

Attachment 9
Waterfront Design Water Levels and Wave Conditions



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November 15, 2023

TECHNICAL MEMORANDUM

To: Carlos G. Peña, P.E., Foth Infrastructure & Environment, LLC

From: John Ramsey, P.E. and Sean W. Kelley, P.E.

Re: Hingham waterfront design water levels and wave conditions

Storm generated flood inundation is not a new challenge for communities surrounding Boston Harbor. Flood records dating back to the mid-1800s detail episodic storm events that have generated catastrophic storm surge and subsequently causing damage to residential and commercial infrastructure, roadways, and the natural environment. However, rising sea levels threaten to increase the occurrence of these events as well as chronic nuisance flooding from periodic spring tide cycles. East facing shorelines are most susceptible to flooding induced by extratropical storms (or Nor'easters), which may last as long as multiple days, creating prolonged exposure to atypical water elevations over and above normal astronomical tide levels, as well as storm wave action. The timescale of these storms often results in longer duration flooding that may persist until the storm has passed.

Due to the existence of the Nantasket Beach barrier complex and the series of Harbor Islands, the mainland shoreline of Hingham Harbor is protected from storm wave conditions often associated with the open Atlantic Ocean, providing relatively safe conditions for development of communities along the shoreline. However, this stretch of coastline is particularly susceptible to coastal flooding due to the low-lying topography in some areas. Based upon the topography of the Hingham downtown shoreline, much of the site is presently between 7 and 11 feet NAVD. The FEMA Stillwater 100-year flood elevation for the area is 10 feet NAVD, where nearby Boston Harbor recorded water elevations of 9.6 feet NAVD in both February 1978 and January 2018. A portion of the effective FEMA Flood Insurance Rate Map (FIRM) is shown in Figure 1. Portions of the downtown area with FEMA Zone AE at elevation 11 feet NAVD indicate low-lying areas that are inundated to approximately 10 feet NAVD with a 1-ft storm wave envelope above the still water flood elevation.

With the above understanding, most coastal flood mitigation efforts for site improvements can be focused specifically on elevating the infrastructure. As depicted in Figure 1, the seaward edge of the site is exposed to storm wave action; however, according to FEMA, waves impacting the site are relatively small and wave action within developed areas is limited, with only minor influence in the developed areas beyond the

wharves. Elevating the flood protection infrastructure along the harbor shoreline can provide effective means for eliminating storm tide pathways through the downtown Hingham area.

To quantify design requirements for both coastal storm surge and wave action, an analysis of potential future sea level rise impacts and storm wave impacts was performed for the site-specific conditions. In support of the coastal engineering design analysis, and the development of management alternatives, past and future sea level rise (SLR) trends were analyzed. The analysis of projected SLR is necessary to understand appropriate design levels for future infrastructure improvements. In addition, an assessment of storm wave conditions associated with existing and future storm surge levels is necessary to design the level of shore protection necessary for future conditions.

