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New York District

Storm Surge Barrier Sub-Appendix

Annex C – Storm Surge Barrier Water Level Rise on the Interior

DRAFT

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

Annex B2.C

September 2022

Table of Contents

List of Tables

List of Figures

1 System Description

1.1 Storm Surge Barriers and Basins

A select set of storm surge barriers, located in accordance with the NYNJHATS study alternatives described in the SSB Sub-Appendix and the enclosed areas behind, referred to as basins, are the subject of this analysis. Storm surge barriers and basins considered for this analysis are shown in Figure 1-1 and listed below. It should be noted that at the time this analysis was performed, not all SSBs described in the SSB Sub-Appendix were yet defined. The storm surge barriers selected are considered a representative set of the conditions and processes that could be encountered at the SSBs that are part of the NYNJHAT Study alternatives

The surface areas for the enclosed basins are based on data provided by ERDC and included in the stage storage relationship (see Table 1-1). In and outflows were applied to the basins based on the examined alternatives.

Table 1-1: Enclosed basins and storm surge barriers.

ID	Enclosed Basin	Enclosed by storm surge barriers	Study Alternative	Area [acre]
A1	New York and New Jersey Harbor (incl. Hudson River)	Outer Harbor & Throgs Neck	2	198,901
A2	New York and New Jersey Harbor (incl. Hudson River)	Verrazano Narrows, Arthur Kill & Throgs Neck	3A	105,466
B	Jamaica Bay	Jamaica Bay	3A, 3B, 4	12,004
C	Newark Bay + Arthur Kill	Arthur Kill & Kill van Kull	3B	11,908
D	Hackensack River	Hackensack River	4	256
E	Newtown Creek	Newtown Creek	4	168
F	Gerritsen Creek	Gerritsen Creek	3A, 3B, 4	261

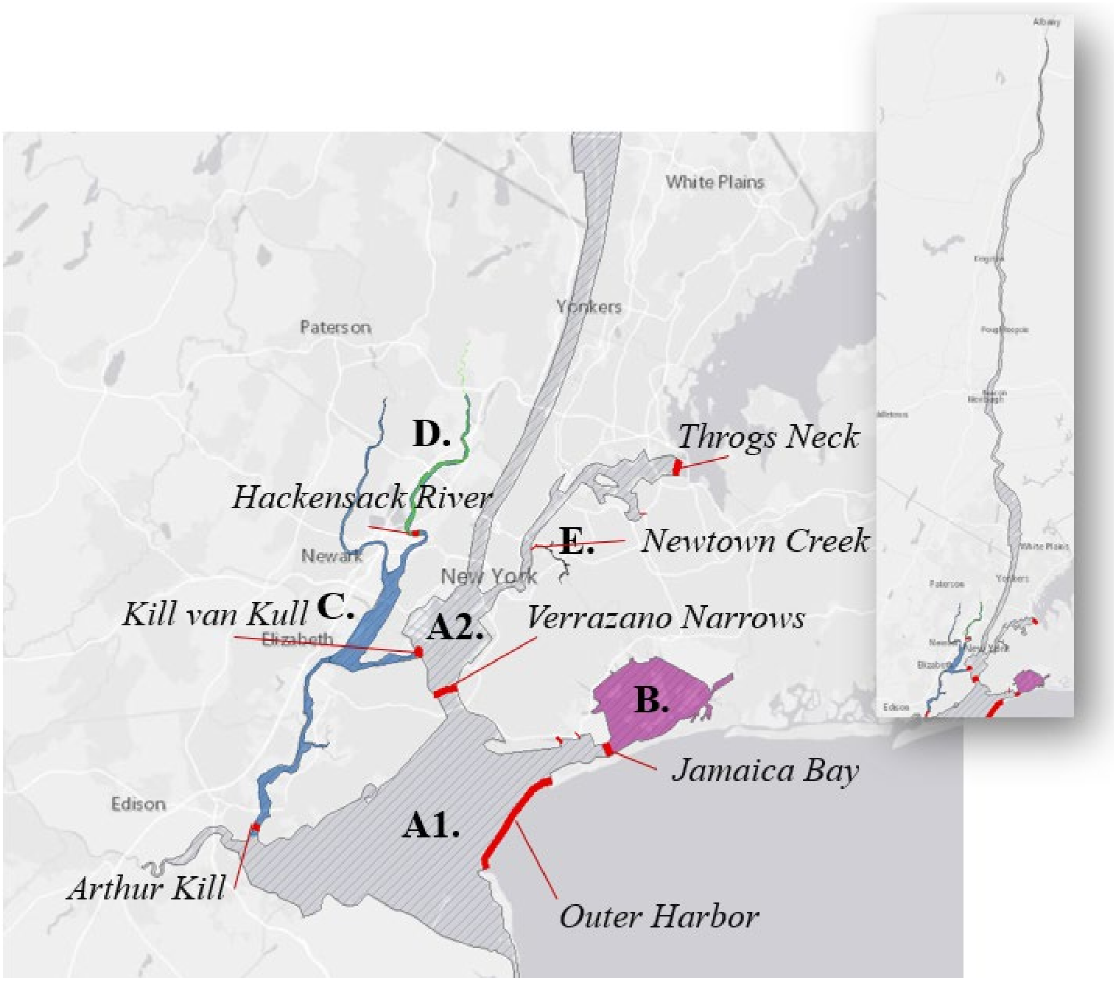

Figure 1-1: Storm Surge Barriers and Basins.

1.2 River Inflows

Riverine inflows were considered for the Hudson, Passaic, Hackensack and Raritan River. Yearly peak flow data is available from USGS. Gauges and associated numbers are shown in Table 1-2.

Table 1-2: USGS gages used for inflow analysis.

USGS Gage	USGS Gage Number
Hudson River at Green Island, NY	01358000
Passaic River at Little Falls, NJ	01389500
Raritan River below Calco Dam at Bound Brook, NJ	01403060
Hackensack River at New Milford, NJ	01378500

Table 1-3 shows various maximum percentile flow rates in cfs over the entire year and return periods in cfs. This data was obtained from previous analyses completed by ERDC.

Table 1-3: Average Recurrence Interval (ARI) flows and Percentile Flow Rates [cfs].

Attribute	Hudson	Passaic	Hackensack	Raritan
Average Recurrence Interval				
1-Year	37,398	2,048	49	10,601
2-Year	93,697	7,490	1,956	21,298
5-Year	118,152	10,439	3,101	30,113
10-Year	138,356	15,800	3,934	39,542
Percentile of Maximum Annual Flows				
Max. 50	35,798	2,200	155	1,801
Max. 75	48,999	3,390	343	3,418
Max. 80	55,200	3,931	420	4,111
Max. 90	69,898	5,491	735	7,211
Max. 95	91,401	7,681	1,300	18,399

1.3 Precipitation

Precipitation input data is available as precipitation frequency estimates from NOAA Atlas 14 Volume 10 Version 3 and is listed in Table 1-4 for average recurrence intervals (ARI) and rainfall storm duration.

Table 1-4: NOAA Precipitation Frequency Estimates [inches].

Storm duration	1 Year ARI	2 Year ARI	5 Year ARI	10 Year ARI	25 Year ARI	50 Year ARI	100 Year ARI
5-min:	0.370	0.444	0.561	0.657	0.791	0.892	0.997
10-min:	0.528	0.629	0.794	0.932	1.12	1.26	1.41
15-min:	0.621	0.74	0.935	1.1	1.32	1.49	1.66
30-min:	0.852	1.02	1.28	1.5	1.81	2.04	2.28
60-min:	1.08	1.29	1.63	1.91	2.3	2.59	2.89
2-hr:	1.43	1.68	2.09	2.43	2.89	3.25	3.61
3-hr:	1.65	1.94	2.41	2.8	3.33	3.74	4.16
6-hr:	2.05	2.44	3.07	3.591	4.31	4.84	5.41
12-hr:	2.46	2.99	3.85	4.57	5.56	6.3	7.09
24-hr:	2.83	3.5	4.6	5.51	6.76	7.69	8.68

Notes:

1. *This highlighted cell shows 10-Year rainfall event with 6-hour duration*

1.4 Municipal Inflows

Municipal Inflows were identified and included in this analysis (NYC.gov, 2019, Authority B. C., 2019, Authority L. R., 2019). The locations of major wastewater treatment plants were mapped and are included per basin (Figure 1-2). The design capacities are summarized in Table 1-5.

Table 1-5: Design Capacity of Major Waste Water Treatment Plants in the Area of Interest.

