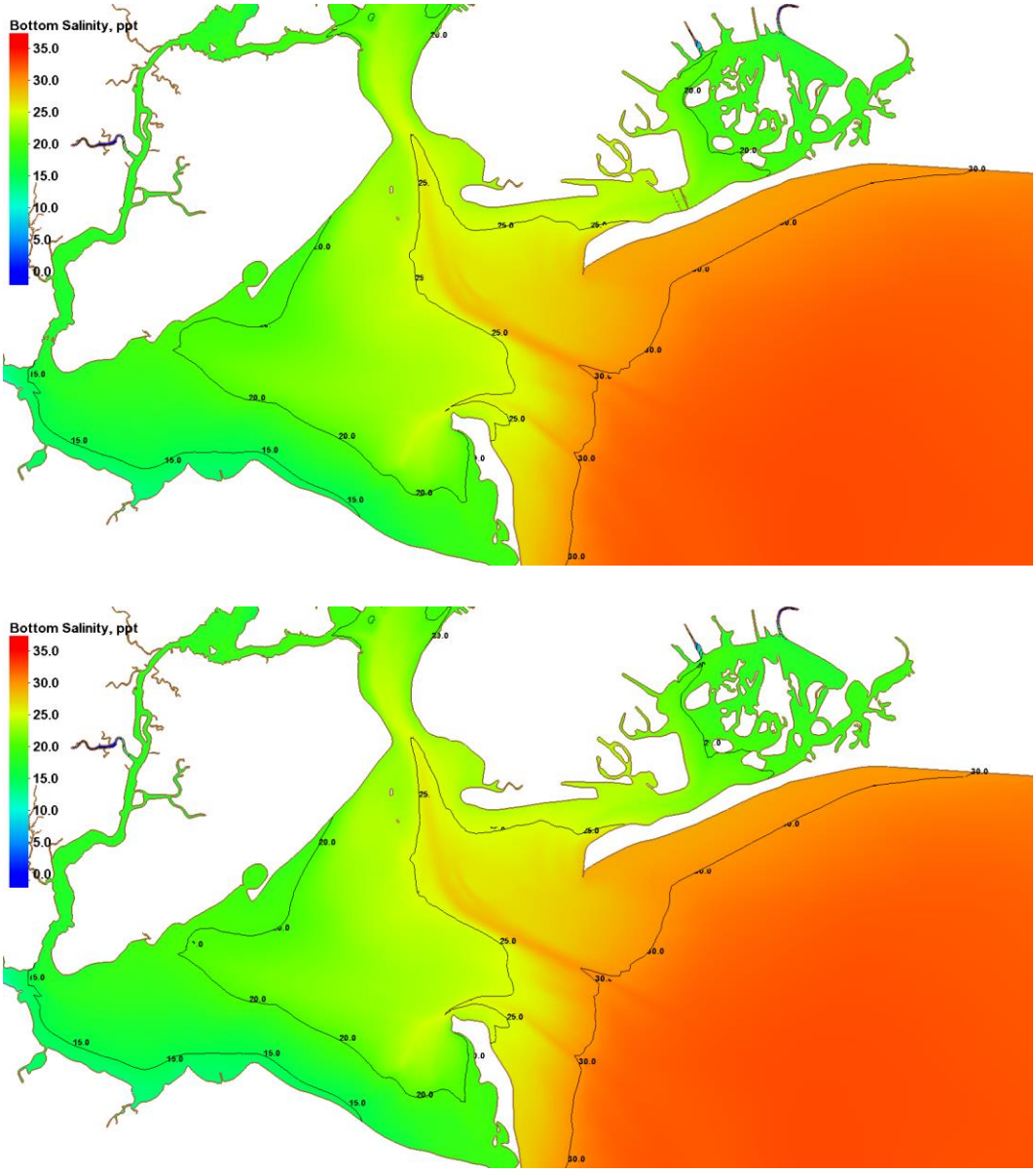


Figure 252: Set 2 Bottom Salinity 75th percentile in Lower Bay (Alternative 3b top, Base Bottom)

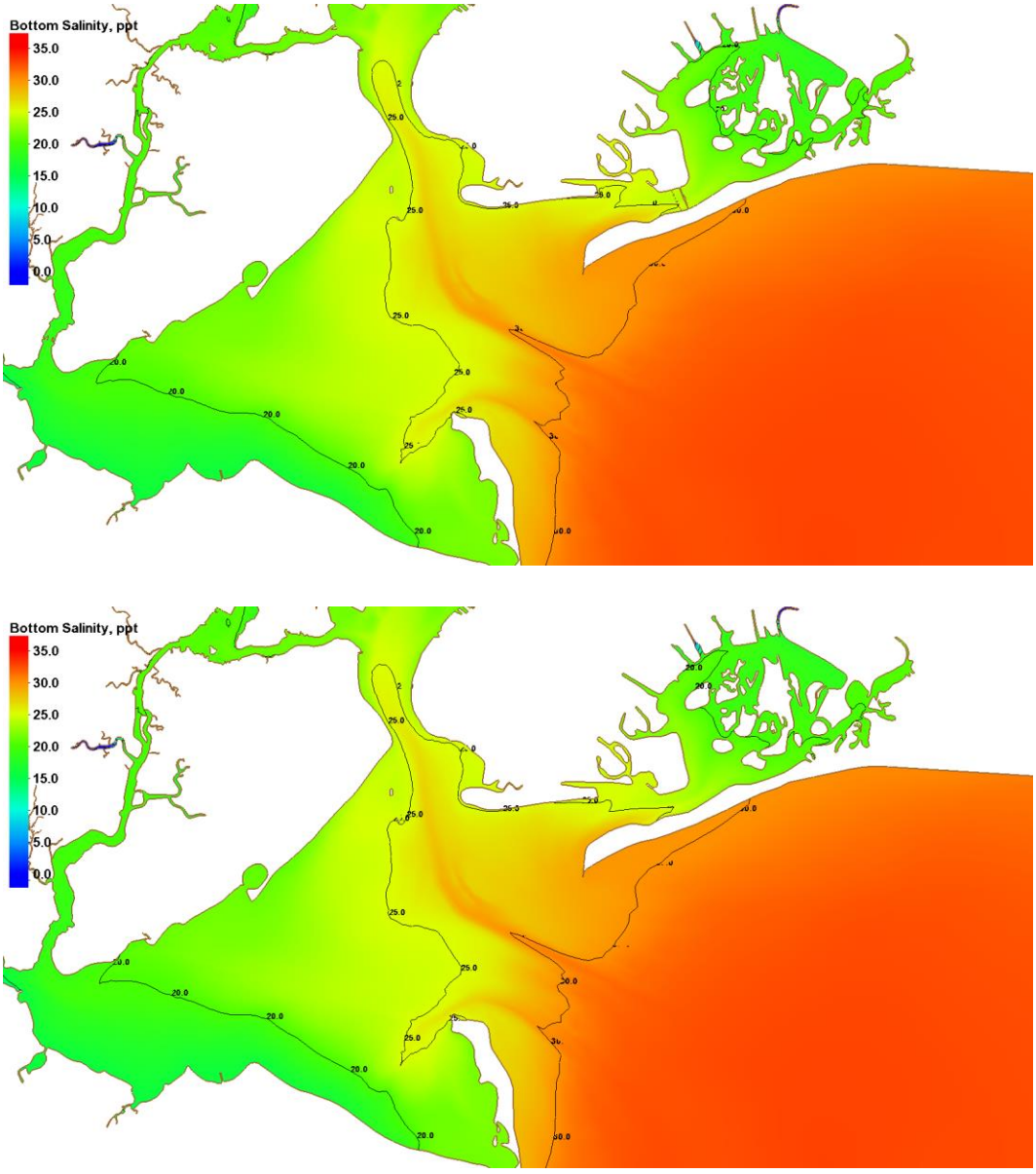


Figure 253: Set 2 Bottom Salinity 50th percentile in Upper Bay (Alternative 3b top, Base Bottom)

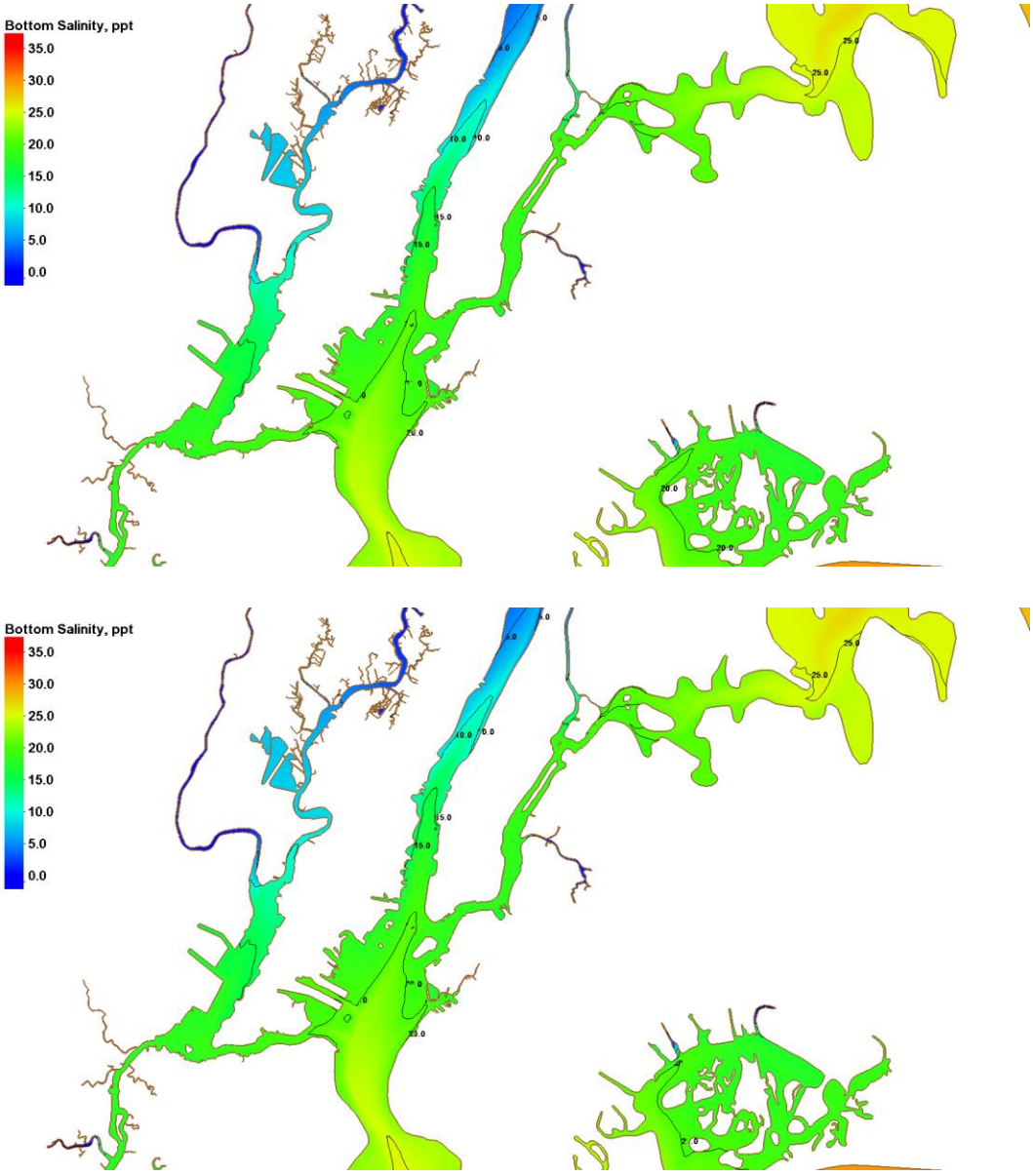


Figure 254: Set 2 Bottom Salinity 75th percentile in Upper Bay (Alternative 3b top, Base Bottom)

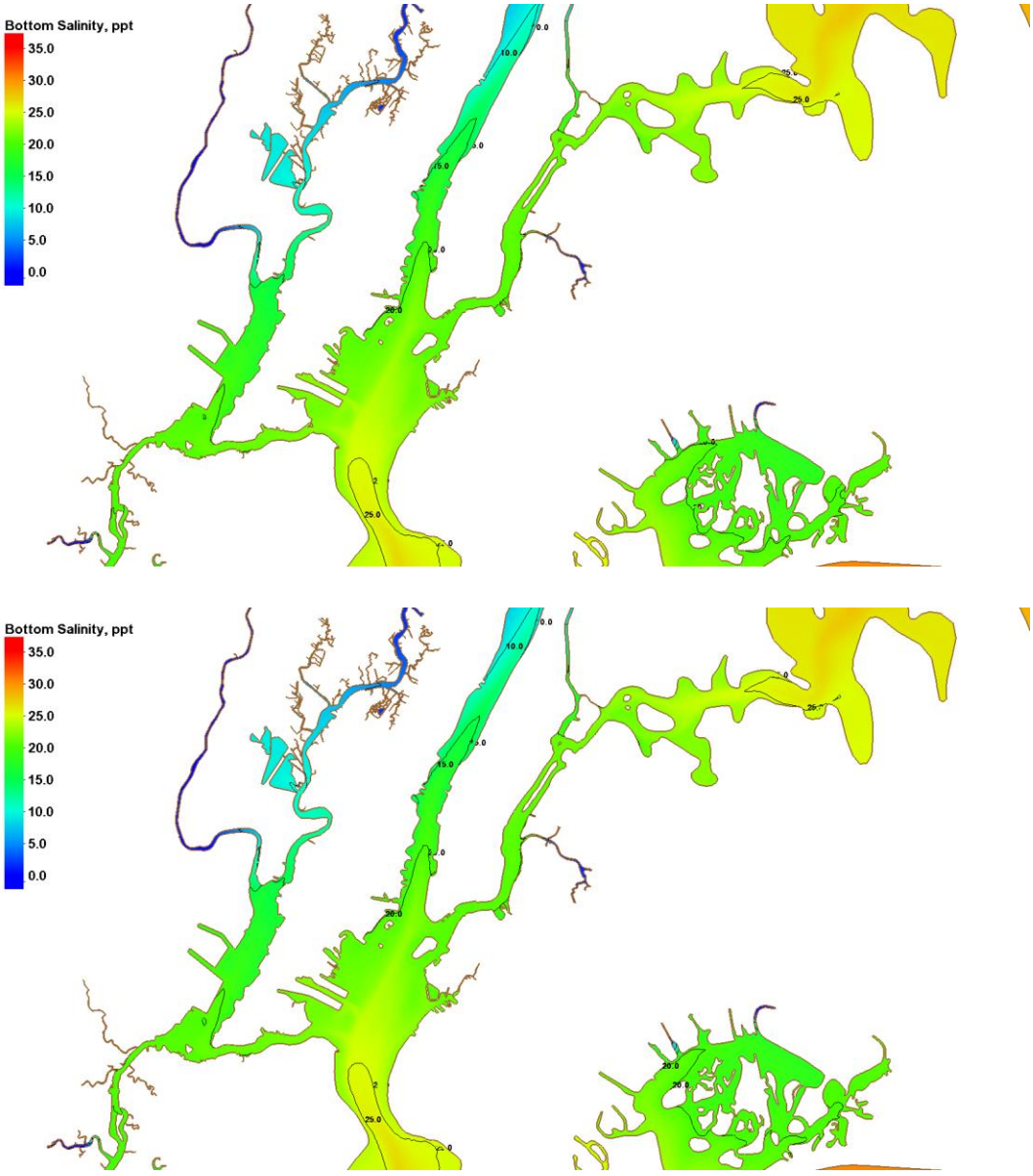


Figure 255: Set 2 Bottom Salinity 50th percentile in Lower Bay (Alternative 4 top, Base Bottom)

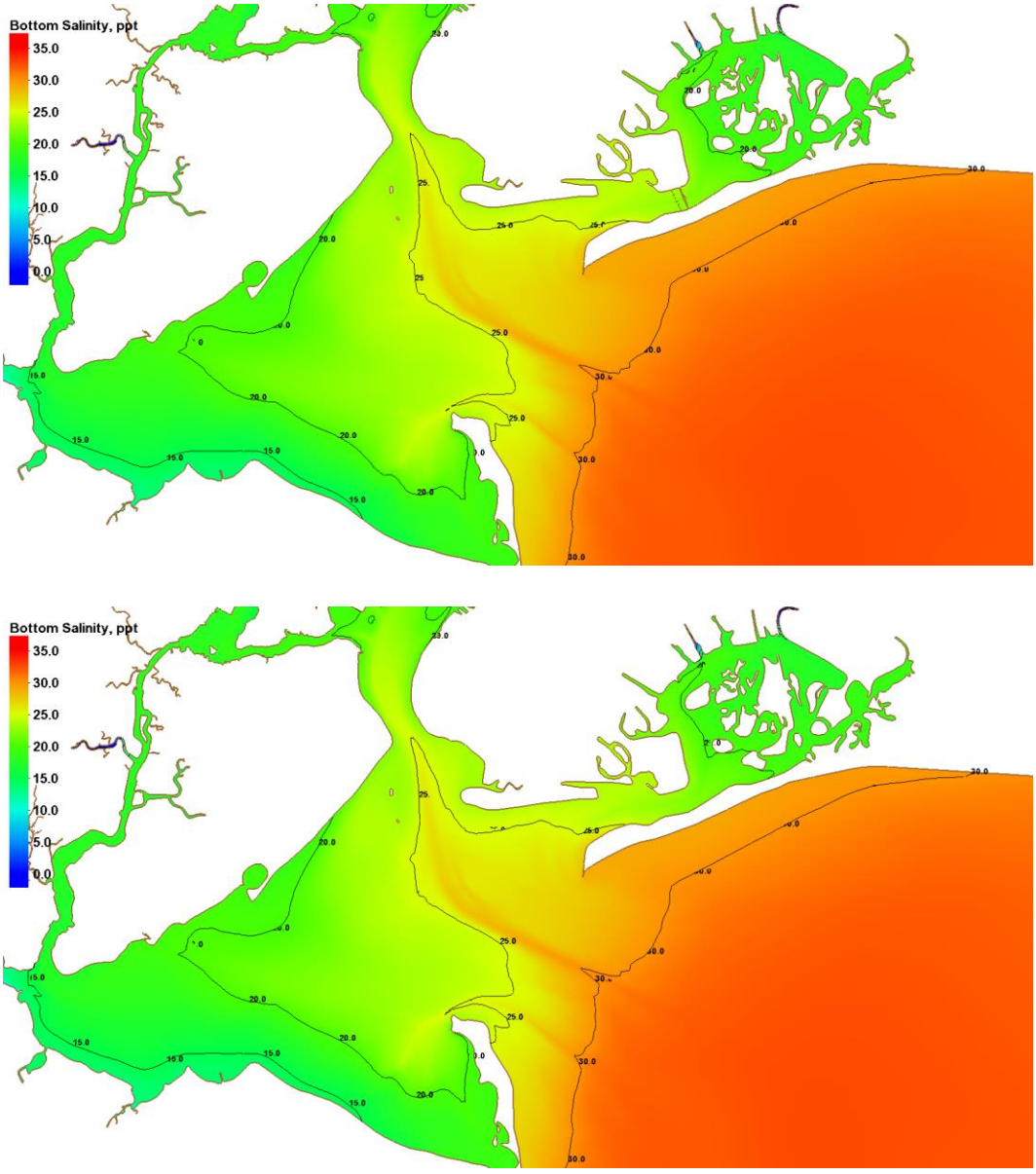


Figure 256: Set 2 Bottom Salinity 75th percentile in Lower Bay (Alternative 4 top, Base Bottom)

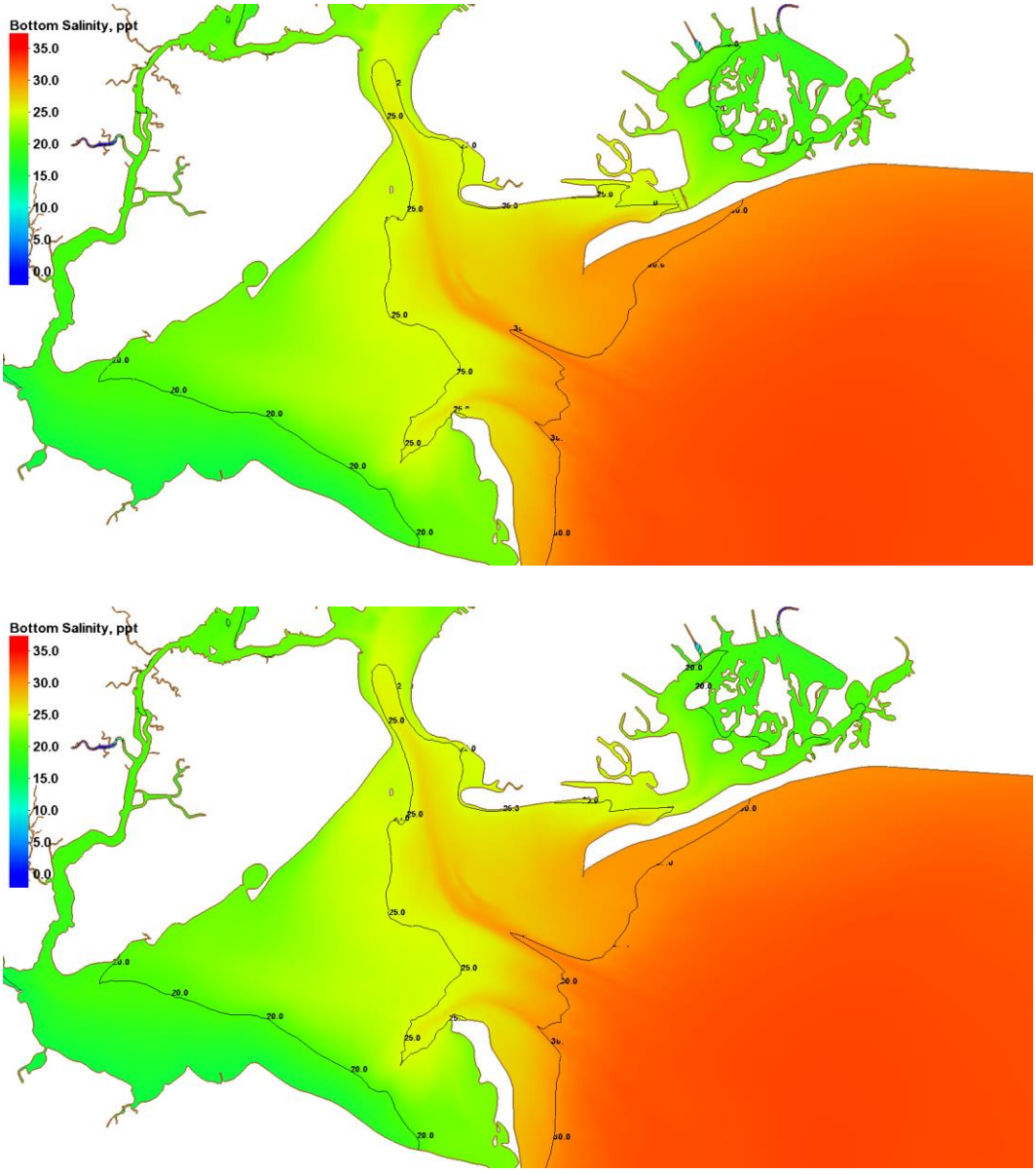


Figure 257: Set 2 Bottom Salinity 50th percentile in Upper Bay (Alternative 4 top, Base Bottom)

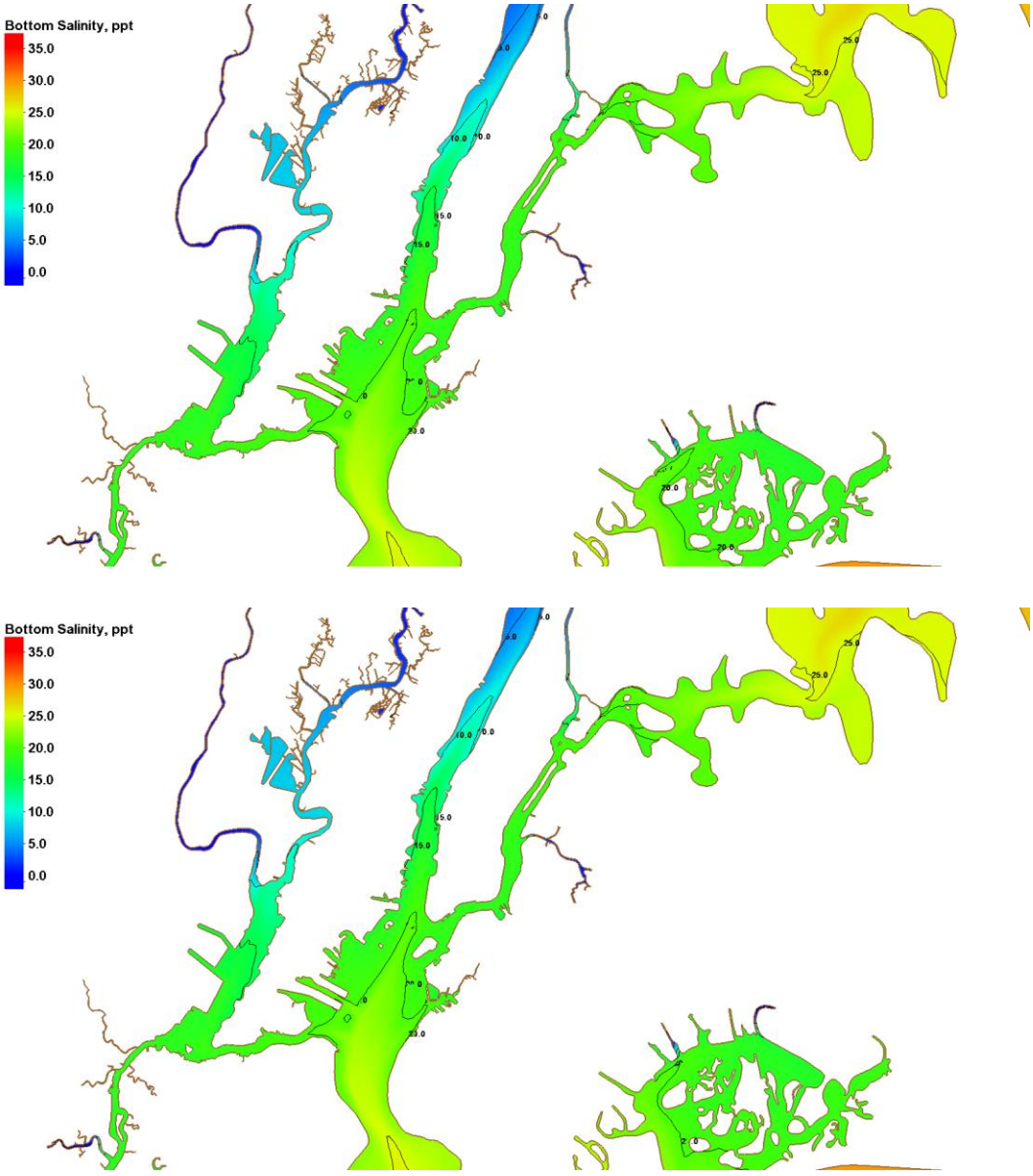
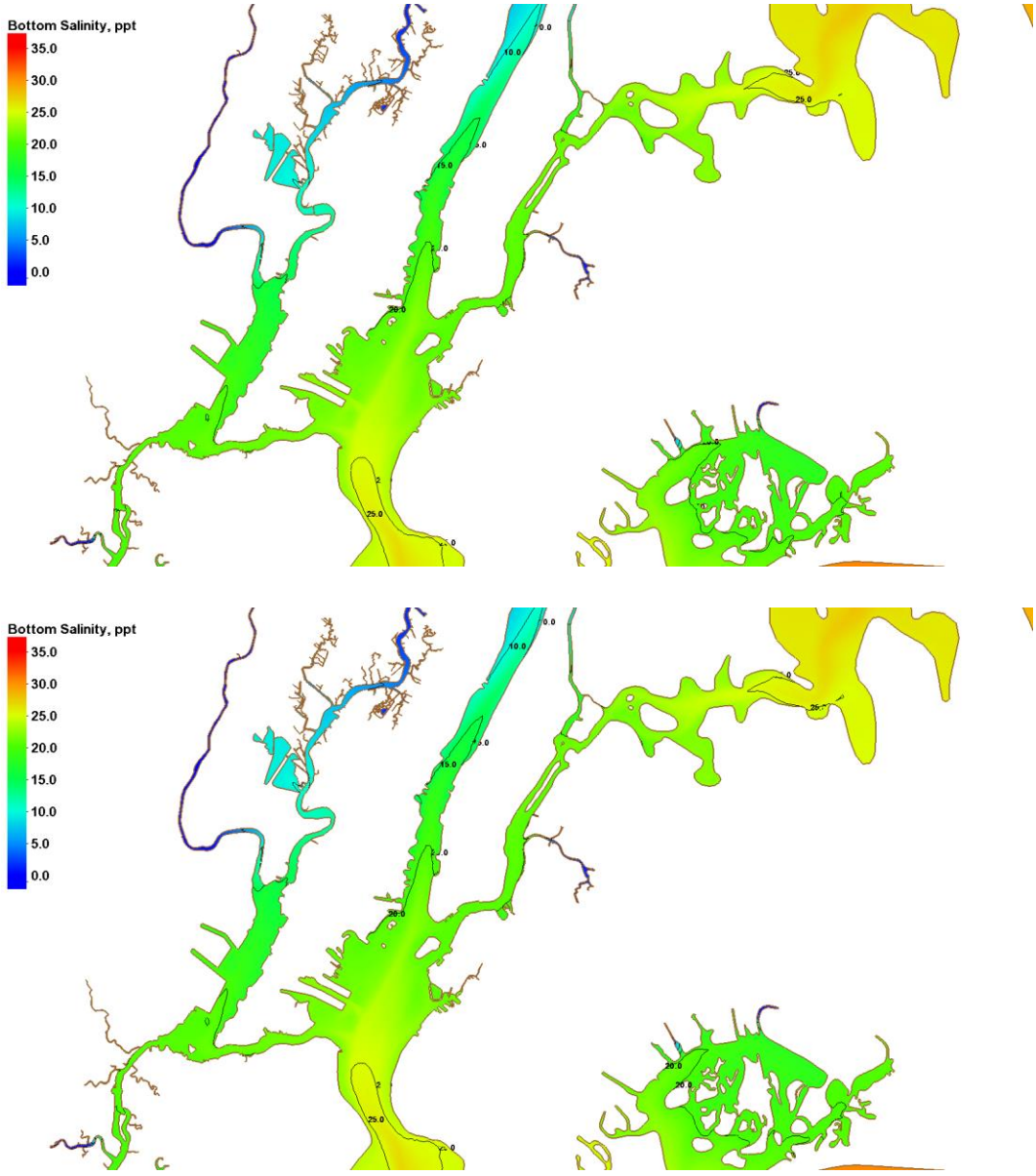


Figure 258: Set 2 Bottom Salinity 75th percentile in Upper Bay (Alternative 4 top, Base Bottom)



3.2.3 Tidal Prism Analysis

A tidal prism percentile analysis was performed for the areas in the mesh that are impounded by the barriers and then compared to the tidal prism for the same areas in the without project.

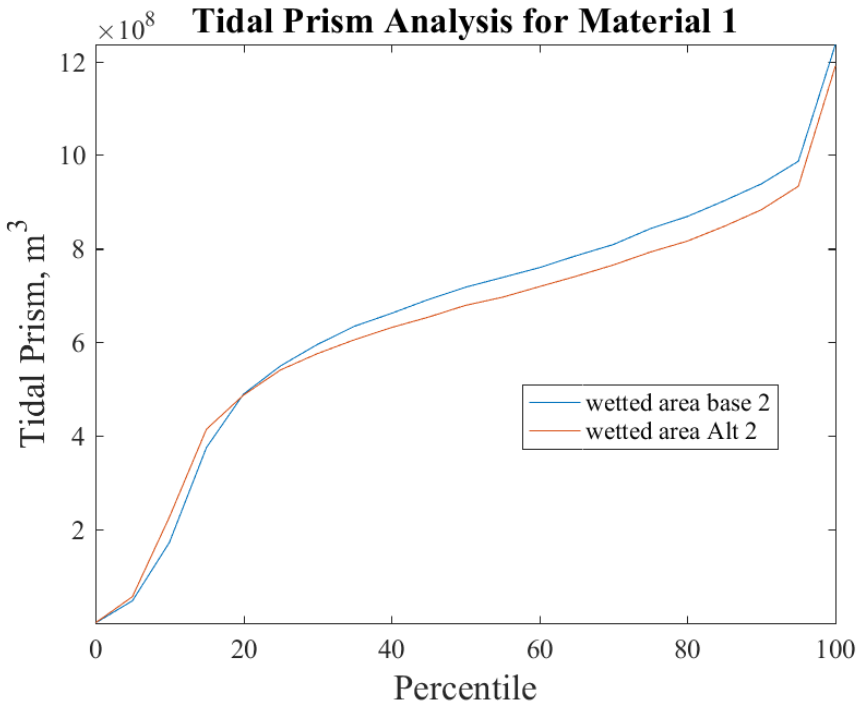
3.2.3.1 Alternative 2

Figure 259 is a schematic of the area used to calculate the tidal prism, and Figure 260 is the percentile analysis.

Figure 259: Set 2 Alternative 2 Tidal Prism Material Schematic



Figure 260: Set 2 Alternative 2 Tidal Prism Material 1



3.2.3.2 Alternative 3a

Figure 261 is a schematic of the area used to calculate the tidal prism, and Figure 262 and Figure 263 is the percentile analysis.

Figure 261: Set 2 Alternative 3a Tidal Prism Material Schematic



Figure 262: Set 2 Alternative 3a Tidal Prism Material 1

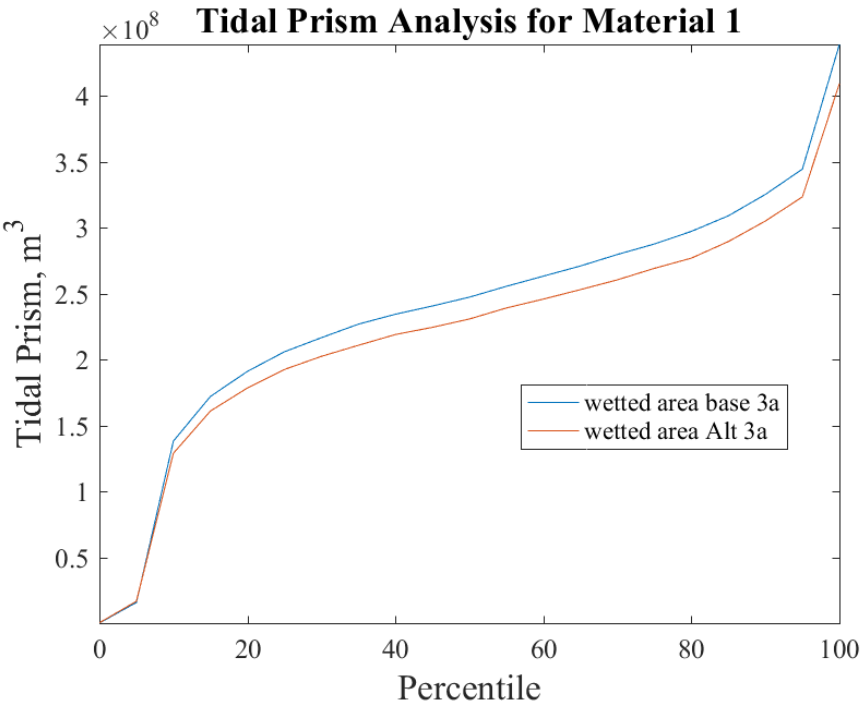
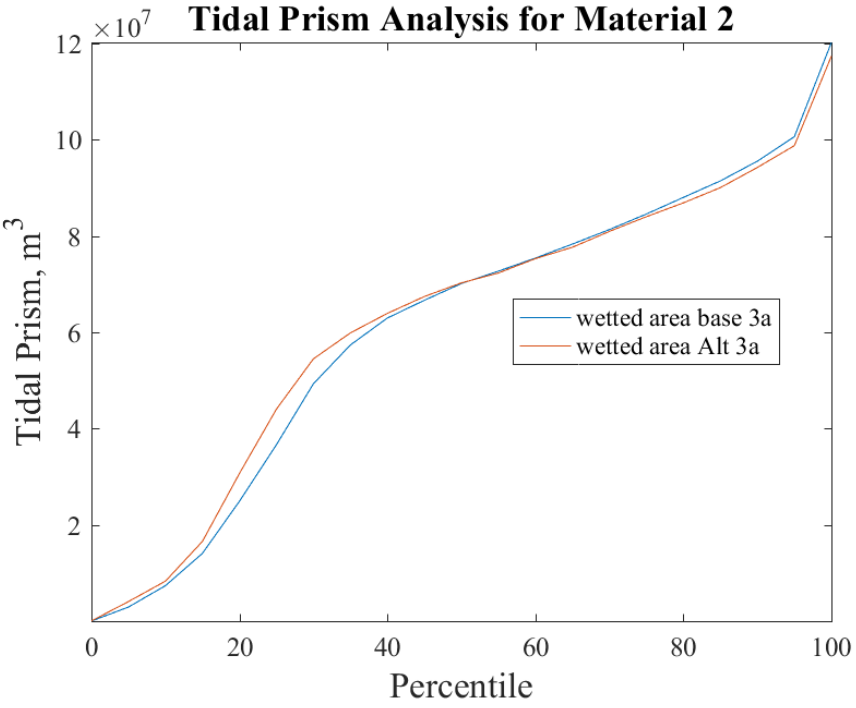


Figure 263: Set 2 Alternative 3a Tidal Prism Material 2



3.2.3.3 Alternative 3b

Figure 264 is a schematic of the area used to calculate the tidal prism, and Figure 265 and Figure 266 is the percentile analysis.