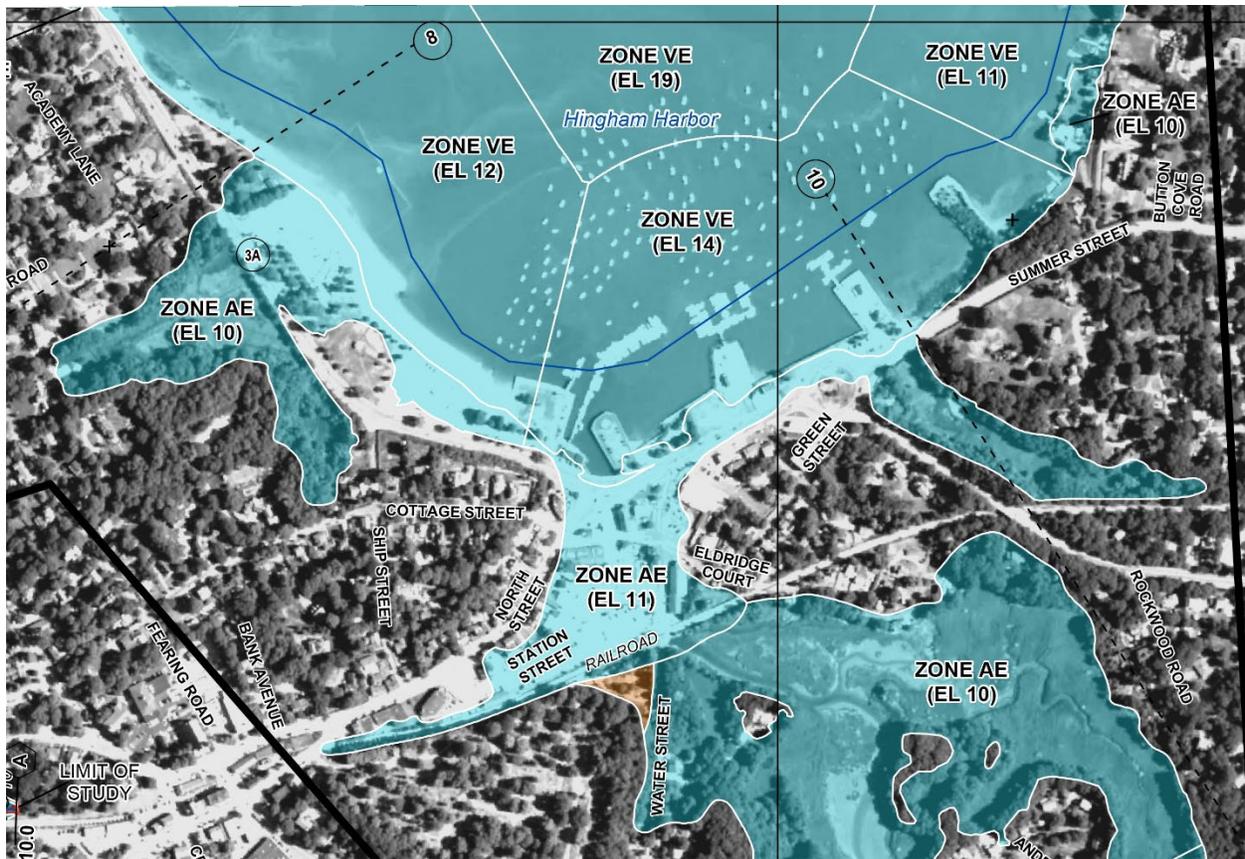


Figure 1. Portion of updated FEMA Flood Insurance Rate Map (FIRM) for Hingham, last updated on August 14, 2015

A. Updated Sea Level Rise Analysis

The exposure of population and infrastructure to flooding along the Hingham Harbor shoreline has significantly increased over the last several decades. Several factors including coastal urbanization, aging infrastructure, alterations to the natural environment, and sea level rise have all contributed to the increase in flood exposure

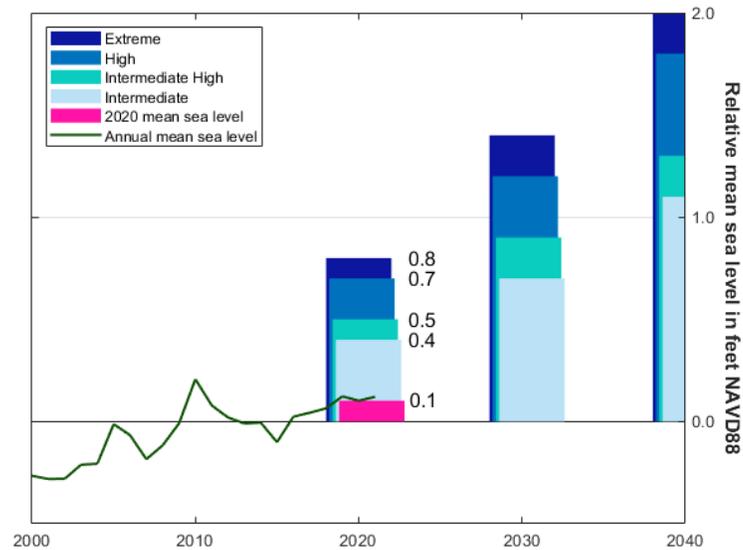


Figure 4. Comparison of probabilistic sea level rise projections from Resilient MA (DeConto and Kopp, 2017) and measured annual mean sea level for Boston Harbor, Massachusetts.

More recent sea level rise projections from NOAA (Sweet, et al., 2022) suggest significantly lower projected future sea level rise rates for Boston (downscaled from the full U.S. analysis), especially between the present and 2050. Figure 5 provides the updated NOAA projections, where the ‘intermediate’ projection represents conditions that are about as likely as not to occur or, in other words, a 50% chance of occurrence. It should be noted that the NOAA utilization of the term ‘intermediate’ follows standard statistical terminology, where the intermediate result represents the middle curve between the two extremes (high and low) or the 50% chance of occurrence. The Resilient MA documents use a different definition of the ‘intermediate’ scenario, which likely leads to further confusion when attempting to compare the various sea level rise projections. In the case of Resilient MA, the ‘intermediate’ sea level rise projection represents a more unlikely scenario, i.e., the ‘unlikely to exceed’ threshold or a 17% probability of exceedance, rather than the 50% probability of exceedance used by NOAA.

As illustrated in Figure 5, the ‘intermediate’ NOAA sea level rise projection generally matches the ‘observed trajectory’ projection to 2050, which was based upon extrapolating the observed sea level rise trends between 1970 and 2020. Further, Figure 6 demonstrates the applicability of utilizing more moderate sea level rise projections, as the observed sea level rise in Boston between 2000 and 2020 (shown in gray) is below all of the projections evaluated by Sweet, et. al. (2022). Based on the NOAA tide data, the Boston sea level rose 0.33 feet between 2000 and 2020; therefore, in 2020, the mean sea level was 0.03 feet NAVD88 since the mean sea level in 2000 was -0.30 feet NAVD88.