Location	Design Capacity [mgd]	Design Capacity [cfs]
26th Ward	85	131.5
Bowery Bay	150	232.1
Coney Island	110	170.2
Hunts Point	200	309.4
Jamaica	100	154.7
Newtown Creek	310	479.6
North River	170	263.0
Oakwood Beach	39.9	61.7
Rockaway	45	69.6
Owls Head	120	185.7
Wards Island	275	425.5
Tallman Island	80	123.8
Port Richmond	60	92.8
Red Hook	60	92.8
BCUA Hackensack	77	119.1
PVSC Outfall	300	464.2
Linden Roselle	13	20.1

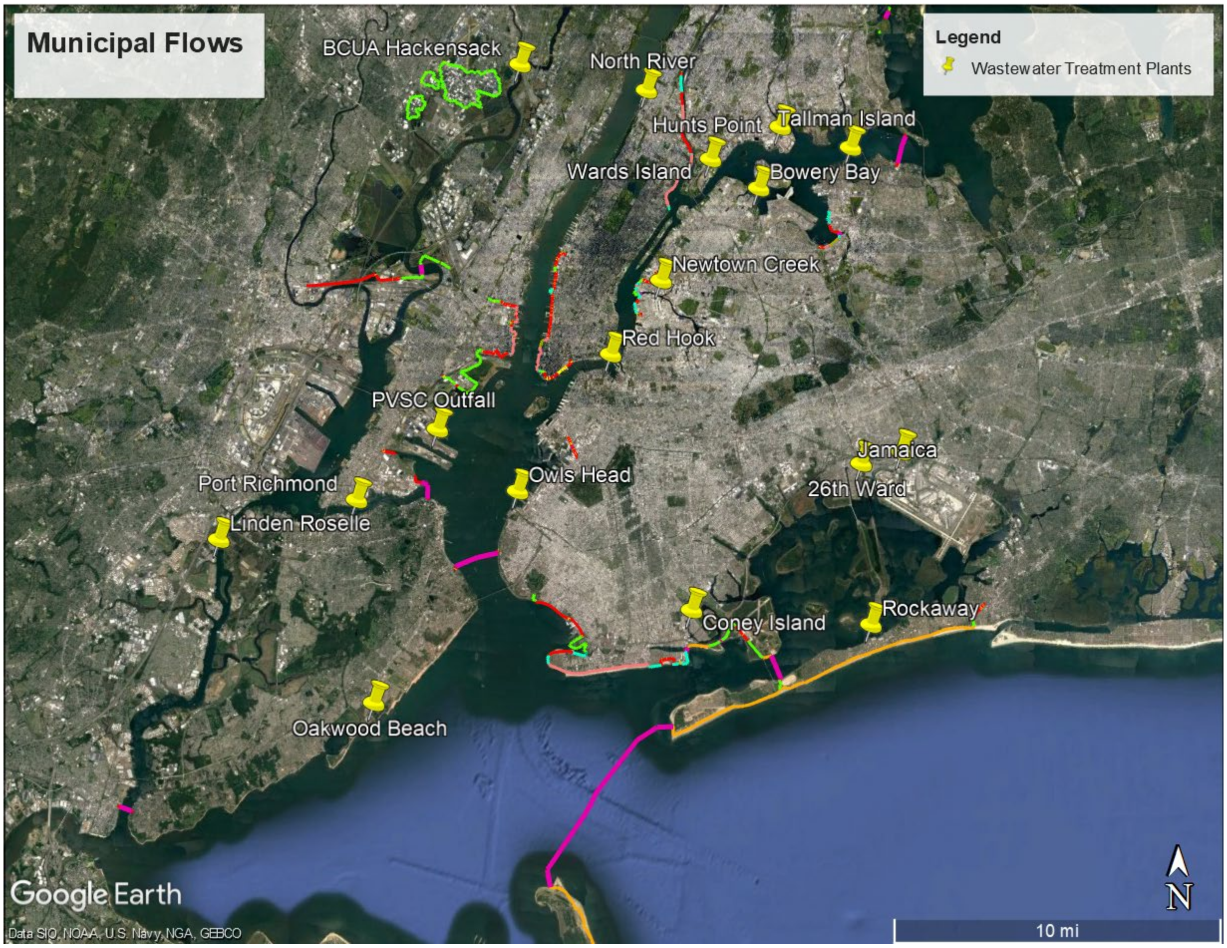

Figure 1-2: Locations of wastewater treatment plants for municipal inflows.

1.5 Storm Surge Barriers

Preliminary determined SSB crest elevations and lengths are summarized in Table 1-6 below and are further described in the SSB Sub-Appendix.

Table 1-6: Storm Surge Barrier Crest Elevations and Lengths.

Storm Surge Barrier	Barrier Crest Elevation [ft NAVD88]	Barrier Length [ft]
Verrazano Narrows	+19	7,160
Throgs Neck	+18	4,510
Arthur Kill	+19	2,140
Outer Harbor	+29	34,590
Kill van Kull	+19	3,320
Jamaica Bay	+18	4,120
Hackensack River	+19	1,570
Newtown Creek	+17	510
Gerritsen Creek	+17	303

1.6 SSB Pump Station Capacity

Pump stations were included in the analysis for two surge barrier complexes. The preliminary proposed pumping rates are based on the technical analysis as documented in the interim report (USACE, 2019). The preliminary estimates for pumping rates are listed in Table 1-7.

Table 1-7: Estimated Pumping Rate for Pump Stations associated with Storm Surge Barrier Complex

Storm Surge Barrier	Pumping Rate [cfs]
Hackensack River	1,664
Newtown Creek	1,240

2 Methodology

2.1 General Principal

The stage on the protected side of a storm surge barrier is based on a stage storage relationship ("bucket approach"), considering the impacts of river inflows, precipitation, municipal inflows (i.e., wastewater treatment plants), pump stations and barrier overtopping during the time of barrier closure. A closure criterion as well as operating closure elevation need to be defined.

- The closure criterion is the forecasted water level for which operation of a storm surge barrier is mandated to reduce flood risk for the region behind it.
- The operating closure elevation is the observed water level at which the mechanical closure procedure is executed. This elevation could be defined as low tide at the occurrence of high riverine discharges or be defined as a fixed water level when riverine discharge is moderate or low.

This analysis centers around the exceedance of 1% AEP water levels (or lower probability events) and still water level hydrographs were selected upfront I.e., the closure criterion is met a priori and only the influence of the closure elevation is investigated.

Once the storm surge barrier is closed, the direct relationship between interior water levels and exterior water levels is removed and other physical processes become the governing drivers for the interior water level. These processes are: riverine inflow, rainfall, municipal inflows, inflow as a result of SSB overtopping and outflow as a result of pumping. The basic principle is illustrated in Figure 2-1.

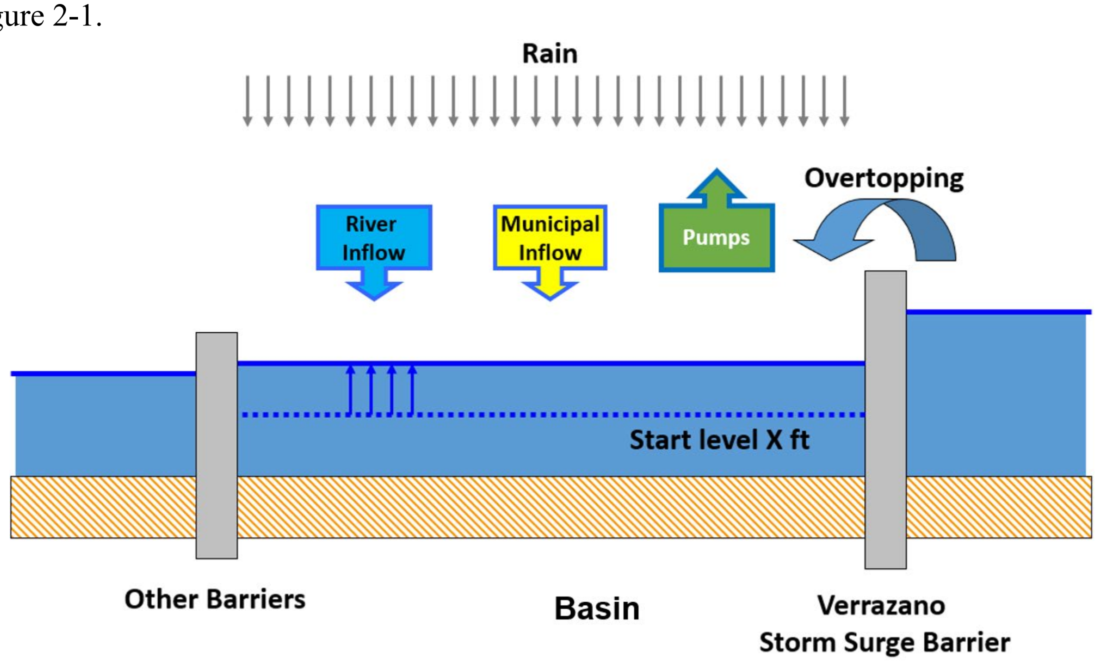

Figure 2-1: Example of schematization of stage storage relationship for basin behind Verrazzano Narrows storm surge barrier (Alternative 3A).

2.2 Basin Stage Increase

The basin stage increase is calculated as the sum of in- and outflows over the basin area for every time step during the storm event.

$$Stage = \sum_{TIME} \frac{(QOT + QRIV + QPREC + QM - QP) \cdot \Delta t}{BASIN\ AREA}$$

Where

QOT = $Inflow\ Overtopping$
$QRIV$ = $River\ Inflow$
$QPREC$ = $Precipitation$
QM = $Municipal\ Inflow$
QP = $Pump\ Rate$
Δt = $Time\ step\ difference\ during\ storm\ event$

2.3 Closure of Barrier

Closure elevations of +4, +5, +6, +7, +8, +9 and +10 ft NAVD88 were tested, in order to calculate the influence on the stage inside. Closure occurs when the water level is equal to the closure elevation prior to the peak of the storm. The closure time depends on the closure elevation as well as shape of the storm surge hydrograph.

2.4 Inflows and Outflows

Rainfall was applied uniformly over the whole basin area; no surface runoff was considered. River inflows, overtopping, precipitation, and municipal inflows were added positively in the stage increase equation; pump outflows were taken as negative input. Boundary conditions are further detailed in Section 2.6.

2.5 Opening of Barrier

For simplification the barrier is assumed to be opened after the peak of the storm has occurred and opening occurs when water levels on the inside and outside of the barrier are equal.

2.6 Boundary Conditions

The following sections describe previously discussed input parameters. These inputs are used as boundary conditions for the interior stage analysis. For each input parameter, a base value is selected to represent the expected condition for design. Additional variations are considered as well to investigate the sensitivity of the basin stage to the parameter.

2.6.1 AEP SWL and Wave characteristics

The annual exceedance probabilities (AEP) for still water level levels as well as wave characteristics were taken from the closest NACCS output points in proximity to the barriers (compare with Table 2-2). The base value is selected for the 1% AEP still water level. Sensitivity is tested for the 0.2% AEP and 0.1% AEP event.