Figure 264: Set 2 Alternative 3b Tidal Prism Material Schematic

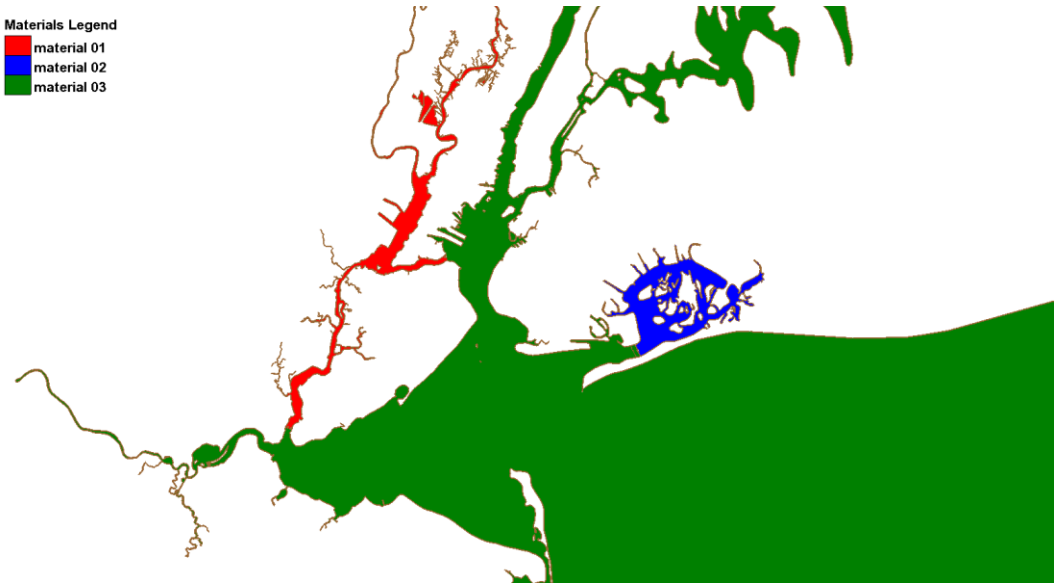


Figure 265: Set 2 Alternative 3b Tidal Prism Material 1

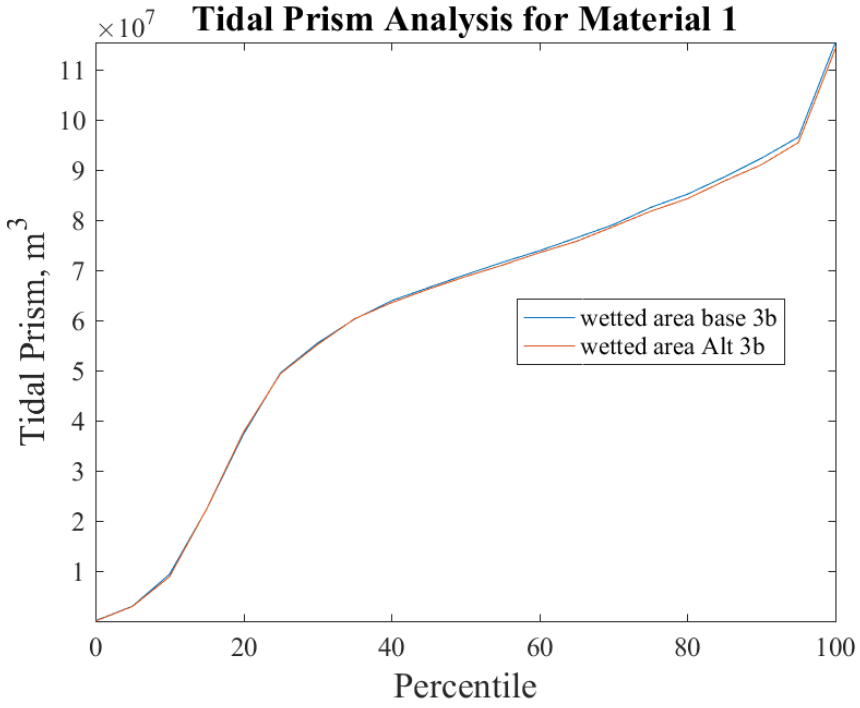
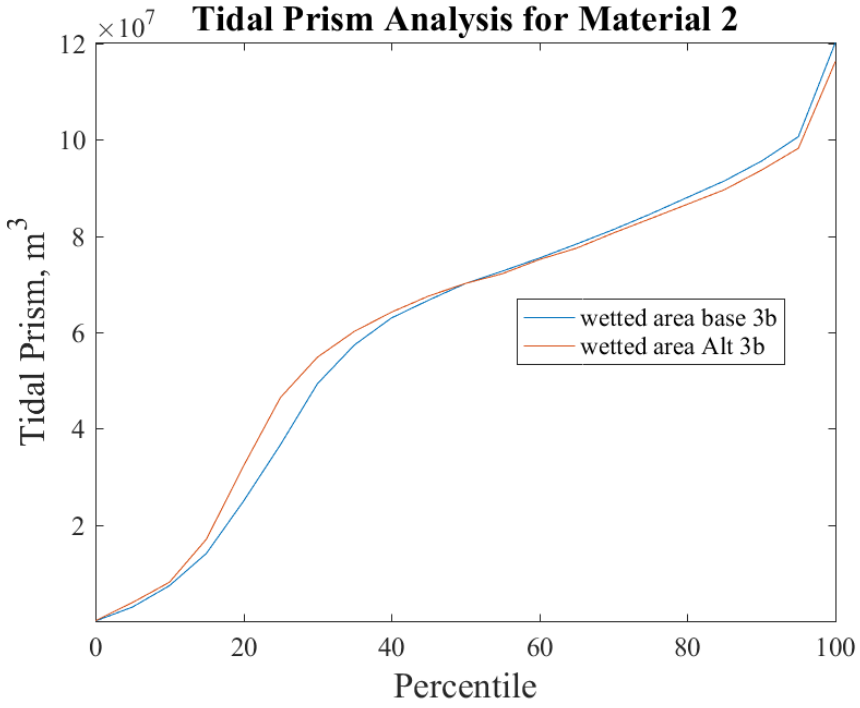


Figure 266: Set 2 Alternative 3b Tidal Prism Material 2



3.2.3.4 Alternative 4

Figure 267 is a schematic of the area used to calculate the tidal prism, and Figure 268 and Figure 269 is the percentile analysis.

Figure 267: Set 2 Alternative 4 Tidal Prism Material Schematic



Figure 268: Set 2 Alternative 4 Tidal Prism Material 1

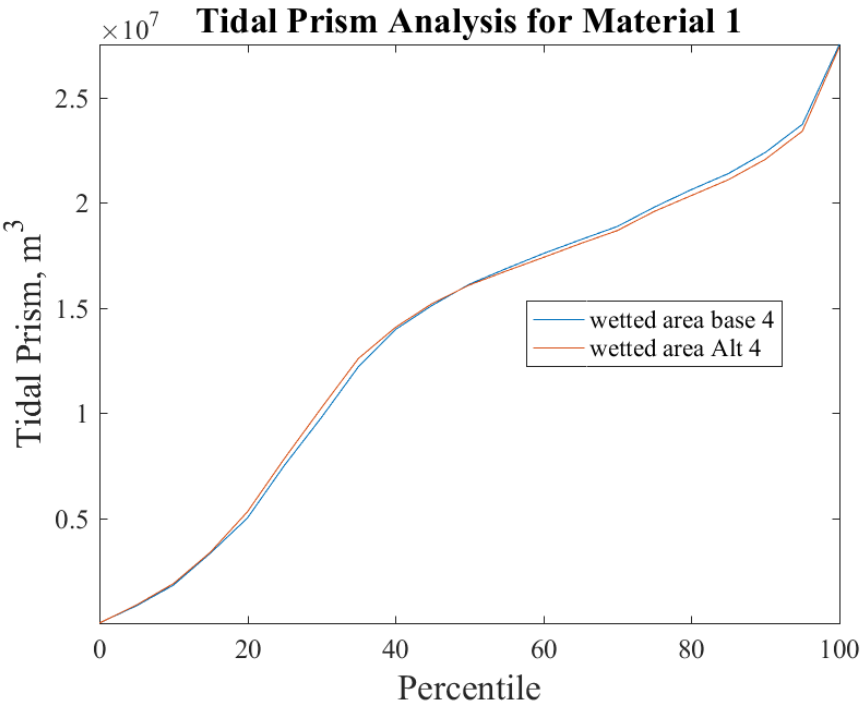
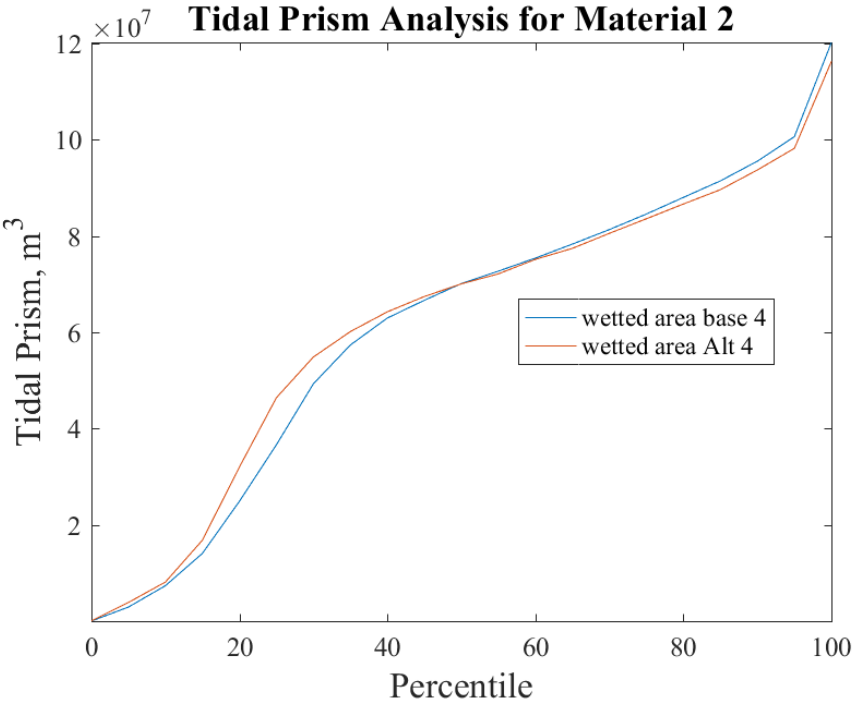


Figure 269: Set 2 Alternative 4 Tidal Prism Material 2



3.2.4 Summary of Results

As with Set 1, the biggest impacts on water surface elevation occur in Alternatives 2 and 3a. For Alternative 2, the tidal ranges decrease slightly at all of the analysis points. Alternative 3a has the largest impact on water surface elevation. Within the impounded areas, Alternative 3a has a slightly higher decrease in tidal range than Alternative 2. There are, however, negligible effects in the areas not impounded by the proposed structures in Alternative 3a. Alternatives 3b and 4, as with Set 1, have negligible effects on the water surface elevations at the analyzed point locations. Again the velocities do follow the same general trend. The velocities do increase at the points that are at a proposed structure, and doesn't deviate much at the points with no structure for a particular alternative. An example of this would be point S2. This point is in-between one of the sector gates for Alternative 3a, and the velocity for Alternative 3a is much higher compared to the without project velocities. At the same point (S2) the velocities for Alternatives 2, 3b, and 4 does not deviate much from the without project velocity since there is no structure at that point for these alternatives. The velocity percentile analysis plots are another representation of this same behavior. Again, this behavior is also noticeable in the spatial velocity figures. In Figure 228, for example, the velocities are higher at the

barrier than the without project, and then moving away from the barriers spatially, the velocity starts to match the without project velocities. The tidal percentile analysis represents how much water is moving into or out of a certain area. In theory, it's expected that the areas impounded by the barriers have a slightly lower tidal prism than the without project assuming a reduction in cross-sectional area due to the proposed structure, and this does end up being the case here for Set 2.

4 Conclusions

Two sets of barrier alignments were simulated using AdH with each set consisting of four alternatives. The model output from the first set of alignment simulations along with ship sizes/frequency/transit path information were utilized to determine improved structure configurations in terms of gate sizes, locations, and elevations for the second set of simulations. The analysis and model results of both sets includes point velocity and water surface elevation model comparisons, velocity percentiles, spatial figures of the 50th and 75th percentile velocities and salinity, and tidal prism percentiles for the areas affected by the structures. For both Set 1 and 2, the barriers had similar effects on the water surface elevation. There were minor impacts in Alternative 2 for Set 1 and 2. For Alternative 3a there were negligible impacts in the non-impounded areas and noticeable impacts in the areas impounded by the proposed structures for both sets. Alternatives 3b and 4 had negligible impacts on tidal range for both sets as well for the analyzed locations. Set 1 and Set 2 did have a similar effect on the velocity. In general, the velocity increased at or near any of the structures, but quickly returns to the same velocity time series values as the without project as you move further away from the structures. The tidal prism percentile analysis do show the barriers slightly constrict flow.

There are some limitations to this study that need to be considered. The original mesh used and modified to include the barriers for this study was used to investigate the sediment transport behavior in the harbor and as such was not focused on resolving some peripheral areas. For example, the representation of the New Jersey Meadowlands marsh areas and associated schematization were an engineering decision made to reduce the computational burden that was appropriate for the sedimentation study but may not be for evaluating tidal exchange associated with a structure on the Hackensack River. Further investigation is needed in order to correctly predict tidal exchange in the peripheral extents of the mesh like the Hackensack River. The bathymetry utilized for the base mesh was also representative of the depths in the bay in 2004 with the exception of the harbor deepening in the channels which were incorporated into the mesh to quantify the impact of the harbor deepening on the sedimentation.

5 References

McAlpin, Tate O, Joseph V Letter, Mary Bryant, Gary L Brown, Gaurav Savant, Bryce W Wisemiller, Jamal A Sulayman, Corey J Trahan, and Anthony G Emiren. 2017. "New York/New Jersey Harbor Sedimentation Study: Numerical Modeling of Hydrodynamics and Sediment Transport." Technical Report.

6 Appendix A – Profile View of structures for Set 1

Figure 270: Set 1 Sandy Hook to Breezy Point Cross Section

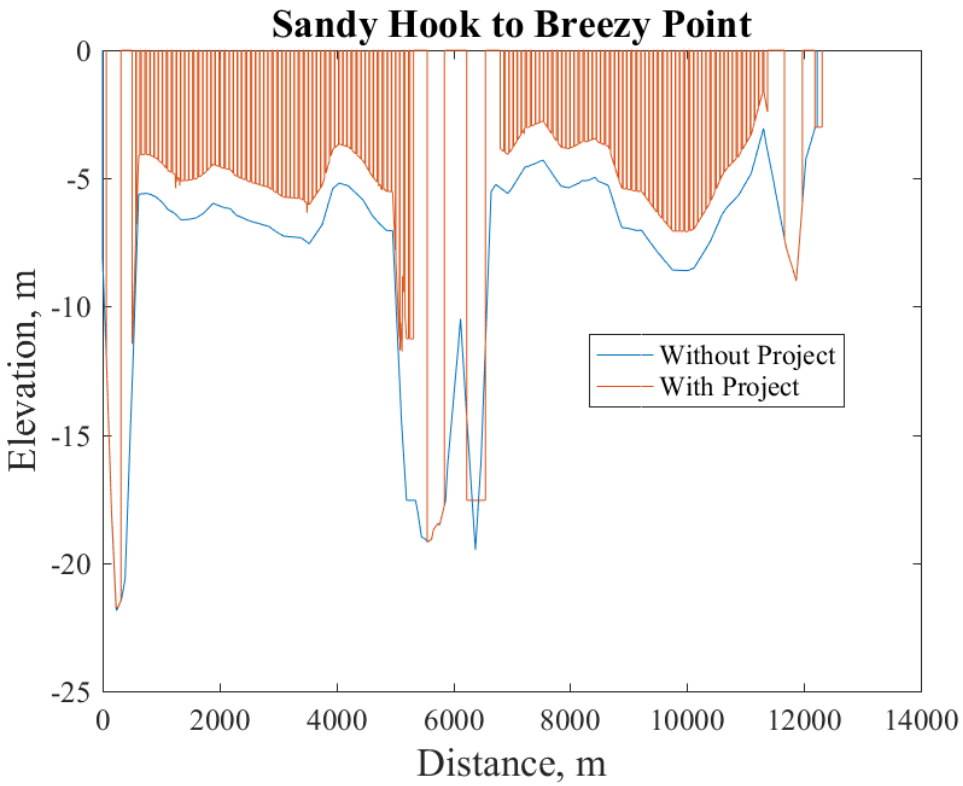


Figure 271: Set 1 Throgs Neck Cross Section

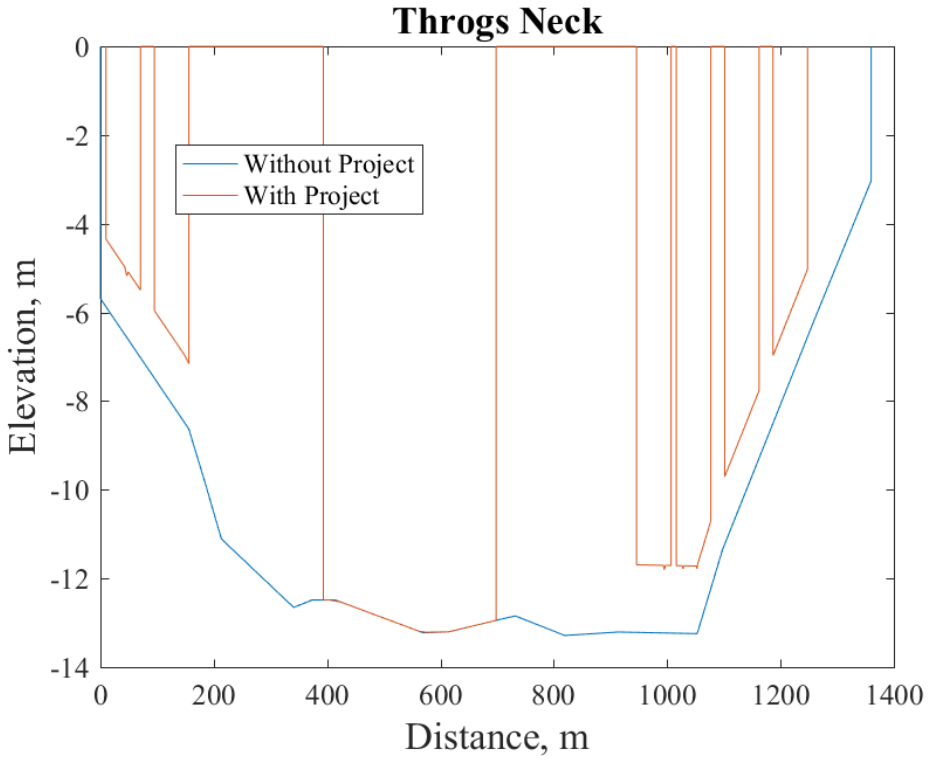


Figure 272: Set 1 Arthur Kill Cross Section

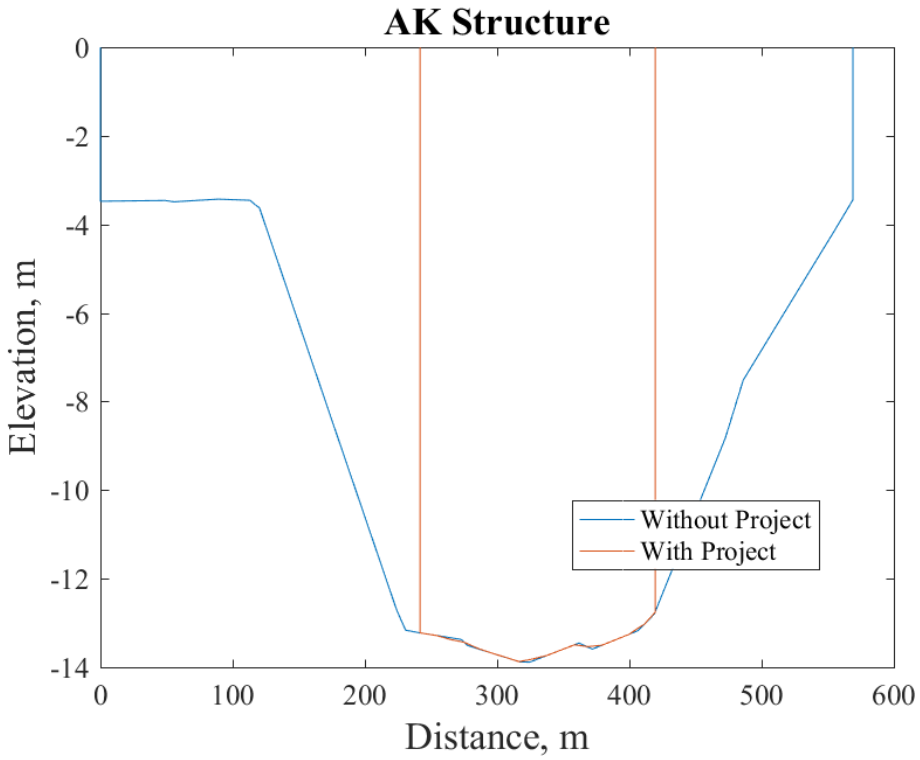


Figure 273: Set 1 Jamaica Bay Cross Section

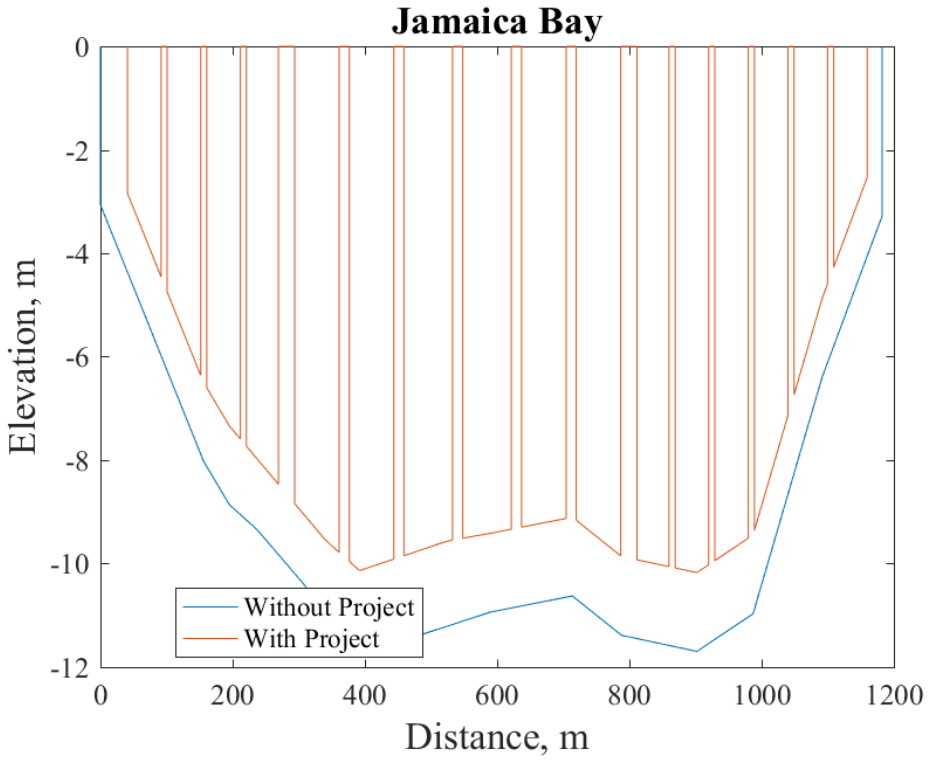


Figure 274: Set 1 Narrows Cross Section

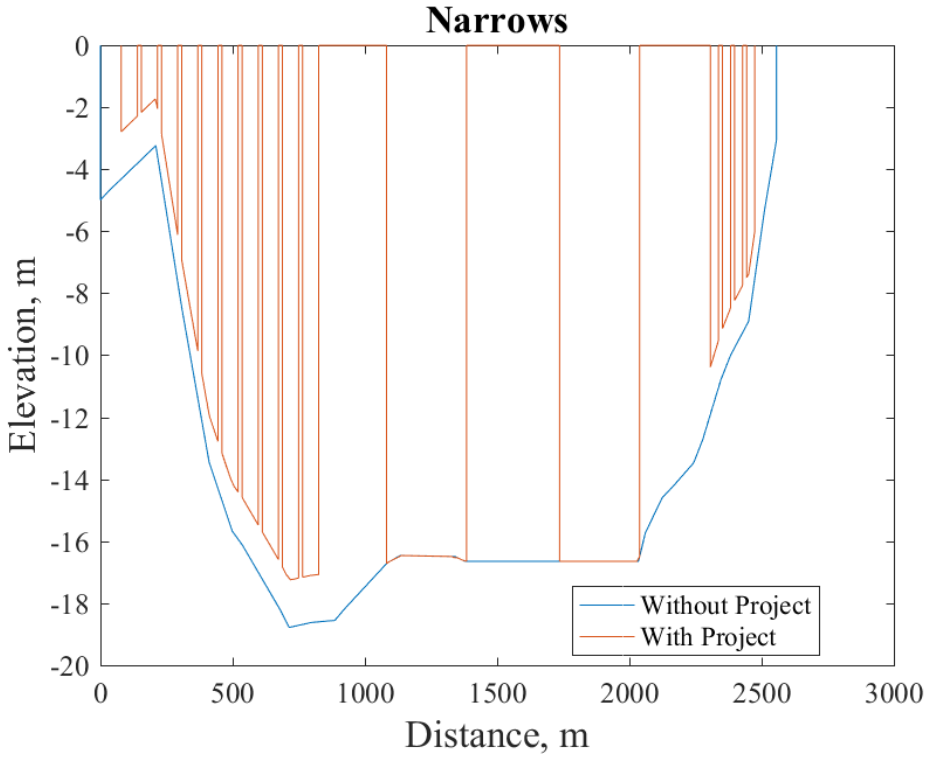


Figure 275: Set 1 Kill van Kull Cross Section

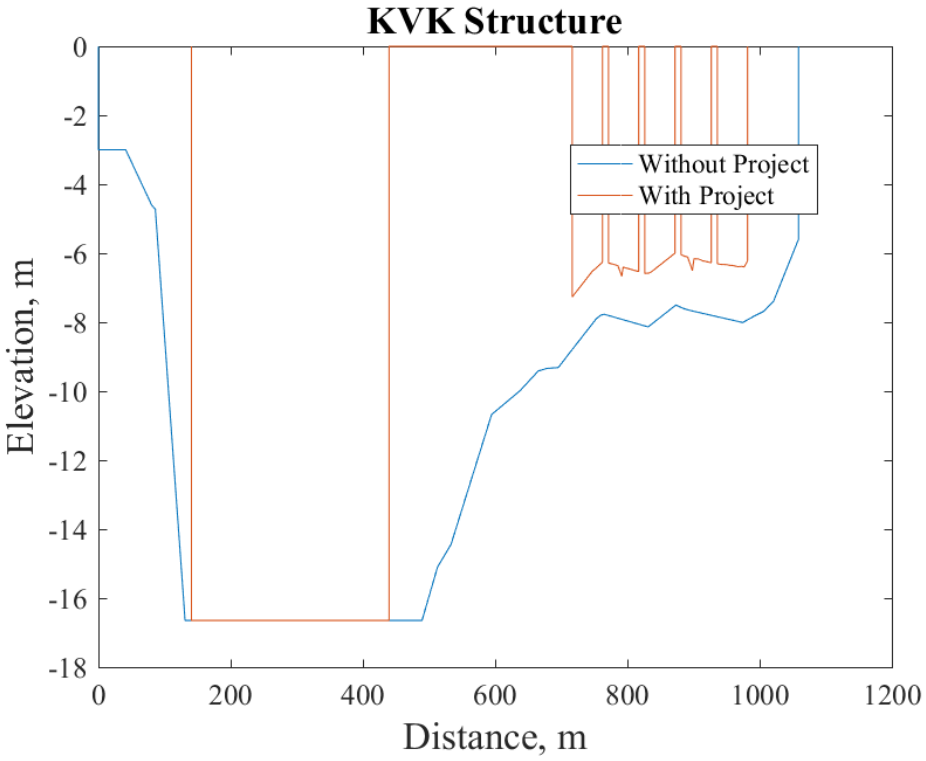
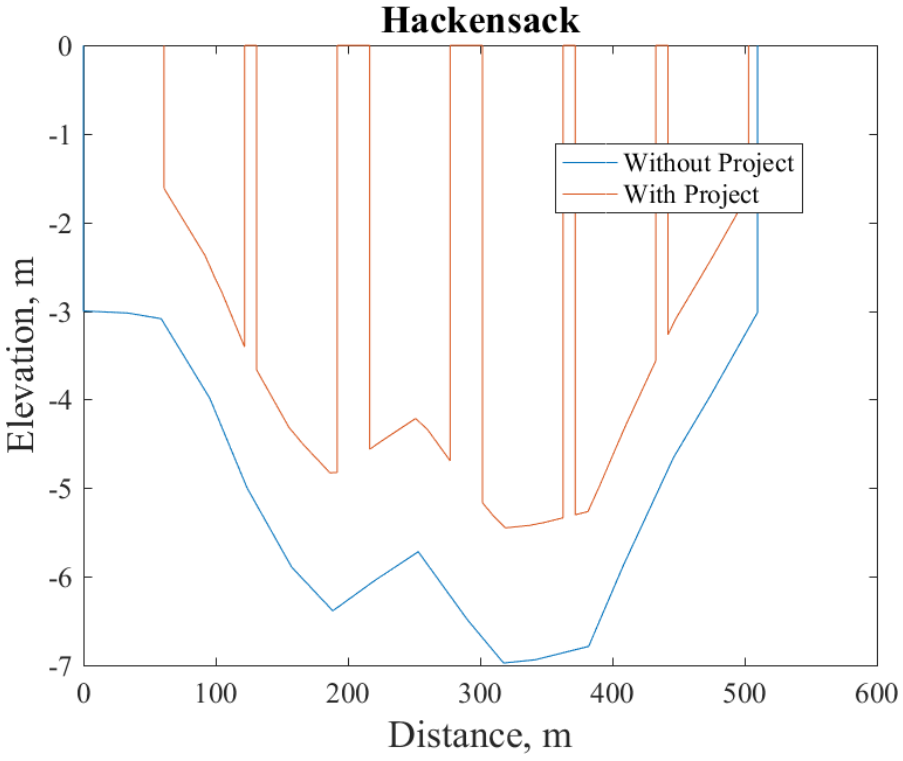
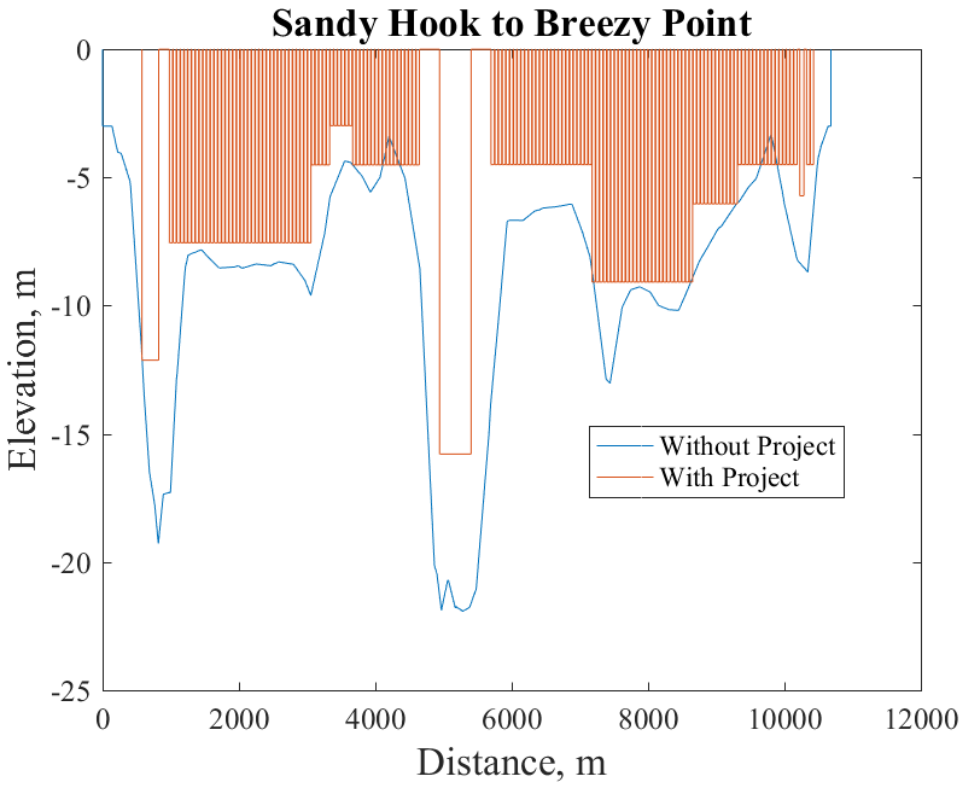
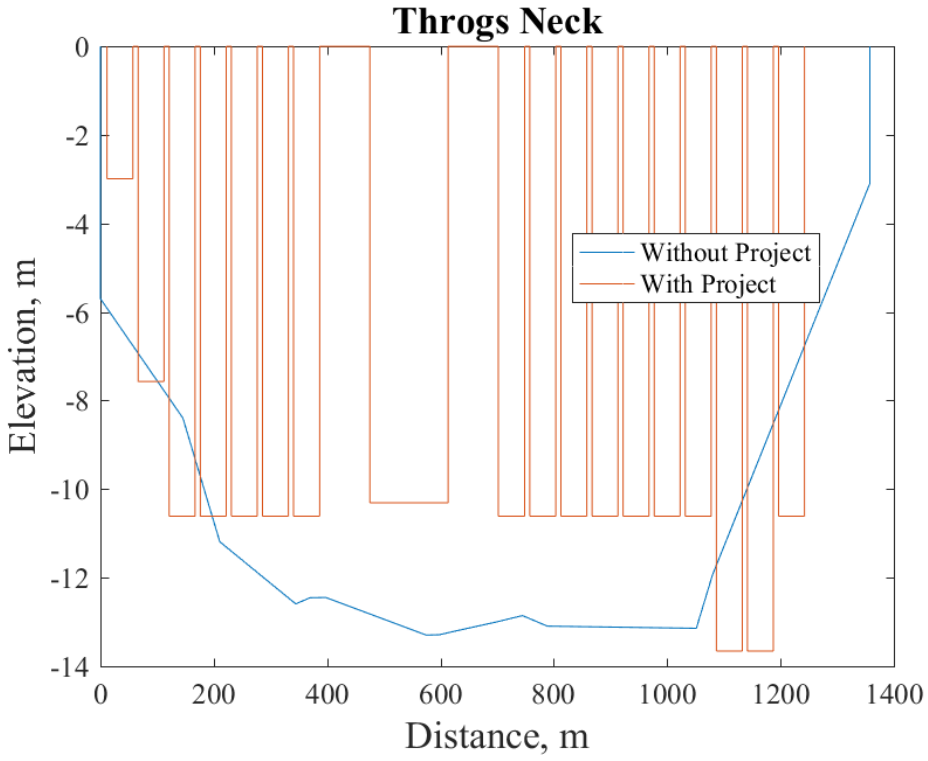