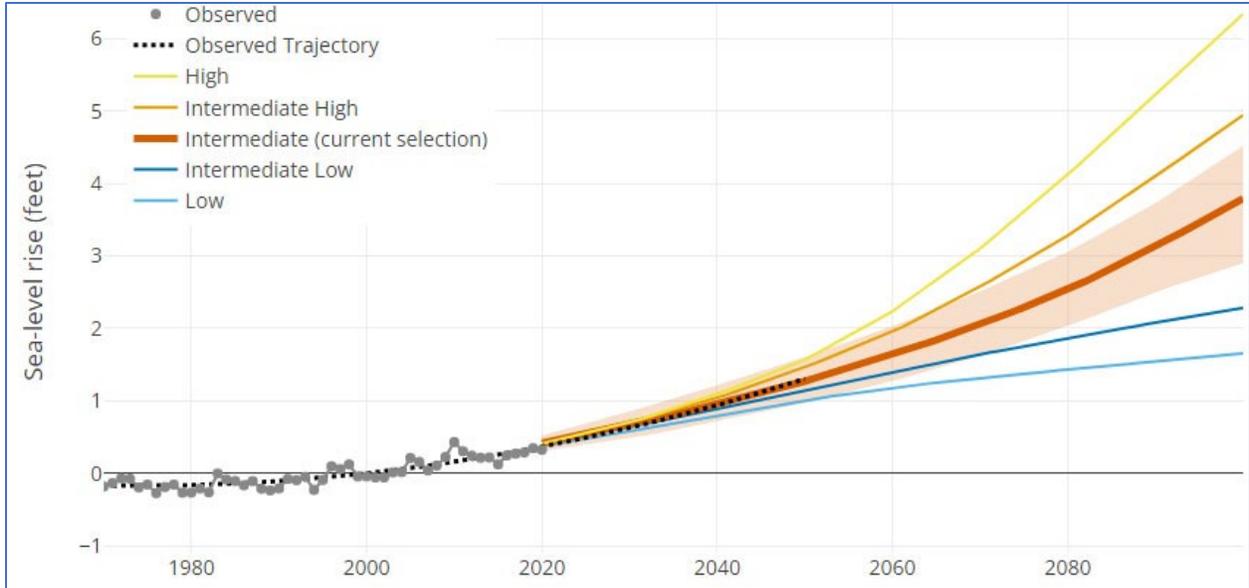


Figure 5. Projected sea level rise for Boston Harbor, Massachusetts based upon modeling analyses performed by NOAA (Sweet, et. al., 2022). Results for a full range of scenarios can be found at: <https://sealevel.nasa.gov/flooding-analysis-tool/projected-flooding?>

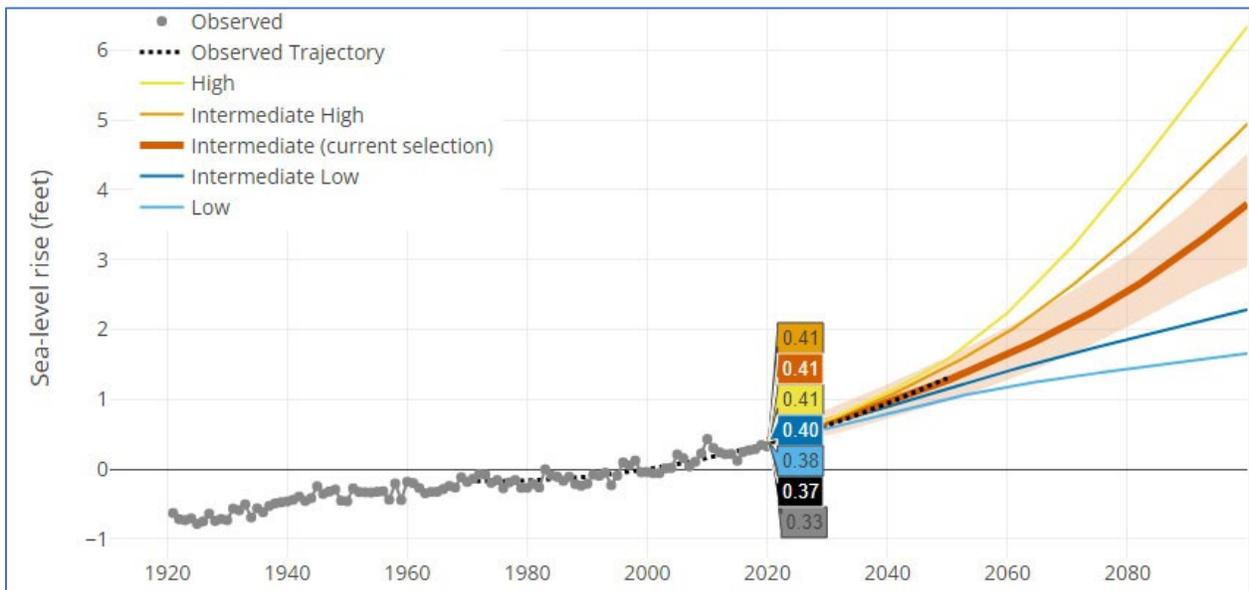


Figure 6. Projected sea level rise for Boston Harbor, Massachusetts based upon modeling analyses performed by NOAA (Sweet, et. al., 2022). The colored numbers represent the modeling results for the various scenarios for 2020, as well as the observed mean sea level. Results for a full range of scenarios can be found at: <https://sealevel.nasa.gov/flooding-analysis-tool/projected-flooding?>

For Boston, the NOAA projections for 2050 are shown in Figure 7. According to Sweet, et al. (2022):

As a result of improved science and the updated framework and procedure for generating the Global Mean Sea Level (GMSL) scenarios, the time path of the scenarios - particularly the higher scenarios - is now more realistic and consistent with current process-based understanding. In this report, the range between the Low and High scenarios in 2020, 2030, 2040, and 2050 is now 0.02 m [0.07 feet], 0.06 m [0.20 feet], 0.15 m [0.49 feet], and 0.28 m [0.92 feet], respectively. In other words, there is less divergence between the GMSL scenarios in this near-term time period, which reduces uncertainty in the projected amount of GMSL rise up to the year 2050. The Low scenario remains largely the same between this report and Sweet et al. (2017); this range reduction reflects a downward shift in the higher scenarios in 2050 and times prior, as discussed above. As an example, the projected value in 2050 for the High scenario in this report (~0.4 m [1.31 feet]) is the same as that for the Intermediate-High projected value in 2050 in Sweet et al. (2017).