2.6.2 Sea Level Change

Sea Level Change scenarios were included. The closest tide gauges for each storm surge barrier location are listed in Table 2-1 and SLR scenario projections were based on the gauge specific projection using the regional SLC rate.

Table 2-1: Storm Surge Barrier Gauges used for Sea Level Change.

Storm Surge Barrier	Closest Gauge	Station Number
Verrazano Narrows	The Battery	8518751
Throgs Neck	Kings Point	8516945
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	853069
Gerritsen Creek	Sandy Hook	8531680

The selected base value for the analysis is the USACE intermediate sea level change prediction. Sensitivity was tested for the NY State high sea level change prediction.

2.6.3 Storm Surge Hydrographs

Storm hydrographs are needed to describe the temporal behavior of the hydrodynamic conditions on the flood side of the storm surge barrier. Storm hydrographs were selected at the designated NACCS output points. The selection was based on the still water level levels for 100-year storm events with sea level change for the years 2055, 2105 and 2155, as well as 100-year wave heights and is further described below.

2.6.4 Selection of Synthetic Storms

The storm selection is based on the statistical annual exceedance probability values for still water level levels and wave heights. Storms shown in Table 2-2 have peak water levels that closely match the statistical 1% AEP still water level levels at the selected NACCS output points.

Table 2-2: Storm Surge Hydrograph Selection.

Storm Surge Barrier	NACCS Output Point	Selected Synthetic Storm
Verrazano Narrows	11781	398
Throgs Neck	4347	525
Arthur Kill	11650	409
Outer Harbor	3900	525
Kill van Kull	11766	399
Jamaica Bay	3592	208
Hackensack River	11816	208

Newtown Creek	13898	184
Gerritsen Creek	14085	398

Both still water level and wave height time series were taken from the selected synthetic storm. No variations in storm hydrographs were investigated.

2.6.5 Adjustment of Storms

Synthetic NACCS storm hydrographs were selected as described above. However, input values vary as described earlier based on year of evaluation, sea level change and return period. In order to match the input values for water levels and waves, the selected hydrographs were scaled. One example for the Verrazano Narrows barrier is shown in Figure 2-2. Here, target values for significant wave height and still water level are indicated through horizontal lines, the unscaled hydrograph for storm #398 is shown in grey and the scaled version to match the input values in blue. This demonstrates that with small modifications the selected hydrograph can be used as representative input.

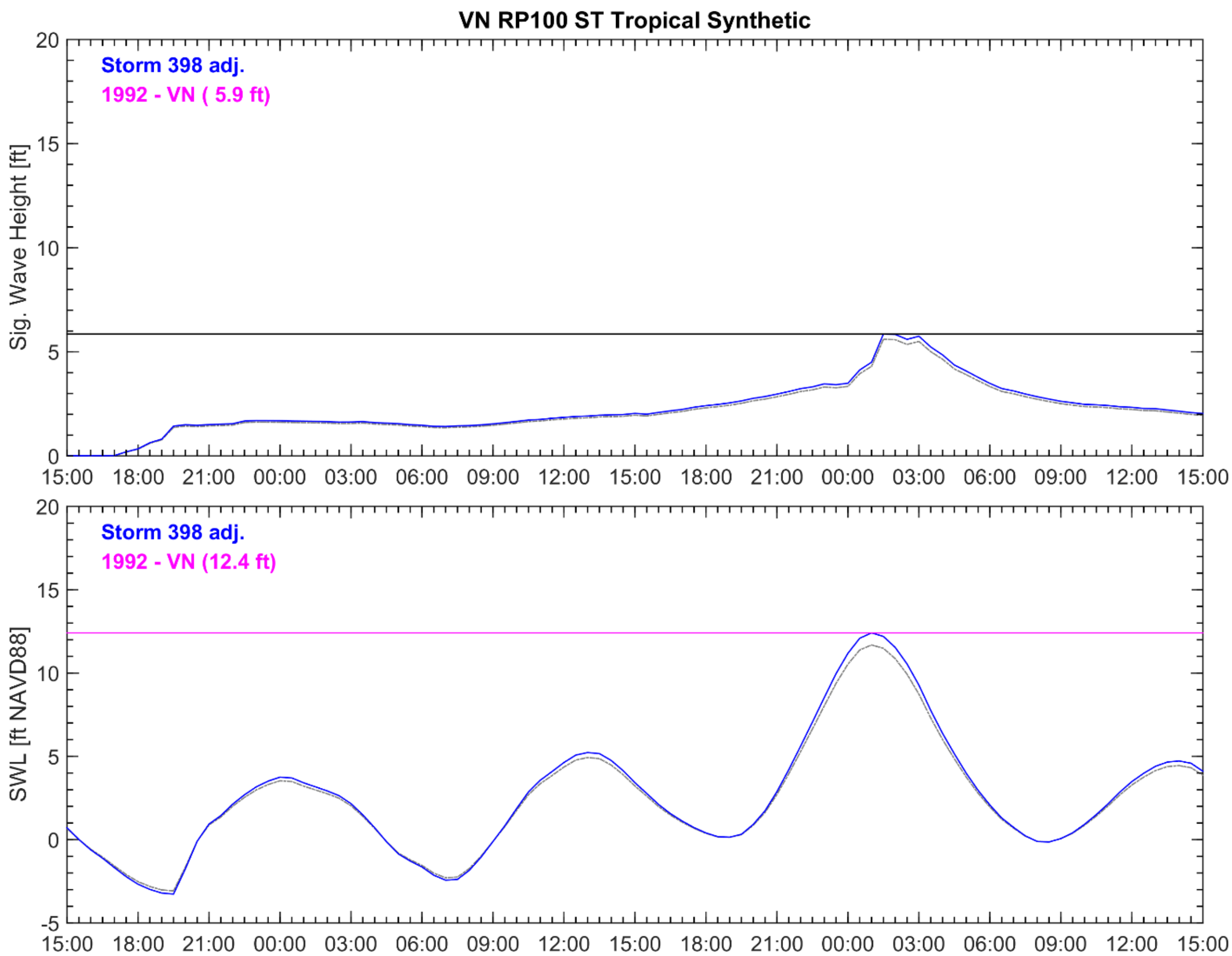

Figure 2-2: Synthetic NACCS storm #398 for Verrazano Narrows (scaled in blue and unscaled in light grey).

2.7 Overtopping Discharge Rate

For the stage increase calculations, the deterministic design & assessment approach as described in the EurOtop manual was applied (EurOtop, 2018). Wave inputs and still water level levels were used from the storm hydrographs (Section 2.6.3), resulting in a time dependent overtopping rate. The overtopping rate was multiplied with the length of the barrier to reflect the wave overtopping volume. Weir overtopping and overflow of the barrier was included for negative freeboards.

2.7.1 Riverine Discharge

Riverine discharges for multiple AEP events were shown in the previous section. The selected base value is the 90th percentile of the annual discharge. Investigated variations include the 50th, 75th, 80th, 95th percentile of the annual discharge and the 10-year return period discharge.

2.7.2 Municipal Inflows

The municipal inflows are assumed to be equal to the design capacity of the relevant WWTPs as listed earlier (see Table 1-5). This is a conservative assumption as flows will be time varying and will likely not be constant at peak design capacity during the closure of the barrier. No variations in municipal inflows were investigated.

2.7.3 Rainfall

Rainfall and storm surge for coastal storms generally do not show a good correlation. A high precipitation rate was selected to be conservative in this analysis. The 10-year return period 6-hour rain event (3.59 inches per 6 hours) was applied on the basin surface as base value. Variations were investigated for a 1-, 2-, 5-, 25-year return period rainfall 6-hr event.

2.7.4 Pump Station Capacity

The preliminary estimated pumping capacity (see 1.6) was used as base value. No variations were investigated.

2.7.5 Application and Limitations of Methodology

This is a feasibility level type analysis to investigate the potential for stage increase in the basins enclosed by the storm surge barriers. The methodology presented here is a simplified schematization of the problem and the following limitations are noted. So far, the analysis has only been performed using tropical event storm surge hydrographs. Non tropical events generally have a storm surge of lesser magnitude but could be of longer duration. The results of this analysis are dependent on the storm duration and shape of the storm surge hydrograph (i.e., selected storm ID) which affect the closure duration. Furthermore, the analysis does not account for wind effects on the basin and increases beyond the values presented here could be observed due to wind setup within the enclosed basin. Note that for the extreme events flanking and flow into the basin that may occur at locations beyond the Storm Surge Barrier structure are not included. Finally, only one storm surge barrier is assumed to be overtopped during the peak of the storm to calculate the contribution of wave overtopping to the basin stage.

3 Results

3.1 Run Cases

Figure 3-1 is shown to explain the basic principle of the stage increase analysis. The storm surge barrier is closed, when still water level elevations exceed the predetermined closure elevations prior to the arrival of the storm peak. In Figure 3-1, a closure elevation of +6 ft NAVD88 was selected (indicated by the red horizontal line). The stage on the inside of the barrier is indicated in blue, on the outside in black. The blue line follows the outside water level prior to the storm surge barrier closure, it increases at a low rate after barrier closure and follows the outside water level after barrier opening, which occurs when interior and exterior water levels equalize. The gray area in the background indicates the selected barrier crest elevation (in this case +19ft NAVD88).