Figure 276: Set 1 Hackensack Cross Section



7 Appendix B – Profile View of structures for Set 2

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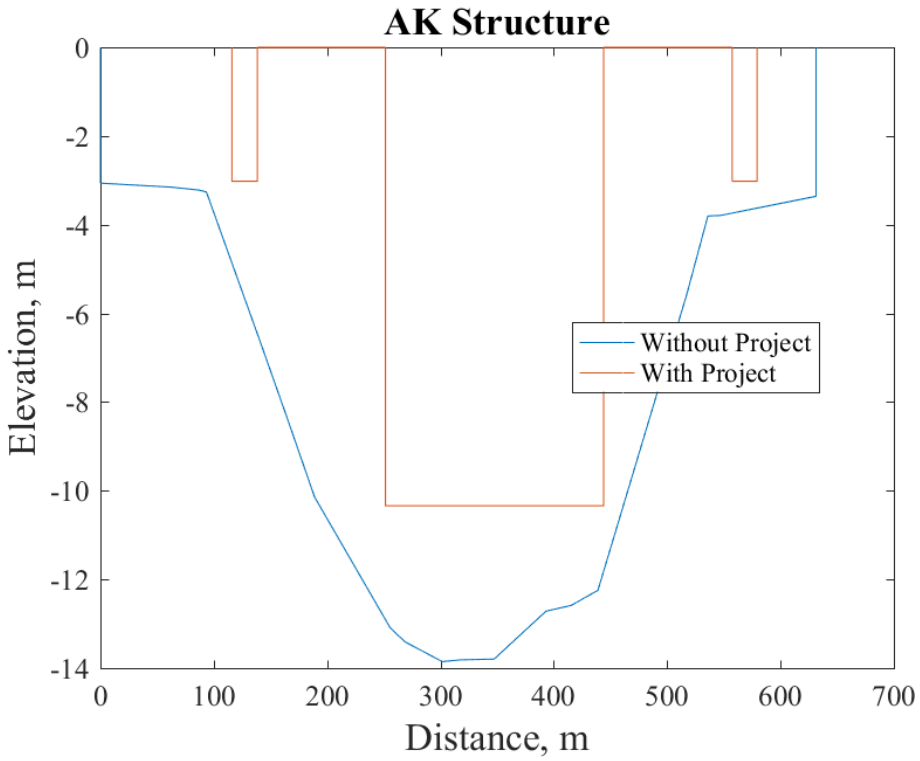
AK Structure
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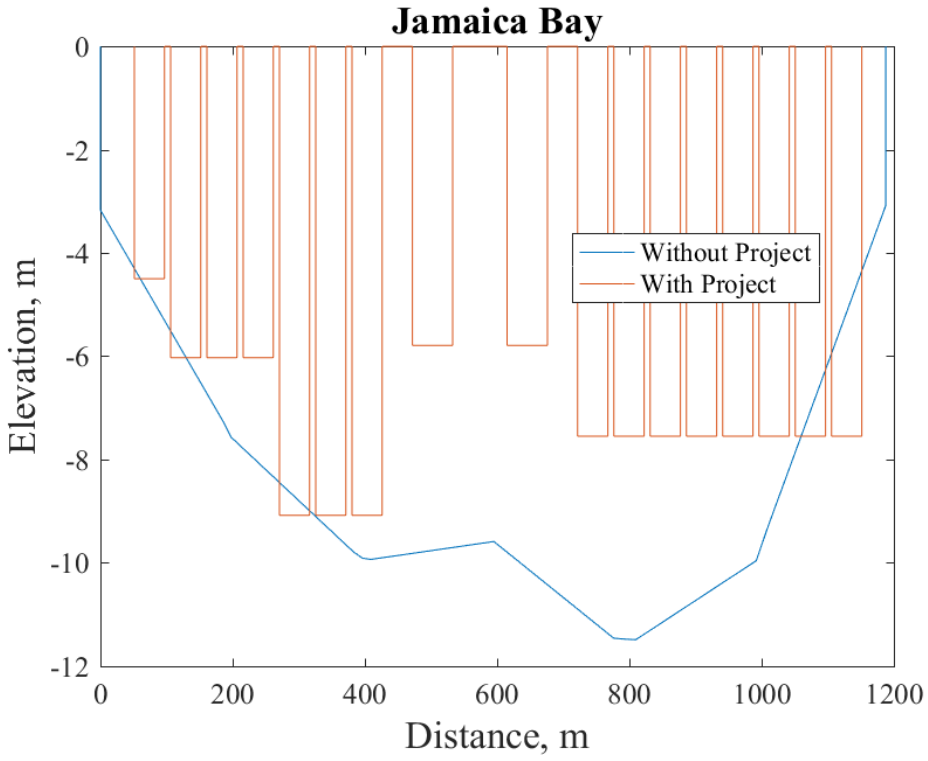
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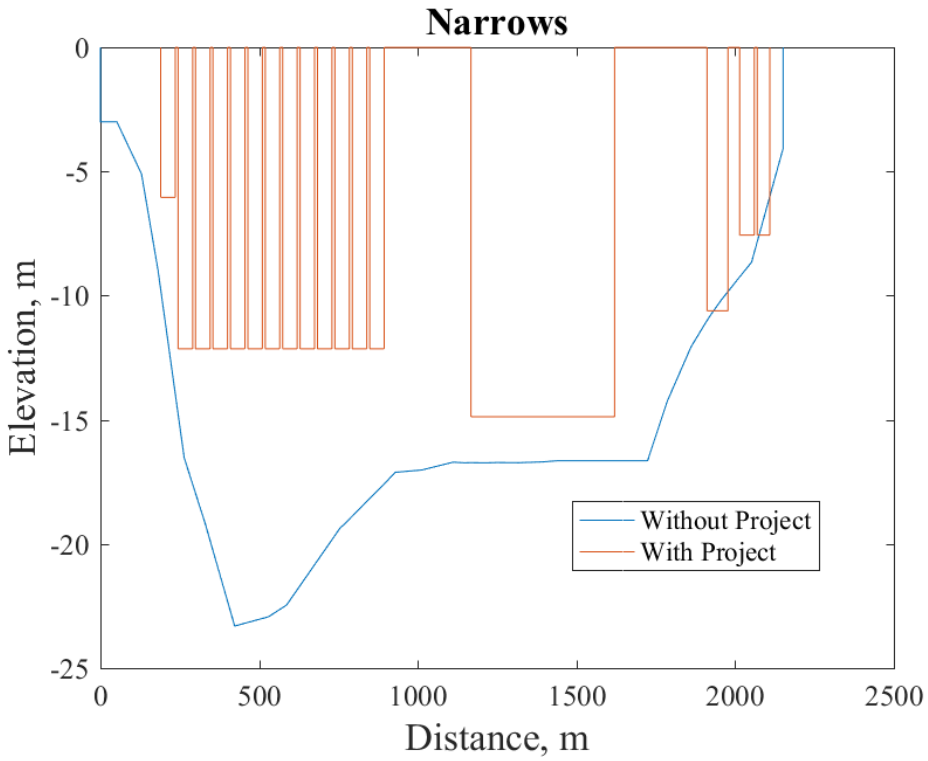
Narrows
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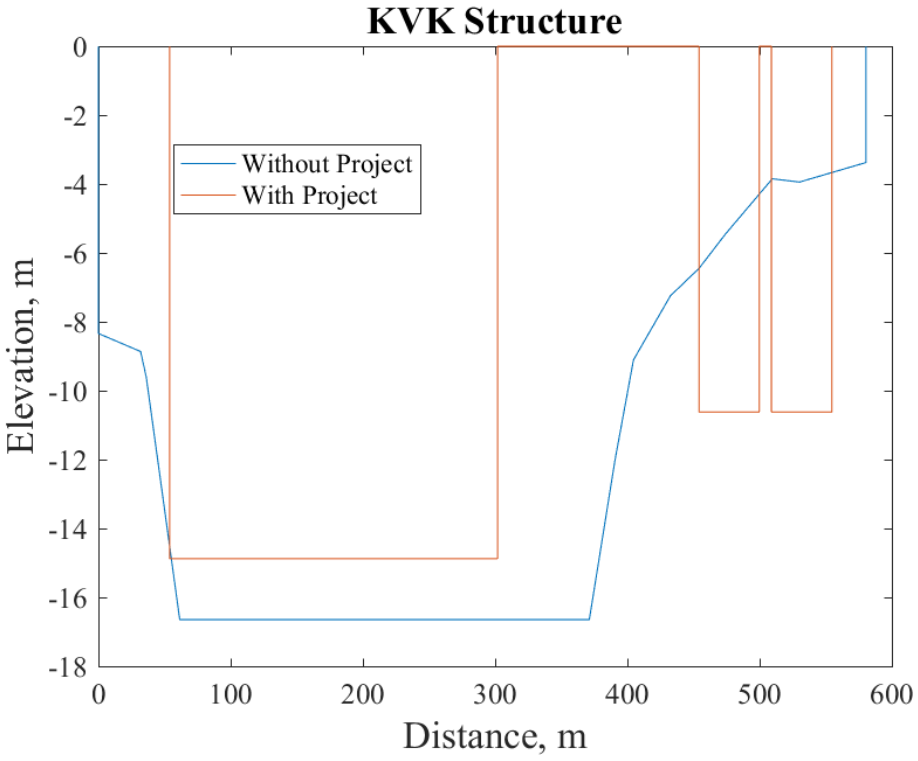
KVK Structure
Without Project
With Project
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Distance, m
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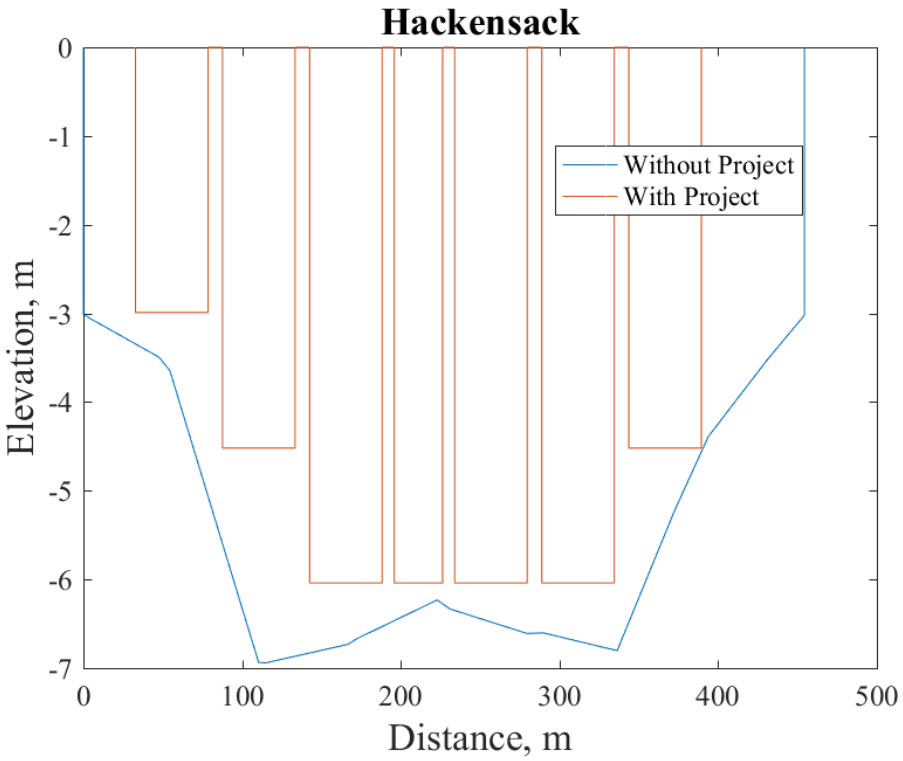
Hackensack
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Elevation, m
Distance, m
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JAMAICA BAY

STORM SURGE BARRIER WATER QUALITY MODELING ANALYSIS

Prepared for the

New York City

Department of Environmental Protection

and the

US Army Corps of Engineers

New York District

Revised

June 2018

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Figure 37. Time-Series Results for Grassy Bay, for the 1-in-25 Year, Eastern Barrier 96 Hour Closure Scenario. 34

EXECUTIVE SUMMARY

When Hurricane Sandy hit New York and New Jersey it brought incredible storm surges and severely damaged areas along the coastal areas. Particularly hard hit by the storm surge and resulting flooding were areas in the Rockaways and the Broad Channel communities. In response to this event and to help prevent such flooding in the future, the USACE is considering the construction of a storm surge barrier to be located near the Rockaway Inlet to Jamaica Bay. This barrier could be closed during a storm event to prevent high tides associated with a storm surge from entering Jamaica Bay. In order to assess the potential impact of a barrier closure on water quality within the Bay, a modeling study was conducted using the Jamaica Bay Eutrophication Modeling system, known as JEM.

JEM was developed for the New York City Department of Environmental Protection (NYCDEP) in the early 2000's to help support the development of a nitrogen control plan for Jamaica Bay. JEM is comprised of a coupled hydrodynamic model and a water quality model, which is capable of simulating eutrophication (nutrients, phytoplankton biomass and dissolved oxygen) and pathogenic bacteria. The original JEM model has undergone several revisions in recent years improving its spatial resolution and adding functionality that allowed the model to forecast nutrient impacts on the Bay's biological communities, including bivalves (*Mercenaria mercenaria*), macroalgae (*Ulva*) and benthic algae. Also available for use with the JEM modeling system is a watershed or sewershed model, which relates rainfall that falls over the upland drainage basin to determine the pollutant loadings of nutrients and pathogens delivered to the Bay via combined sewer overflows (CSOs), separate sewer overflows (SWOs) and direct runoff to the Bay.

As part of this analysis, the sewershed model was run to generate loadings associated with a 1-in-10 year (or a 0.1 annual exceedance probability (AEP)) and a 1-in-25 year (or a 0.04 AEP) rainfall event. It should be noted that damage to the coastal communities during the Sandy event were associated with storm surge as opposed to rainfall. Therefore, larger rainfall events (e.g. 1-in-50 years or 1-in-100 years) were not considered. Analysis of long-term rainfall records for the New York area was performed to determine the magnitude of the 1-in-10 and 1-in-25 year events. Two actual observed rainfall events that were closest in magnitude to the 1-in-10 and 1-in-25 year events were selected. These events were then inserted into the 2008 rainfall record (based on the JFK airport rain gauge) in late August to provide a time history of rainfall to be used as input to the sewershed model to generate a time history of associated runoff volumes and pollutant loading to be used in the JEM modeling system. Late August was selected as the time to evaluate water quality response to a potential storm event and storm surge barrier closure because this is a period of time, when bottom water dissolved oxygen (DO) within the Bay tends to be at its minimum values and perhaps might be most susceptible to adverse impacts from a storm event and storm surge barrier closure. The intent was to model the impacts of a worst case scenario on water quality and DO, in particular, in order to establish a range of potential impacts resulting from the construction and operation of the proposed storm surge barrier.

The JEM model was then used to assess a series of model runs that investigated the water quality impacts from a potential storm event and barrier closure. Three conditions were considered:

(1) a base case, i.e., existing conditions with no storm surge barrier in place,
(2) the presence of a storm surge barrier, but with the storm surge barrier always open, and
(3) the presence of a storm surge barrier but with the barrier being closed during the late August storm events.

The analysis included closures of 48 hour and 96 hour durations and the 1-in-10 and 1-in-25 year rainfall events. It was believed that a closure of 48 hours is perhaps the most reasonable closure period, but the analysis was extended to include a 96 hour closure, which was considered to be very conservative. The 96 hour closure was in response to a consideration that it might not be possible to re-open the storm surge barrier gates, perhaps in response to a power outage, if a separate power source was not considered in the design of the storm surge barrier system. These conditions were again chosen in order to establish a range of potential impacts, with the barrier that is never closed being the "best case" range for impacts from an operational standpoint, and the August closure when existing DO conditions are the worst, being the "worst case" scenario. When analyzing duration of closure, again the choice of 48 hours and 96 hours was intended to capture a range of impacts for closure, with 96 hours being the worst case.

The modeling analysis considered both long-term and short-term effects. The long-term effects analysis was meant to evaluate the potential impact of the storm surge barrier system itself, i.e., the physical structure, on water quality. In this case, the analysis compared water quality projections with the barrier in place, but with the gates of the storm surge barrier always in the open position, versus existing conditions, i.e., no storm surge barrier. Model results indicate that the long-term effects include a slight reduction in the tidal range within the bay. Changes to salinity, pathogens, nutrients, chlorophyll, and DO concentrations were generally smaller than could be accurately measured using analytical laboratories. These changes are unlikely to have any impact on aquatic habitat within Jamaica Bay.

The short-term impacts of closing the storm surge barrier were shown to have minimum effects on levels of pathogenic bacteria in the open waters of the Bay and that the attainment of associated standards would be met. However, calculated results for barrier closure scenarios were shown to have a potentially greater impact on aquatic habitat, especially related to bottom water DO concentrations. Concentrations of bottom water DO are reduced to between 1.7 and 2.0 mg/L in the open waters of the Bay for the 1-in-25 year storm event for a 96 hour closure. This represents a reduction of between 0.5 and up to 4 mg/L at worst relative to baseline conditions. It is important to note, however, that the JEM consists of 10 vertical layers and the non-attainment is typically limited to the bottom one or two layers of the water column, which represent between 3 and 8 percent of the total vertical water column. Reductions in DO in other upper portions of the water column are much less severe, on the order of a few tenths of a mg/L on average and resulting levels are capable of sustaining aquatic life. In order to minimize the impact of gate closures on DO concentrations it is recommended to minimize the time the storm surge barrier gates are closed. Mobile aquatic life could potentially escape the low bottom DO concentrations, since DO levels in the mid-depth and surface waters of the Bay are capable of supporting pelagic life. However, benthic organisms would likely not be able to avoid low DO and would be subject to detrimental impacts. However, it should be noted that the effects of winds on surface DO re-aeration and vertical mixing, and sediment resuspension were not considered. These factors can potentially impact DO and chlorophyll concentrations and should be incorporated into future phases of the modeling.

Other notable water quality impacts were increases in total nitrogen of between 0.1 and 0.9 mg N/L, depending on location in the Bay and proximity to WWTP discharges (between 10 and 50% increases from baseline), but these increases are not anticipated to detrimentally impact water quality or result in additional phytoplankton growth. This, in part, is due to the fact that current nutrients concentrations are in excess of levels that would limit phytoplankton growth. Model computations also showed

changes in salinity with the top layer typically decreasing by about 1-3 ppt up to almost 9 ppt in a few open water cells and in the CSO tributaries. Salinity, on the seaward side of the storm surge barrier, increased by about 2 ppt. The changes in the water quality parameters were indeed short term, with concentrations of the various water quality parameters of interest returning to near baseline conditions within a few days to a week after the storm surge barrier gates were re-opened.

Summary Conclusion

Preliminary results of the modeling, as presented in this report, indicate the installation and operation of the storm surge barrier could conceivably impact the water quality (e.g., bottom water dissolved oxygen) and habitat in the interior tidal tributaries and shallow areas of the Bay. Consequently, additional model refinement and analysis should be conducted by the USACE prior to construction of the proposed storm surge barrier across Jamaica Bay to better quantify and conclusively address any possible impacts of a storm surge barrier on water quality and fish and benthic species and their habitats in the Bay. Design considerations for the proposed storm surge barrier could include measures to mitigate for potential water quality impacts, such as backup mechanisms to ensure timely re-opening of the barrier after a storm in the event of power failure.

INTRODUCTION

The U.S. Army Corps of Engineers is considering the construction of a storm surge barrier across the width of the Rockaway Inlet in New York City to protect Brooklyn and Queens from tidal surges associated with events such as Hurricane Sandy. To support the evaluation of the storm surge barrier, a modeling study was conducted to assess the impact of the storm surge barrier on water quality under varying rainfall and barrier operating conditions. The hydrodynamic analysis focused on potential changes to the tide range and salinity concentrations. The water quality analyses focused on compliance with existing New York State Department of Environmental Conservation (NYSDEC) pathogen (fecal coliform and enterococcus) and DO water quality standards, as well as other constituents such as nutrients and chlorophyll.

MODELING CONDITIONS

The modeling baseline condition was the year 2008 with rainfall for that year based on data collected at the John F. Kennedy International Airport. The rainfall for this period is considered a “typical” annual rainfall with near average annual rainfall, number of storms and rainfall intensity (HHS, 2011). The model projections considered two storm surge barrier gate structures: an Eastern Option located just east of the Marine Parkway Bridge stretching from Barren Island to Jacob Riis Park; and a Western Option stretching from Manhattan Beach to Rockaway Point. The structures are presented in Figure 1. The portions represented by a line are solid walls that prevent the passage of flow at all times. The ovals represent locations in the structure that can be opened and closed, with the larger openings used for navigation.

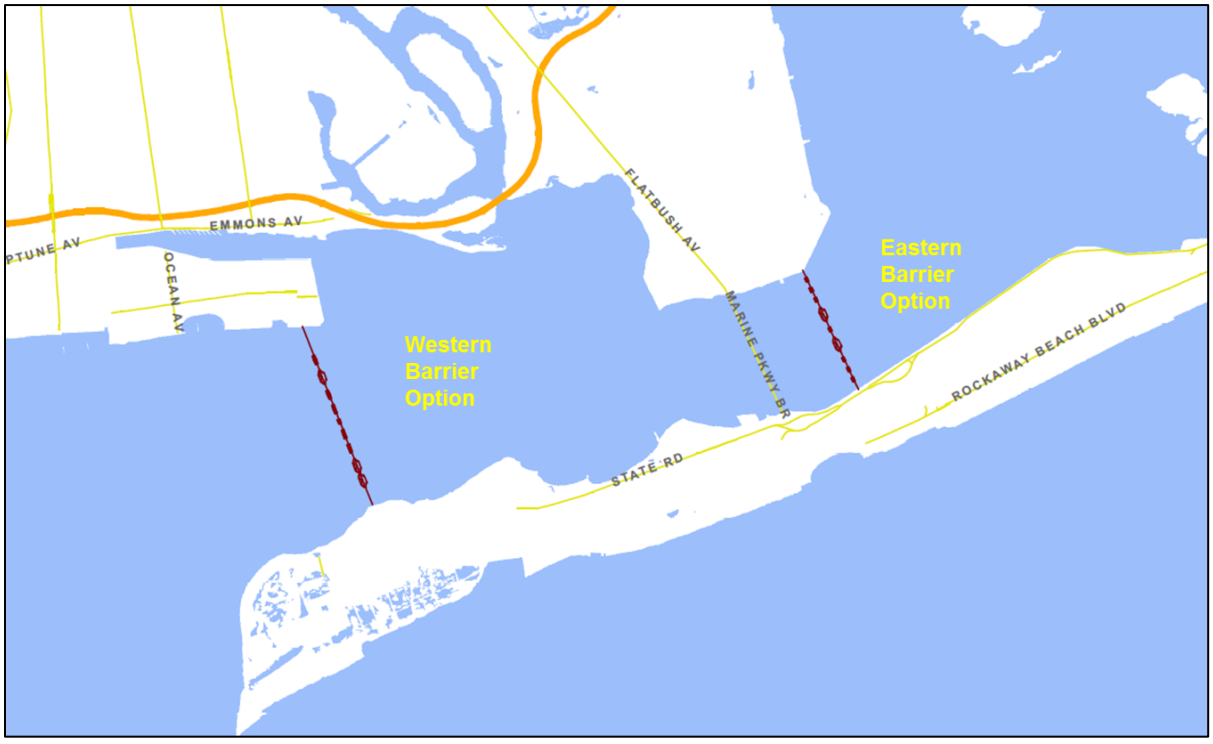


Figure 1. Location of Western Storm Surge Barrier (left) and Eastern Storm Surge Barrier (right).

In addition to the baseline rainfall conditions, two storm conditions were added to the rainfall pattern: a 1-in-10 year (or a 0.1 AEP) rainfall based on a 24 hour period, and a 1-in-25 year (or a 0.04 AEP) rainfall based on a 24 hour period. The 1-in-10 year rainfall event contained 5.78 inches of rainfall and was based on a storm that occurred at LaGuardia Airport (LGA) on August 27, 2011. The 1-in-25 year rainfall event contained 6.60 inches of rainfall and was based on a storm that occurred at LGA on August 14, 2011. Figure 2 presents the rainfall conditions. It should be noted that the 1-in-10 year event had the highest intensity during the middle of the storm, while the 1-in-25 year storm had the highest intensity during the beginning of the storm. These rainfall patterns result in timing differences in the model response to the two different storms. For both the 1-in-10 year and 1-in-25 year conditions the storm rainfall was added to the base 2008 rainfall condition on August 26th.

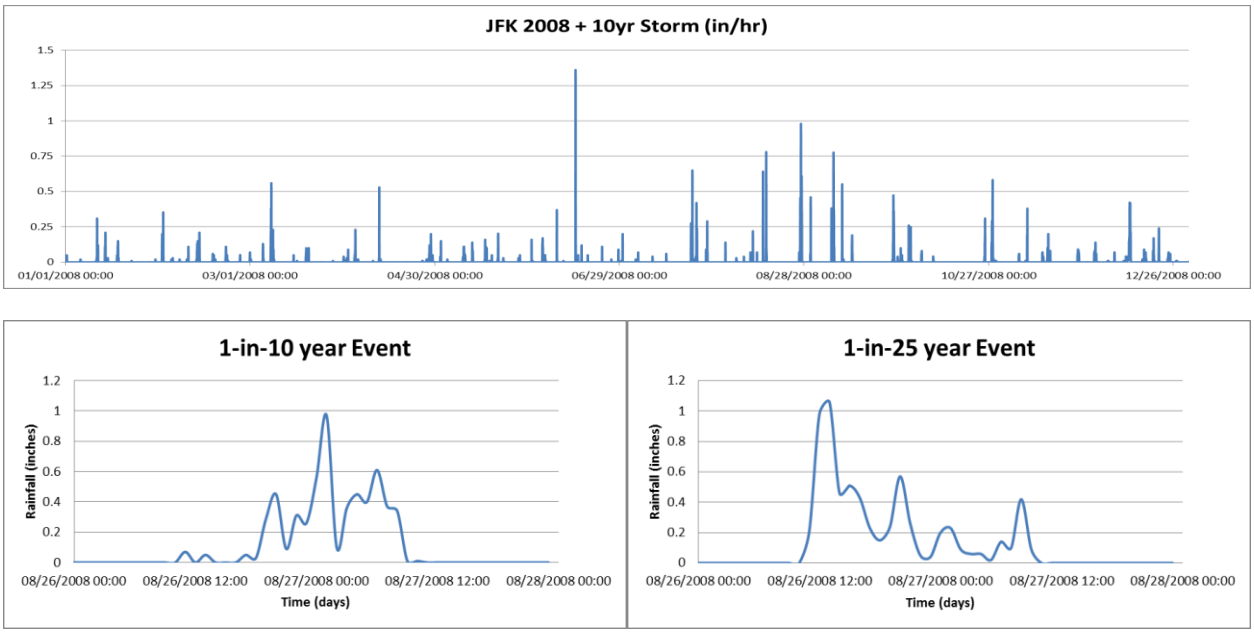


Figure 2. Model Rainfall Conditions.

Modeling was conducted based on four conditions (or scenarios) related to the storm surge barrier:

(1) a no action (or no barrier) scenario,
(2) a least impact scenario - a barrier with its gates always open,
(3) a most likely scenario - 48 hour gate closure, and
(4) a worst case scenario - 96 hour gate closure.

The 48 hour closure was assumed to represent a more typical operating condition, as elevated water levels associated with large storms do not generally last more than a day. The 96 hour gate closure was associated with a conservative length of time that might be associated with longer storm surge events or a power outage, or other factors that might disrupt the ability to open the gates. In addition to modeling a range of duration closures, Table 1 summarizes the range of impact based on differing

rainfall events, varying in timing and intensity and presents a listing of the 14 model scenarios evaluated.

Table 1. Modeling Scenarios

No Barrier	Eastern Barrier	Western Barrier
1-in-10 year storm	Always open – 1-in-10 year	Always open – 1-in-10 year
1-in-25 year storm	48 hour closure – 1-in-10 year	48 hour closure – 1-in-10 year
	96 hour closure – 1-in-10 year	96 hour closure – 1-in-10 year
	Always open – 1-in-25 year	Always open – 1-in-25 year
	48 hour closure – 1-in-25 year	48 hour closure – 1-in-25 year
	96 hour closure – 1-in-25 year	96 hour closure – 1-in-25 year

The modeling analysis was based primarily on the additional rainfall from an extreme rainfall event. In addition, increased cloud cover was assumed during the previously sunny days of August 26th and August 27th resulting in a reduction of solar radiation by 30 percent to simulate the possible conditions associated with large storm systems. The additional cloud cover would tend to reduce algal primary production on these two days of increased cloud cover. The model analysis did not, however, consider higher winds that might be associated with hurricanes or other large storms. Typically, these increased winds would result in increased vertical and horizontal mixing and increased waves. These winds would also likely increase atmospheric reaeration (i.e., the exchange of DO between the atmosphere and the surface waters of the Bay), and possibly increase sediment resuspension. The latter could result in reduced water clarity that might reduce phytoplankton growth. Increased vertical mixing might increase the exchange of surface water DO with oxygen deficient bottom waters, thus reducing the projected hypoxic conditions.

THE JEM MODEL

The model used for this analysis was the Jamaica Bay Eutrophication Model (JEM). The JEM modeling system actually includes several models applied in the analysis. The first component of the JEM modeling system is a landside or sewershed model of the upland drainage basin. The second component is stand-alone hydrodynamic model, using the Estuarine Coastal and Ocean Model – Sediment Transport framework (ECOMSED), which provides water movement, as well as temperature and salinity information, to the water quality models. The two other components include separate water quality models: a pathogen model applied for fecal coliform and enterococcus; and a geochemical or eutrophication model applied for nutrients, chlorophyll-a, and DO. The water quality models use the Row-Column AESOP (RCA) model framework.

RCA was developed by HydroQual Inc. in the late-1980s and is the most recent extension of the family of water quality models that originated as the Water Quality Analysis Simulation Program (WASP) used by the United States Environmental Protection Agency (Di Toro et al., 1983). The eutrophication form of RCA has been applied to a number of coastal and estuarine systems (Isleib et al., 2007; Testa et al., 2014; Xue et al., 2014; Zhang & Li, 2010). Included in the standard eutrophication form of RCA is a biogeochemical sediment nutrient flux submodel, known as SFM. Initially developed for the US Army Corps of Engineers and the USEPA Chesapeake Bay Program by HydroQual, Inc. in the early

1990's (Di Toro & Fitzpatrick, 1993), it has been applied, together with the water column biogeochemical version of RCA in a number of coastal and estuarine systems (CEC, 2015; HydroQual, 1995, 1996, 2001; JEI, 2014). Details of the RCA biogeochemical may be found in HDR|HydroQual (2014) and Testa et al. (2014), while the details of the SFM may be found in (Brady et al., 2013; Di Toro, 2001; Testa et al., 2013). Figure 3 presents a conceptualization of the major state-variables and transformation processes included in RCA.

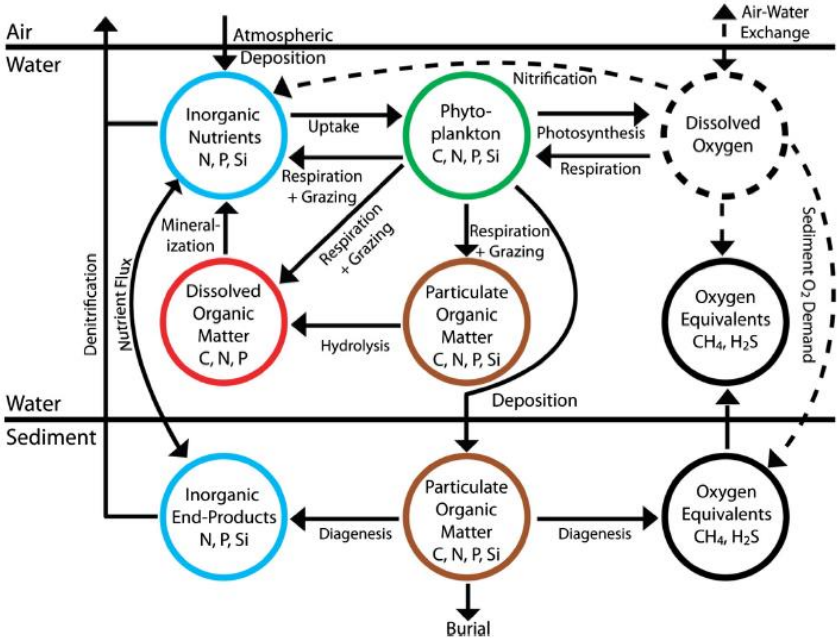


Figure 3. Diagram of the major state-variables and transformation processes in RCA (from Testa et al., 2014)

The original JEM model (model grid shown on Figure 4) was calibrated against an extensive field and laboratory program (Jamaica Eutrophication Study - JES) conducted between July 1995 and June 1996 and funded by the NYCDEP (field sampling locations also shown on Figure 4). Field sampling included measurements of temperature, salinity, tidal currents and water elevations, primary production/community respiration, *Spartina* biomass, *Ulva* biomass, benthic filter feeder biomass, benthic algal biomass, zooplankton biomass (EEA, 1996), and sediment chemistry and sediment nutrient flux and sediment oxygen demand (Cornwell and Owens, 1998). Additional monitoring and field efforts included sampling of the wastewater flows and effluent nutrient concentrations from the four NYCDEP waste water treatment plants (WWTPs) that discharge into Jamaica Bay and also included wet weather event sampling of a number of the combined sewer overflows (CSOs) and storm water overflows (SWOs) that discharge into the Bay.

The original JEM model was calibrated against the 1995/1996 JES data set (HydroQual, 2002) and was reviewed by an external Model Evaluation Group or MEG consisting of a scientist familiar with Jamaica Bay biology (Dr. David Franz, Brooklyn College), an estuarine hydrodynamicist (Dr. William Boicourt (University of Maryland), a marine ecologist (Dr. Jay Taft, Harvard University), an experienced water quality modeler (Dr. Mark Dortch, USACE Waterway Experiment Station) and a staff member for the NYS Department of Environmental Conservation (Mr. Philip O'Brien).