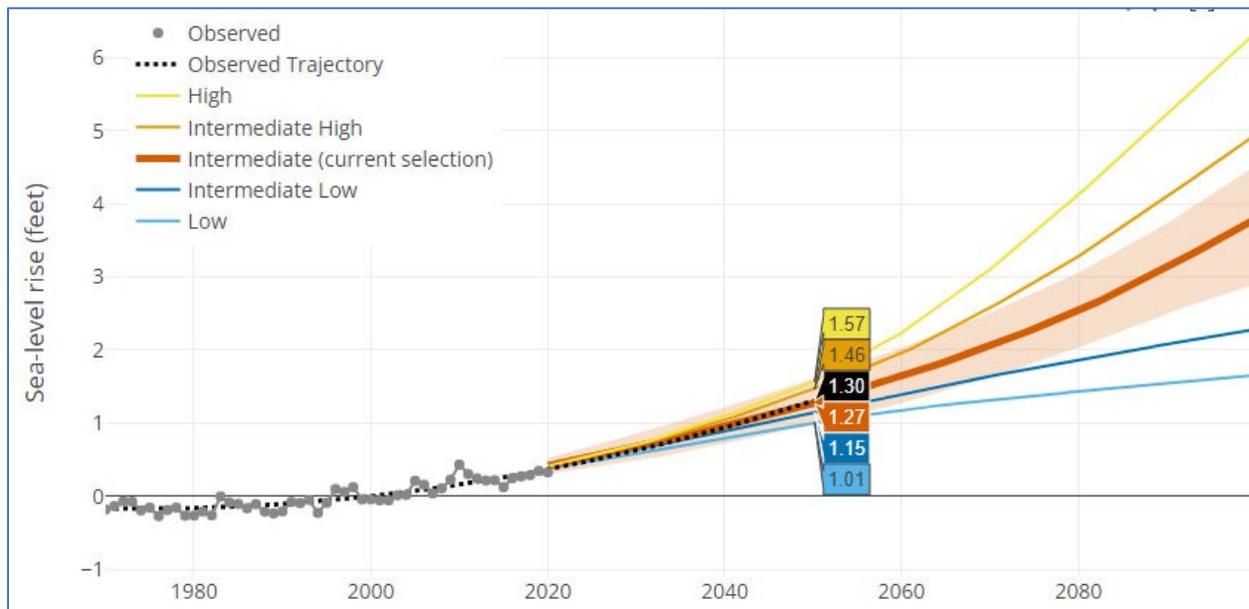


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Based on this updated information, a reasonable expectation for near-term (through 2050) sea level rise in the Boston region, inclusive of the project area, is within the range of sea level rise projections illustrated in Figure 7. In this case, the 2050 mean sea level can be expected to be approximately 1.3 feet above the 2000 level or approximately 1.0±0.3 feet NAVD88. This value is substantially lower than the projections provided in the Resilient MA documentation (Table 1). Specifically, the updated NOAA evaluation indicates that expected sea level rise in Boston by 2050 is ~40% of the value recommended for planning by Resilient MA.

For planning of future infrastructure, incorporating a safety factor to accommodate potential future sea level rise is warranted; therefore, the Resilient MA ‘High’ sea level rise projections are useful to ensure that future development is safe from the impacts of sea level rise. However, when developing flood mitigation strategies for existing infrastructure, designing for future sea level conditions that are ‘extremely unlikely to occur’ can be both cost-prohibitive and unnecessary. Specifically for the sites evaluated along the Hingham Harbor shoreline, appropriate design levels for flood mitigation strategies should be based upon expected future sea levels, which NOAA project to be approximately 1.0 feet NAVD in 2050 and 1.8 feet NAVD in 2070. As the proposed flood mitigation strategies involve elevating seawalls, revetments, and coastal dunes, it will be a simple process to modify the design if future sea level rise exceeds the intermediate projections developed by NOAA (Sweet, et al., 2022). Table 2 provides expected future sea level rise for 2030, 2050, and 2070, based upon NOAA estimates (Sweet, *et al.*, 2022). Figure 8 provides both the 2022 NOAA projections and the projections that have been utilized for project planning by SCS engineers over the past decade that was based on Intergovernmental Panel on Climate Change (IPCC) modeling with the addition of ice sheet contribution from Rignot et al., 2011. Good agreement between these two sets of projections indicates that this pragmatic approach continues to provide a valid science-based methodology for evaluating future sea level rise, especially in the near-term (next 30 to 40 years).