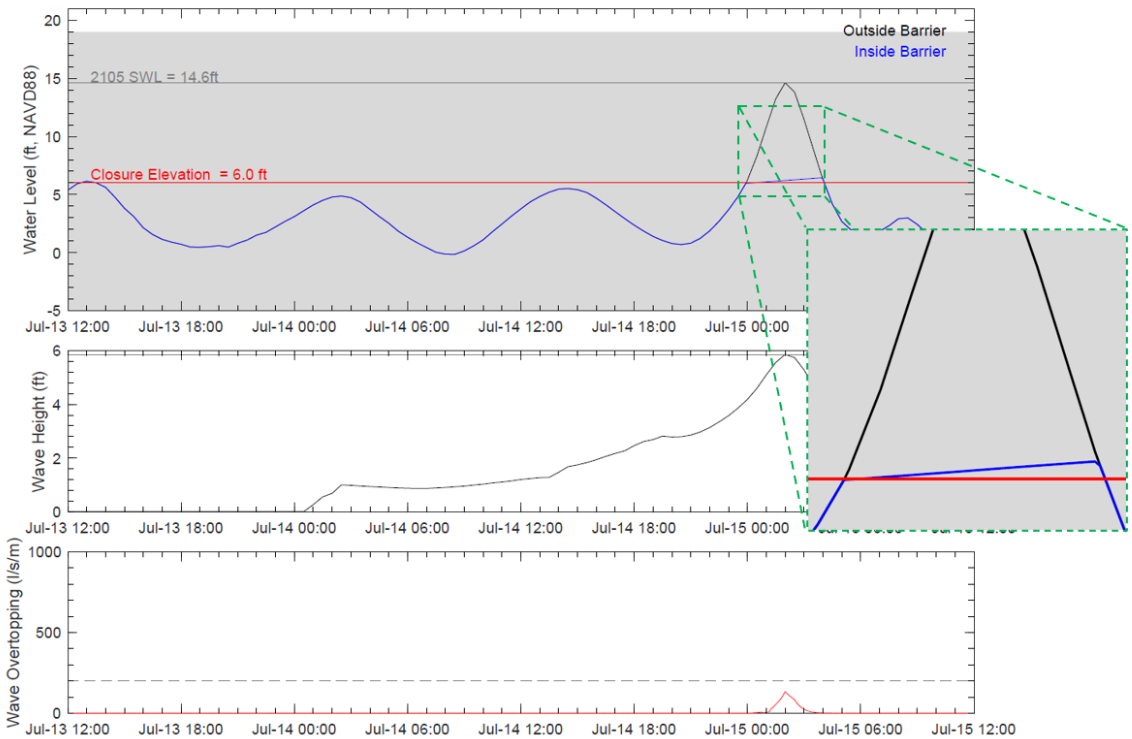

Figure 3-1: Outside and inside water level showing stage increase for Verrazano Narrows at closure elevation +6ft NAVD88 (top panel). Exterior wave height (middle panel). Overtopping discharge rate (lower panel).

Constant riverine, precipitation induced and municipal inflows as well as pump induced outflows were applied where applicable during barrier closure. Constant inflows lead to a constant water surface increase, time dependent overtopping leads to unsteady increase and the summation determines the time dependent interior stage.

Approximately 500 run cases were created varying in closure elevations, evaluation year, sea level change, precipitation rate, return period, river inflow percentiles, pump rates and municipal flow rates for the locations. In order to find a realistic estimate for stage increase, a set of basic input parameters were selected conservatively: 100-year return period storm hydrographs were used together with the maximum 90th percentile of the yearly river inflows, 6 hour 10-year return period rains and the 2105 USACE intermediate SLC estimate. The basic design run cases used for evaluation are summarized in Table 3-1.

Table 3-1: Basic run cases with closure elevation of +5, +6, +7, and +8 ft NAVD88

SSB	Alt.	Eval. Year	SLR	Riv. Flow (Percentile)	Precip. RP (Yr)	Precip. Duration (h)	Storm ID
Outer Harbor	2	2105	INT	90	10	6	525
Throgs Neck	2,3a	2105	INT	90	10	6	525
Verrazano Narrows	3a	2105	INT	90	10	6	398
Arthur Kill	3a,3b	2105	INT	90	10	6	409
Jamaica Bay	3a,3b,4	2105	INT	90	10	6	208
Kill van Kull	3b	2105	INT	90	10	6	399
Hackensack River	4	2105	INT	90	10	6	208
Newtown Creek	4	2105	INT	90	10	6	184
Gerritsen Creek	3a,3b,4	2105	INT	90	10	6	398

3.2 Increases in Basin Stage

The interior basin stage evaluation results using the above basic run cases are summarized in Table 3-2. The total interior stage increase is below 1ft for each closure elevation and storm surge barrier apart from the Arthur Kill and Kill van Kull SSB. For those two storm surge barriers and for that particular basin that is part of Alternative 3B the stage increase is 1.9ft and 2.2ft for a closure elevation of +5ft NAVD88 respectively. Increase in stage for the interior basis is directly dependent on closure duration since longer closure durations result in a larger net inflow and therefor net larger stage increase. The selected storm surge hydrographs influence the closure duration. A higher closure elevation results in a shorter closure duration because the closure will occur later, and the opening will occur sooner (the point of equal inside and outside water levels will occur earlier after the storm).

Table 3-2: Stage increase at closure elevations from +5ft to +8ft NAVD88

Storm Surge Barrier	Increase for 1% AEP Total Stage Increase (ft)*	Closure Dur. (hrs)	Increase by River Inflow (ft)	Increase by Municipal Inflows (ft)	Increase by Rainfall (ft)	Increase by Overtop (ft)	Decrease due to Pumping (ft)
Closure Elevation of +5 ft NAVD88							
Verrazano Narrows	0.6	4.7	0.3	<0.1	0.3	<0.1	<0.1
Throgs Neck	0.9	7.9	0.5	<0.1	0.4	<0.1	<0.1
Arthur Kill	1.9	13.3	1.2	<0.1	0.7	<0.1	<0.1
Outer Harbor	0.6	6.3	0.2	<0.1	0.3	<0.1	<0.1
Kill van Kull	2.2	15.4	1.4	<0.1	0.8	<0.1	<0.1
Jamaica Bay	0.3	4.8	<0.1	<0.1	0.2	<0.1	<0.1
Hackensack River	<0.1	7.6	1.8	<0.1	0.4	0.1	2.3
Newtown Creek	<0.1	17.5	<0.1	4.1	0.9	<0.1	5.1
Gerritsen Creek	0.9	8.2	<0.1	0.4	0.4	<0.1	<0.1
Closure Elevation of +6 ft NAVD88							
Verrazano Narrows	0.5	4.0	0.3	<0.1	0.2	<0.1	<0.1
Throgs Neck	0.8	7.3	0.5	<0.1	0.4	<0.1	<0.1
Arthur Kill	1.6	11.1	1.0	<0.1	0.6	<0.1	<0.1
Outer Harbor	0.5	5.4	0.2	<0.1	0.3	<0.1	<0.1
Kill van Kull	1.5	10.2	1.0	<0.1	0.5	<0.1	<0.1
Jamaica Bay	0.2	4.2	<0.1	<0.1	0.2	<0.1	<0.1
Hackensack River	<0.1	7.6	1.6	<0.1	0.3	0.1	2.1
Newtown Creek	<0.1	16.0	<0.1	3.8	0.8	<0.1	4.6
Gerritsen Creek	0.8	7.2	<0.1	0.4	0.4	<0.1	<0.1
Closure Elevation of +7 ft NAVD88							
Verrazano Narrows	0.4	3.6	0.2	<0.1	0.2	<0.1	<0.1
Throgs Neck	0.7	6.4	0.4	<0.1	0.3	<0.1	<0.1
Arthur Kill	1.4	9.8	0.9	<0.1	0.5	<0.1	<0.1

Storm Surge Barrier	Increase for 1% AEP Total Stage Increase (ft)*	Closure Dur. (hrs)	Increase by River Inflow (ft)	Increase by Municipal Inflows (ft)	Increase by Rainfall (ft)	Increase by Overtop (ft)	Decrease due to Pumping (ft)
Outer Harbor	0.4	4.2	0.1	<0.1	0.2	<0.1	<0.1
Kill van Kull	0.9	6.2	0.6	<0.1	0.3	<0.1	<0.1
Jamaica Bay	0.2	3.5	<0.1	<0.1	0.2	<0.1	<0.1
Hackensack River	<0.1	6.1	1.5	<0.1	0.3	0.1	1.9
Newtown Creek	<0.1	14.2	<0.1	2.8	0.6	<0.1	3.4
Gerritsen Creek	0.6	6.0	<0.1	0.3	0.3	<0.1	<0.1
Closure Elevation of +8 ft NAVD88							
Verrazano Narrows	0.4	3	0.2	<0.1	0.2	<0.1	<0.1
Throgs Neck	0.7	5.6	0.4	<0.1	0.3	<0.1	<0.1
Arthur Kill	1.2	8.3	0.8	<0.1	0.4	<0.1	<0.1
Outer Harbor	0.3	3	0.1	<0.1	0.2	<0.1	<0.1
Kill van Kull	0.7	4.9	0.5	<0.1	0.3	<0.1	<0.1
Jamaica Bay	0.2	3	<0.1	<0.1	0.2	<0.1	<0.1
Hackensack River	<0.1	5.3	1.3	<0.1	0.3	0.1	1.6
Newtown Creek	<0.1	4.9	<0.1	1.2	0.3	<0.1	1.5
Gerritsen Creek	0.6	5.2	<0.1	0.3	0.3	<0.1	<0.1

Note:
**Conservative estimate*

Stage increase and proportions of pump-, overtopping-, river inflow- and municipal inflow-induced stage for closure elevations of +5ft NAVD88 to +8ft NAVD88 are shown in Figure 3-2 to Figure 3-5. This is a graphical depiction of the tabulated results.