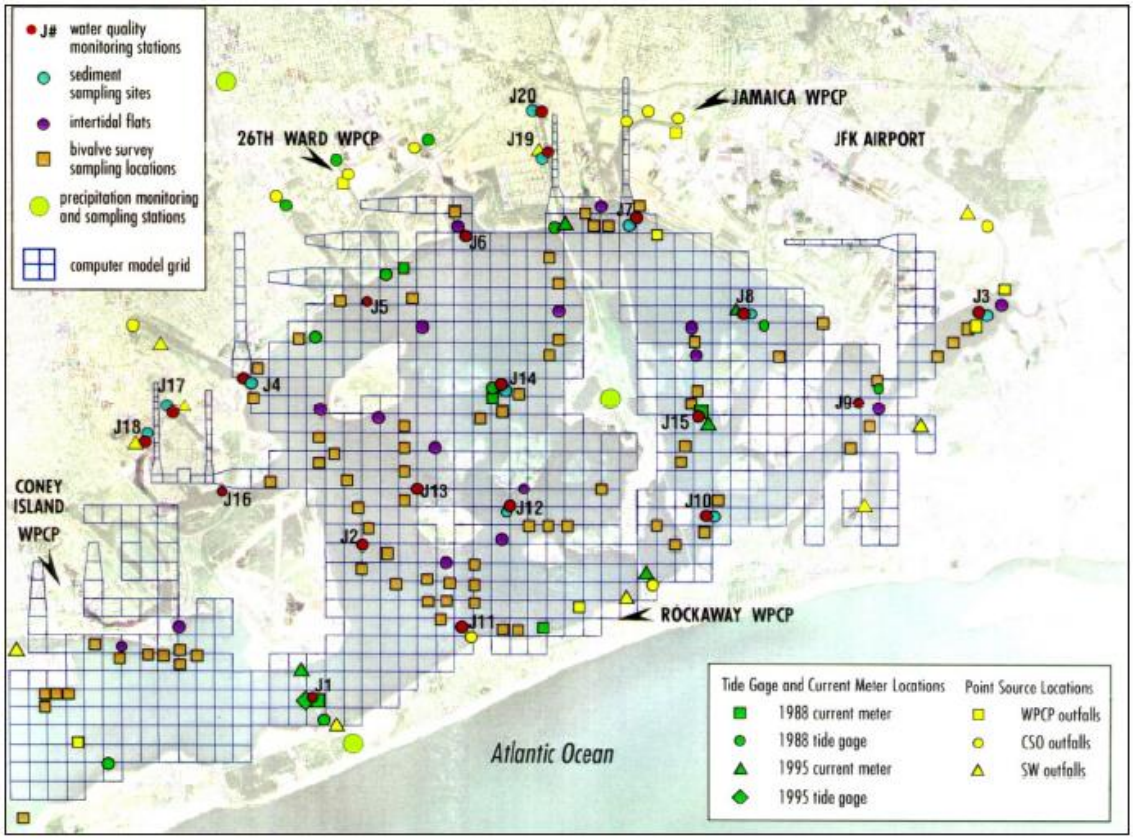


Figure 4. Original JEM Grid and WWTP Locations and 1988 and 1995/1996 Field Sampling Locations.

As part of recent project funded by the National Park Service and the Science and Resilience Institute at Jamaica Bay (Orton et al., 2017), the JEM was expanded both in terms of processes included in the eutrophication model and it's spatial resolution. The expanded eutrophication model includes the original water column eutrophication submodel, sediment nutrient flux submodel and suspension feeder submodel, and new macroalgae (*Ulva*) and benthic algae submodels (Figure 5). The revised model is known as JEM2016.

Figure 6 presents the model segmentation and bathymetry used in the JEM2016 model. The model segmentation applied for this analysis has approximately four times the number of grid cells as the original JEM. The model includes 10 vertical layers. The new segmentation has been applied for larval transport calculations and is being used as part of NYCDEP's CSO Long-Term Control Plan process. The eutrophication model has been re-calibrated to the 1995-1996 JES data and was also calibrated against 2015 NYCDEP Harbor Survey data (Orton et al., 2017). The Harbor Survey program monitors 13 stations in the open waters of the Bay and 11 tributary stations (Figure 6). The Harbor Survey program includes bi-weekly to monthly sampling of physical and chemical water quality parameters, including temperature, salinity, dissolved oxygen, phytoplankton biomass (as chlorophyll-a), and various forms of nutrients, but does not include components of the 1995/1996 JES study such as measurements of primary production *Ulva*, benthic algae, etc.

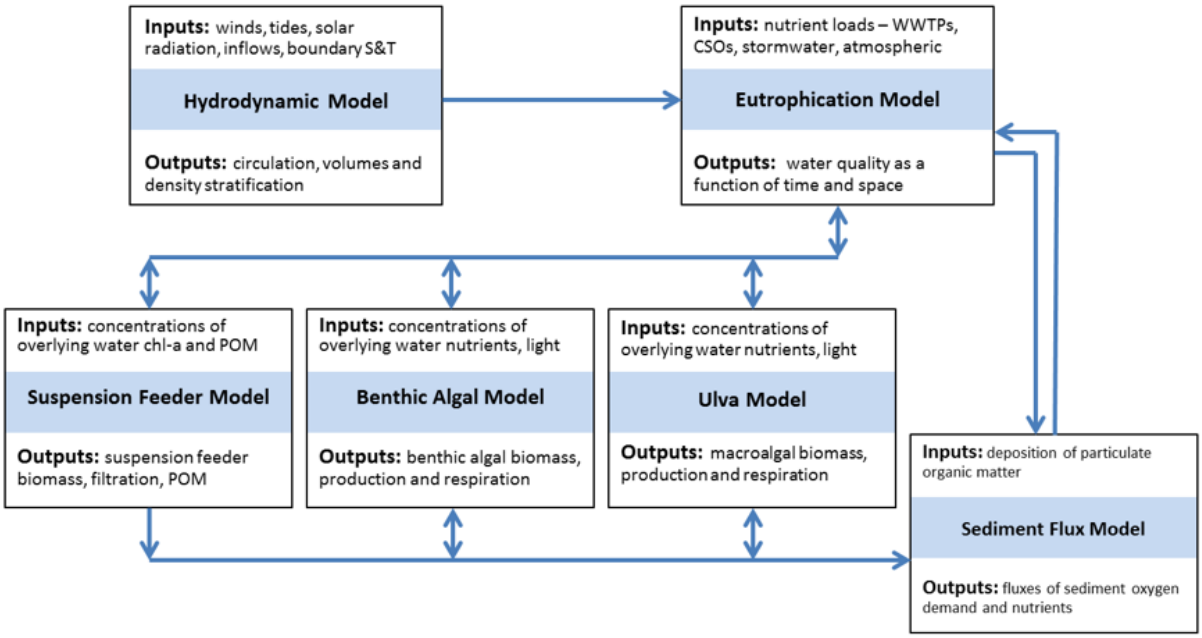


Figure 5. JEM2016 Hydrodynamic and Water Quality Model.

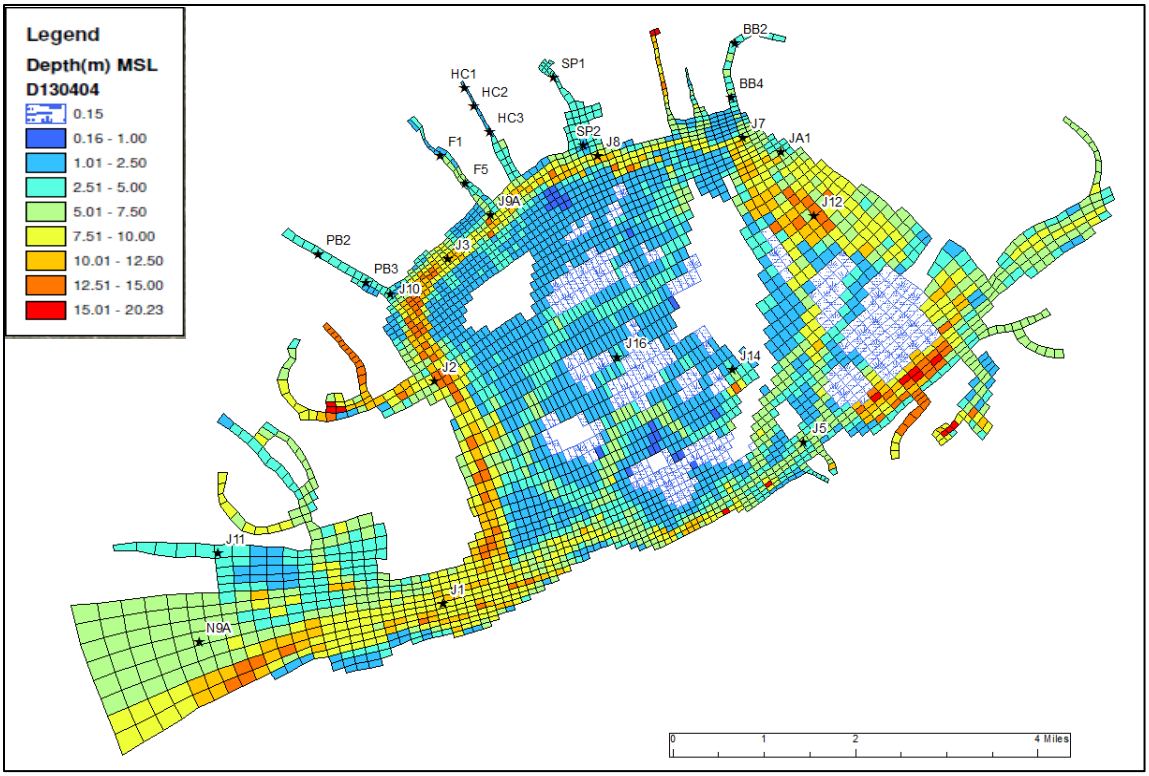


Figure 6. Jamaica Bay, New York, JEM2016 Model Segmentation, Bathymetry, and 2015 Harbor Survey station locations. (Each grid cell has ten vertical layers.)

METHODOLOGY

The modeling process began with the application of InfoWorks-based landside models for the five sewershed drainage areas (Coney Island, 26th Ward, Jamaica, Rockaway and a portion of Nassau County) that discharge into Jamaica and provided runoff volume estimates for CSOs, storm sewers and direct runoff to the Bay. Based on a recent analysis of rainfall records for the period 2002-2011 from JFK airport, which showed a 10-year average of about 46 inches/year, the NYCDEP has selected the year 2008, with a rainfall of 46.3 inches/year, as being a typical year (HHS, 2011) and is using this rainfall record for all sewershed and water quality modeling for the City's Long-Term Control Plan for CSOs. As a point of comparison, the 1955-2011 long-term average rainfall is about 41 inches/year. To be consistent with other NYCDEP modeling efforts, this study also used the 2008 rainfall record as input to the InfoWorks models. However, for this study, the 2008 rainfall conditions were supplemented with either the 1-in-10 year or 1-in-25 year storms.

For scenario runs with the storm surge gates closed, hydrodynamic model computations accounted for the changes in water level in the bay due to freshwater inputs. Changes in water levels in the bay affect when tide gates that cover CSOs or storm sewers, open or close and were included in the InfoWorks analysis. InfoWorks outputs were then used as input for the hydrodynamic model runs.

The model segmentation used by both the hydrodynamic and water quality models is not of a sufficient resolution to precisely reproduce the proposed storm surge barrier structures as shown in Figure 7. However, it is believed that the spatial resolution is sufficient for this screening level analysis. In order to reproduce the effect of the storm surge barrier under conditions with the storm surge barrier gates open, model segments were blocked off to leave only enough area to reproduce the open area of the storm surge barrier design, i.e., the choice of open gates versus closed support structures was chosen such that the cross-sectional areas of the open gates of the actual barrier design is approximately matched in the hydrodynamic model. In Figure 7, open gates are shown in light blue and the support structures that would not allow the passage of flow are shown in green.

InfoWorks also provided information regarding the fraction of CSO volume that contained sanitary versus stormwater volume in order to determine the pathogen concentrations for the CSOs. NYCDEP provided information concerning 2015 wastewater treatment plant (WWTP) flows and loading. The volumes of water from each discharge source (WWTPS, CSOs, separate storm sewers (SWOs), direct runoff to the bay and rainfall directly impinging onto the water surface of the Bay), during the storm surge barrier closure periods and for each storm event are shown in Figure 8. As can be seen, CSOs, storm water, and direct precipitation to the Bay dominate the water volume delivered to the Bay for these storms, with direct precipitation comprising 20 to 25 percent of the total volume.

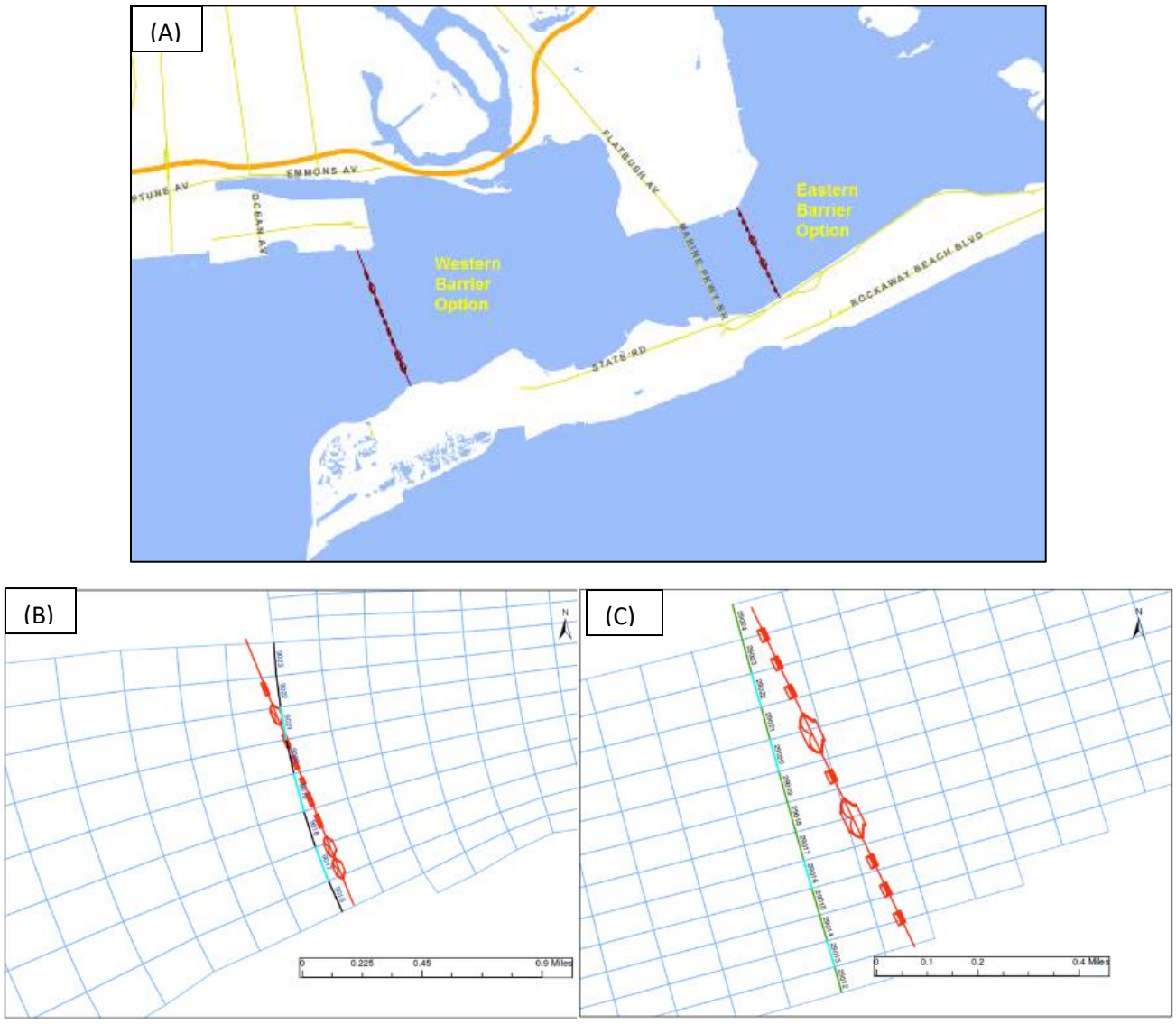


Figure 7. (A) Location Map for Western and Eastern Storm Surge Barrier Options and (B) Western Barrier Option and (C) Eastern Barrier Option in Comparison to the Model Segmentation.

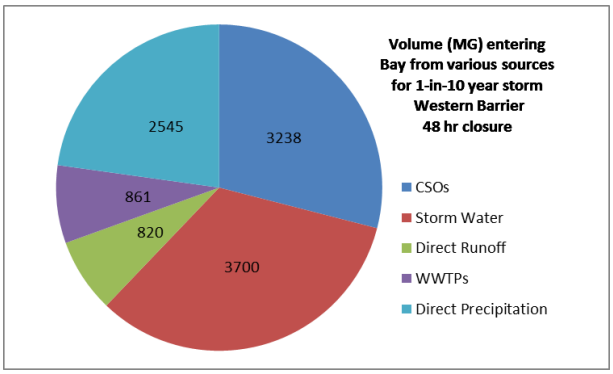
During the modeled storm events, the gates were closed for either 48 or 96 hours. The gate closure began during a low tide period before the storm. This allows for the largest storage capacity for storm related runoff and discharge from the four NYCDEP WWTPs within the bay behind the storm surge barrier. Figure 9 presents the behavior of water levels behind the Western Barrier during the 96 hour closure for the 1-in-10 year storm in the upper panel and the 96 hour closure, 1-in-25 year storm for the Eastern Barrier in the lower panel. Time in the figure is shown in Julian days or days from January 1st, 2008, the beginning time of the model simulations. The timing of the gate closure is shown as the vertical red line. The vertical blue line shows the timing of the beginning of the rainfall event. As the storm begins, the water level rises (note that the two storms vary in terms of the timing relative to one another - see Figure 2) and then gradually levels off as the precipitation ends and CSOs and SWOs finish discharging. The majority of the increase in water elevation is due to runoff from the surrounding watershed, as the watershed area is several times larger than the surface area of Jamaica Bay. The gates are then re-opened when the water levels inside the barrier and outside the barrier are similar. This approach would avoid high current velocities that could occur if the gates are opened with vastly different water elevations on each side of the barrier.

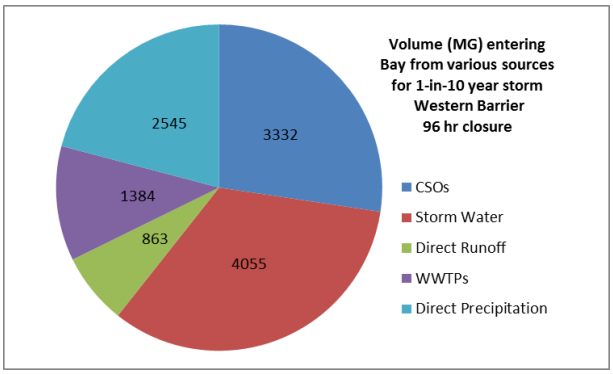
The results from the hydrodynamic model were then used as input for the water quality models to calculate pathogen, nutrient, chlorophyll and DO concentrations. Pollutant loads for the water quality model were developed by assigning pollutant concentrations to the CSO, SWO and WWTP flows calculated by the InfoWorks models. Nitrogen loads from the WWTP were based on the First Amended Nitrogen Consent Judgment concentrations for each of the WWTP.

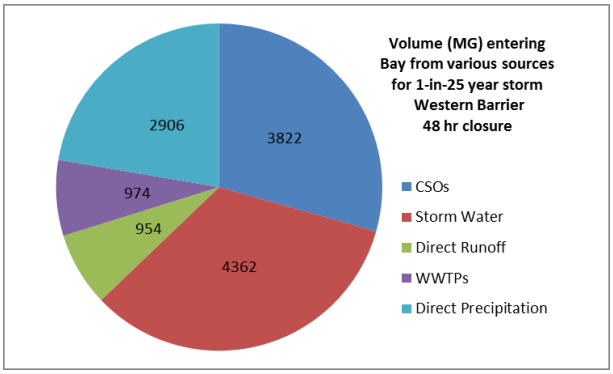
RESULTS

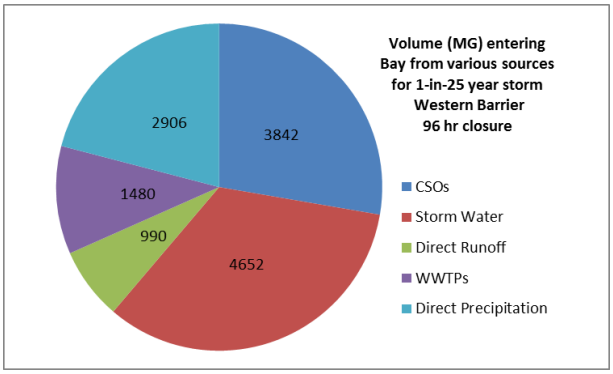
The effects of the storm surge barrier will be presented for both long-term and short-term responses. Under most circumstances, the storm surge barrier once constructed will have its gates open allowing tidal exchange between the nearby Atlantic Ocean and Jamaica Bay via the Rockaway Inlet. The structure will effectively reduce the cross-sectional area of the inlet, which has the potential to reduce the tidal range, the tidal prism and thus reduce overall flow velocities inside Jamaica Bay. While not addressed in this study, sediment transport patterns and estuary morphology could be potentially altered by channels filling in with sediment. Also, the increased local flow velocities at the barrier structures could result in increased sediment transport and deepening of flow channels through the gates. These impacts would likely be more chronic in nature.

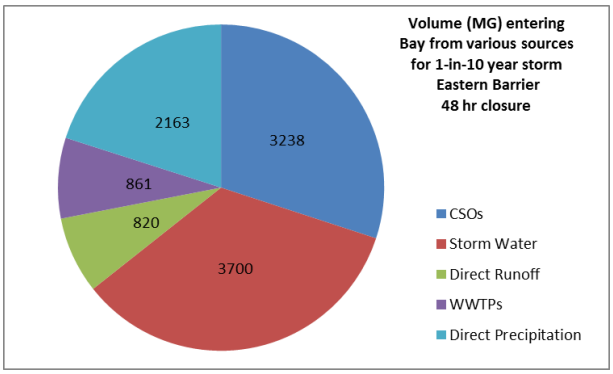
Short-term water quality effects occur when the storm surge gates are closed. Presumably the storm surge gates would be closed on only the rare occasion when tidal flooding is predicted to occur. Closing the storm surge gates eliminates tidal flushing, which would allow the buildup of contaminants in the waters of Jamaica Bay behind the barrier. This has the potential to negatively impact water quality and attainment of water quality standards. These impacts would tend to be more acute in nature and may vary during the times of year where DO can drop to low levels due to algal and bacterial respiration and sediment oxygen demand coupled with high temperatures and stratification, which can prevent warm surface and cooler bottom waters from mixing and replenishing deeper waters' oxygen.

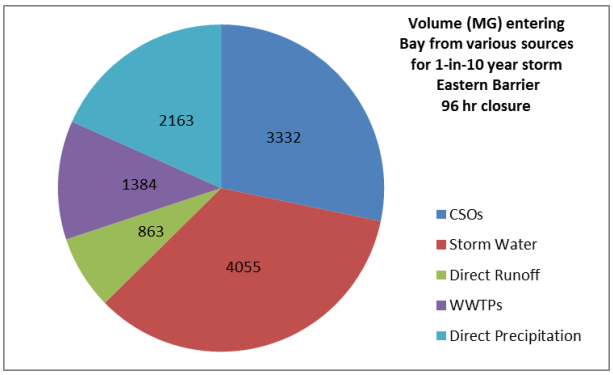


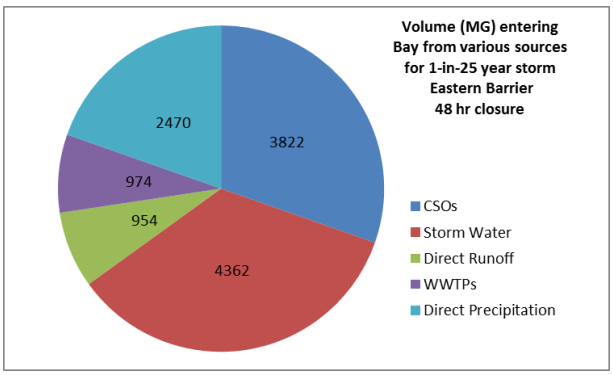












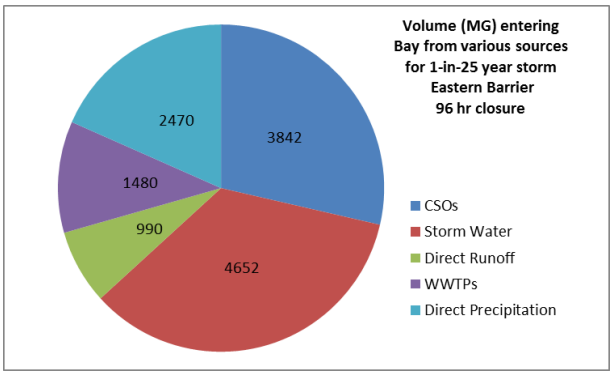


Figure 8. Runoff Volumes for the Various Modeling Scenarios.

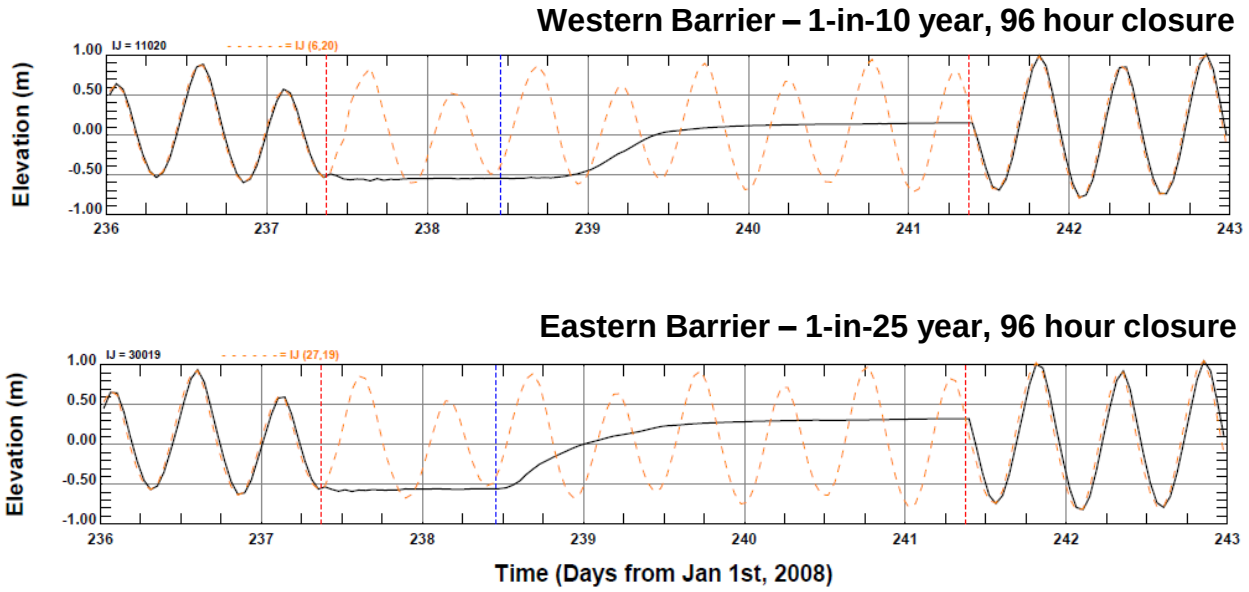


Figure 9. Water Elevations During the 96 hour Closure of the Storm Surge Barrier Gates.

Storm Surge Barrier Open Results

Water Elevation

Changes in tidal elevation range were assessed by tabulating the maximum differences in the tide range on an annual basis. The annual calculation included the extremes for the 2008 conditions. The results will be presented for seven locations within the bay (Figure 10): (1) near the eastern storm surge barrier, (2) near Paerdegat Basin in the North Channel in the northwestern portion of the bay, (3) near Spring Creek in the North Channel in the northeastern portion of the bay, (4) in Grassy Bay, (5) in Grass Hassock Channel, south of Jo Co Marsh, (6) in Beach Channel near Cross Bay Boulevard in the southern portion of the bay, and (7) in Pumpkin Patch Channel south of Duck Point Marsh.

Table 2 presents the maximum annual tidal elevation range that was computed for 2008 for the baseline (no storm surge barrier) and for the eastern and western barrier (assumed to be always open) at each of the seven locations. The model suggests that the implementation of a storm surge barrier will reduce the maximum tidal range. In the case of the construction of an eastern barrier, which is located at a narrower location in the Rockaway inlet as compared to the western barrier, the tidal range is reduced between 6.4 to 7.6 cm or about 3 percent. In the case of the construction of the western barrier, the tidal range is reduced between 1.4 and 2.3 cm or between 0.6 and 1 percent. These changes are fairly small.

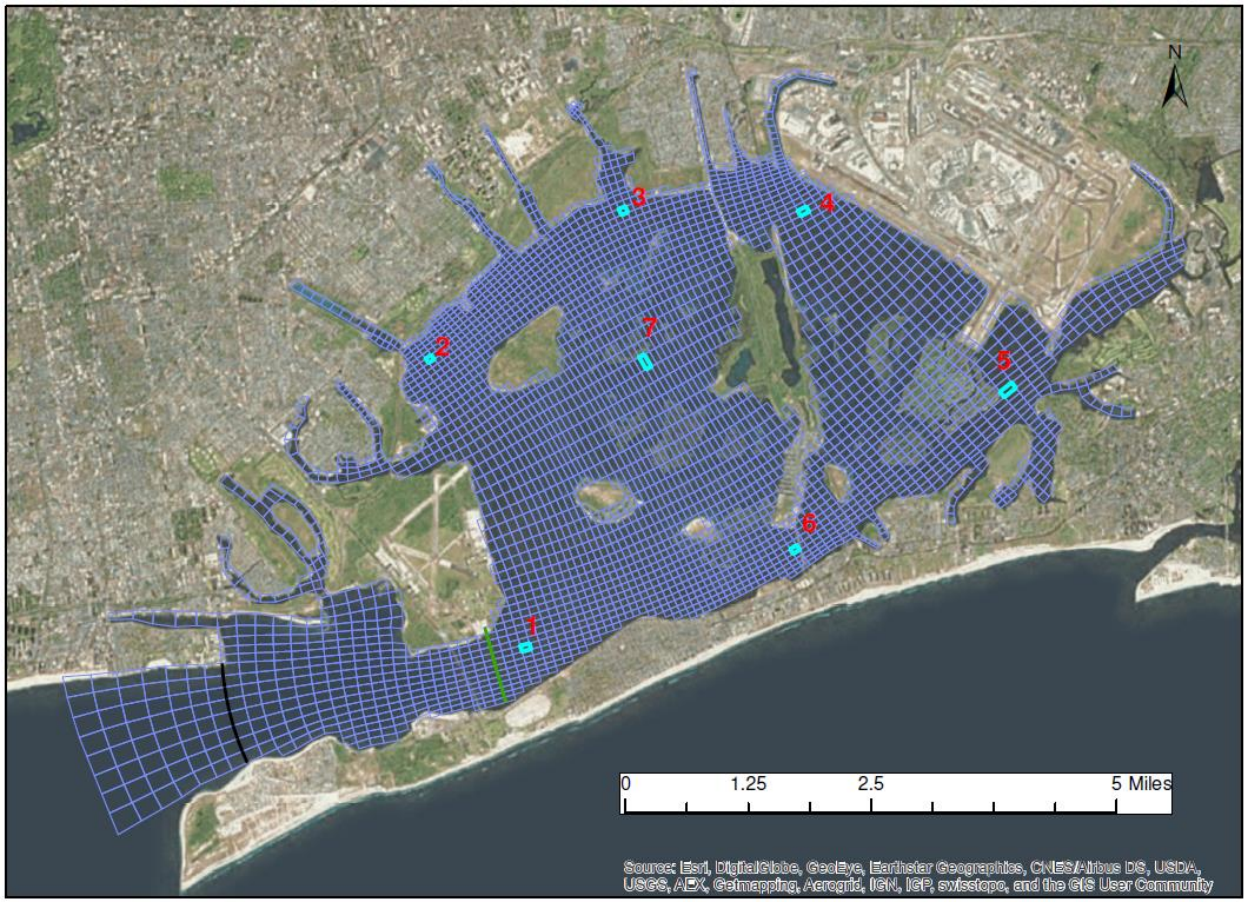


Figure 10. Location Map of Cells Used to Report Tidal Ranges.

Table 2. Maximum Annual Tidal Elevation Range (cm)

Location	Baseline	East Barrier	Difference	West Barrier	Difference
Near east storm surge barrier	231.5	224.8	**-6.7**	230.1	**-1.4**
Near Paerdegat Basin	242.3	235.4	**-6.9**	240.2	**-2.1**
Near Spring Creek	245.3	238.6	**-6.7**	243.0	**-2.3**
Grassy Bay	245.2	238.0	**-7.2**	243.0	**-2.2**
Grass Hassock Channel	245.4	237.8	**-7.6**	243.1	**-2.3**
Beach Channel	242.2	235.3	**-6.9**	240.1	**-2.1**
Pumpkin Patch Channel	242.1	235.7	**-6.4**	240.7	**-1.4**

Water Quality

To assess the changes due to the construction of the storm surge barriers on an annual basis, daily average model concentration results for all model segments were aggregated for the full calendar year. Then, as a way to summarize the variability or range of the model computations in tabular form, the 10th (representing the low range in computed concentrations), 50th (median), and 90th (representing the high range in computed concentrations) percentile concentrations are presented.

Salinity

The storm surge barriers have the potential to restrict tidal flow when the storm surge gates are open. Table 3 presents the calculated salinity for the baseline condition, as well as the East Barrier and West Barrier scenarios. The percentiles shown represent the low (10%), median (50%) and high (90%) range in the model computed salinities. The results are presented for the 1-in-25 year scenarios, but the results for the 1-in-10 year scenarios are similar. The model results indicate small changes in long-term salinity levels, on the order of 0.1 ppt due to the construction of the storm surge barriers. However, some short term changes can be observed, particularly during rainfall events. Figure 11, presents contour plots of surface salinity for the Baseline condition, i.e., no storm surge barrier scenario, and for the storm surge barrier gates open scenario. (Contour plots will also be presented for other water quality parameters of interest on the plots to follow. The format for these plots is as follows: the Baseline condition or scenario will be presented on the left-hand panels, the storm surge barrier gates open scenario on the right-hand panels and the computed differences between the two scenarios in the center panels.) Figure 11, shows the model computed salinities for the Baseline (left panel) and the barrier open (left panel) as a function of concentration, from 0-4 ppt (blue) to 28-32 ppt (red). The center panel shows the difference between the two scenarios and indicates that there is a 1-4 ppt difference or reduction in salinity on Day 240 (which is about 1.5 days into the rainfall event) with the implementation of the storm surge barrier. The differences are likely due to freshwater inputs associated with the rainfall events and restrictions in flow caused by the closure of the storm surge barrier. However, within a few days after the rainfall event and re-opening of the barrier the differences in salinity begin to be reduced.