Scenario	Probabilistic projections	2030	2050	2070
NOAA - Intermediate	Conditions that are about as likely as not to occur or, in other words, a 50% chance of occurrence (RCP 8.5)	0.4	1.0	1.8

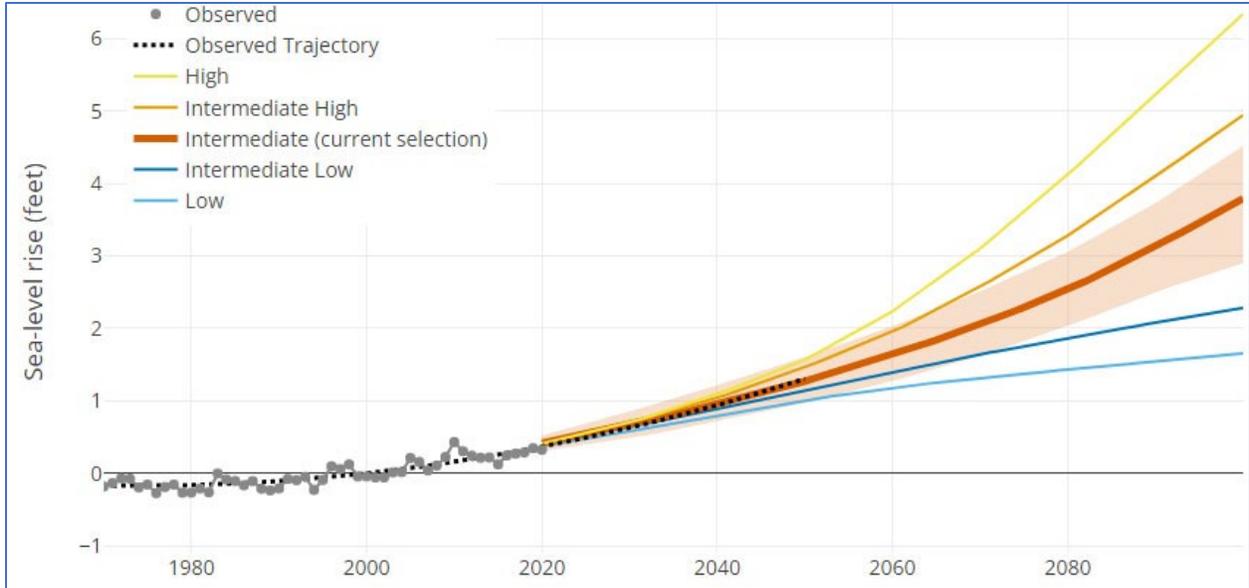


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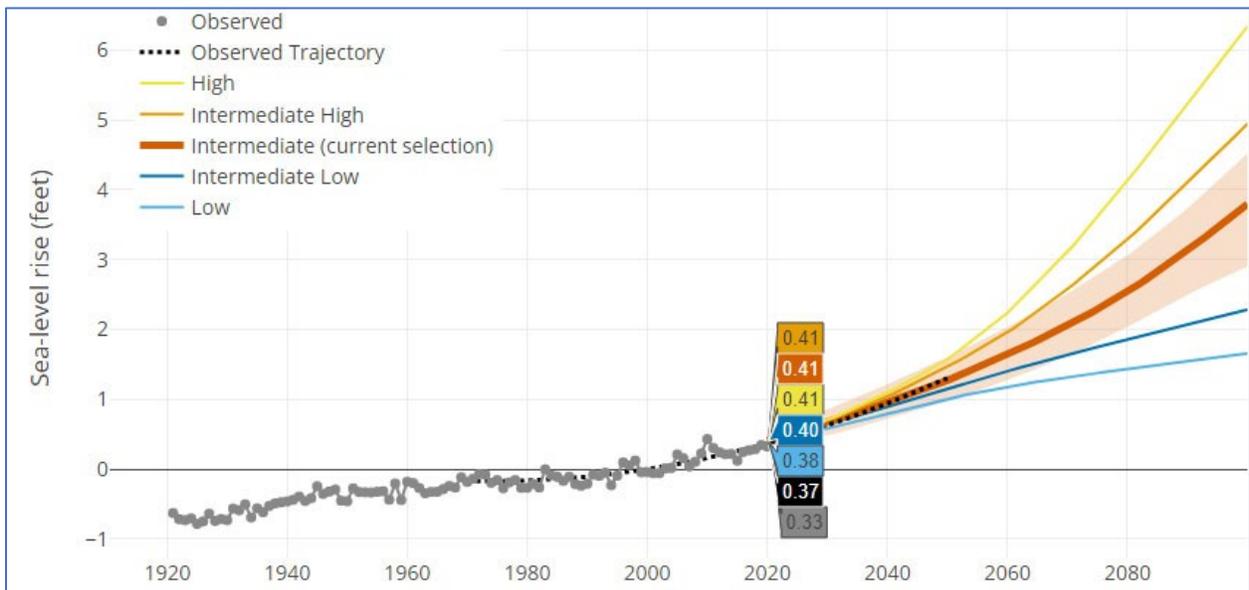