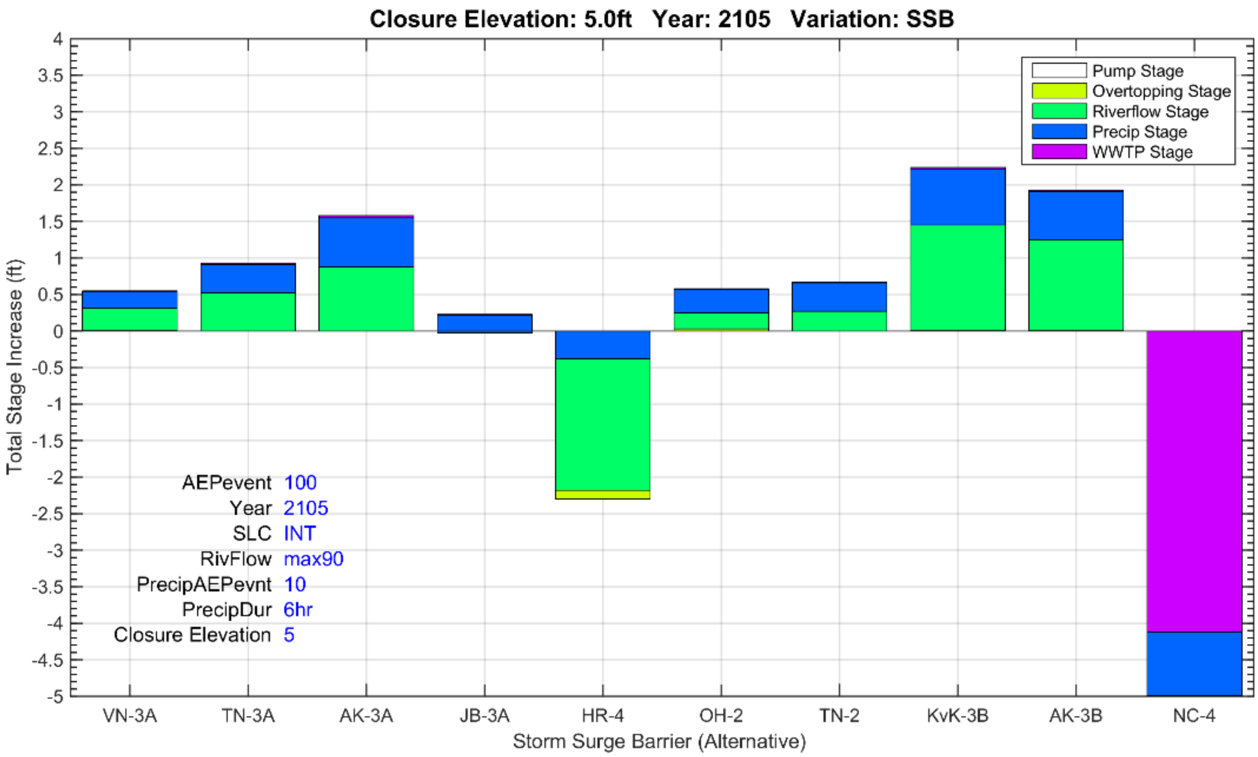

Figure 3-2: Interior stage increase caused by pumping, overtopping, river inflow, precipitation and municipal inflows at a closure elevation of +5ft NAVD88.

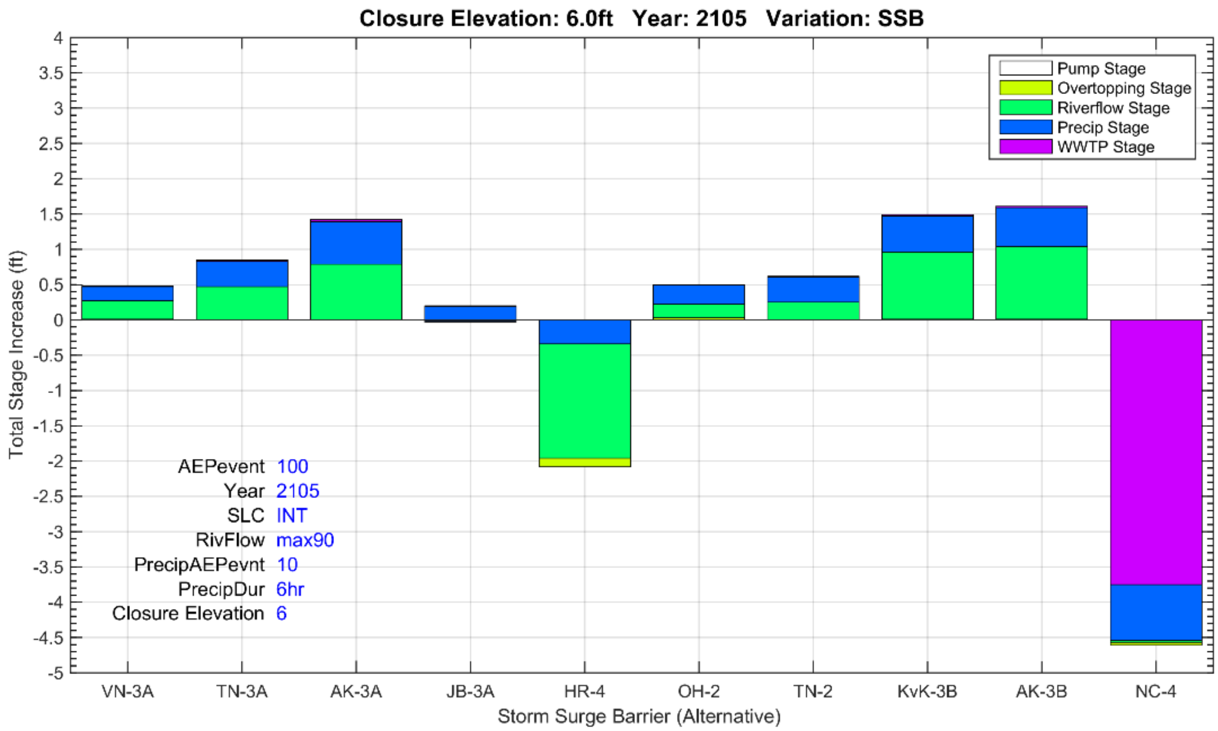

Figure 3-3: Interior stage increase caused by pumping, overtopping, river inflow, precipitation and municipal inflows at a closure elevation of +6ft NAVD88.

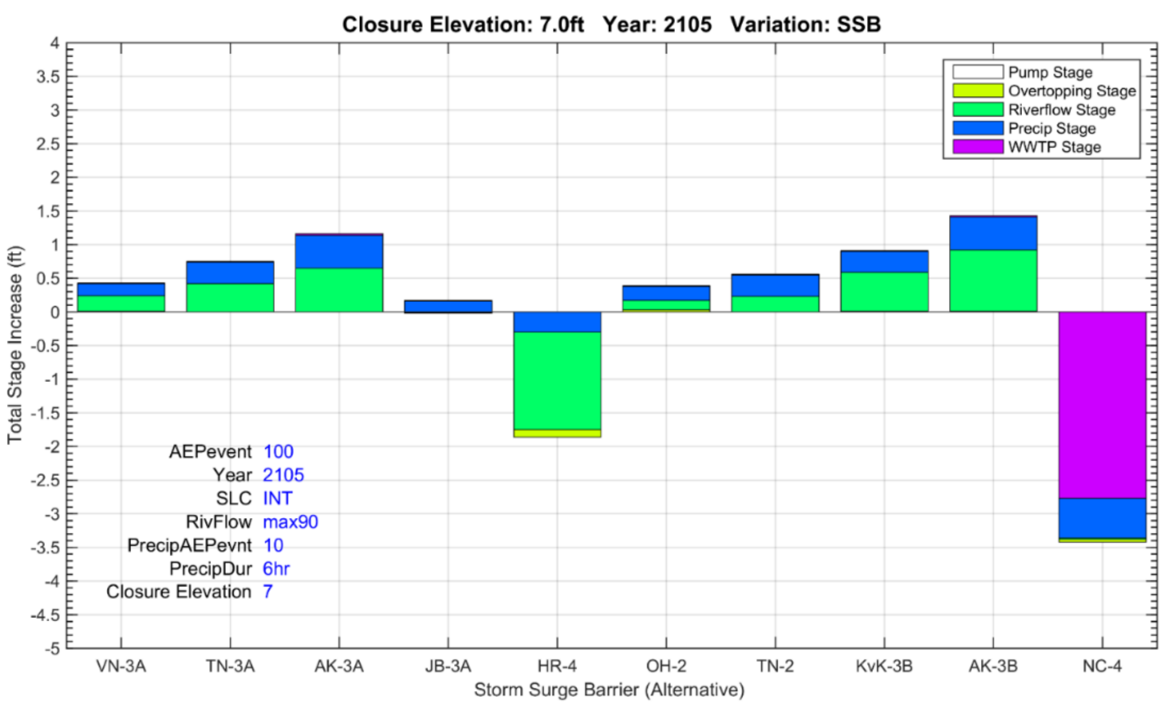

Figure 3-4: Interior stage increase caused by pumping, overtopping, river inflow, precipitation and municipal inflows at a closure elevation of +7ft NAVD88.