Table 3. Long-Term Salinity (ppt) Results

Scenario	10th Percentile	50th Percentile	90th Percentile
Baseline	23.51	25.61	27.43
East Barrier	23.25	25.47	27.39
West Barrier	23.54	25.74	27.57

Pathogens

Fecal coliform and enterococci were analyzed on an area-weighted basis using both a geometric and arithmetic annual mean for both the 1-in-10 and 1-in-25 year storm scenarios using surface layer results. The fecal coliform results are presented in Table 4 and the enterococci results in Table 5. The model results indicate that long-term changes to the pathogen concentrations will be small (~5-8% for geometric means and ~13-19% for arithmetic means). Based on these results there should not be any impact on the long-term attainment of the pathogen water quality criteria, since NYSDEC fecal

and enterococci water quality standards are based on a monthly geometric mean not to exceed 200 cfu/100 mL for fecal coliform and a not to exceed 104 cfu/100 mL for a single sample or not to exceed a monthly geometric mean of 35 cfu/100 mL for enterococci.

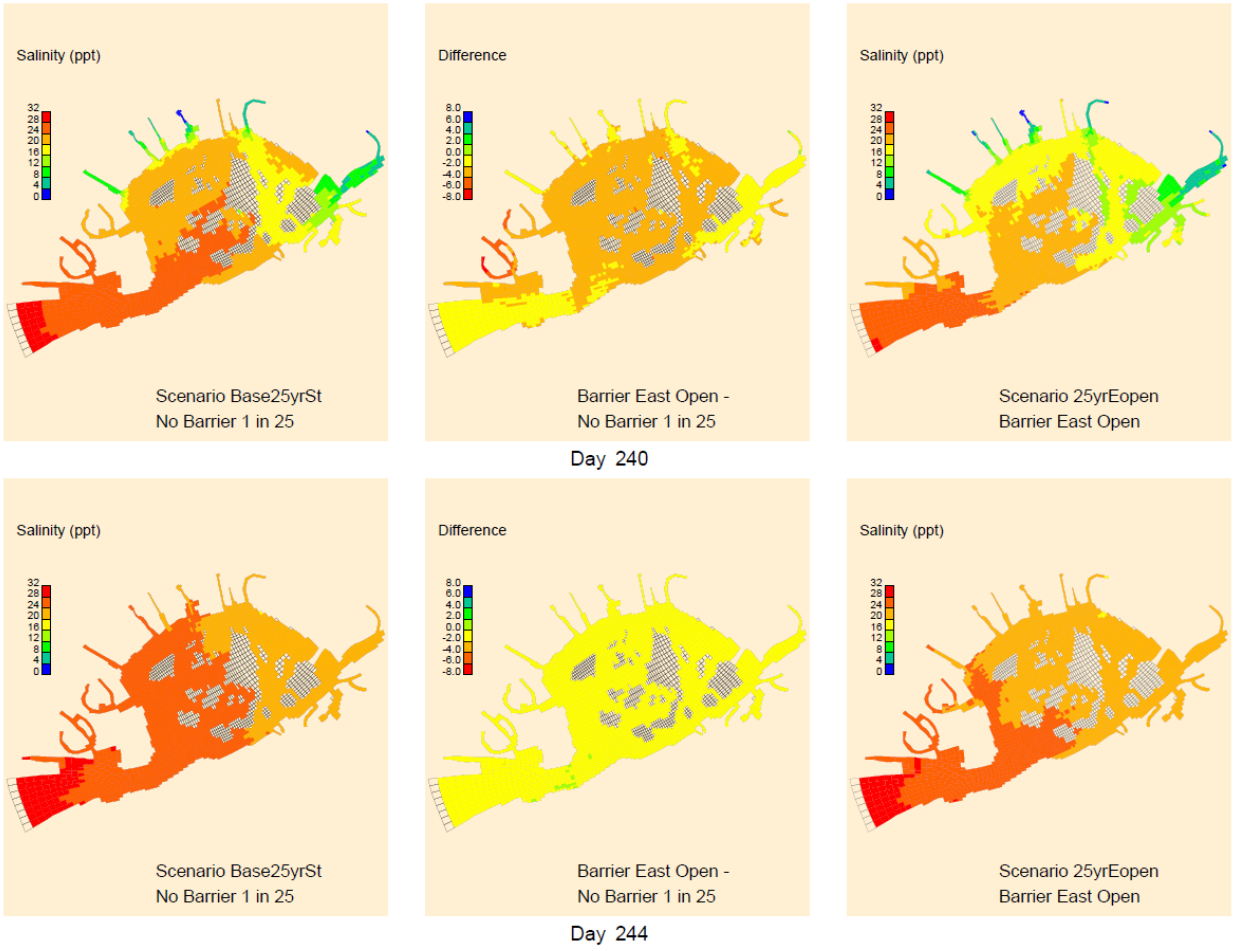


Figure 11. Comparison of Calculated Surface Salinity for the Baseline and Eastern Storm Surge Barrier Open, 1-in-25 year Scenarios.

Table 4. Long-Term Fecal Coliform (cfu/100mL) Results

	Annual Geometric Mean		Annual Arithmetic Mean	
Scenario	**1-in-10 year**	**1-in-25 year**	**1-in-10 year**	**1-in-25 year**
Baseline	7.10	7.68	116.93	138.07
East Barrier	7.13	7.72	117.56	138.96
West Barrier	7.14	7.71	116.99	138.12

Table 5. Long-Term Enterococci (cfu/100mL) Results

	Annual Geometric Mean		Annual Arithmetic Mean	
Scenario	**1-in-10 year**	**1-in-25 year**	**1-in-10 year**	**1-in-25-year**
Baseline	4.44	4.65	41.32	46.74
East Barrier	4.45	4.66	41.52	47.02
West Barrier	4.49	4.69	41.36	46.80

Nutrients

The eutrophication model includes six nitrogen state-variables and five phosphorus state-variables. Model results for total nitrogen (TN) and dissolved ammonium (NH_4) are presented as examples. Table 6 presents the calculated TN concentrations and Table 7 presents the calculated NH_4 concentrations for the baseline condition, as well as the East Barrier and West Barrier scenarios. The results are presented for the 1-in-25 year scenarios, but the results for the 1-in-10 year scenarios are similar. The model output is presented to three or four decimal places in order to show the differences between the scenario results. The NH_4 results do not represent the unionized ammonia (NH_3) concentrations. The NH_3 concentration is a fraction of the NH_4 concentration based on the temperature, salinity and pH. The change in NH_3 can be inferred from the change in NH_4 in that small changes in NH_4 likely indicate small changes in NH_3. The model results indicate small changes in TN concentrations, on the order of 0.01 mg/L or less, due to the construction of the storm surge barriers. NH_4 concentrations are not only affected by dilution, but are also affected by algal growth, nitrification and fluxes from the sediment, so there are multiple factors that influence the change in NH_4 concentrations. Differences in NH_4 concentrations are calculated to be on the order of 0.001 mg/L.

Table 6. Long-Term Total Nitrogen (mg N/L) Results

Scenario	10th Percentile	50th Percentile	90th Percentile
Baseline	0.452	0.814	1.309
East Barrier	0.450	0.815	1.313
West Barrier	0.443	0.809	1.297

Table 7. Long-Term Dissolved Ammonium (mg N/L) Results

Scenario	10th Percentile	50th Percentile	90th Percentile
Baseline	0.0347	0.0812	0.1983
East Barrier	0.0348	0.0805	0.2003
West Barrier	0.0351	0.0821	0.1926

On shorter time scales, on the order of a day, the impacts of the barrier are slightly larger, as shown on Figure 12 for dissolved ammonium. This figure compares the concentrations (0.0-0.1 mg N/L (blue) to 0.7-0.8 mg N/L (red)) for the Baseline (i.e., existing conditions - no barrier), shown in the left panel, versus the East barrier (gates open), shown on the right panel, for Julian day 240 (second day of the rainfall event) and day 244 (three days after reopening storm surge barrier gates) for the 1-in-25 year storm event. The middle panel presents the difference (-0.12 to -0.16 mg N/L (red) to 0.12 to 0.16 mg N/L (blue)) between the two model scenarios (East Barrier open - Baseline). As can be seen short-term differences lie within a +0.04 and -0.04 mg N/L range.

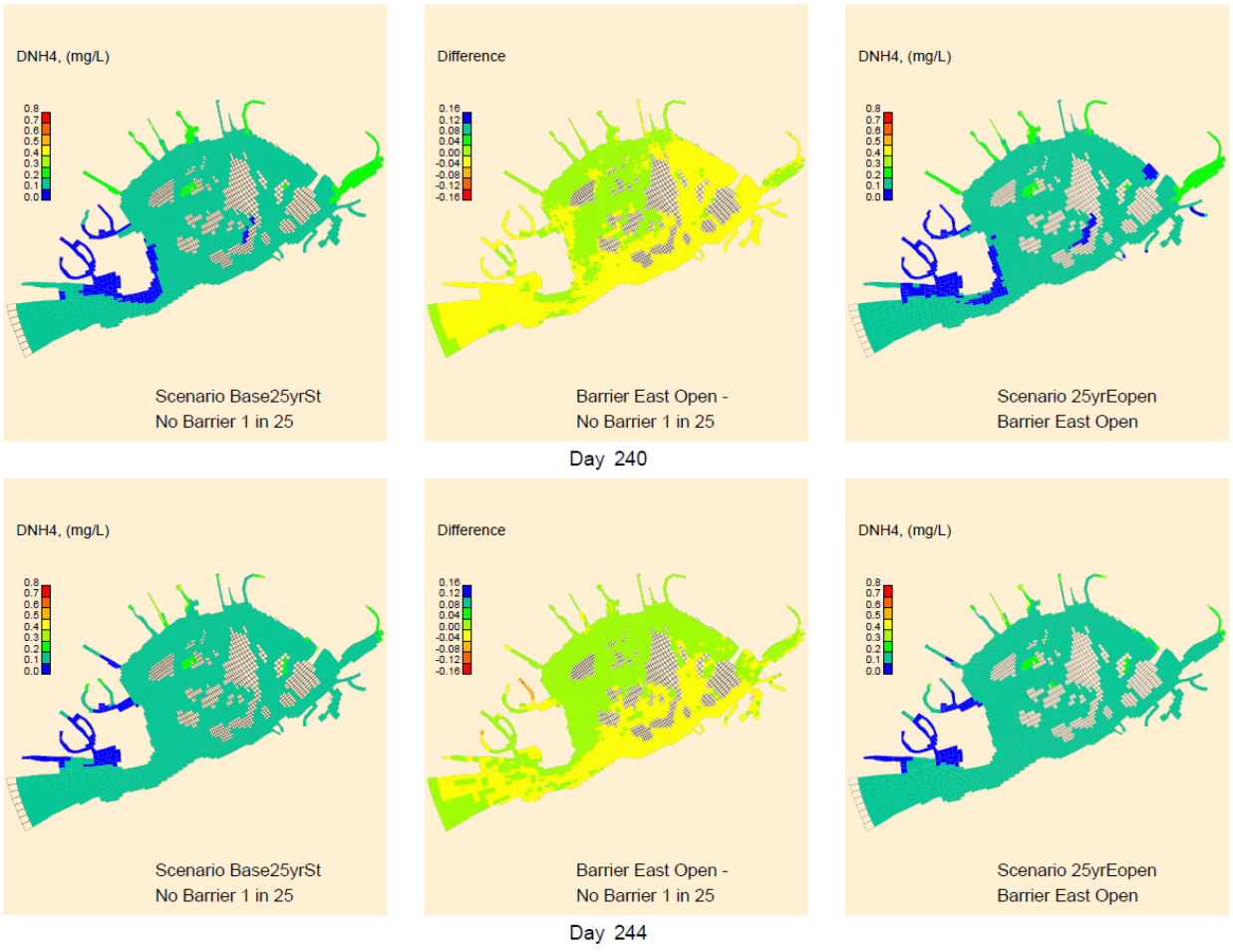


Figure 12. Comparison of Calculated Surface Dissolved Ammonium (DNH4) for the Baseline and Eastern Storm Surge Barrier Open, 1-in-25 year Scenarios.

Chlorophyll

Chlorophyll annual results are presented in Table 8. The model results indicate that like the other constituents, the annual changes resulting from the construction of the storm surge barriers are small (less than 1 ug/L when comparing concentration differences between the scenarios for a fixed percentile).

Table 8. Long-Term Chlorophyll (ug/L) Results

Scenario	10th Percentile	50th Percentile	90th Percentile
Baseline	10.00	24.12	38.64
East Barrier	10.01	24.14	38.57
West Barrier	10.15	24.39	39.09

However, on shorter time scales, the changes between the baseline and the storm surge barrier scenarios are larger than computed on an average annual basis. Figure 13 presents a comparison between the Baseline and the open Eastern Barrier scenarios (the layout of the figures is as described for Figure 11 above). Generally, the model shows that the barrier reduces chlorophyll on the order of 2-4 ug/L, except in the Gerritsen Creek, Mill Creek, and Shell Bank Creek portion of the Bay, where chlorophyll decreases as much as 8 ug/L. The chlorophyll reductions in the open waters of the Bay represent a reduction of between 7 to 15% from ambient levels.

Dissolved Oxygen

Long-term or annual bottom DO results are presented in Table 9. The model results indicate that the long-term changes in DO concentrations, due to the construction of the storm surge barriers, will be negligible, on the order of 0.01 mg/L or less.

However, changes in bottom water DO over shorter time intervals are larger than those computed on an annual basis. Figure 14 presents a comparison of bottom water DO for the Baseline and open East Barrier for the 1-in-25 year storm even for Julian days 240 (second day of storm) and 244 (three days after storm surge barrier gates re-opened). The layout of the figure is a described above for Figure 11, with computed DO values ranging from 0 to 8 mg/L and computed differences ranging from -2.0 to 2.0 mg/L. While, in general, the bottom water DO for both scenarios show a high degree of similarity, the difference plots indicate that there are some areas of the Bay, mainly in Grassy Bay and in the channels that lie between the marsh islands in the middle region of the Bay, where bottom water concentrations of DO decrease between 1 to 2 mg/L. However, in these marsh area channels, the bottom DO values are still above the NYSDEC water quality standards. Over the larger area of the Bay the changes in DO tend to be smaller, between +0.5 and -0.5 mg/L. It is important to recognize, however, that for this JEM model, the bottom layer of the model represents only 3 percent of the total water column depth. As will be shown later, the upper portions of the water column (representing greater than 90 percent of the total water column depth) do not show as the same low levels of DO as do the bottom two layers of the model.

It is important to note that the model runs performed in this analysis did not include the effects of storm related winds that might be expected to accompany the types of storm that are likely to trigger a closure

of the proposed barrier, i.e., Hurricane Sandy type events. Since the JEM modeling done to date does not include storm related wind, it likely overestimates the DO impacts at the Bay bottom.

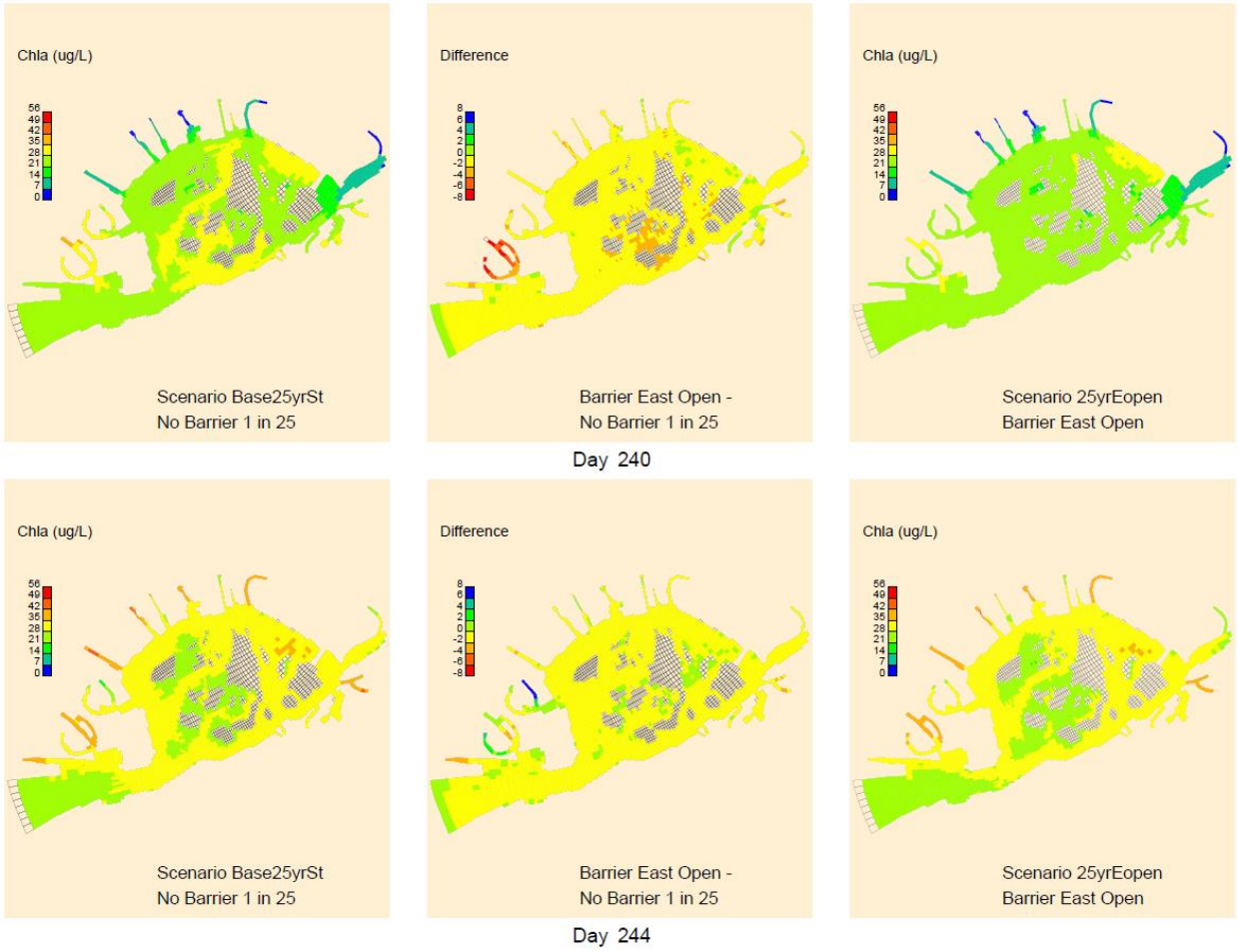


Figure 13. Comparison of Calculated Surface Chlorophyll for the Baseline and Eastern Barrier Open, 1-in-25 Year Scenarios.

Table 9. Long-Term Daily Average Dissolved Oxygen (mg/L) Results

Scenario	10th Percentile	50th Percentile	90th Percentile
Baseline	6.58	9.06	12.20
East Barrier	6.57	9.07	12.21
West Barrier	6.61	9.07	12.17

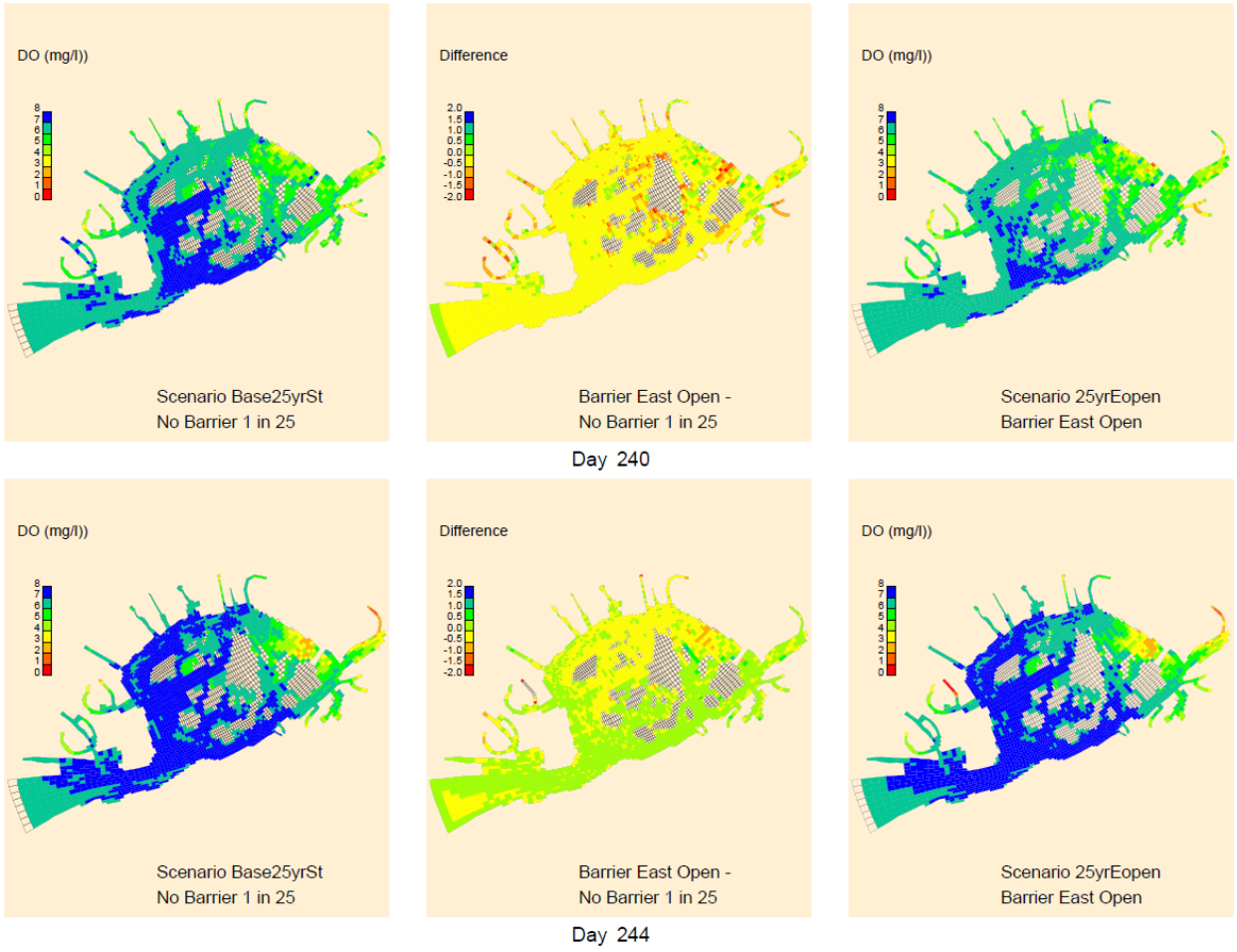


Figure 14. Comparison of Calculated Bottom Dissolved Oxygen for the Baseline and Eastern Barrier Open, 1-in-25 Year Scenarios.

Storm Surge Barrier Closure Results

Short-term changes in water quality in response to closures of the storm surge barrier gates were analyzed both spatially and temporally. Spatially, results provide an overview of which areas of the Bay would expect to be impacted by closure of the storm surge barrier on a daily average basis. The temporal analysis, based on time-series figures, present hourly results at specific locations to help assess the magnitude of impacts at specific locations and over how long a period of time the impacts would occur.

Water Elevation

Water elevation increases in Jamaica Bay, when the storm surge gates are closed, due to freshwater inputs from WWTPs, CSOs, stormwater, direct runoff, and precipitation falling directly on the water surface as previously shown in Figure 9. In the scenarios that were analyzed, the smallest increase in water level occurs for the 1-in-10 year storm, 48 hour closure, Western Barrier scenario. This scenario has the least amount of precipitation, the shortest duration of closure and the largest surface area to accept water. The scenario with the greatest change in elevation is the 1-in-25 year, 96 hour closure, Eastern Barrier scenario. The water elevations within the Bay increase by 2 to 3 feet during the time

the storm surge gates are closed in the scenarios that were analyzed, which is about half of the normal tidal range. Table 10 presents the increase in water elevation for each scenario during the period of storm surge barrier closure. The increase in water elevation due to 5.78 to 6.60 inches of rain is largely due to runoff from the Jamaica Bay watershed, which is several times larger than the wetted surface area of the bay, as well as precipitation delivered to the surface waters of the Bay. Roughly, the watershed runoff (CSOs, stormwater, and direct drainage) contributes about 65-75 percent of the increase in elevation, rainfall falling directly on the bay is the source for 20-25 percent of the increase, and WWTPs account for 7-12 percent of the increase in water elevation.

Table 10. Water Elevation (feet) Increase During Storm Surge Barrier Closure

Scenario	Western Barrier	Eastern Barrier
1-in-10 Year, 48 Hour Closure	2.31	2.44
1-in-10 Year, 96 Hour Closure	2.38	2.65
1-in-25 Year, 48 Hour Closure	2.66	2.83
1-in-25 Year, 96 Hour Closure	2.70	3.00

Salinity

Surface salinity was significantly impacted by the closure of the storm surge gates. Figure 15 presents contours of surface salinity comparing the Eastern Barrier Open scenario to the 96 hour closure scenario. (The contour plots presented in Figure 15 and following are similar in layout as for Figures 11-14, but now the panels on the left present results for the open barrier scenario, while the panels on the right present the results for the closed barrier scenario. The center panel presents the difference in concentrations between the two scenarios, i.e., the closed barrier scenario concentrations minus the open barrier scenario concentrations.) Surface salinity decreases by as much as 8 ppt near the storm surge barrier inside the bay. Conversely, salinity increases by more than 2 ppt outside the storm surge barrier as it is cut off from the freshwater in the bay. Salinity within the bay as a whole is reduced due to the large rainfall event even for the scenario with the storm surge gates open. Since periods of lower salinity are periodic naturally occurring events, it would not be expected that changes in salinity due to the closure of the storm surge barrier would have a significant long-term impact on the existing habitat quality of the bay.

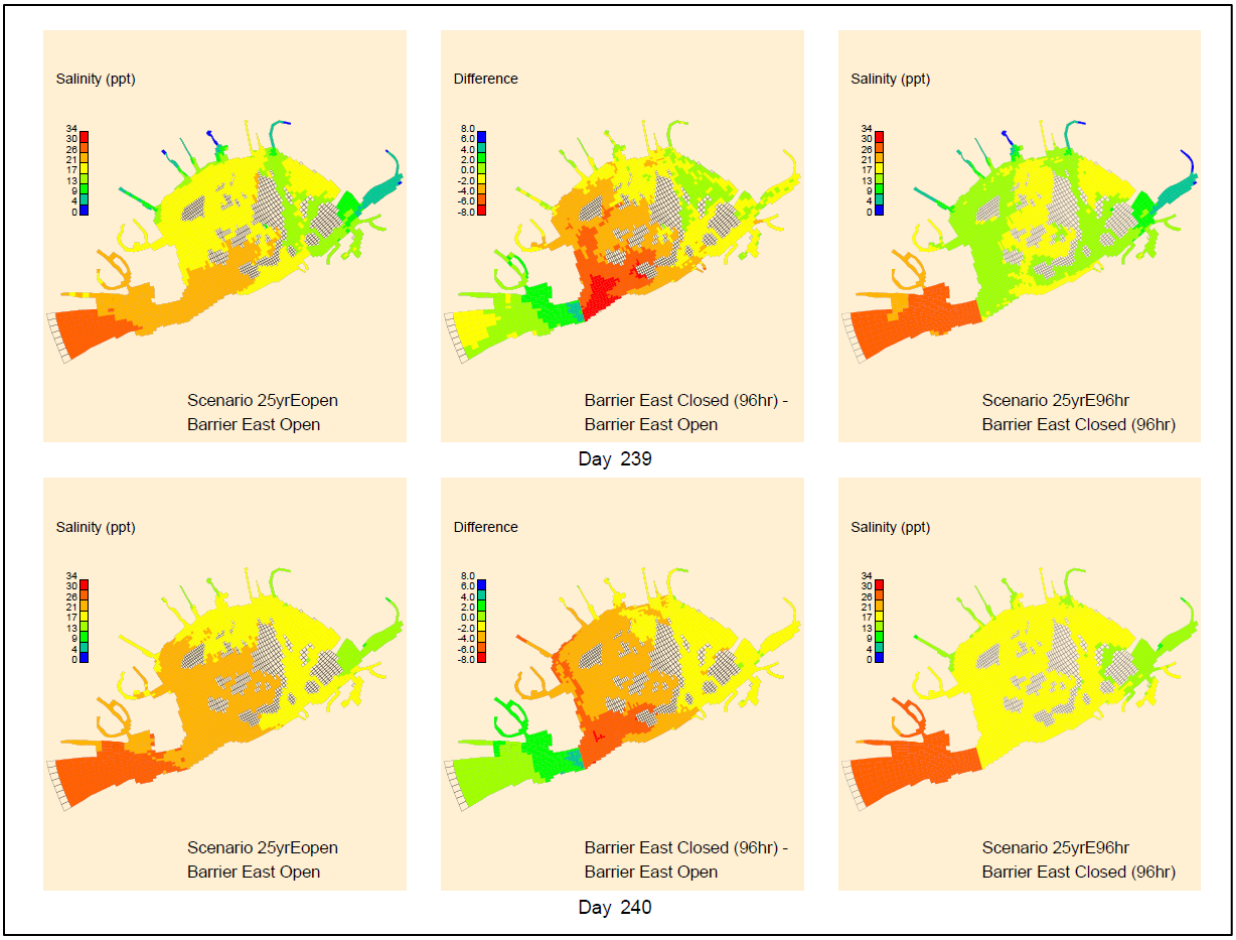


Figure 15. Comparison of Calculated Surface Salinity for the Eastern Barrier Open and 96 hour Eastern Barrier Closed, 1-in-25 Year Scenarios During and After Storm Surge Barrier Gate Closure.

Pathogens

In this analysis the gates were closed for a period in late-August for 48 to 96 hours. As shown in Figures 16 (barrier always open) and 17 (barrier closed for the 96 hour closure) for a location in the North Channel near Bergen Basin, both the fecal coliform and enterococci geometric means were below the existing water quality criteria for these pathogen indicators, i.e., attainment was met. Since the water quality criteria are based on 30-day geometric means, there is a limited response to the 30-day geometric mean concentration due to a perturbation that lasts two to four days (i.e., the influence of an event which spans two to four days does not have a significant impact on the computation of a 30-day geometric mean) as shown in Figures 16 or 17. Additionally, the die-off rate for bacteria is much faster than the flushing of the bay, so the consequences of closing the storm surge barrier has minimal impact on bacteria. The flushing time of Jamaica Bay is on the order of two to three weeks. Since at 20°C the model assumes a die-off rate of approximately 1.4 day^{-1}, which includes natural die-off and salinity induced toxicity effects, only 24.7 percent of the bacteria discharged into the bay would remain after a single day due to die-off.

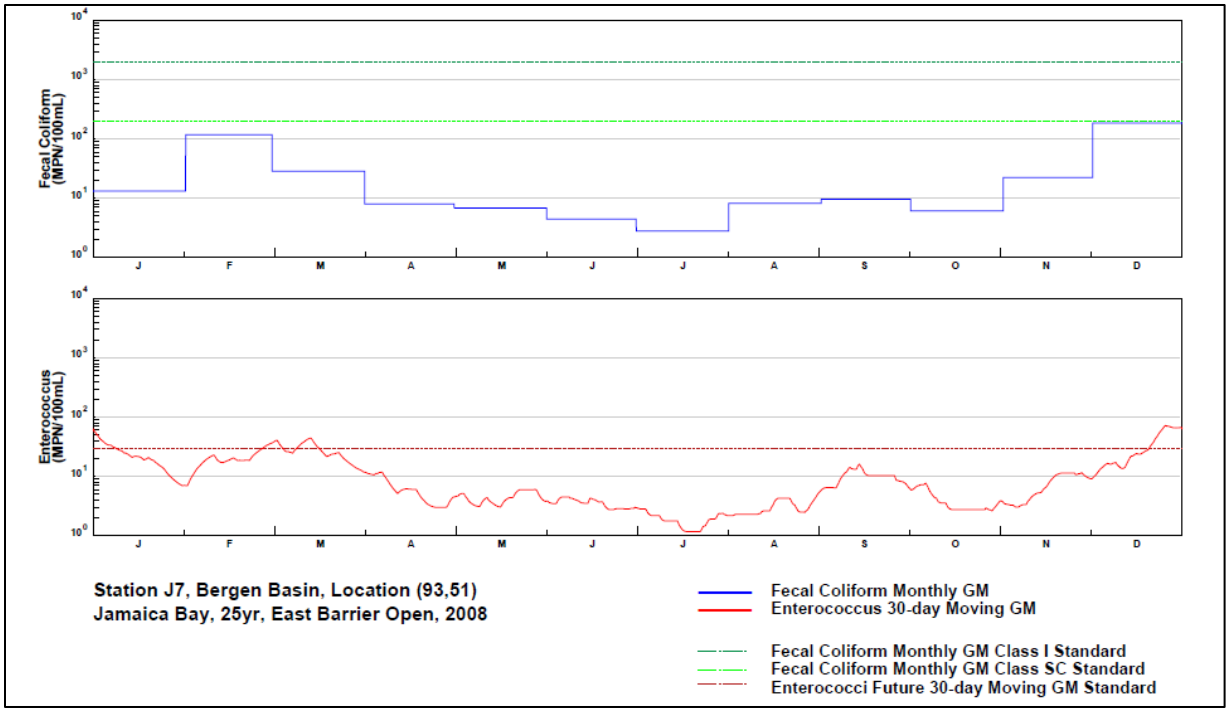


Figure 16. Comparison of Fecal Coliform and Enterococci Model Results Near the Mouth of Bergen Basin to Existing Water Quality Criteria for the Eastern Barrier Open, 1-in-25-year Scenario.

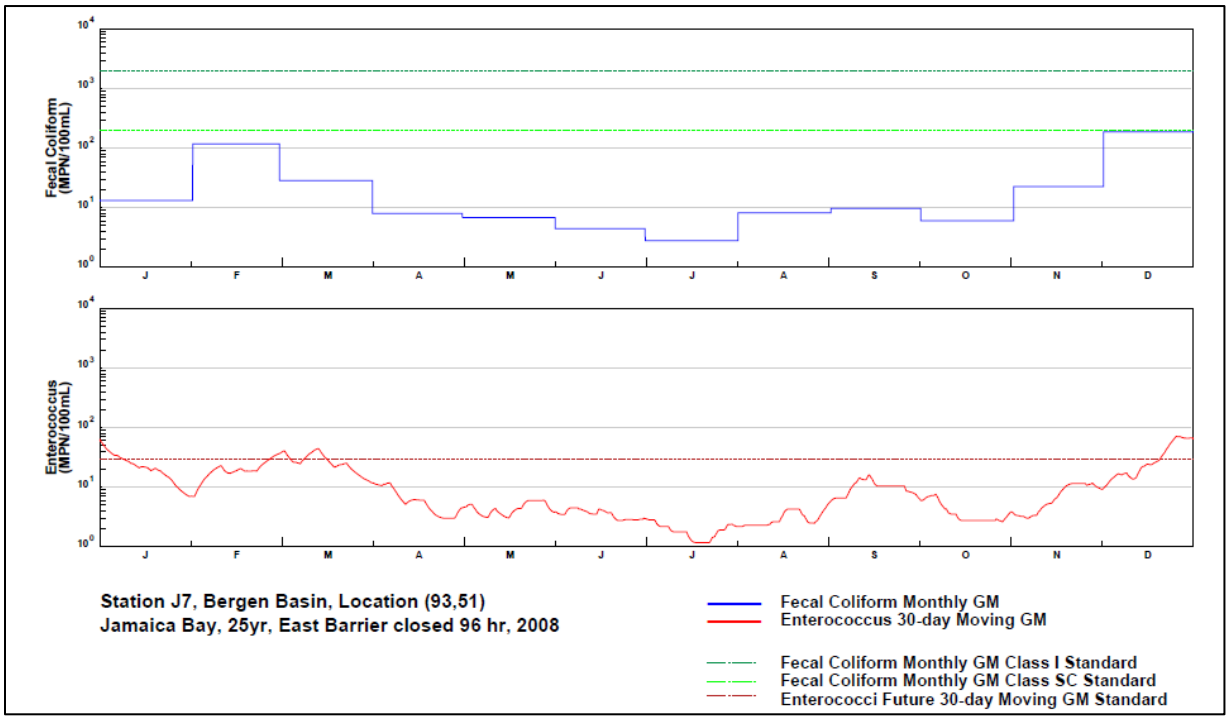


Figure 17. Time-Series of Fecal Coliform and Enterococci Model Results Near the Mouth of Bergen Basin to Existing Water Quality Criteria for the Eastern Barrier 96 hour Closure, 1-in-25-Year Scenario.