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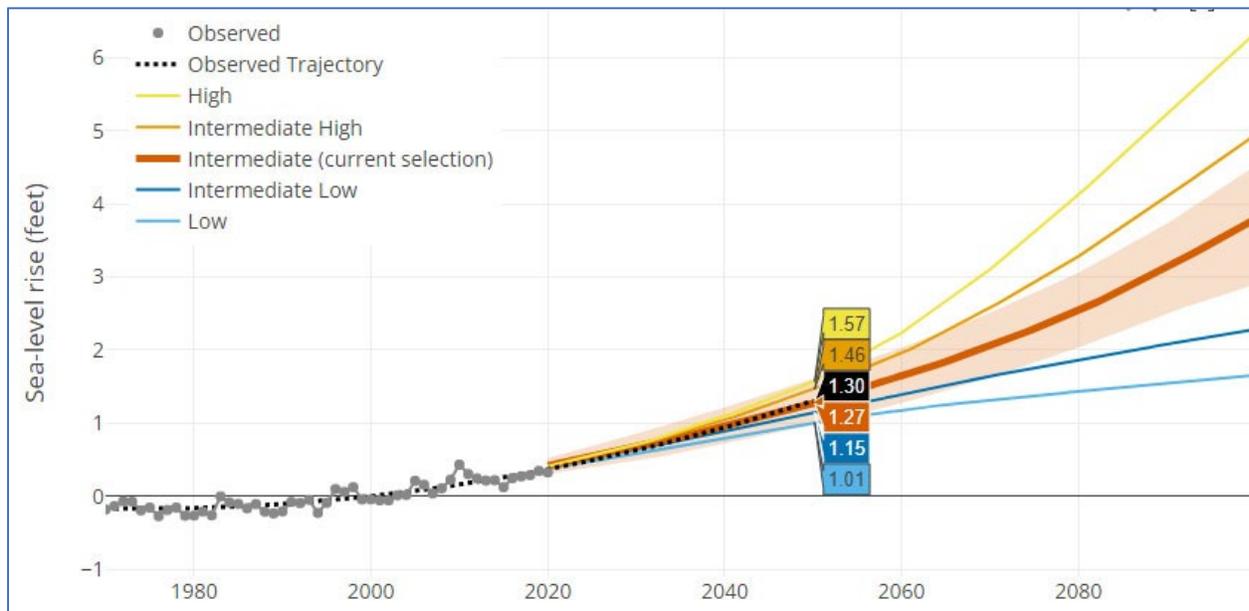


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Table 2. Relative mean sea level (feet, NAVD88) projections for Boston, MA as presented in Sweet, et al., 2022				
Scenario	Probabilistic projections	2030	2050	2070
NOAA - Intermediate	Conditions that are about as likely as not to occur or, in other words, a 50% chance of occurrence (RCP 8.5)	0.4	1.0	1.8

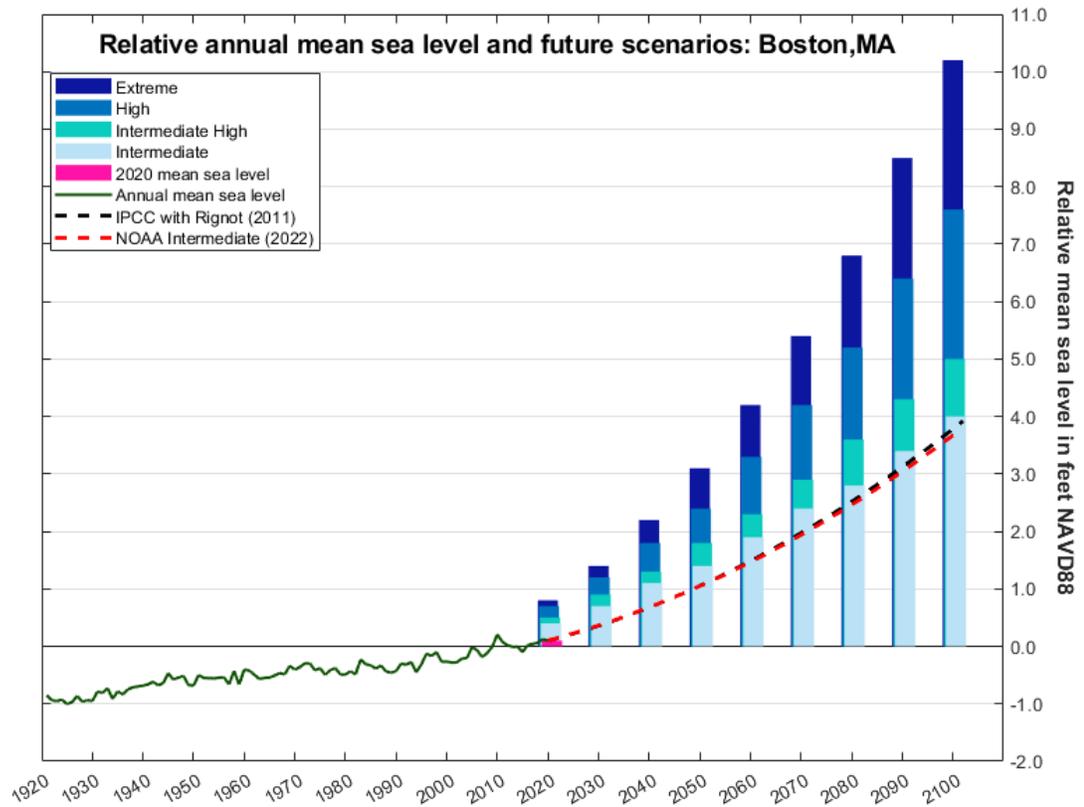


Figure 8. Sea level rise projections with the latest NOAA projections (adjusted to account for current mean sea level; dashed red line) and a curve representing flood projections from the IPCC augmented by sheet ice contributions determined by Rignot et al. (2011; dashed black line). The bar plot represents the sea level rise projections presented in Resilient MA.