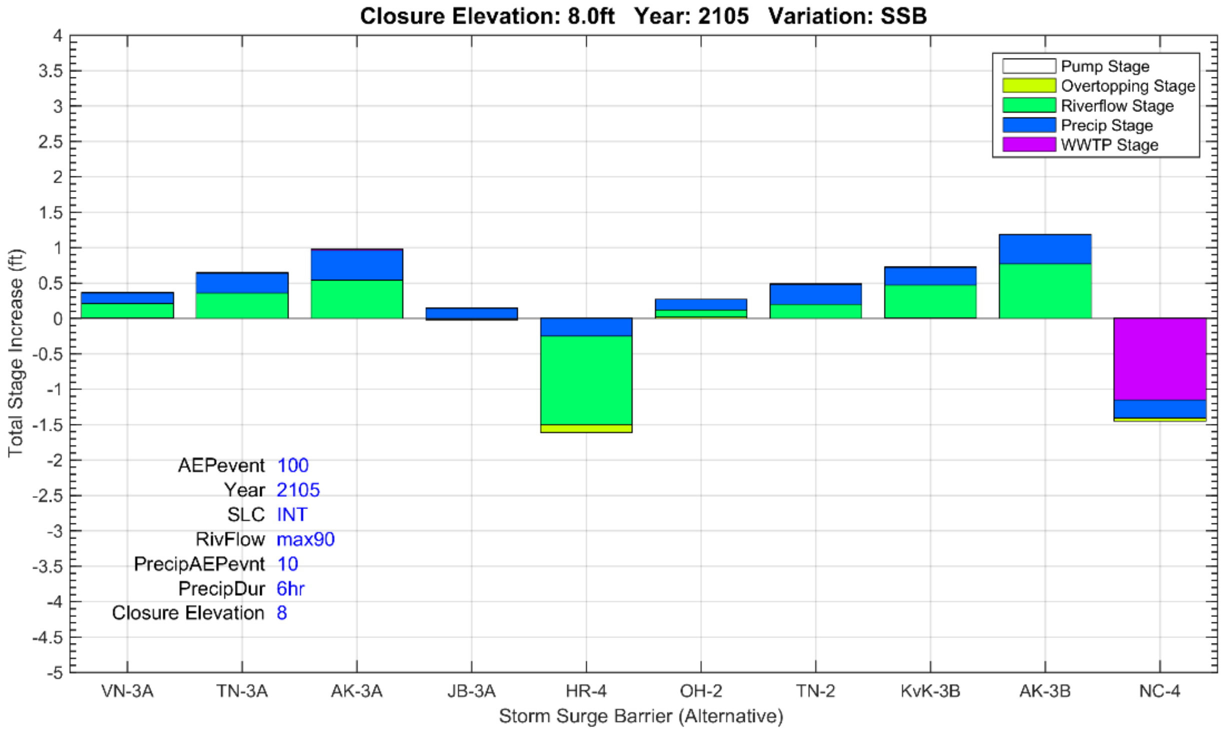

Figure 3-5: Interior stage increase caused by pumping, overtopping, river inflow, precipitation, and municipal inflows at a closure elevation of +8ft NAVD88.

It can be observed that river inflow and precipitation influence the interior basin stage the most for almost all cases. The influence of overtopping is negligible. For both, the Hackensack River and Newtown Creek basin, pumps are used to counteract the otherwise high interior stage increase. For those two basins, the interior stage increases with a much higher rate than for the other basins caused by high basin inflows compared to a small storage volume. For the Hackensack River basin, riverine inflows are the driving factor for the accelerated stage increase. For Newtown Creek and Gerritsen Creek, the stage increases faster due to municipal inflows caused by wastewater treatment plants. For the Arthur Kill and Kill van Kull storm surge barrier the stage increase is different for the analysis of the HAT Study Alternative 3B. This is an example where the storm hydrograph influences the outcome. A shorter closure duration leads to a smaller increase in basin stage.

3.3 Closure Durations

Storm surge barrier closure durations depend on the closure elevation and time of the storm induced peak water elevations. The barrier is closed as soon as the closure elevation is reached during a storm event and remains closed until the water level on both sides of the barrier equates. Closure durations for the barrier locations and alternatives were calculated for run cases and are shown for closure elevations from +5ft to +8ft NAVD88 in Figure 3-6 to Figure 3-9. A more detailed analysis of SSB closure probability and closure duration is provided in Annex D.

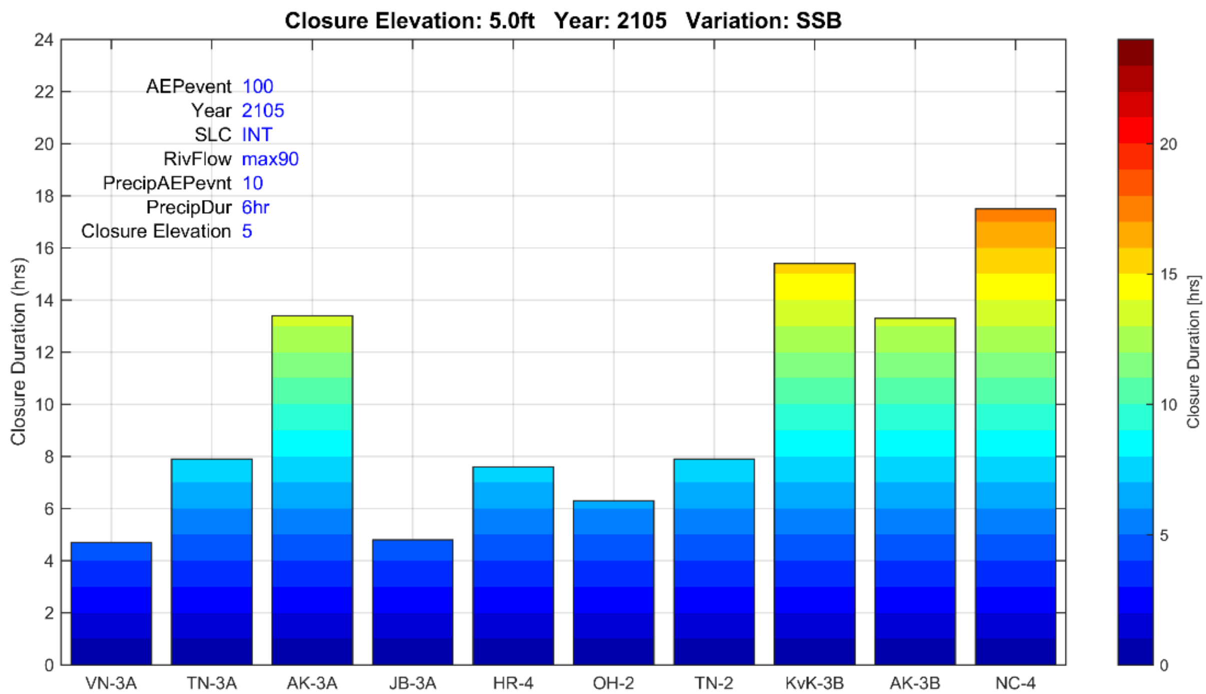

Figure 3-6: Barrier closure durations for a closure elevation of +5ft NAVD88 during a 2105 100-year storm event.

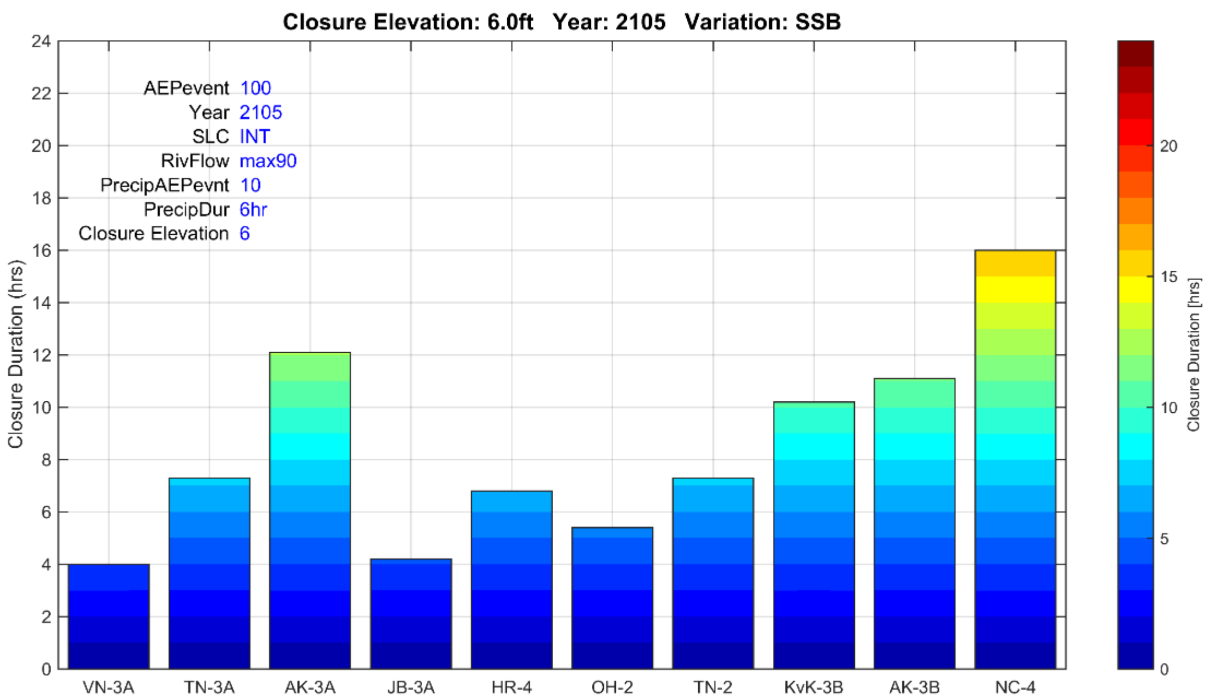

Figure 3-7: Barrier closure durations for a closure elevation of +6ft NAVD88 during a 2105 100-year storm event.