Figure 18 presents a spatial comparison of fecal coliform and enterococci attainment during the 30-day period when the western storm surge barrier is open and during the 96 hour closure for the 1-in-25 year event. The figure shows attainment of the criteria virtually everywhere (exceptions are the head of Bergen Basin and the head of Thurston Basin), and no difference between the two scenarios. Similar results were observed for the other scenarios. The analysis represents a period during late-August, when pathogen concentrations tend to be low due to high die-off rates that accompany higher water temperatures. There is, however, the potential for the storm surge barriers to contribute to non-attainment of the pathogen standards if the closure were to occur during a winter/spring nor'easter when water temperatures are cooler.

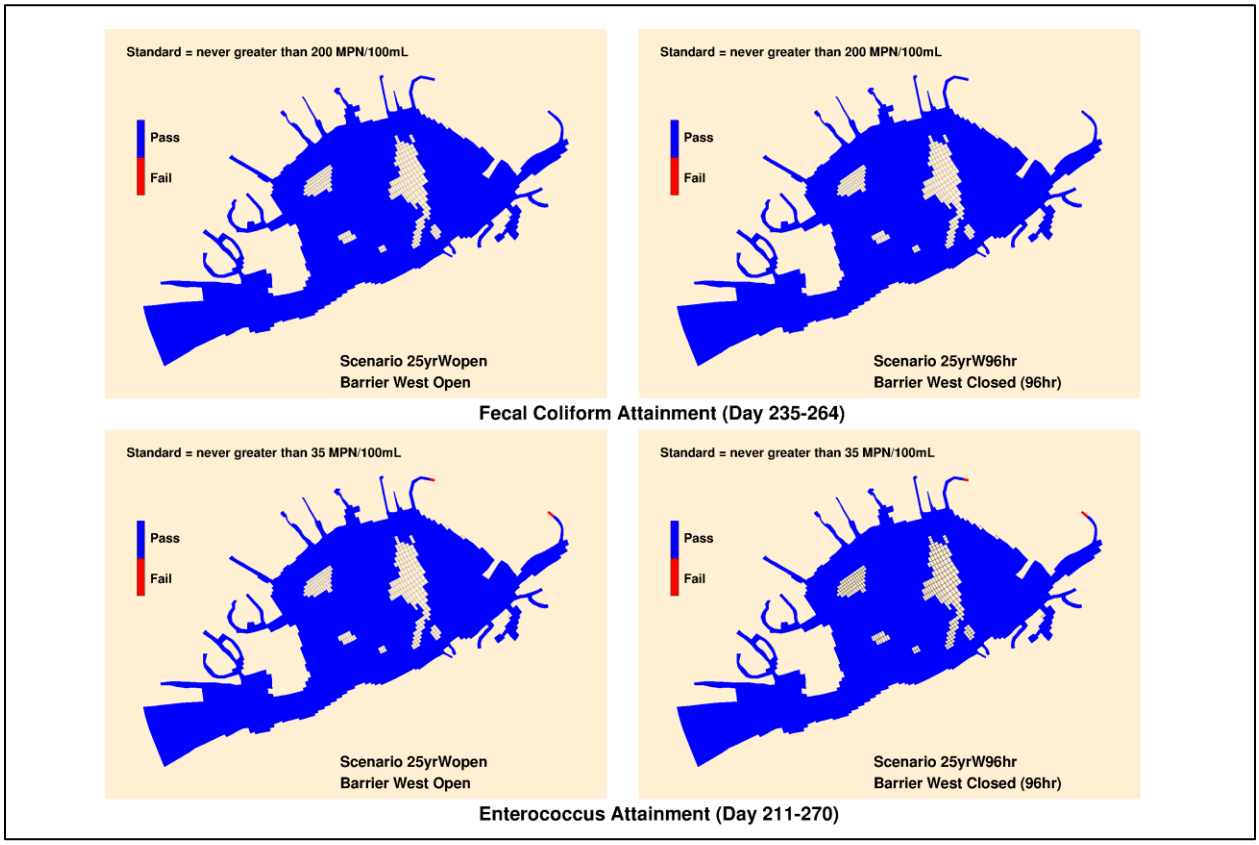


Figure 18. Comparison of Surface Fecal Coliform and Enterococci Model Calculated Attainment for the Western Barrier Open and 96 hour Western Barrier Closed, 1-in-25 Year Scenarios During the Period of Storm Surge Barrier Gate Closure.

Changes in pathogen levels between scenarios with storm surge gates open and closed are greater in the tributaries, which is where the pathogen sources originate. In the open waters of the bay the changes in the pathogen levels are relatively small. The changes in pathogen concentrations are not consistently higher or lower due to the impact of the tides in the open gate scenarios. Time-series Figures 19 through 21 present fecal coliform concentrations for the Barrier East, 1-in-25 year, open barrier and 96 hour closure scenario and the difference between these two scenarios at three locations in the bay (near the MTA railroad bridge trestle in Beach Channel, in the North Channel south of Bergen Basin and at the head of Paerdegat Basin). These time-series figures present surface, mid-depth and bottom fecal coliform concentrations for the period of a few days before the storm surge

gates are closed, the period when the gates are closed as delineated by the vertical lines, and a period after the gates are opened after the storm event.

As mentioned above, the biggest impacts on estimated pathogens occur in the tributaries. Figure 19 shows fecal coliform bacteria in the middle of Paerdegat Basin (a CSO tributary). During the storm event and for a period of about 24 hours after the rainfall ends, fecal coliform in mid-Paerdegat Basin are generally higher during the closure period as compared to the open barrier scenario. There are periods, however, wherein the mid-Paerdegat Basin fecal coliforms are lower during the closure period as compared to the open period. It depends on the point in the tidal cycle. However, the closure high differences are greater in magnitude than the closure low differences are. It is important to note that the maximum differences during closure are only about a factor of two when compared to the actual pathogen cell counts and that the differences between open barrier and closed barrier generally disappear within a few days after the tidal-barrier gates are re-opened. Figures 20 and 21, Harbor Survey Station J7 near Bergen Basin and Station J16, Horse Channel, in the center of the bay, show that the impacts of closure in the open waters of the Bay are much less than computed in the tributaries; maximum differences are less than 50 percent. In all cases, the impact of the storm surge barrier on pathogen concentrations quickly dissipates, so in the long-term impact of the storm surge barrier is minimal. In the case of pathogens, the storm surge gate closure time of 48 hours versus 96 hours is not that important because the die-off rate is much higher than the loss of pathogens due to tidal flushing.

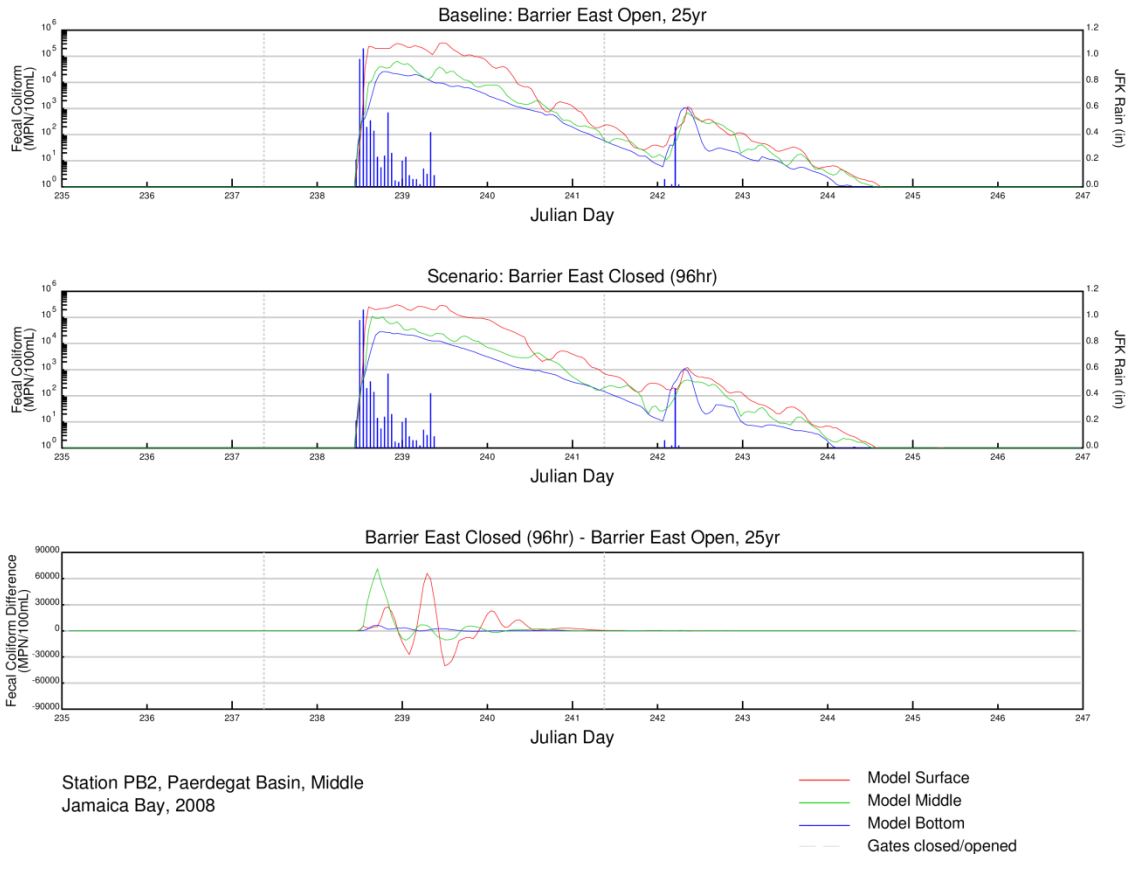


Figure 19. Computed Time-Series for Fecal Coliform in Mid-Paerdegat Basin for the 1-in-25 Year Storm Event with a 96 Hour Closure of the Eastern Storm Surge Barrier.

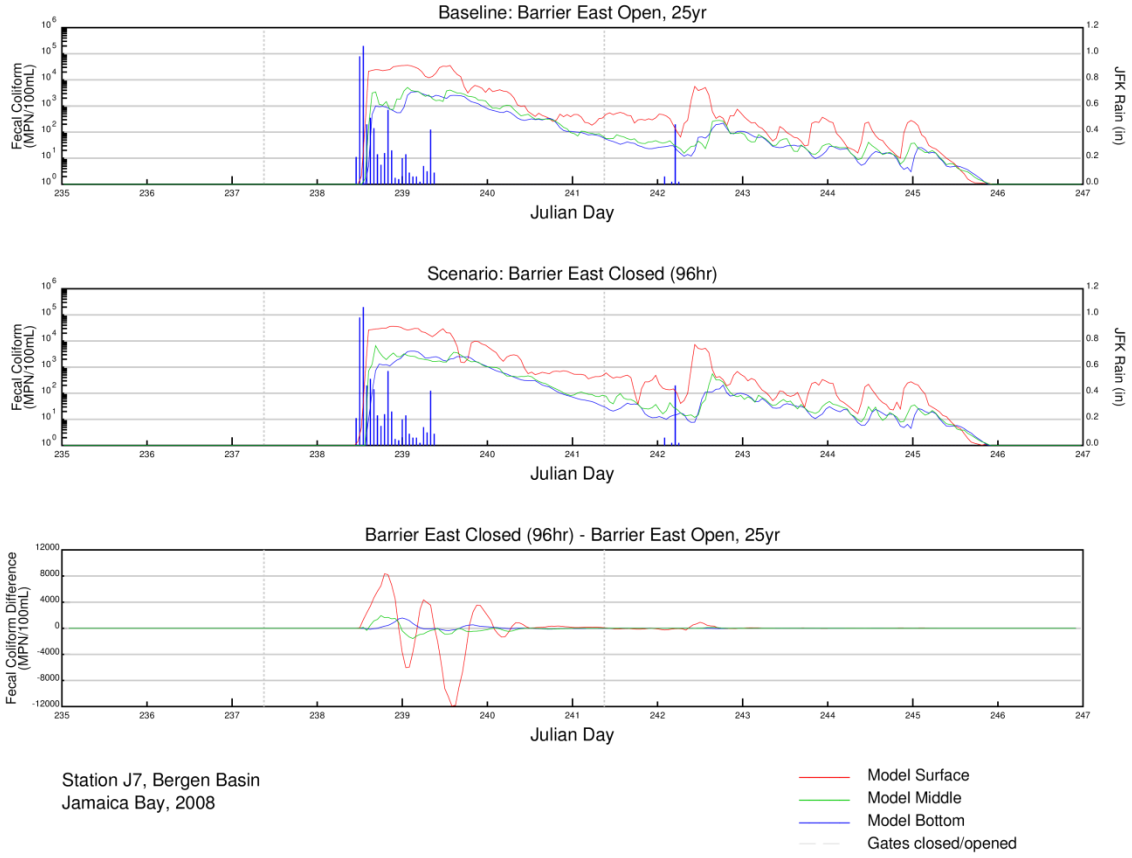


Figure 20. Computed Time-Series for Fecal Coliform at Harbor Survey Station J7 Near Bergen Basin for the 1-in-25 Year Storm Event with a 96 Hour Closure of the Eastern Storm Surge Barrier.

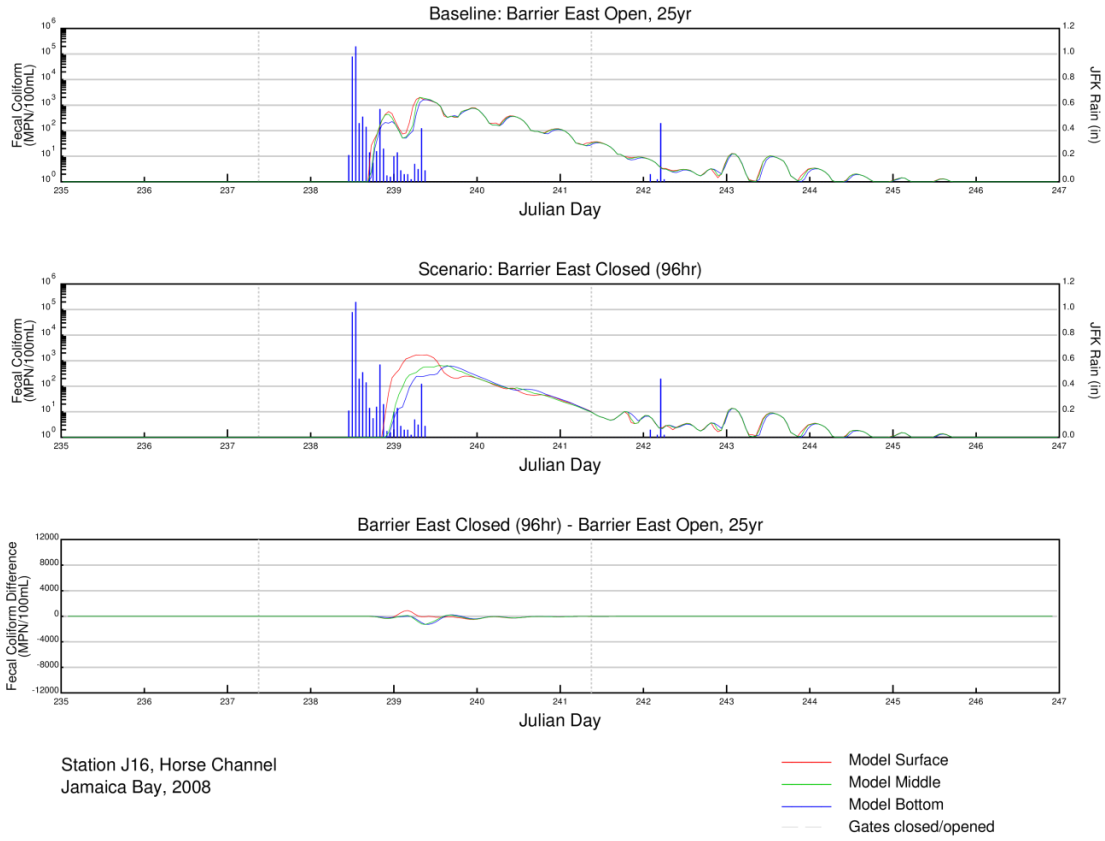


Figure 21. Computed Time-Series for Fecal Coliform at Harbor Survey Station J16 in Horse Channel for the 1-in-25 Year Storm Event with a 96 Hour Closure of the Eastern Storm Surge Barrier.

Nutrients

There are moderate impacts on nutrient concentrations as a consequence of the storm surge barrier gate closure. Figure 22 presents an example comparing the Eastern Barrier Open (left-hand panels) to the Eastern Barrier Closed for 96 hours (right-hand panels) during the 1-in-10 year storm event scenarios for surface dissolved NH_4 (0.0-0.8 mg N/L) during the last day of the gate closure and on the third day after reopening the storm surge barrier gates. The middle panels show the differences (-0.16 to 0.16 mg N/L) between the scenarios. In general, there are calculated reductions in ammonium near the storm surge barrier, and there are small increases further away from the barrier. Most of the changes in the calculated nutrient concentrations are small in the various scenarios analyzed and once the gates are re-opened conditions quickly return to levels previously calculated before the gates were closed. Changes to the surface nutrient concentrations would be expected to have the greatest impact on phytoplankton growth as there is no light limitation for growth at the surface. While the surface nutrient concentrations would be expected to increase more due to the less dense freshwater discharge from WWTPs, the model calculates a greater increase of inorganic nutrients in the bottom layer of the model. This may be due to the release of dissolved ammonium from the sediment bed that results as bottom DO levels decrease in the bottom waters of the Bay and due to vertical stratification of the water column (as will be described below).

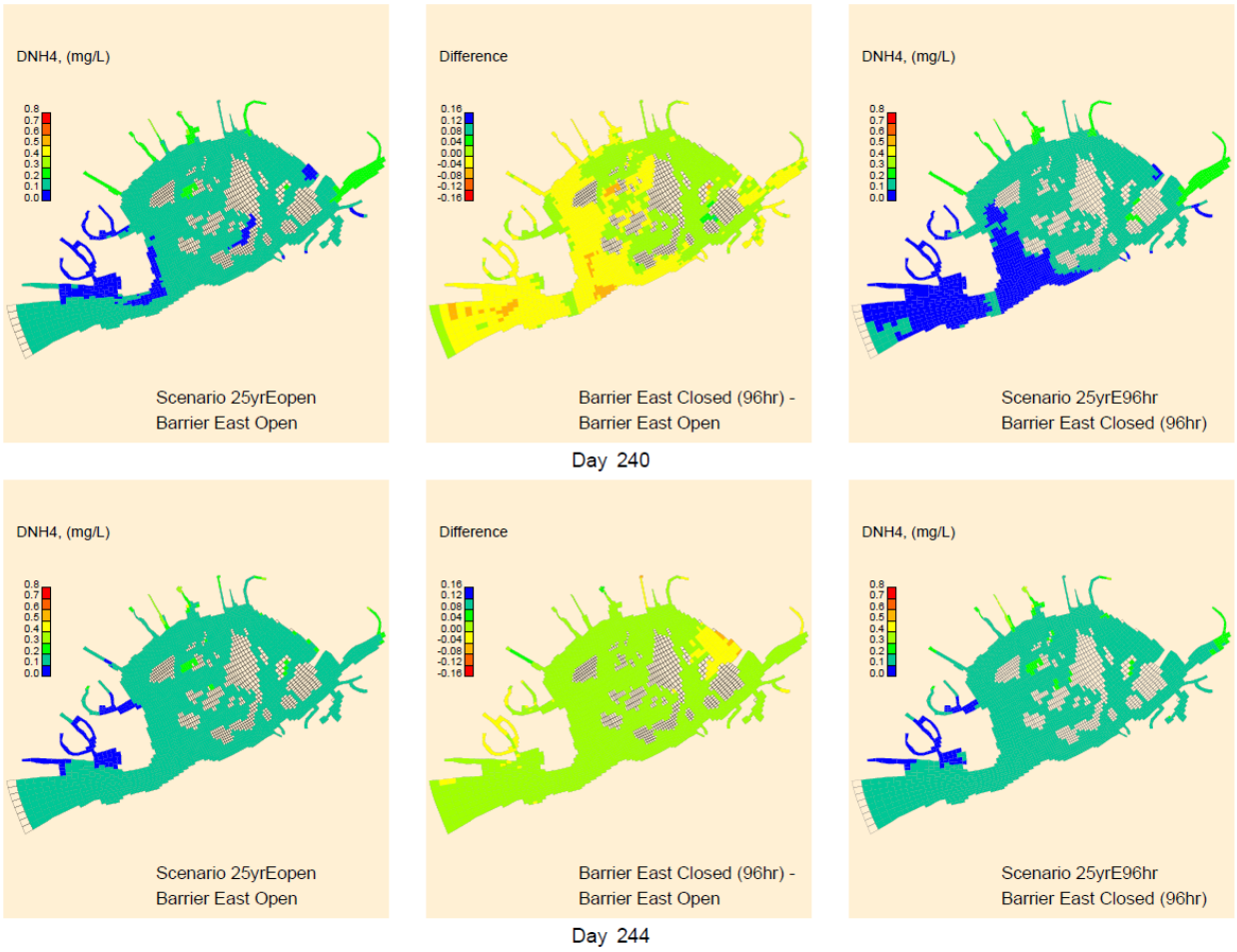


Figure 22. Comparison of Calculated Surface DNH4 for the Eastern Barrier Open and 96 hour Eastern Barrier Closed, 1-in-10 Year Scenarios During and After Storm Surge Barrier Gate Closure.

Times series of total nitrogen (TN) are shown in Figures 23 through 25 at Paerdegat Basin, near Bergen Basin and in the center of the bay, respectively. While TN increases in both the Barrier East Open and Barrier East Closed (96 hour), 1-in-25-year storm scenarios after the rainfall event, the TN continues to increase after the event with the storm surge gates closed, especially in the bottom layer. As will be discussed later, the bottom water DO concentrations decrease with time when the gates are closed. Low DO concentrations result in increased ammonia and phosphate fluxes from the sediment bed. This suggests that a 96 hour gate closure could have negative impacts on water quality and a shorter gate closure would be recommended as the higher TN concentrations are not observed in the first 48 hours of the gate closure. The time-series figures show that the high concentrations calculated just before the gates re-open dissipate rapidly after the opening of the gates, but slightly higher TN concentrations, relative to the open gate scenario,` linger for some time. As Jamaica Bay is generally not nutrient limited, the short period of elevated nutrient concentrations would not be expected to result in significant changes to phytoplankton growth.

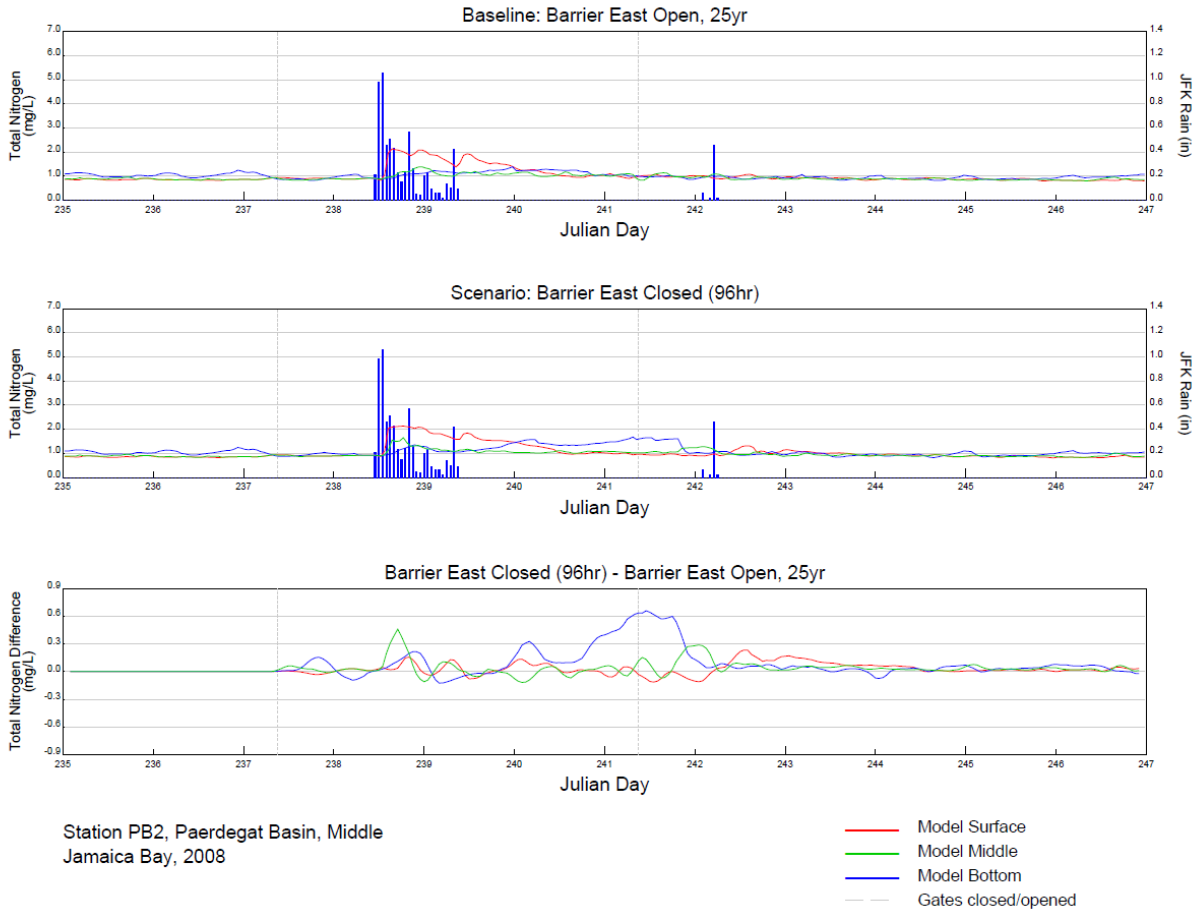


Figure 23. Computed Time-Series for Total Nitrogen in Mid-Paerdegat Basin for the 1-in-25 Year Storm Event with a 96 Hour Closure of the Eastern Storm Surge Barrier.

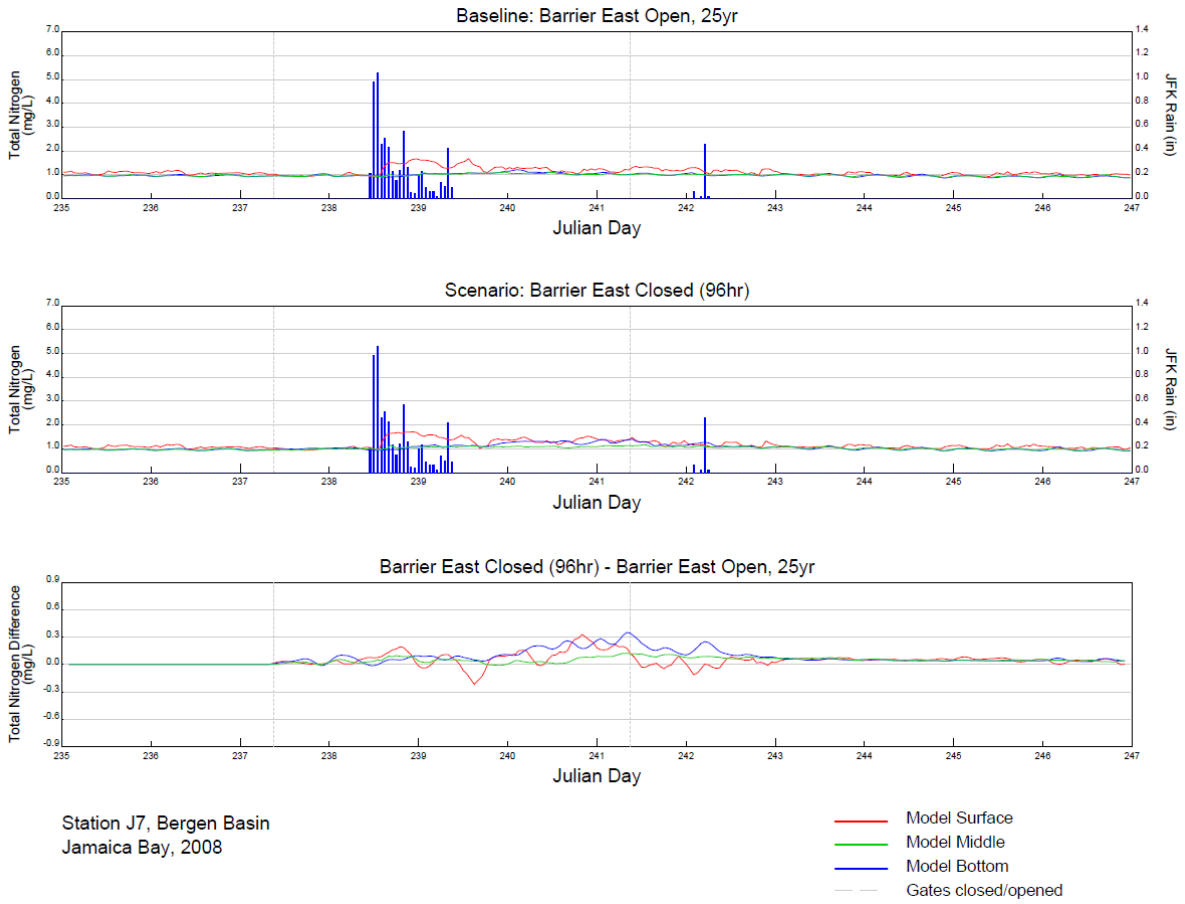


Figure 24. Computed Time-Series for Total Nitrogen at Harbor Survey Station J7 Near Bergen Basin for the 1-in-25 Year Storm Event with a 96 Hour Closure of the Eastern Storm Surge Barrier.

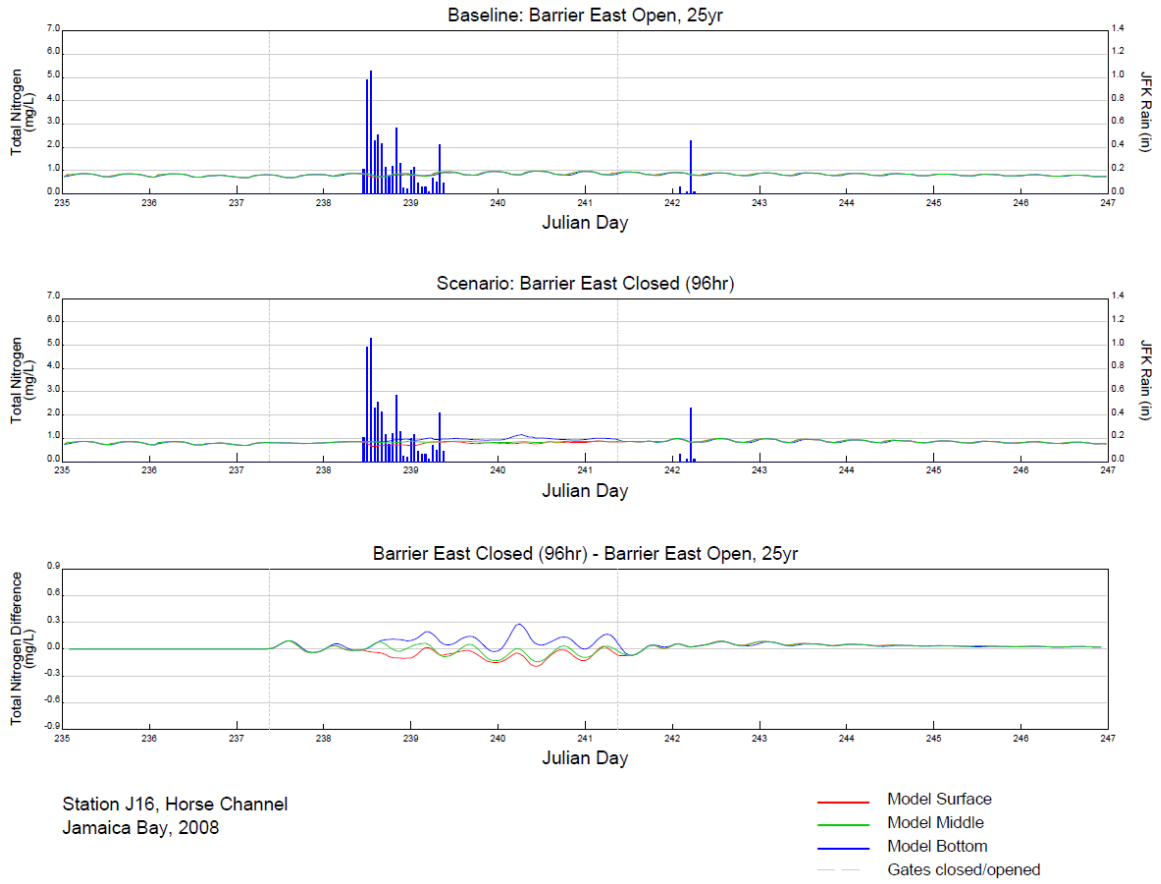


Figure 25. Computed Time-Series for Total Nitrogen at Harbor Survey Station J16 in Horse Channel for the 1-in-25 Year Storm Event with a 96 Hour Closure of the Eastern Storm Surge Barrier.

Chlorophyll

Figure 26 presents a comparison of the Eastern Barrier Open (left-hand panels) and the 96 hour Eastern Barrier Closure (right-hand panels) 1-in-25-year scenario for chlorophyll (0-56 ug/L) during the third day of the gate closure and on the third day after reopening the storm surge barrier gates. As can be observed in the middle panels (-8 to 8 ug/L) , there are small decreases in chlorophyll in the North Channel in the inner portion of the bay and small increases in chlorophyll in the near shore sections of Island Channel and just to the east of the barrier. Presumably, in the interior shallow portions of the bay the growth rate of phytoplankton is lower than the loss rate of the phytoplankton that are settling out of the water column. Within a few days after reopening the gates, the surface chlorophyll concentrations become quite similar, except for a portion of Grassy Bay. Times series for chlorophyll are presented in Figures 27 through 29 for Paerdegat Basin, near Bergen Basin, and in the center of the bay. The maximum changes in chlorophyll levels are on the order of 10-15 µg/L, and vary in magnitude likely due to tidal influences that are present in the open gate scenario, which are not present when the gates are closed. In other words, much of the difference appears to be related to the timing of the tides rather than actual changes in phytoplankton growth. As the increase in nutrients due to the gate closure is short lived, there is no subsequent algal bloom during the period after the gates are re-opened.