B. Design Wave and Overtopping Analysis

A coastal engineering analysis was performed at 10 transects along the Hingham Harbor waterfront (Figure 9) to determine specifications for structural and soft-engineering interventions to improve storm resiliency in this area. General information about each transect is provided in Table 3. For this analysis, a two-dimensional (2D) wave model was developed in order to determine storm wave conditions along the Hingham Harbor waterfront. For shoreline reaches where hard engineering structures are in place, a wave overtopping analysis was performed in order to determine structure heights that would reduce wave overtopping discharges to levels that would be safe for paved surfaces during storms. At the two analysis transects placed at Bathing Beach, a cross-shore morphological model analysis was performed in order to determine attributes of a dune that would be necessary to withstand storms and be an effective barrier to storm surges and waves in Boston Harbor.

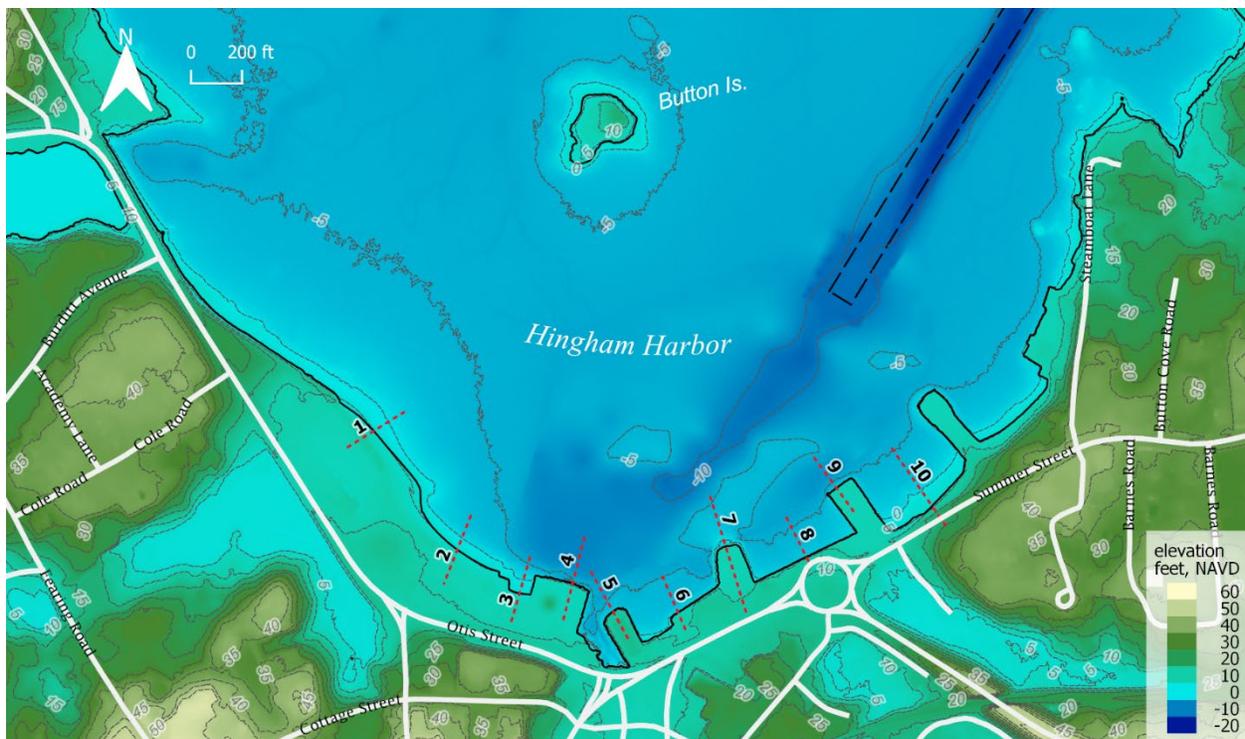


Figure 9. Map of Hingham Harbor waterfront with the location of the ten coastal analysis transects along the study area shoreline. The US Army Corps of Engineers' navigation channel limits are indicated by the black-dashed line.

Table 3. Coastal analysis transects (as mapped in Figure 9).

Transect no.	Transect description	Foth plan wall station	present condition	present crest elev. ft, NAVD	proposed intervention
1	Bathing Beach W, dune		beach	10.8	dune
2	Bathing Beach E, no dune		beach	8.5	dune
3	Otis Street	0+50	beach	8.7	dune/berm
4	Town Wharf	3+25	vertical wall	7.2	wall crest extension
5	Witney Wharf	11+50	vertical wall	10.5	wall crest extension
6	Summer Street, W of rotary	15+00	vertical wall	7.3	revetment
7	Kimball's Wharf	18+25	vertical wall/toe revetment	8.6	wall crest extension
8	Rotary	21+00	vertical wall	10.6	wall crest extension
9	Barnes Wharf	25+75	vertical wall	7.2	wall crest extension
10	Summer Street, E of rotary	31+00	vertical wall	9.6	wall crest extension

Data Sources

Several data sets were compiled as part of this analysis. These data are intended to represent the present, site-specific physical conditions in Hingham Harbor and along the shoreline reach of this study. Most data used in this analysis were retrieved from public sources of quality-controlled data (for example, bathymetry, tide, and wind data). Some data used in this analysis were available from other work funded by the Town (for example, sediment grain size data and an elevation survey of the waterfront).