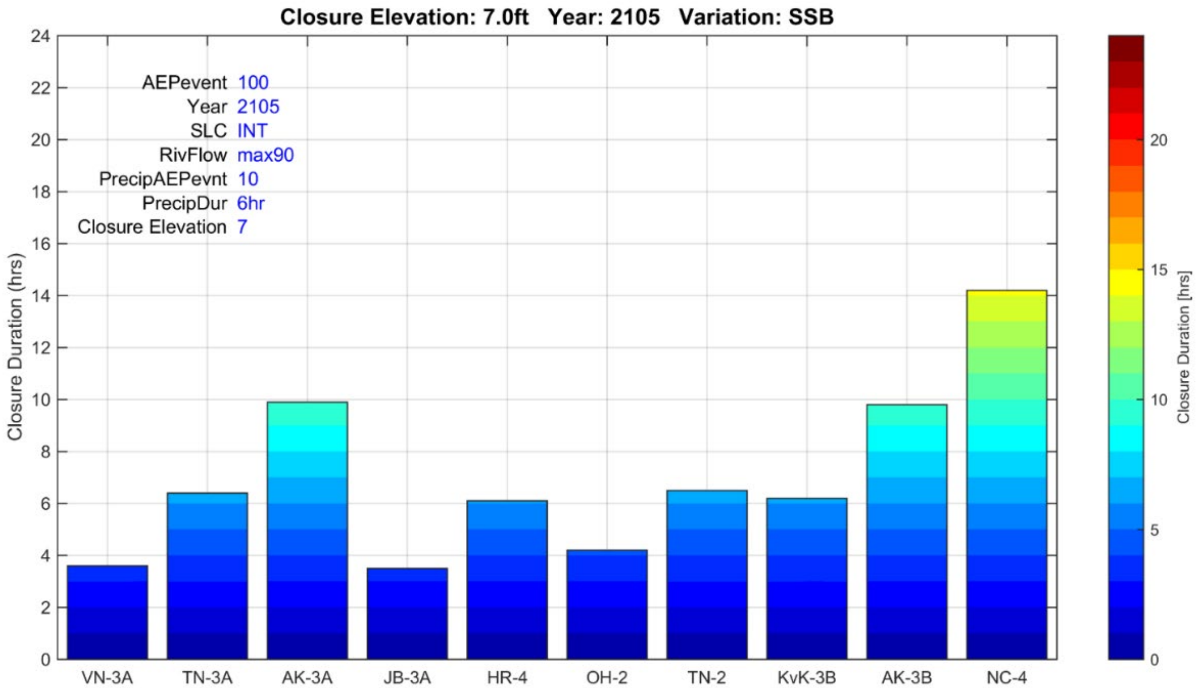

Figure 3-8: Barrier closure durations for a closure elevation of +7ft NAVD88 during a 2105 100-year storm event.

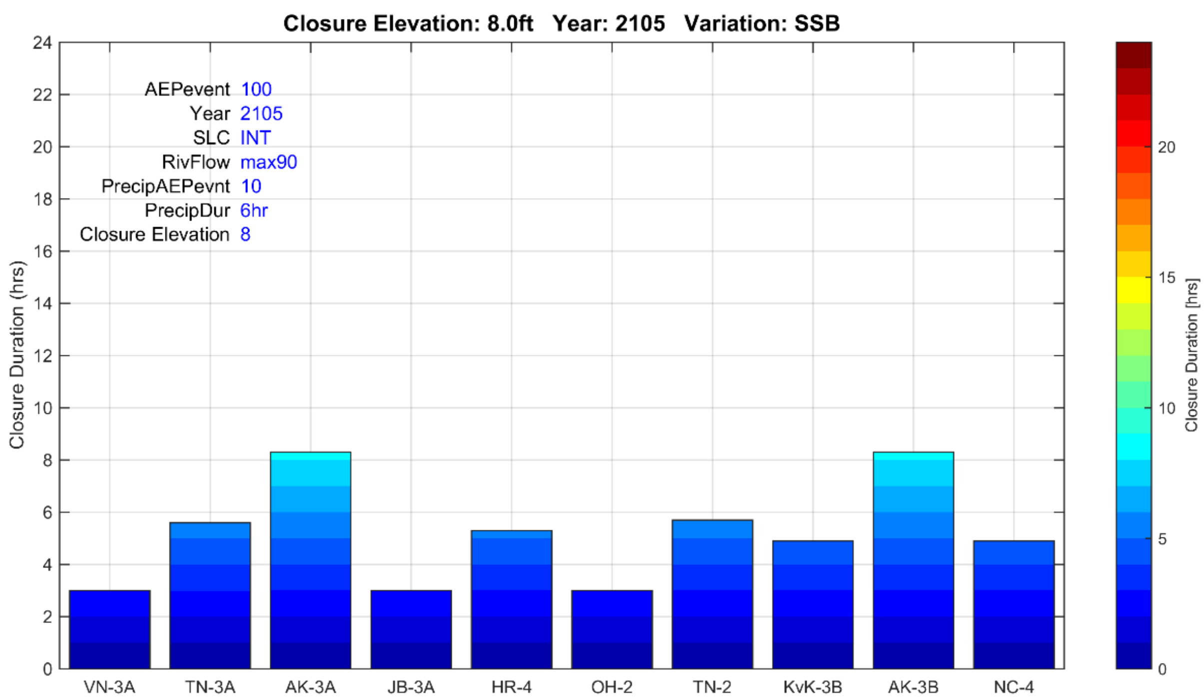

Figure 3-9: Barrier closure durations for a closure elevation of +8ft NAVD88 during a 2105 100-year storm event.

3.4 Sensitivity to Variations in Boundary Conditions

Sensitivity was examined for variations in river flow, precipitations, evaluation year, sea level change and AEP events. The variations examined are listed in Table 3-3.

Table 3-3: Variations for input parameters.

Input Value	Variation in
River Flow	10-yr RP, max 50%, max 75%, max 80%, max 90%, max 95%
Precipitation	1-yr RP, 2-yr RP, 5-yr RP, 10-yr RP, 25-yr RP
Year	2055, 2105, 2155
Year + Sea Level Change	High SLC 2055, 2105
AEP Event	100-yr RP, 500-yr RP, 1000-yr RP

Variations in river flow have an observable impact, however, the larger the basin size, the smaller the sensitivity to river inflow. The analysis revealed that variations in years for a high sea level change scenario potentially impact the interior stage the most. This analysis also indicates that the other examined input parameters have a lesser impact on the basin stage. The following figures show the results of the sea level (Figure 3-11) change and river inflow (Figure 3-10) sensitivity calculations for the Verrazano Narrows barrier.

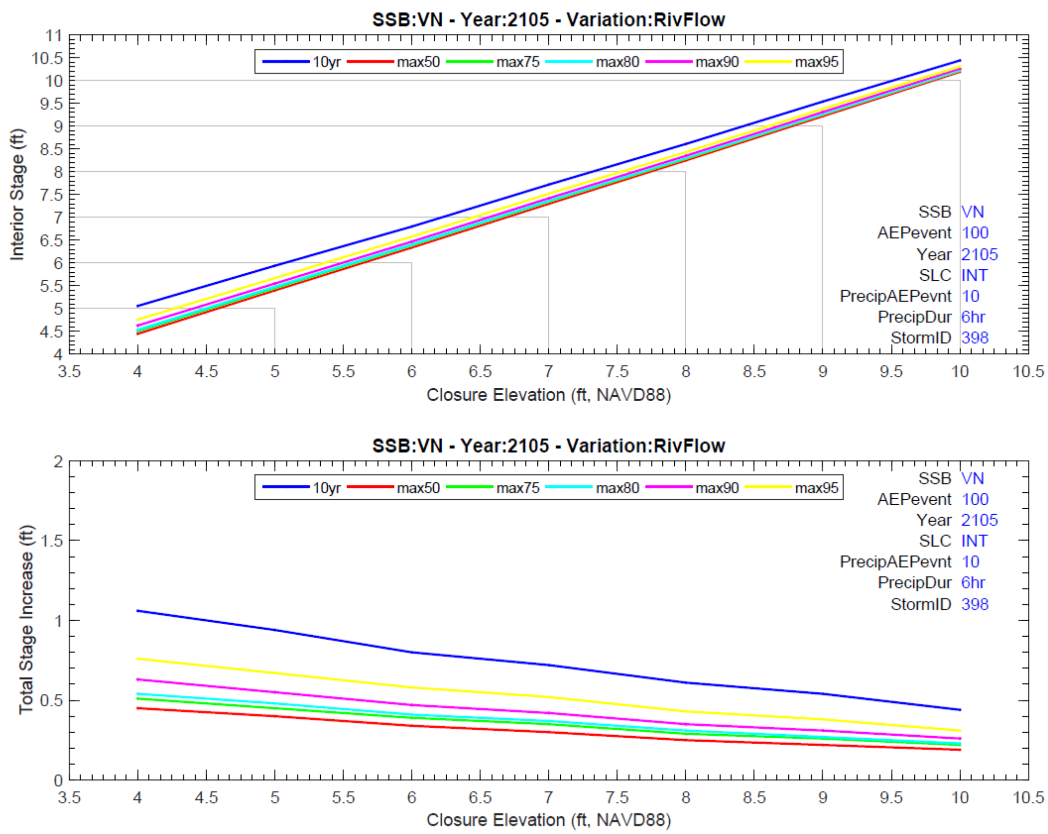

Figure 3-10: Sensitivity: Influence of river inflow variations on the stage (protected side) of the closed barrier during a 100-year RP storm event (Here: Verrazano Narrows).