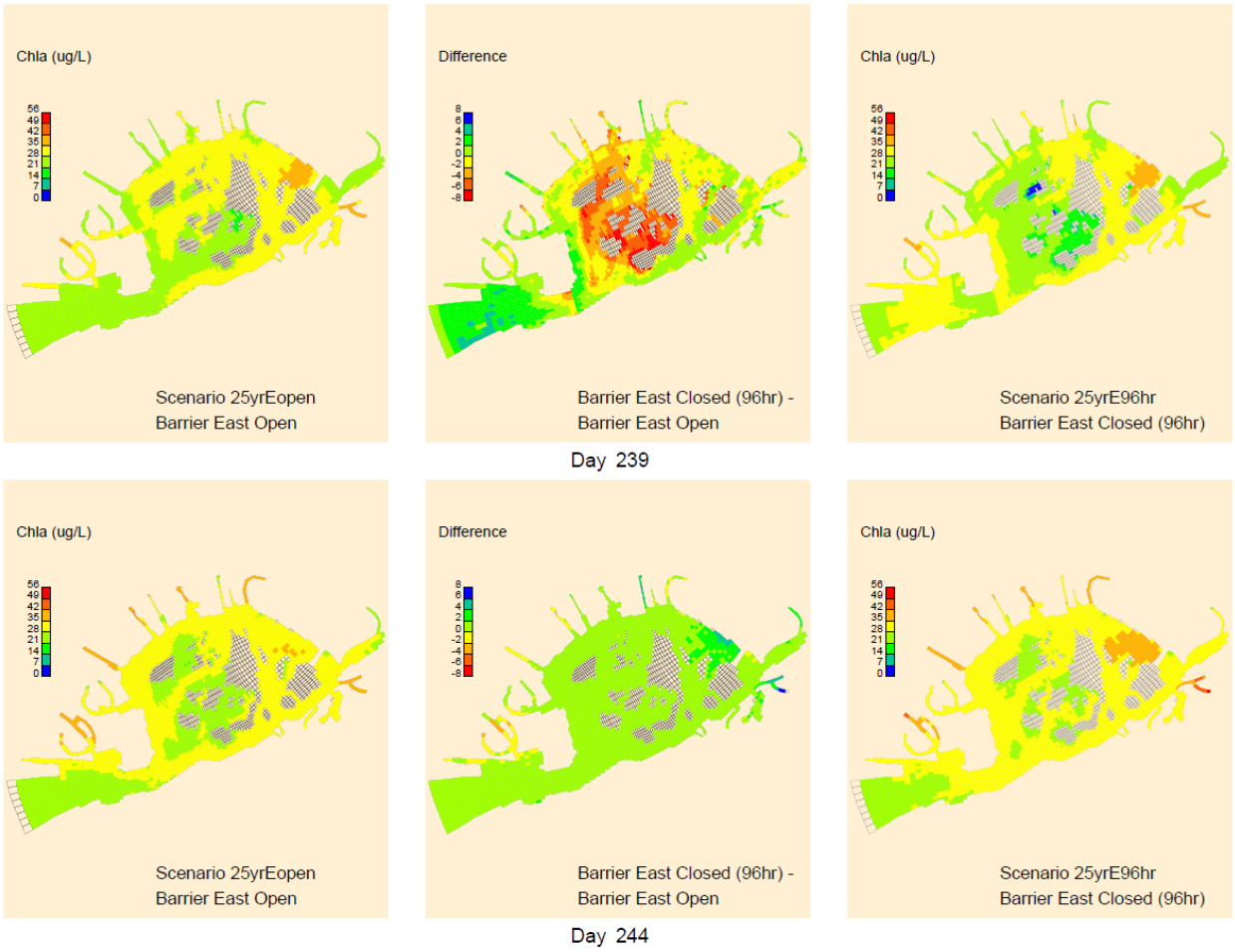


Figure 26. Comparison of Calculated Surface Chlorophyll for the Eastern Barrier Open and 96 hour Eastern Barrier Closed, 1-in-25 Year Scenarios During and After Storm Surge Barrier Gate Closure.

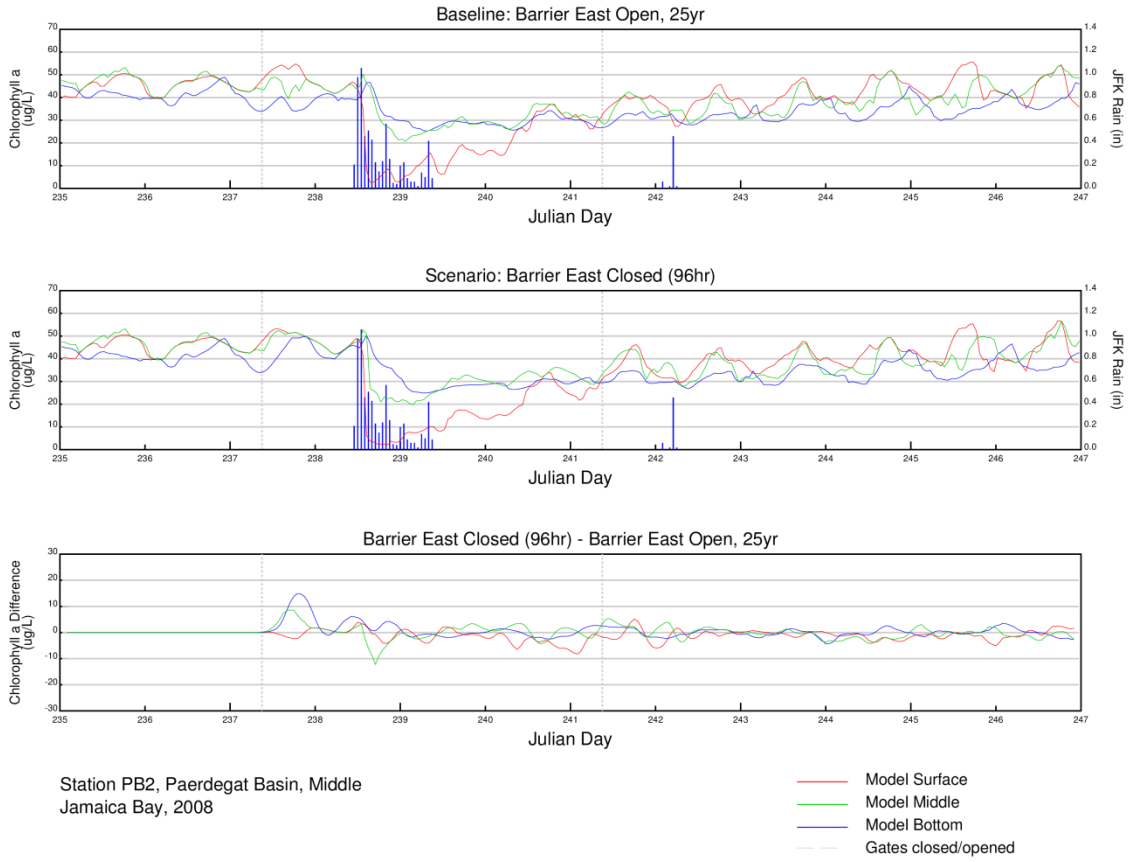


Figure 27. Computed Time-Series for Phytoplankton Chlorophyll in Mid-Paerdegat Basin for the 1-in-25 Year Storm Event with a 96 Hour Closure of the Eastern Storm Surge Barrier.

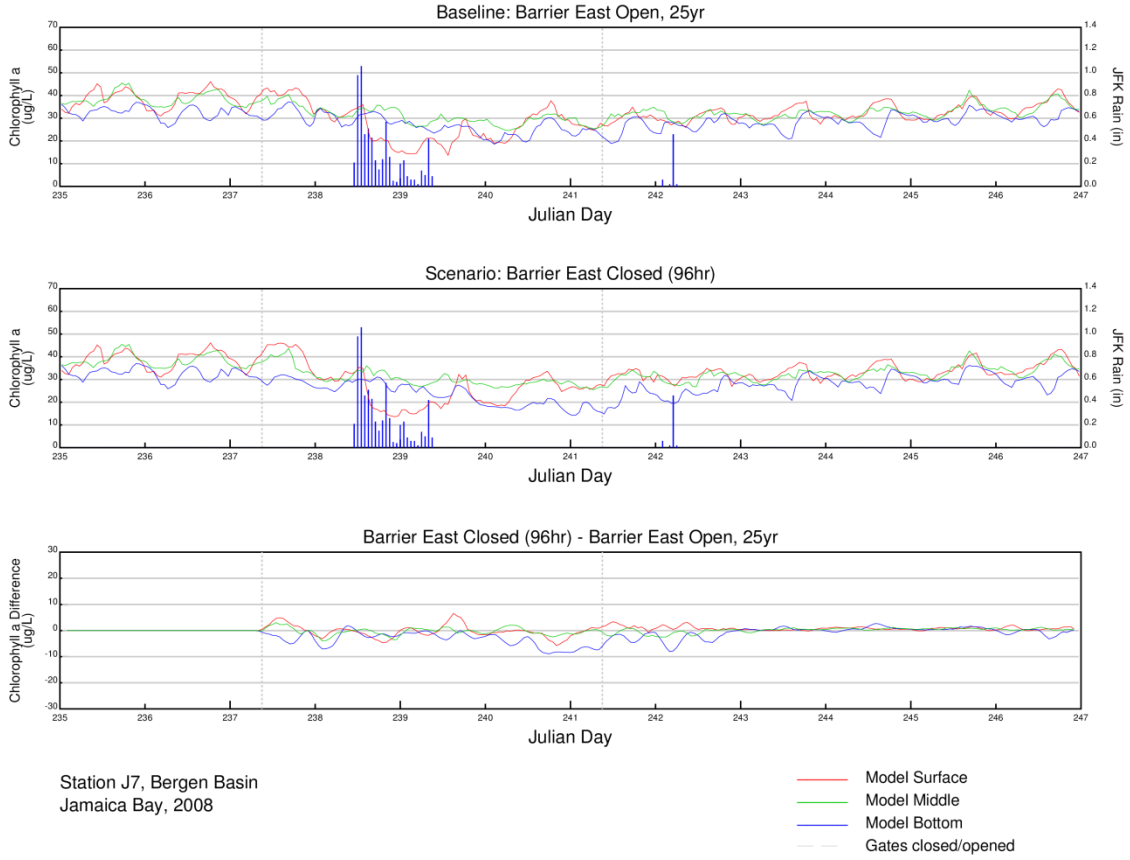


Figure 28. Computed Time-Series for Phytoplankton Chlorophyll at Harbor Survey Station J7 Near Bergen Basin for the 1-in-25 Year Storm Event with a 96 Hour Closure of the Eastern Storm Surge Barrier.

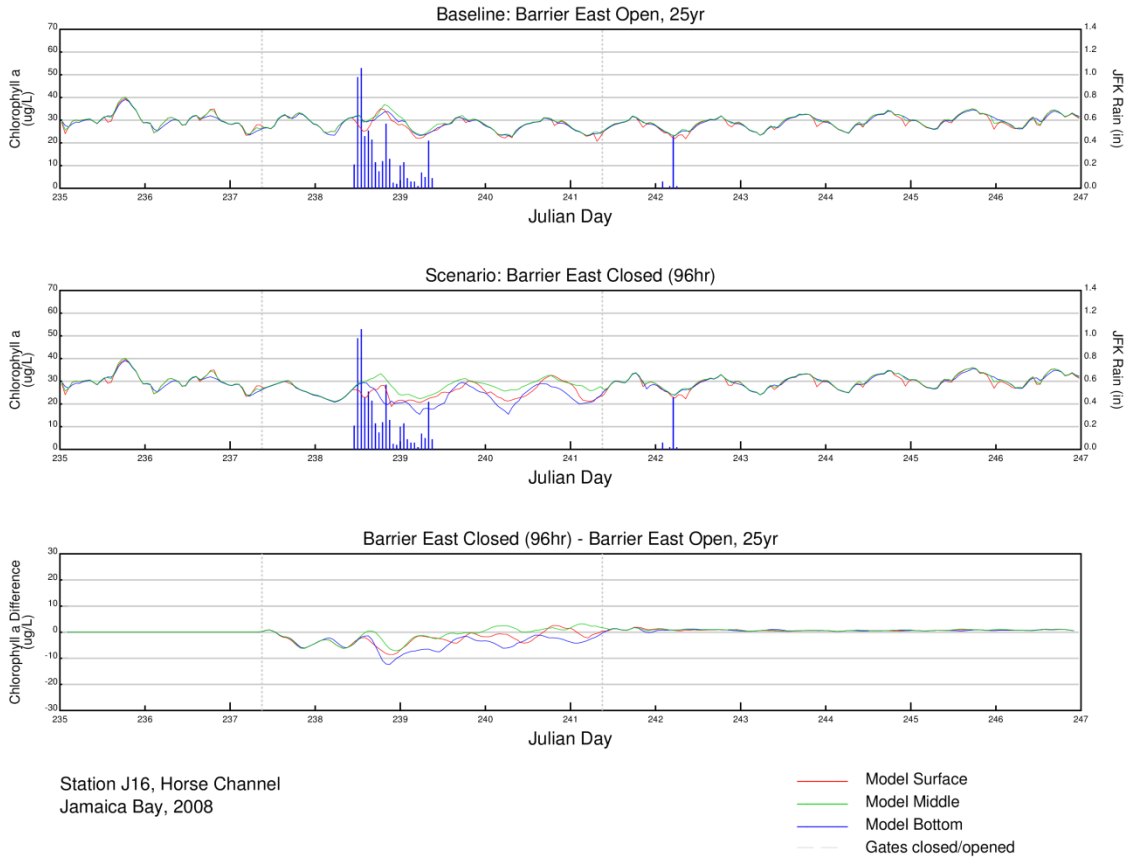


Figure 29. Computed Time-Series for Phytoplankton Chlorophyll at Harbor Survey Station J16 in Horse Channel for the 1-in-25 Year Storm Event with a 96 Hour Closure of the Eastern Storm Surge Barrier.

Dissolved Oxygen

DO is the one constituent that showed the greatest impact from the closure of the storm surge gates. With increasing time of gate closure the area where calculated DO concentrations were less than 3.0 mg/L increased significantly, although there was also a rapid recovery in DO concentrations once the gates were re-opened. Figure 30 presents the area of DO attainment for the chronic standard (daily average DO greater than 4.8 mg/L) and Figure 31 presents the attainment of the acute standard (DO never less than 3.0 mg/L) comparing the Eastern Gate Open scenario (left-hand panels) to the Eastern Gate Closed 48 hour scenario (right-hand panels), 1-in-25 year condition for the last day of gate closure (day 241) and a few days after the gate being re-opened (day 244). The middle panels presents the concentration differences between the two scenarios. Figures 32 and 33 show similar results but for the 96 hour, 1-in-25 year condition. Day 241 shows the maximum extent of standards violation that results from low DO concentrations and by day 244, three days after re-opening, the extent of bottom water DO non-attainment begins to show recovery.

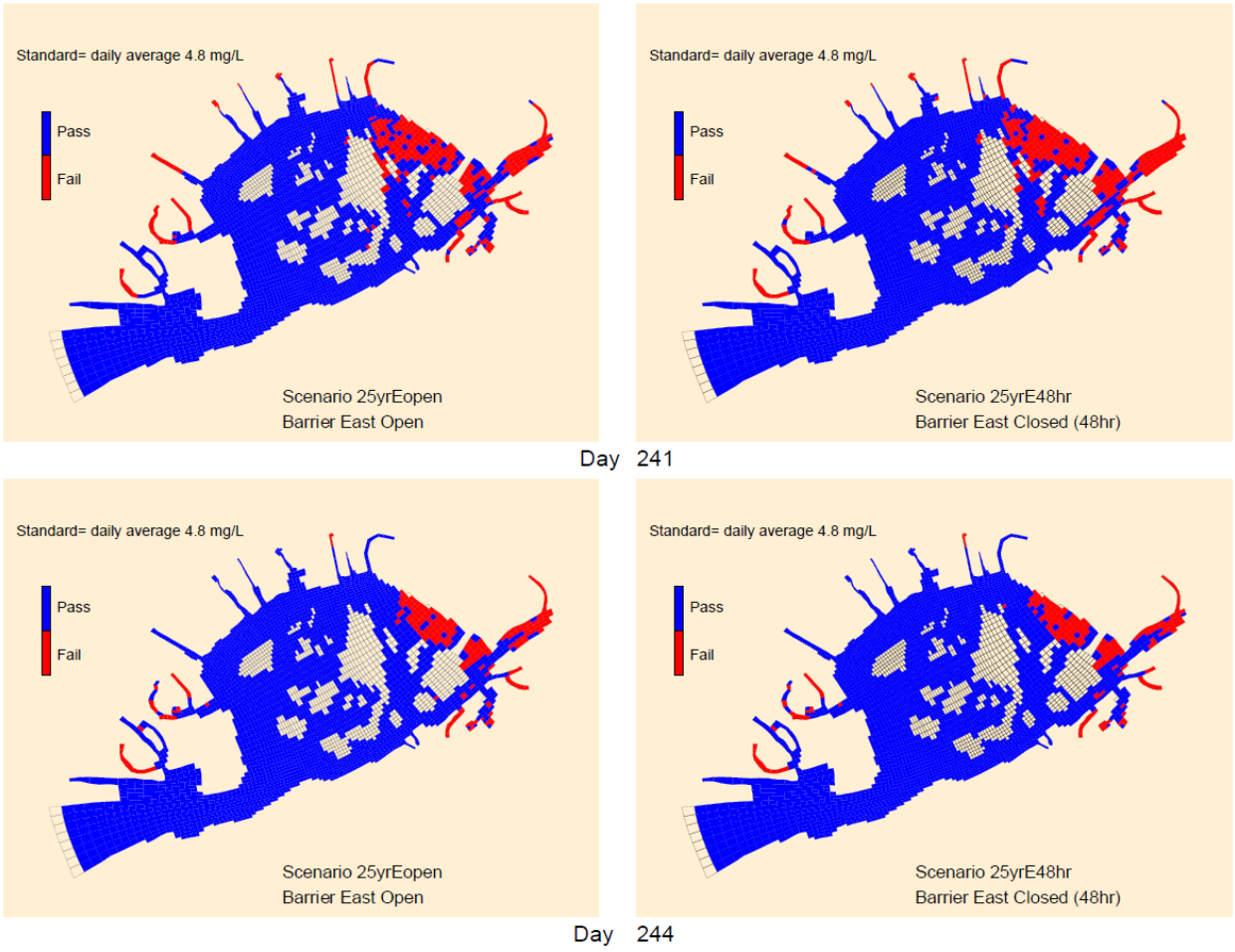


Figure 30. Comparison of Bottom Dissolved Oxygen Model Calculated Attainment of the Chronic Criterion for the Eastern Barrier Open and 48 hour Eastern Barrier Closed, 1-in-25 Year Scenarios.

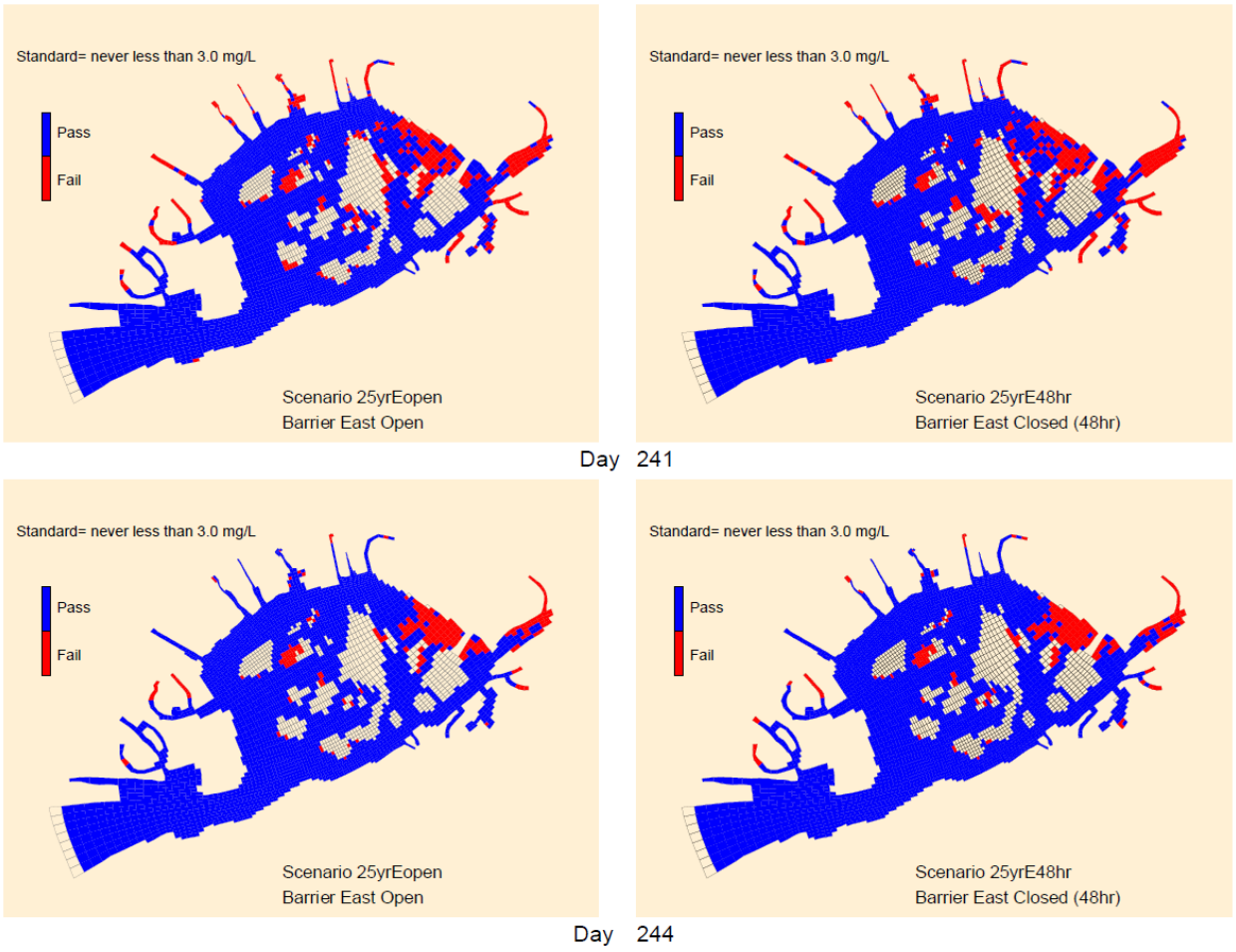


Figure 31. Comparison of Bottom Dissolved Oxygen Model Calculated Attainment of the Acute Criterion for the Eastern Barrier Open and 48 hour Eastern Barrier Closed, 1-in-25 Year Scenarios.

For the 48 hour gate closure conditions, the results for the chronic and acute criteria show minimal changes in the area of non-attainment of either standard when the gates are closed. Upon re-opening the spatial extent of non-attainment for both criteria are virtually the same. As shown in Figure 33, the spatial extent of non-attainment is much larger for the 96 hour closure. However, it is important to note, that the recovery of bottom DO attainment for both criteria is very rapid after the storm surge barrier gates are re-opened. It is also worth noting, that a 96 hour closure is probably not necessary and may be extremely conservative, if the barrier design were to include a back-up or alternative power source. Furthermore, both the 48 hour and 96 hour closure runs did not include storm related wind that would likely result in an increase in the surface water-air exchange (i.e., increased reaeration) and an increase in vertical mixing, which in turn would increase the exchange of surface water oxygen with oxygen deficient bottom waters. Therefore, the results presented in Figures 32 and 33 represent a very conservative estimate or worst-case conditions. In addition, it is important to note that this evaluation of non-attainment is for the bottom-most layer of the water column and for the current JEM grid, this represents only the lower 3 percent of the water column depth.

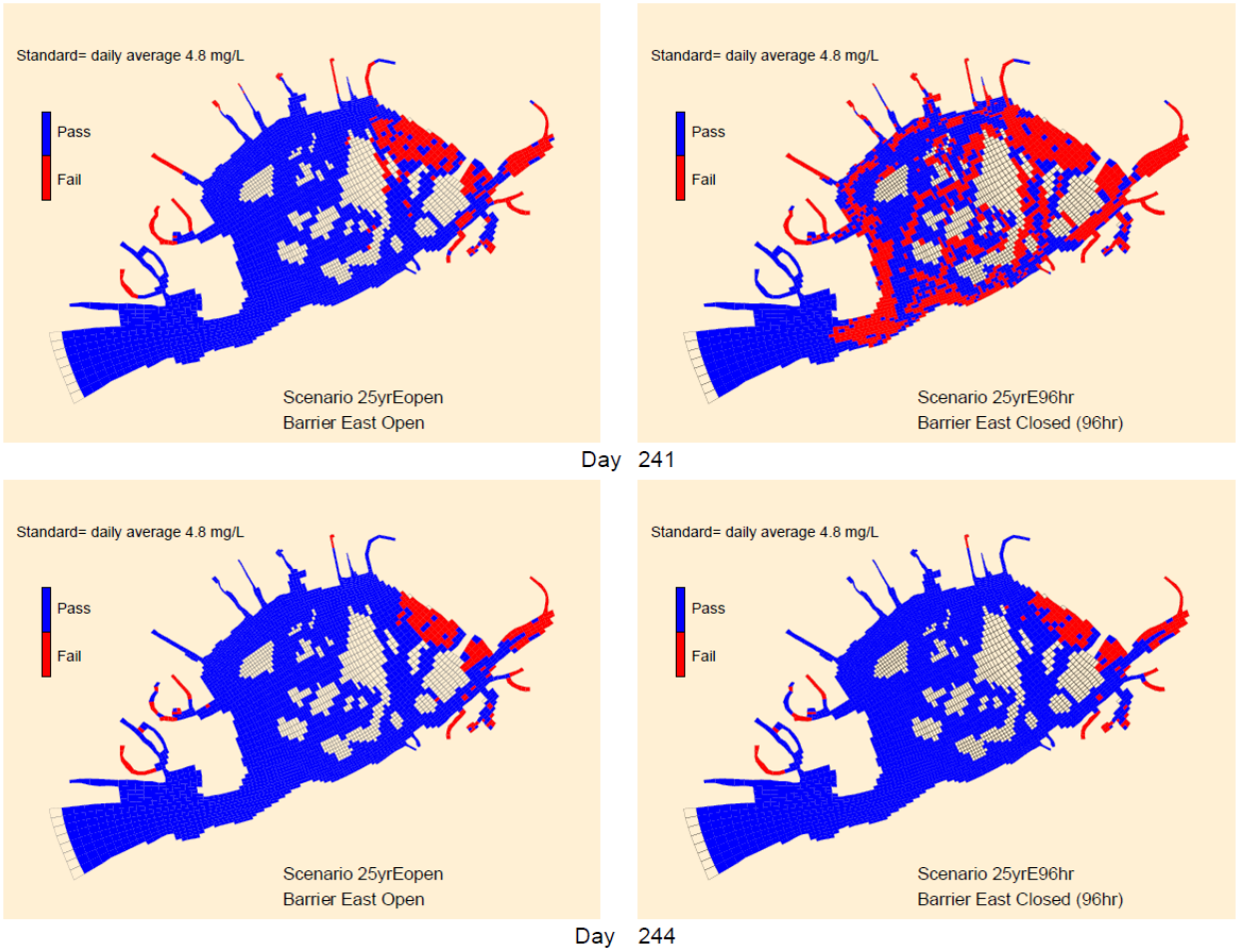


Figure 32. Comparison of Bottom Dissolved Oxygen Model Calculated Attainment of the Chronic Criterion for the Eastern Barrier Open and 96 hour Eastern Barrier Closed, 1-in-25 Year Scenarios.

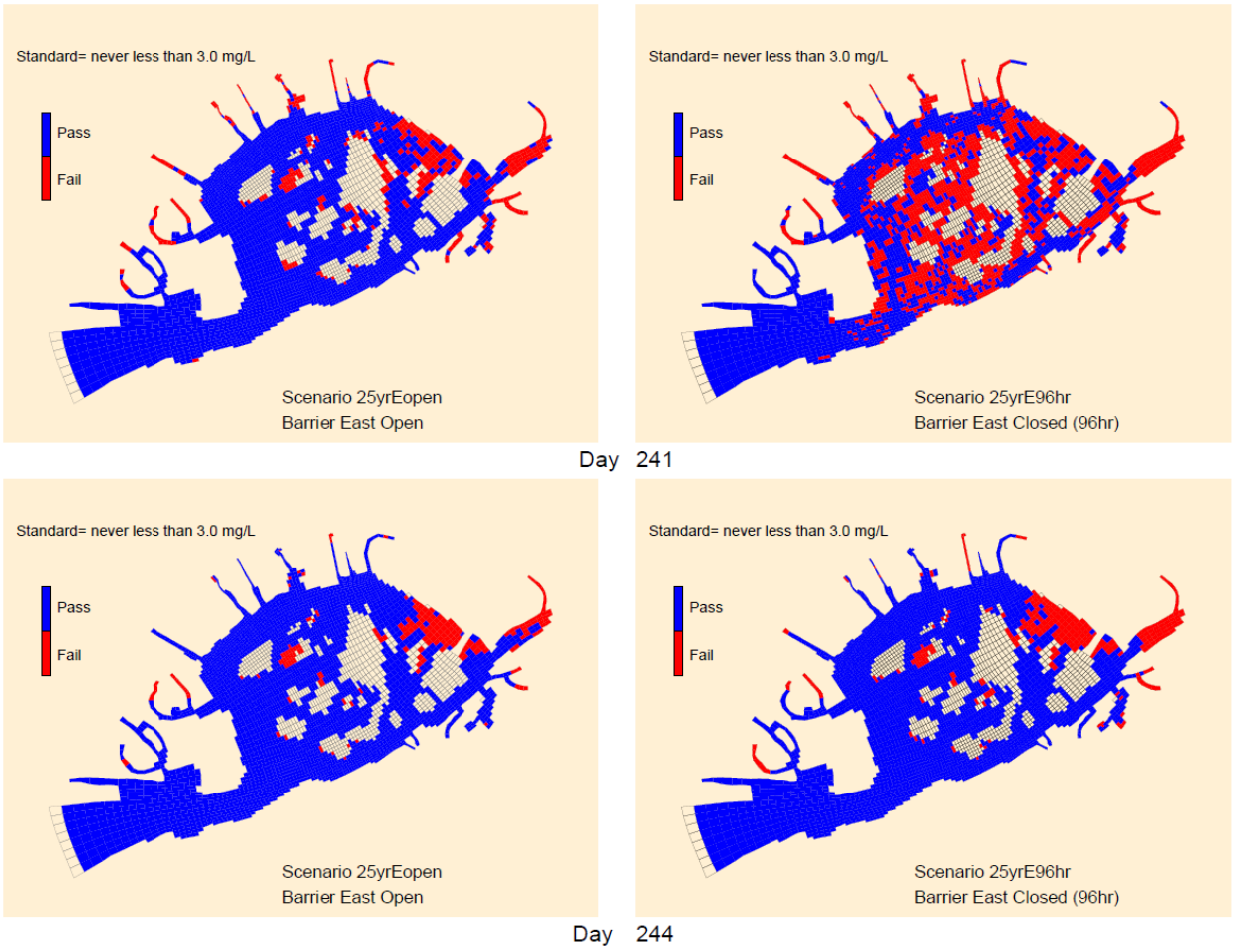


Figure 33. Comparison Of Bottom Dissolved Oxygen Model Calculated Attainment of the Acute Criterion for the Eastern Barrier Open and 96 hour Eastern Barrier Closed, 1-in-25 Year Scenarios.

Figure 34 presents a spatial plot of acute DO attainment for the 1-in-25 year, 96 hour closure for the Eastern storm surge barrier as a function of total water column depth. In this plot, attainment of the acute standard is evaluated in each vertical layer of the water column (for JEM the water column is divided into 10 depth layers). The plot shows the number of vertical layers in attainment. As can be seen, on day 241 of the 96 hour closure scenario, the model indicates that, in most of the bay, 9 or 10 layers of the water column are in attainment. In the case where 9 layers are in attainment it implies that only the bottom layer (i.e., the lower 3 percent of the water column) is in non-attainment. There are, however, several model grid cells, in the shallower interior sections of the bay, where up to 4 layers, corresponding to the bottom 25% of the water column, are not in attainment, meaning they do not meet water quality standards for dissolved oxygen set by the State of New York. Day 244 begins to show the fairly rapid response of the Bay's water coming back into attainment once the gates are re-opened. The recovery response is, however, somewhat delayed in Thurston Basin, likely due to the longer residence time in this tributary of the Bay.

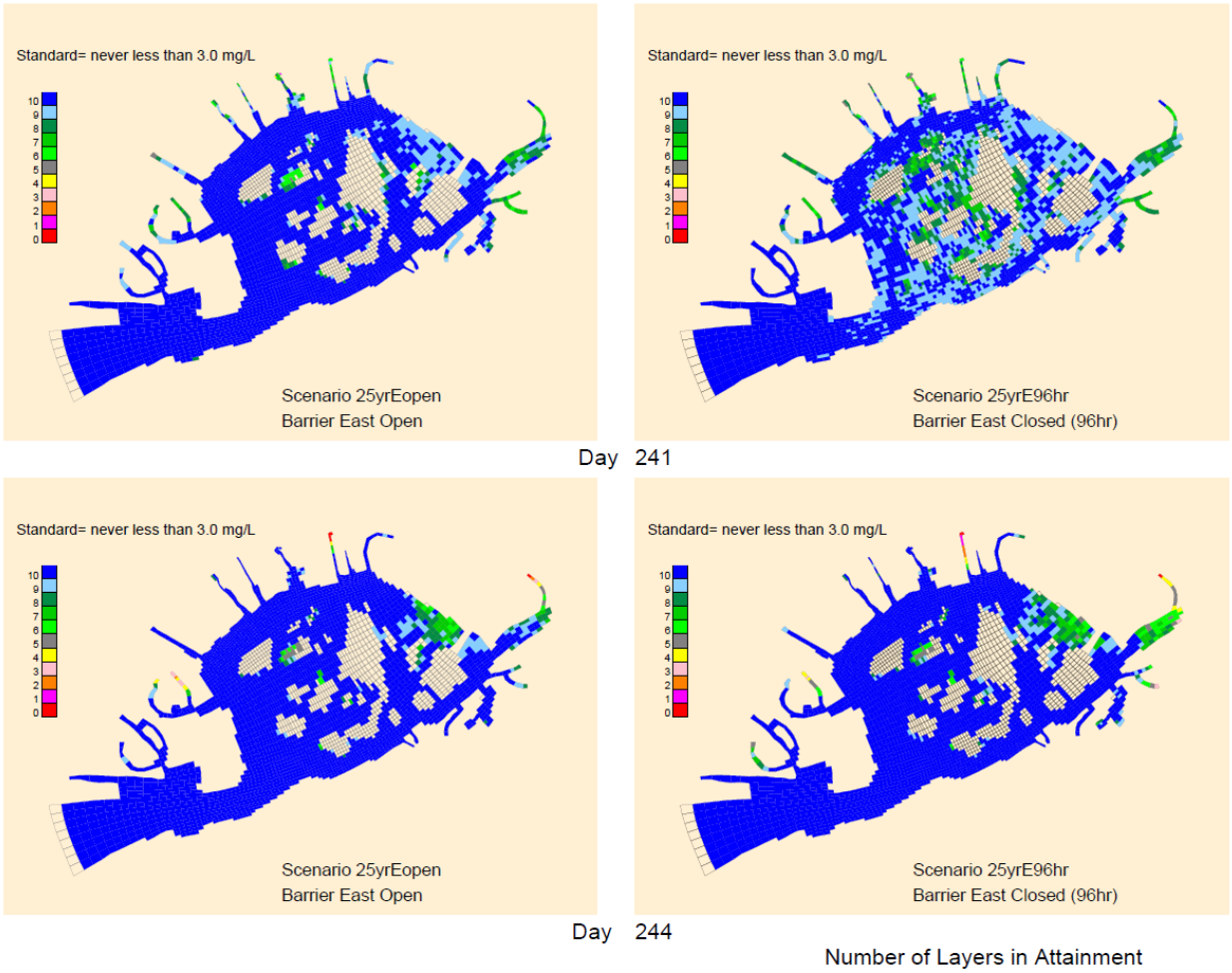


Figure 34. Comparison of Dissolved Oxygen Model Calculated Attainment of the Acute Criterion for the Entire Water Column for the Eastern Barrier Open and 96 hour Eastern Barrier Closed, 1-in-25 Year Scenarios

The processes that affect bottom water DO are influenced by a number of factors, including horizontal water movement due to tides, vertical stratification, bottom water algal respiration and sediment oxygen demand (SOD). The closure of the gates and the freshwater entering the Bay, as a result of the storm event, exert some degree of influence each of these factors. With the gates closed horizontal water movement is greatly reduced and this in turn acts to reduce vertical mixing, since tidal velocities influence vertical shear stresses which influences vertical mixing. Freshwater entering the Bay, due to the storm event, tends to remain on the surface of the water column and tends to increase vertical stratification and reduce vertical mixing. Reductions in vertical mixing tend to exacerbate the influence of bottom water algal respiration and SOD in lowering bottom water DO. Once the gates are re-opened and freshwater inputs decline, vertical mixing tends to increase and enhances the transfer of surface rich oxygenated waters to the bottom waters deficient in DO. Time-series figures illustrating the processes are presented in Figures 35 through 37 for a few model cells in the Bay. These figures show computed vertical mixing coefficients between layers 1 and 2 (surface) and layers 9 and 10 (bottom) of the water column. Minimum vertical mixing is molecular mixing (10^{-6} m^2/sec), while high vertical mixing is on the order of 10^{-3} to 10^{-2} m^2/sec. Also shown on these plots are time-series plots of salinity for a number of layers in the water column to help visualize the vertical stratification. Finally,

time-series plots of surface, mid-depth, and bottom DO are presented to illustrate that the low DO concentrations are mainly limited to the bottom waters of the Bay.