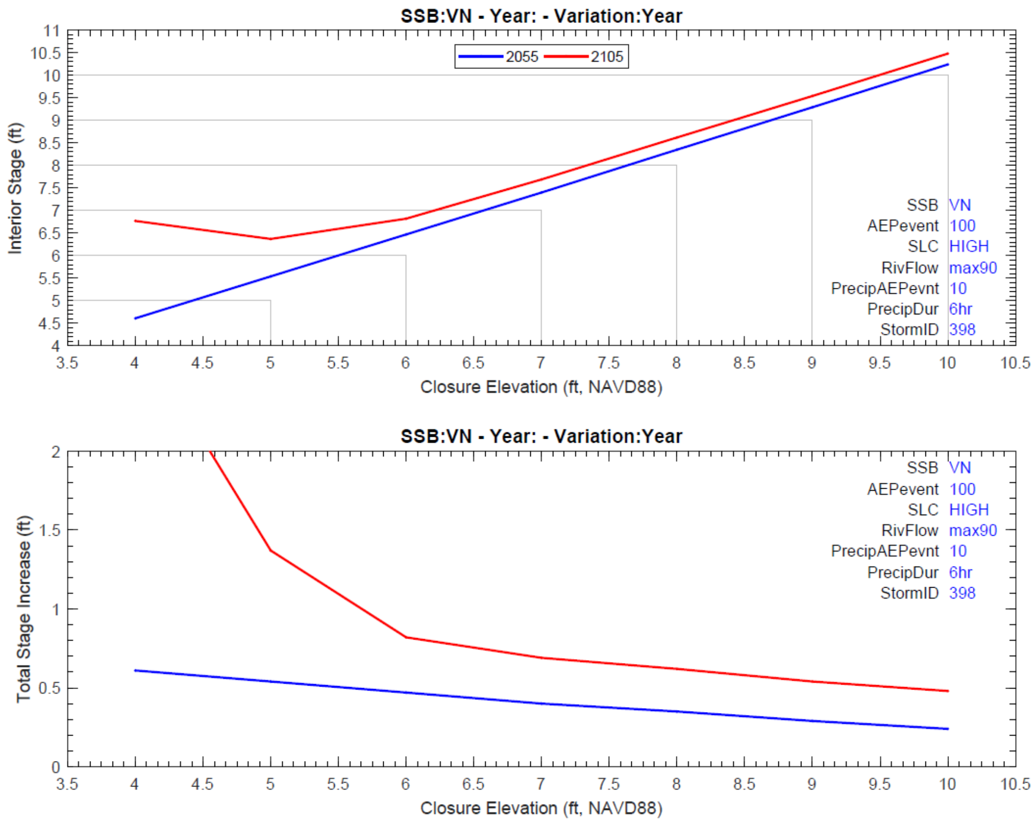

Figure 3-11: Sensitivity: Influence of evaluation year for a high SLC scenario (protected side) during a 100-year RP storm event (Here: Verrazano Narrows).

4 Summary and Conclusions

Stage increase was examined using various parameters and variations. The interior stage analysis revealed that besides sea level change scenarios, river inflow variations have the largest impact on the interior barrier stage (protected side). Barrier overtopping is expected to have no significant impact. River inflow and rainfall have the highest contribution on stage increase for the large basins (Hackensack River basin and larger). Newtown Creek however experiences a significant stage increase due to wastewater treatment inflows. Small basins which high inflows such as Newtown Creek or Hackensack are generally more susceptible to a high stage increase during closure. However, the preliminary established pumping outflow rates for those smaller basins are expected to be sufficient to ensure interior stage increase does not occur during the storm event. Stage increase for Verrazano Narrows is expected to be less than half a foot during a 1% AEP storm event considering the previously mentioned conservative estimates and a +7ft NAVD88 closure elevation. The largest stage increase for those conditions is expected at the Arthur Kill barrier and is anticipated to be approximately 1.2ft.

4.1 Design Water Levels for a closed Storm Surge Barrier

The Storm Surge Barrier has the function to impede storm surge and reduce the risk of flooding for the area behind it. Without the storm surge barrier in place, storm surge water levels have an unconstrained floodplain to flood. The presence of a barrier structure causes the storm surge elevations on the flood side to increase. At the same time, as discussed earlier, once the storm surge barrier is closed, the water levels of the interior basin can increase due to inflows from various sources as summarized in section 3.

4.1.1 Direct Head

To establish the direct head for each storm surge barrier, a set of ADCIRC simulations was analyzed. ERDC completed a set of ADCIRC simulations to represent the possible future with project conditions (FWP) to assess the increase in storm surge elevations on the flood side of the storm surge barriers as a result of the project (for project details, the reader is referred to the storm surge barrier appendix (USACE, 2022)). These model runs were analyzed and peak floodside water levels for each run were compared to the 1% AEP SWL for each SSB location. Figure 4-1 shows an example of ADCIRC model run results for Storm 525. Water level time series are shown for output points on the flood side and protected side of the Verrazzano Narrows SSB. The dashed line indicates the 1% AEP SWL.

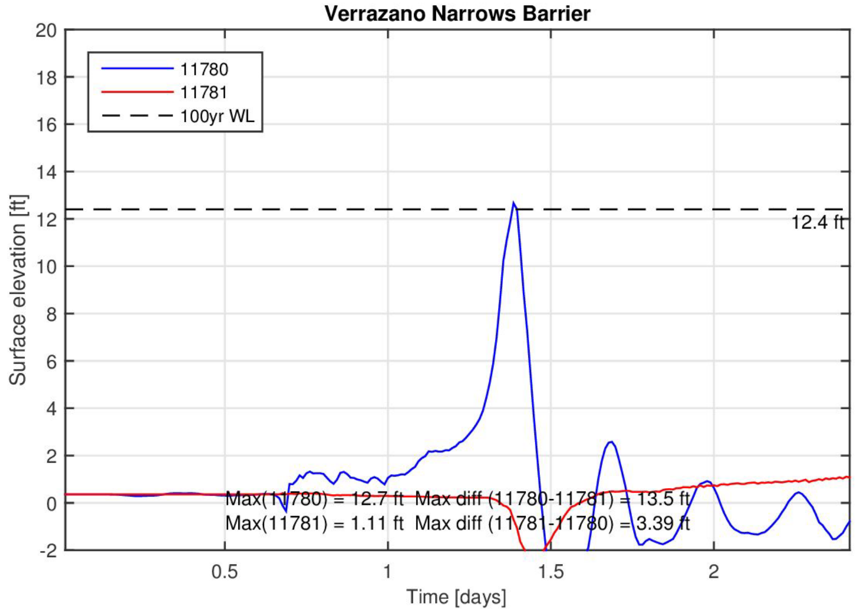

Figure 4-1: Example ADCIRC model run with project conditions for Storm 525 water level time series are shown for output points on the flood side and protected side of the Verrazzano Narrows SSB. The dashed line indicates the 1% AEP SWL

ADCIRC model runs exclude SLR hence a 1% AEP of +12.4ft is shown here. The timeseries depicted in Figure 4-1 shows that as a result of the changing windfield of the storm water levels on the flood side increase, but also that on the protected side water levels can decrease. I.e. water is pushed away from the SSB on the protected side by the storm. The direct head is the maximum difference between simultaneously observed water levels on the flood side and protected side of the SSB.

The storms, during which the maximum head differences occurred, were identified and presented for each alternative in Table 4-1. Floodside water levels were adjusted to include intermediate SLR (up to the year 2105[1]) and a closure elevation of +5ft NAVD88 was assumed to establish the inside water levels. For the Hackensack River storm surge barrier no good coverage of model data is available and data for this location was not processed. Instead, to provide a baseline for the feasibility design, water levels were estimated based on engineering judgement.

[1] The initial anmalysis was performed for storm conditions up to year 2105. The planning horizon was changed since then up to year 2095. The results of this analysis are therefore deemed conservative yet similar to what would be expected in 2095 (the differences in sea level change between these 10 years are less than 0.5ft).

Table 4-1: Direct Head Conditions for 1% AEP conditions for the storm surge barriers

Storm Surge Barrier	Head Difference (ft)	Flood Side Water Level (ft, NAVD88)	Protected Side Water Level (ft, NAVD88)	Notes
Verrazano Narrows	10.7	14.6	4.0	(storm 536 - max flood side WL 14.6ft)
Throgs Neck	14.2	16.1	2.0	(storm 536 - max flood side WL 16.1ft)[1]
Arthur Kill	11.4	17.0	5.6	(storm 363 - max flood side WL 17.0ft)[1]
Outer Harbor	9.7	15.2	5.5	(storm 536 - max flood side WL 15.2ft)
Kill van Kull	10.2	14.8	4.7	(storm 184 - max flood side WL 14.8ft)
Jamaica Bay	11.8	15.0	3.2	(storm 536 - max flood side WL 15.0ft)[1]

Notes:

1. *Difference between flood side water level for selected storm and 1% AEP water level is more than 1ft (compare with the storm surge barrier appendix (USACE, 2022)).*

4.1.2 Reverse Head

To establish the reverse head conditions the interior water level is assumed to reach an elevation of +8ft NAVD88 and the outside water level is assumed to be equal to MLLW while the barrier is closed. This is a conservative assumption as previous analyses have shown that with a closure elevation of +5ft NAVD88 an increase of interior water levels more than 2.0ft is only observed for the Kill van Kull SSB (increase of 2.2ft, see tables in section 3.2) and then only for conditions that are include a series of compounding conservative assumptions. It should further be noted that in ideal circumstances reverse head conditions are avoided altogether. The SSB operation procedures are expected to include rules that allow for opening of the SSB gates when floodside water levels are falling and water levels are equal for the inside and outside. However, for design purposes adverse conditions are considered. At this stage, it is assumed that reverse head conditions will occur and that during the storm the outside water levels will fall to a level equal to MLLW while interior basin levels remain elevated. Table 4-2 list the reverse head conditions for the major storm surge barriers.

Table 4-2: Reverse Head Conditions for 1% AEP conditions for the storm surge barriers

Storm Surge Barrier	Head Difference (ft)	Flood Side Water Level[1] (ft, NAVD88)	Protected Side Water Level (ft, NAVD88)
Verrazzano Narrows	8.4	-0.4	8.0
Throgs Neck	9.9	-1.9	8.0
Arthur Kill	8.0	0.0	8.0
Outer Harbor	8.0	0.0	8.0
Kill van Kull	8.5	-0.5	8.0
Jamaica Bay	8.0	0.0	8.0

Storm Surge Barrier	Head Difference (ft)	Flood Side Water Level[1] (ft, NAVD88)	Protected Side Water Level (ft, NAVD88)
Hackensack River	8.8	-0.8	8.0

Notes:

1. *Floodside water levels represent MLLW conditions in the year 2040 (assumed year for completion of construction).*

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