Figure 35 presents the time-series results for a model cell located in Island Channel, just west of Ruffle Bay. Before gate closure and just before the rainfall event both surface and bottom water mixing is fairly high, although the mixing in the surface layer is highly variable. After the precipitation event, the surface and bottom water mixing coefficients decrease to near molecular levels. Approximately 24 hours after the rainfall ends, the surface mixing increases to near pre-closure, pre-storm levels; however, the bottom layer mixing remains at or near molecular levels (i.e., virtually no mixing). Vertical mixing increases immediately after the gates are re-opened and tidal action begins. As can be seen in the salinity time-series, a vertical gradient develops through the course of the storm event and there is a strong vertical gradient in salinity, which begins to attenuate after the gates are re-opened. Finally, water column DO is fairly uniform throughout the water column before gate closure, but begins to stratify slightly immediately after closure. The bottom water DO declines more significantly and more rapidly after the rainfall event when vertical mixing, and in particular, in the bottom layers of the water column decreases to molecular values. Once the gates are re-opened and vertical mixing increases in all levels of the water column, the vertical gradient in DO dissipates and bottom water DO concentrations rapidly increase.

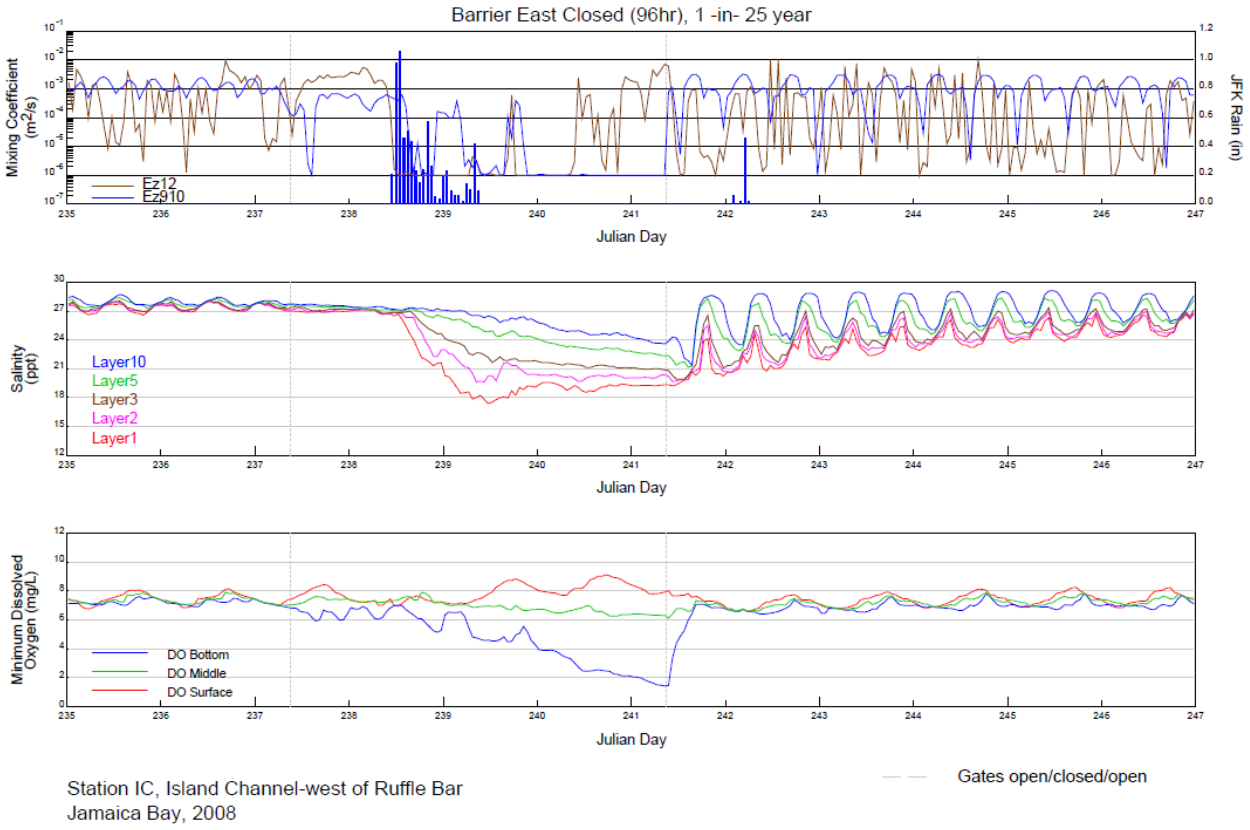


Figure 35. Time-Series Results for Island Channel, West of Ruffle Bar, for the 1-in-25 Year, Eastern Barrier 96 Hour Closure Scenario.

A similar pattern of behavior can also be observed for a model cell located in Big Fishkill Channel, northwest of Ruffle Bar (Figure 36). With the closure of the gates, the bottom water mixing coefficient declines for a short period of time, which leads to the beginning of the decline in bottom water DO. The decline in vertical mixing is more pronounced after the rainfall event, which in turn results in a vertical gradient in salinity and a rapid decline in bottom water DO. One the gates are re-opened vertical mixing increases and the vertical gradients in salinity and DO disappear and bottom DO quickly recovers to levels greater than 5 mg/L.

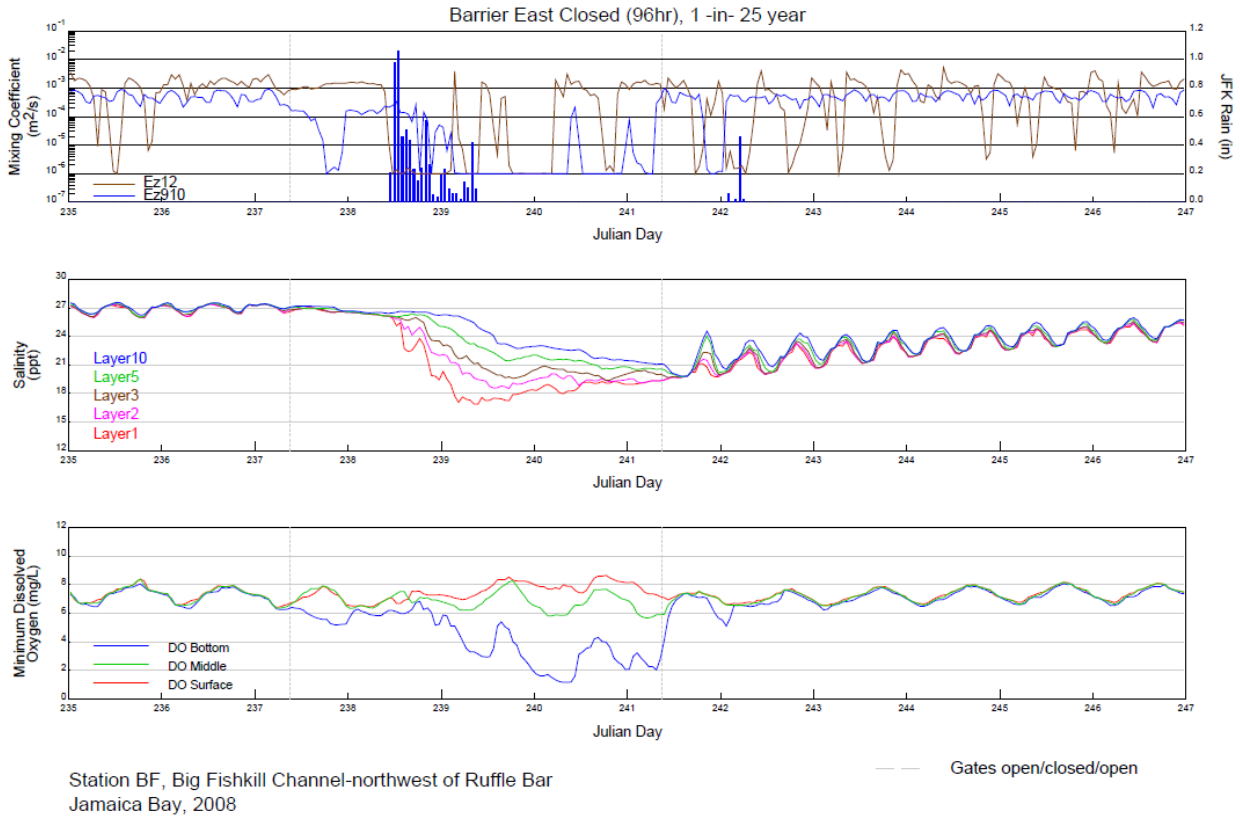


Figure 36. Time-Series Results for Big Fishkill Channel, Northwest of Ruffle Bar, for the 1-in-25 Year, Eastern Barrier 96 Hour Closure Scenario.

Figure 37 presents time-series results for a model cell located in Grassy Bay. While the results are somewhat similar in behavior as observed in Figures 30 and 31, there are some differences. In particular, vertical mixing between the bottom two layers of the water column are already low for some period of time before gate closure. This, in part, explains why the bottom water DO in this cell is already lower than the surface and mid-depth values. With the gate closure and the freshwater associated with the rainfall event, both surface and bottom mixing are significantly reduced and there is an increase in salinity stratification and bottom water DO declines, but at a slower rate than shown for the bottom layers in Island Channel and Big Fishkill Channel. This may result since the Grassy Bay segment is deeper than the other two cells and, therefore, the effects of bottom SOD take longer to exert on a greater depth of water. Also the decline in bottom water DO continues for a period of time, about 2-3 days, after the gates are re-opened. This, in part, may be due to vertical mixing between the bottom layers of the model taking longer to return to pre-closure levels, and perhaps in part due to a greater volume of low DO that needs to be re-oxygenated in the Grassy Bay area.

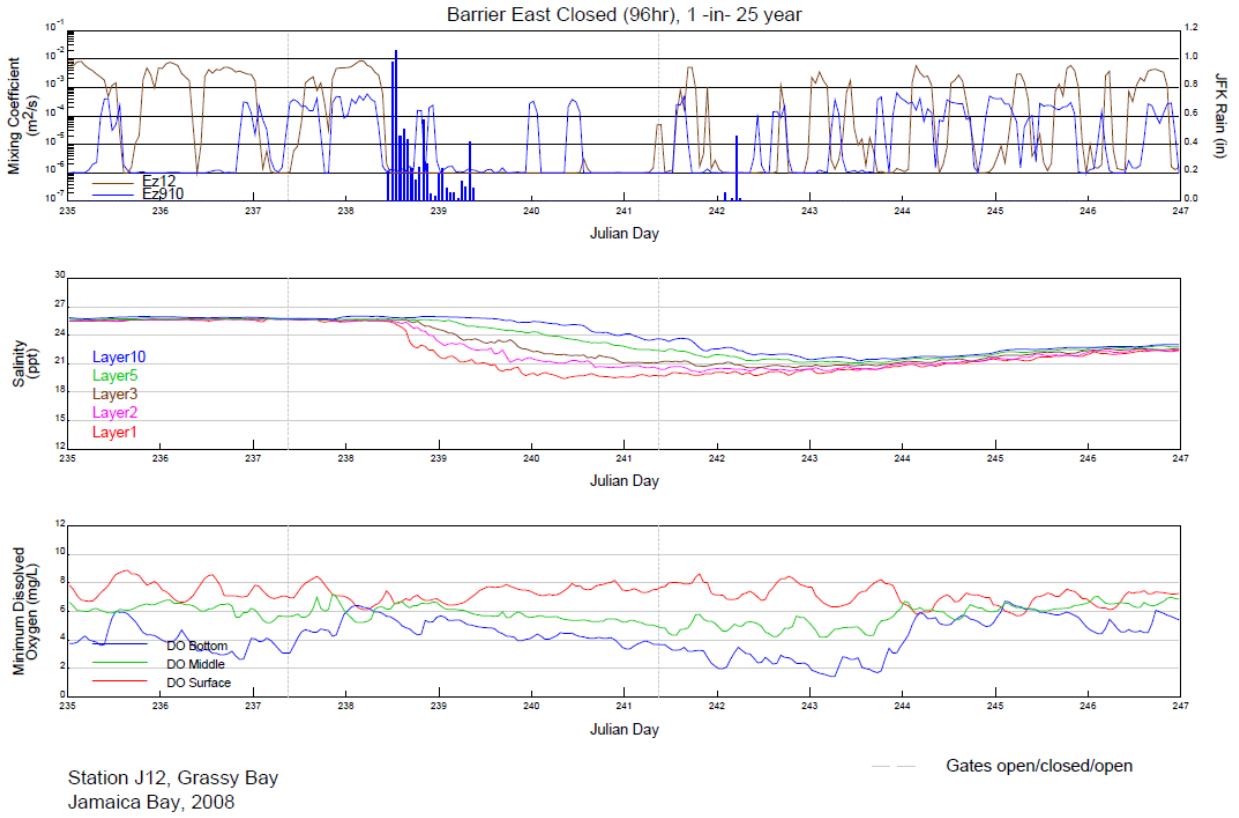


Figure 37. Time-Series Results for Grassy Bay, for the 1-in-25 Year, Eastern Barrier 96 Hour Closure Scenario.

DISCUSSION AND CONCLUSIONS

Modeling was conducted to assess 14 scenarios related to the possible construction of storm surge barriers across the Rockaway Inlet to protect portions of Brooklyn and Queens against flooding during storm surges. The analysis included comparisons of Baseline (i.e., current conditions – no barrier) versus the East and West Barrier options, under different operating conditions, always open versus period of the tidal gates being closed and comparisons of both long-term and short-term effects. For the Baseline versus storm surge barrier gates always open, the long-term effects computed by the model showed a slight reduction in the tidal range within the bay. Changes to salinity, pathogens, nutrients, chlorophyll, and DO concentrations were generally very small. However, the changes were more pronounced over shorter time periods, such as a day. The biggest changes, based on a daily timeframe, were observed in bottom water DO and were generally restricted to Grassy Bay and channels located between the marsh islands in the center of the Bay. It should be noted, though, that the bottom water DO in the channels in the mid-Bay region remained at levels that would not be expected to impact aquatic life. However, the decreases in bottom water DO in Grassy Bay resulting from the construction of the barriers would likely cause additional stresses on aquatic life that might occupy the bottom waters of Grassy Bay.

Short term impacts, introduced by storm surge barrier gate closures, on most water quality parameters considered in the analyses (pathogenic bacteria, salinity, total nitrogen and ammonium nitrogen) were generally small and short lived and would not be expected to adversely affect aquatic life within the Bay. However, short-term impacts of closing the storm surge barrier gates were calculated to have a potentially greater impact on aquatic habitat, especially as related to DO concentrations. Model results indicate that the 96 hour closure results in lower levels of DO and higher levels of non-attainment for the chronic and acute DO criteria than are computed non-closure or for a 48 hour closure, particularly in the bottom waters. Concentrations of bottom water DO are reduced to between 1.7 and 2.0 mg/L in the open waters of the Bay for the 1-in-25 year storm event for a 96 hour closure. This represents a reduction of between 0.5 and up to 4 mg/L at worst relative to baseline conditions. It is important to note, however, that the JEM consists of 10 vertical layers and the non-attainment is typically limited to the bottom one or two layers of the water column, which represent between 3 and 8 percent of the total vertical water column. Reductions in DO in other portions of the water column are much less severe, on the order of a few tenths of a mg/L on average and resulting levels are capable of sustaining aquatic life. In order to minimize the impact of gate closures on DO concentrations it is recommended to minimize the time the storm surge barrier gates are closed. While mobile aquatic life could potentially escape the low DO concentrations by migrating to the upper levels of the water column where DO concentrations are adequate to support life, benthic organisms would not be able to do so and would be more susceptible to DO stress and potential increases in mortality.

It should be noted that this modeling analysis should be considered a screening level analysis, and additional modeling should be conducted should the decision be made to move forward with this project. Currently, the model segmentation is not fine enough to match the size of the gates in the storm surge barriers, although it is believed that this is not a significant detriment in the current analysis. Additionally, the effect of wind and/or sediment resuspension was not considered. These factors can potentially impact dissolved oxygen, via increasing air-water reaeration and vertical mixing, and chlorophyll concentrations by potentially reducing water column clarity.

Consequently, additional model refinement, modeling and analysis should be conducted by the USACE prior to design of the storm surge barrier; to better quantify and conclusively address any possible impacts from construction and operation of a storm surge barrier on water quality and fish & wildlife species and their habitats in the Bay.

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APPENDIX A

ACRONYMS

Acronym	Definition
AEP	annual exceedance probability
cfu/100 mL	colony forming units per 100 milliliters
chla	chlorophyll-a
cm	centimeters
CSOs	combined sewer overflows
day^{-1}	per day
DO	dissolved oxygen
JEM	Jamaica Bay Eutrophication Model
JES	Jamaica Bay Eutrophication Study
JFK	John F. Kennedy (airport)
km^2	square kilometers
MEG	Model Evaluation Group
MG	million gallons
mg/L	milligrams per liter
mg N/L	milligrams of nitrogen per liter
m^2/sec	square meters per second
NH_3	ammonia nitrogen
NH_4	ammonium nitrogen
NYCDEP	New York City Department of Environmental Protection
NYSDEC	New York State Department of Environmental Conservation
ppt	parts per thousand
SWOs	Separate (or storm) sewer overflows
TN	total nitrogen
µg/L	micrograms per liter
USACE	United States Army Corps of Engineers
WWTP	waste water treatment plant

APPENDIX B

RESPONSE TO COMMENTS RECEIVED FROM NYSDOS

NY Department of State Comments on Water Quality Modeling for Rockaway and Jamaica Bay Reformulation Study Draft Hurricane Sandy General Reevaluation Report and Environmental Impact Statement (HSGRR/EIS)

Last updated on October 25, 2017

As a follow-up to the official comments on the HSGRR/EIS, the U.S. Army Corps of Engineers (USACE or Corps) requested additional detail from the commenting agencies (NYSDEC, NYSDOS, and NYC Planning in NYSDEC letter, dated December 14, 2016). Additional detail was requested regarding the water quality modelling objectives that New York State envision and which data they would like to see developed for better understanding in addressing impacts to Jamaica Bay from the presence and operation of a surge barrier component. The following comments were prepared by the New York Department of State (DOS) to help clarify the remaining concerns and data/impact questions on how a proposed storm surge barrier at the entrance to Jamaica Bay could impact water quality in the bay. The modelers who conducted the Jamaica Bay Eutrophic Model (JEM), which was used to analyze impacts in the Draft HSGRR/EIS to water quality for Jamaica Bay, has provided input to the following responses to address DOS comments. Additionally, the memorandum describing the modeling has been edited to add more detail and better address the issues raised in the below comments.

DOS comments on the JEM modeling with responses from USACE:

1. DOS Comment: *All water quality modelling should compare data across the three barrier alignments* as presented in the Alternatives Milestone dated November, 2014. The JEM model at present includes an eastern and a western barrier option, which appear to correspond to the alignments originally presented as barrier alignments numbers 1 and 2. However, the westernmost option, alignment 3, was dropped from further consideration on a cost basis. We see justification for inclusion of this alignment due to the fact that structural shoreline measures would be significantly reduced with this option and it may be a better choice for the environment. The combined costs for full build out of necessary connecting structures may ultimately be less.

   Response: Under USACE's new SMART Planning paradigm, the level of detailed analysis required to make a decision is analyzed, not more. This is to reduce the overall cost and time required to complete a feasibility study in line with priorities of Congress and multiple Executive branches of the United States. Engineering, economic, and public engagement tools were used to screen the barrier alignments down to a smaller number of feasible alignments, which were then analyzed in more detail for environmental impacts in order to better understand how to avoid, minimize, and mitigate for any potential range of impacts resulting from a storm surge barrier. This approach is consistent with SMART Planning and NEPA. As part of the initial screening, the water depth requirements for alignment 3 made it less cost effective than alignments 1 and 2, even when added tie-in structures along the shoreline were considered. Further information regarding the preliminary screening will be added to the Final HSGRR/EIS to expound upon this screening.

2. DOS Comment: *The scope and complexity of the modelling should be improved to reflect the dynamics of an integrated coastal system.* [*Apply a combined rainfall-runoff + surge model for*

the barrier options, not just a surge only model.[1]] The JEM model and its results presented thus far is not sufficiently complex or rigorous to capture the effects of all of the factors that would be realistically and significantly be influencing the system at any given time, duration, or given storm scenario. Thus, to understand water quality changes, there is a need to apply a more holistic approach – i.e. attempt to model the flows entering the basin from the upland in addition to, and simultaneously with, the changes in tidal amplitude as well as the hydrodynamic circulation within the bay. To be meaningful for analysis, the modelling should ideally be a very iterative process conducted for various storm surge and rainfall scenarios and for each alignment.

Response: With respect to the first part of this comment, NYDOS is misinformed, the modeling that HDR performed did include the watershed modeling. The report discusses the application of the InfoWorks watershed/sewershed model to Jamaica Bay (pages 10-12 of the original report) and provides a summary of the results of the modeling effort in Figure 8 (page 12 of the report). The second part of this comment points out that HDR did not look at more than one storm event. This is correct, HDR focused on the impact of a surge and storm on dissolved oxygen during this critical summer period in order to capture a worst case scenario effect for dissolved oxygen, the water quality parameter of biggest concern previously. However, a similar surge and storm event would have impacts on water quality if the event occurred during the earlier part or later part of the recreational season (May-October) when water temperatures are cooler and death rates for pathogens are lower. In order to address this comment and potential pathogen impact during cooler seasons, the study team will look at another storm event during a different season of the year, either as part of the preparation of the Final HSGRR/EIS, or during future study of a storm surge barrier at Jamaica Bay under the New York and New Jersey Harbor and Tributaries (NYNJHATs) Study. However, additional modeling efforts proposed are subject to funding constraints and the study team may alternatively pursue other ways to assess the range of potential impacts.

3. DOS Comment: *The model should also be run to sufficiently demonstrate multiple barrier closure scenarios.* We would like to be able to understand the influence on various water quality parameters of increased residence times of pollutants (sediments, nutrients, chemicals, pathogens) held within the Jamaica Bay during a barrier closure of a given duration and be able to anticipate the circulation and flushing of pollutants post-opening of the gates. Results presented as "small" changes or "minimal" effects should be couched in understanding the potential significance of impacts of seeming small deviations from "normal" and/ or temporal changes on aquatic organisms and the habitats which support them. Scientific support should be provided for any such statements determining minimal effects.

   Response: Since the JEM model does not include suspended sediments nor (toxic) chemicals, HDR could not address those water quality variables. As the Corps studies further impacts

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[1] ***SUPPORT FROM THE LITERATURE AND RESEARCH [For expanding scope and complexity of the modelling] –***

Christian, Jason. Zheng Fang, Torres et al. (2014) American Society of Civil Engineers.

Modeling the Hydraulic Effectiveness of a Proposed Storm Surge Barrier System for the Houston Ship Channel during Hurricane Events. DOI: 10.1061/ (ASCE) NH.1527-6996.0000150.

Mooyaart, Leslie F., Sebastian Jonkman, de Vries, et al. (2014) Coastal Engineering. *Storm Surge Barrier: Overview and Design Considerations*.

Orton, Philip. https://philiporton.com/projects/

associated with the barrier under the NYNJHATs study, the team will consider using the MIKE Suite model (with in-house capacity from Wilmington District) to assess accretion/ trapping of sediments and/ or debris both during barrier operation and also when gates are open. With respect to HDR's comments about "small" or "minimal" changes, these largely refer to an annual basis. Text has been edited to explain this context more. Terms such as "small" or "minimal" effects are based on professional judgment by subject matter experts, and an effort to explain them in the context of the system has been made for the revised memo described above. Effects are larger when looking at the storm event and closure of the proposed barrier and these effects are discussed on pages 21-46 for pathogens, phytoplankton biomass (chlorophyll-a) and dissolved oxygen.

4. DOS Comment: *Additional study/ models should isolate those areas of the bay which are known to be more water quality impaired relative to others during baseline or "normal" conditions. These "hot spots" should be selected for finer scale water quality modelling for each of the barrier and storm scenarios*.

   Response: We believe that the spatial resolution of the current JEM model is sufficient for the purposes of this analysis. The spatial resolution used in for JEM in this analysis (Figure 6) is about a factor of 4.5 times greater than used in the original JEM (Figure 4). Model segment lengths range from about 100 m to 250 m in the open waters of the bay, to as small as 60-70 m in the tributaries. Given the tidal mixing that occurs in the open waters of the Bay, it would not be expected that water quality constituents would vary significantly within spatial areas of 0.025 km^2 (a model segment of 100 m by 250 m).

5. DOS Comment: *Additional study/ modelling should attempt identify extreme or "worst-case scenarios" to fairly understand the risk and level of exposure which may occur. This should include scenarios depicting a potential for operational failure of gates to reopen and a potential failure of WWTPs during a closure.*

   Response: To a certain degree, HDR did try and look at a worst case scenario with the 96-hr closure, i.e., no power to re-open gates. NYDOS does, however, raise a valid point about WWTP failure, however current efforts underway are targeting addressing this risk. The City is in the midst of implementing a resiliency plan for its wastewater facilities. This plan was developed in 2013 and calls for the expenditure of $315 million in upgrades recommended to protect equipment. Therefore the risk of WWTP failure in the future without project condition is considered to be minimal.

6. DOS Comment: *The framework for the models should attempt to integrate with and complement any monitoring network already in place for Jamaica Bay.* "From the monitored parameters, the model will be in a continuously improving process …" Wilfried Michaelis. This will facilitate future water quality management, monitoring, modelling, and research.

   Response: As described in the report (pages 7 and 8), JEM has been calibrated and validated against the NYCDEP JES data set (1995/1996) and NYCDEP Harbor Survey data (2015).

7. DOS Comment: *Areas for further explanation or development within the model report* – Any assumptions should be clearly stated; selected baseline condition (2008 rainfall year at JFK)

should be further explained and justified as the "typical" rainfall year and suitable for modelling future conditions; Model summary statistics should always be included in the report.

Response: The report has been modified to refer to an analysis of rainfall statistics performed for the NYCDEP by Halcrow|Hazen and Sawyer (Technical Note, Nov. 2011, 6 pages), which explain the background for identifying a "typical" rainfall for the area.

Specific data DOS would like the modelling to develop/ specific questions to answer:

8. *Existing condition, without any modeled barrier:*
    a. What water quality parameters have been collected, where have they been collected, and how well does this record document existing flow patterns and water quality conditions around the bay and at the mouth of the inlet?

Response: The report has been modified to describe station locations and the types of data used to calibrate/validate the JEM hydrodynamic and water quality models (pages 7 and 8). A full description of the calibration/validation analysis is contained in reports prepared by HydroQual, Inc. and HDR, Inc. for the NYCDEP (2002) and the Jamaica Bay Science and Resilience Institute (2017).

    b. What are the types, distribution and severity of existing water quality impairments in the bay and at the inlet mouth?

Response: This information (based on 1995/1996) data is contained in the original JEM calibration report prepared by HydroQual, Inc. (2002). More recent data can be found in reports prepared by the NYCDEP Harbor Survey Program
(http://www.nyc.gov/html/dep/html/harborwater/harborwater_quality_survey.shtml)

    c. How well does the operating model agree with existing flow patterns and the type and distribution of existing water quality impairments?

Response: See response to 1a above.

9. *Gates open main channel effects – Due to the confinement of the inlet channel by structural features of the surge barrier:*
    a. What is the change in flow pathways and flow velocity profiles across the inlet opening? In other words, is the flow "channelized" through the inlet in any way, and are there areas of relatively faster or slower flow due to the barrier structure?

Response: Yes, there are some changes in currents near the barriers. The major feature is that flows are restricted through the openings or gates in the barriers. Also, there is some reflection of tidal waters along the northern portion of the Rockaway Inlet away from the tidal barriers during a flood tide.

b. What is the change in velocity of tidal flows through the narrower openings, and is the increased velocity sufficient to affect sediment movement on the bed or in the water column?

Response: This question requires the development of a sediment transport model, such as the MIKE Suite model, which will be considered during future study of the storm surge barrier under the NYNJHATs Study.

c. Would the increased tidal flow velocities and/or the barrier structures have any effects on navigation access or small craft usage?

Response: This comment will be addressed under the NYNJHATs Study.

10. *Gates open, back bay effects:*
    a. As a result of changes in flow velocities and pathways through the barrier structures, are there any changes in hydrologic connectivity, flow velocities or interior flushing in other parts of the bay? What types of changes are there, where do they occur and how significant are they?

Response: Flow velocities are certainly reduced in the immediate area of the tidal barrier gates, and in the channelized areas of western Beach Channel and the southern portion of Island Channel. Flow velocities are slightly reduced in the interior portions of the Bay (North Channel, Grassy Bay, Grass Hassock Channel and Head of Bay), but overall, the general circulation in the interior portions of the bay are similar with or without the tidal barriers.

b. Are there any changes in water circulation near the bay shores as a result of changes in flow pathways and velocities through the barrier structures?

Response: Most of the changes in circulation occur in the vicinity of the tidal barriers, with some reductions in near shore velocities with the implementation of the tidal barriers.

11. *Gates closed, external effects:*
    With the surge barrier gates closed, what is the effect on surge water level to surrounding areas external to the barrier, in comparison to surge levels with no barrier? What is the extent of these effects? Assuming the effects attenuate with distance from the barrier, at what locations do they become negligible?

Response: These questions cannot be addressed with the present JEM as the potential areas of inundation, e.g., Rockaway peninsula overwash from the ocean side or inundation in Coney Island, are outside the JEM model domain. As the Corps studies further impacts associated with the barrier under the NYNJHATs study, the team will consider using the MIKE Suite model (with in-house capacity from Wilmington District) to assess external effects of the gate open and

closed on surge water level and water velocities to surrounding areas external to the barrier, in comparison to surge levels with no barrier and identify the extent of these effects; and identify the change in these effects at different distances from the barrier.

12. *Gates closed, internal effects, assuming a 3 day extra-tropical storm and advance closure time of 6 hours and post-storm opening delay of 6 hours, the gates could be closed 84 hours.*
    a. Is this a reasonable assumption for an extreme event in the project area? How often on average would an event occur that required closure longer that this? If appropriate for potential storms in this region, propose a longer time closure period for estimating water quality impacts.

Response: In choosing a closure time to analyze, HDR looked a 96-hr closure, which is greater than 84-hr closure mentioned here, and which was determined to be a worst case scenario. Future modeling and impact analysis to assess the storm surge barrier impacts will model a wider range of closures to establish a range of potential effects and once more detail is known on the operational and design parameters of the proposed barrier, the team will be able to address duration and likelihoods better. It is early yet to have those details available, which is why the worst case scenario was targeted instead.

    b. Based on an 84 hour closure, or longer time if reasonable based on consideration of potential storms in the region, how will water quality parameters in the vicinity of the inlet channel, interior of the bay, and shoreline areas change?

Response: The report discusses this question in the Results section (pages 13-45) - save for the exterior portion of the model or on the seaward side of the barrier. HDR would not have much faith on the seaward side of the barrier as it is too close to the boundary of the model and the focus of HDR's analysis is what is happening inside of the Bay.

    c. What are the types and sources of water contaminants and pathogens that will enter the bay during the period when surge barrier gates are closed? What effect will these contaminants and pathogens have on water quality, and where will those effects occur?

Response: Both of these questions were addressed in the report: sources - pages 11 and 12 and Figure 8 and impacts – pages 13-45 and Figures 9-37 and Tables 3-10 . The figures, tables and text in the report detail what, where and duration of impacts that might be expected.

    d. For locations around the bay where existing data and modeling indicate significant water quality impairments under existing conditions, how will water quality parameters change during an 84 hour (or longer) closure period?

Response: Again, the report provides this information (pages 21-45 and Figures 15-37).

13. *Gates opening post-storm:* Assuming an 84 hour (or longer) closure period and associated effects on water quality, would water quality parameters return to non-storm conditions throughout the bay? If so, what would be the rate of normalization, and which areas would have diminished water quality the longest.

    Response: This question was also addressed in the report (pages 21-45 and Figures 15-37).

Other information needs that would benefit from further study –

14. *Re: Marshlands* - Separate analyses should superimpose the tidal elevation change projections from the modelling on the marsh islands and coastal wetlands of Jamaica Bay in order to understand and properly frame discussion of impacts to wetlands and habitats.

    Response: While this question is not directly addressed in the report, information concerning inundation of the wetland marshes could be inferred from Figure 9 (page 13) of the report, which shows tidal elevation during barrier closure, as compared to normal tidal elevation range before and after gate closure.

15. *Re: Erosion at the structure* - Separate analyses should directly measure and employ models of hydraulic flow and changes in pattern and flow velocity due to the presence of the structure in order to understand and properly frame a discussion of potential erosional impacts (scouring) which has been known to occur at the foot of the barriers. This will inform future maintenance needs and associated costs.

    Response: As the Corps studies further impacts associated with the barrier under the NYNJHATs study, the team will consider using the MIKE Suite model (with in-house capacity from Wilmington District) to assess potential erosion at the structure to inform future maintenance needs and associated costs.

16. *Re: Trapping of water behind the barrier* – Separate analyses should model the deflection and redirection of water away from the barrier as a consequence of barrier operation (closure) during a large storm to determine the potential field of impact.

    Response: To address this question would require development of a new model. The model would require a greater spatial area on the seaward side of the barriers, since that is where the greater impacts of the question above would be experienced. Again, this was beyond the scope of this analysis.

17. *Re: Factors determining necessity and duration of gate closure* – This should be discussed in detail within the Main HSGRR Report. This is operation and maintenance (to be developed), but critically important to identify upfront and relate directly to the model results.

    Response: Concur. In order to fully address the impacts, operation and maintenance will need to be discussed. However, the study is not far enough along to have this information available. Further study of the barrier, and the operations and maintenance and how they will affect impacts, will now be carried out under the NYNJHATs Study in order to address this and other comments.

18. *Re: Accretion/ trapping of sediments and/ or debris* - Separate analyses should model the potential for trapping sediments and debris during barrier operation and also when gates are open. Implications for ongoing maintenance and also for post-storm response, clearance, and clean-up of debris should be discussed. These issues may impact or delay safe navigation post storm.

    Response: As the Corps studies further impacts associated with the barrier under the NYNJHATs study, the team will consider using the MIKE Suite model (with in-house capacity from Wilmington District) to assess accretion/ trapping of sediments and/or debris both during barrier operation and also when gates are open.