Elevation Data. Though recent LiDAR topography is available for the study area, topobathy LiDAR surveys in this region of Boston Harbor do not provide complete coverage of ocean bottom elevations in Hingham Harbor. Therefore, the main source of topography and bathymetry depended upon as the main source of elevation data is the 2016 USGS Coastal National Elevation Database (CoNED) digital elevation model (DEM), which incorporates several data sets from many government sources to create a continuous topographic surface on a one-meter grid. Sources include recent (up to 2016) LiDAR surveys where coverage is available, and NOAA single-beam fathometer measurements in areas that have no LiDAR data. A contour map of the 2016 USGS DEM data in the vicinity of Hingham Harbor is shown in Figure 10.

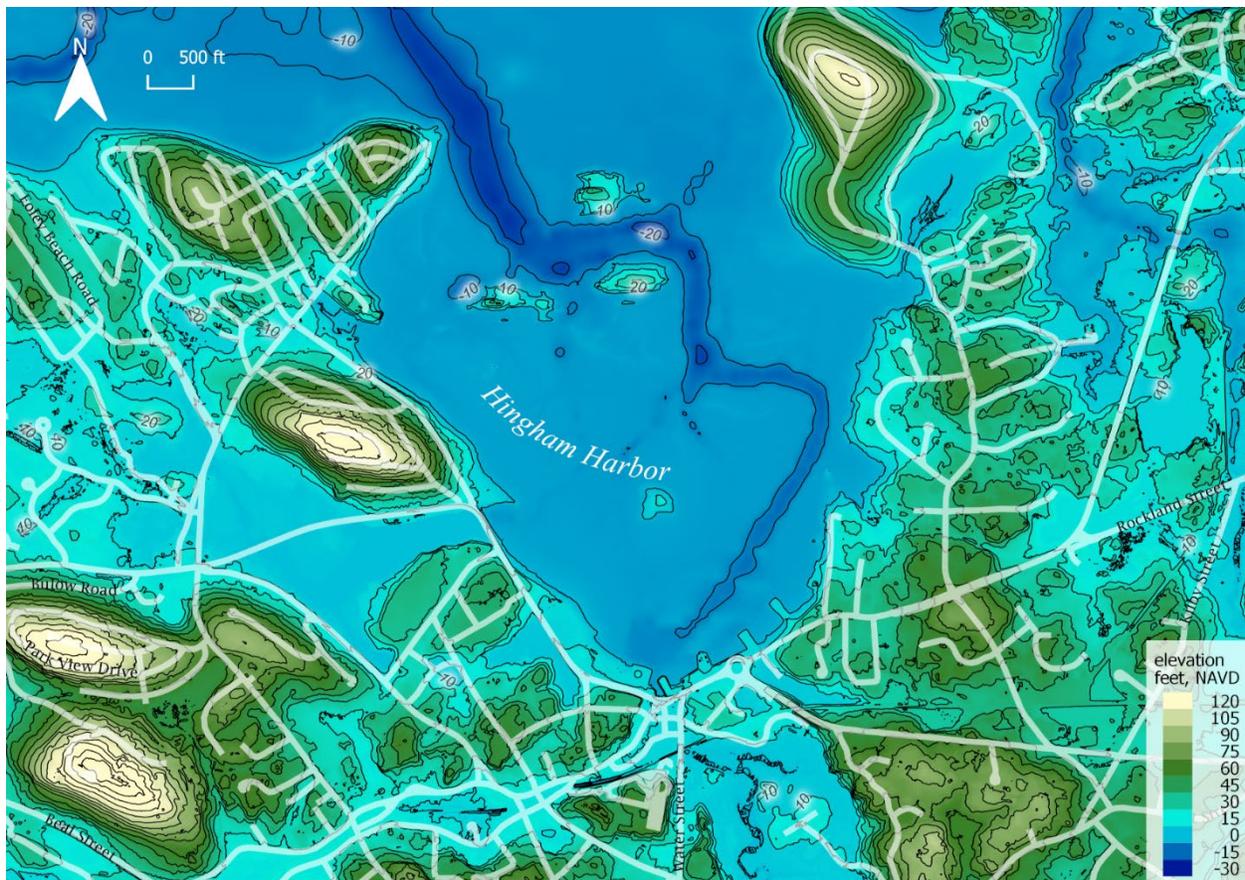


Figure 10. Map of 2016 USGS CoNED elevation data, in the vicinity of Hingham Harbor. Contours lines are shown at 10-foot intervals.

Wave Data. The USACE Wave Information Study (WIS) hindcast provides wave data time series at dozens of stations along the US coastline. Wave parameters (including H_s wave height, T_p Peak Period, and mean direction for sea and swell components of the sea state) are available at a regular hourly interval starting January 1, 1980 through to January 1, 2021. Though NOAA (through its National Data Buoy Center, NDBC) maintains a wave buoy in Massachusetts Bay (station 44013), this record does not have directional wave data until June 2012, and there are significant periods within the time span of the record (1984 to present) where no data are available. Because of this, WIS hindcast is better suited for the development of the extreme wave conditions.

The hindcast record from WIS station 63052 (mapped in Figure 11) was used for this study. This station is about 13 nautical miles northeast of the entrance to Boston Harbor, in Massachusetts Bay, in an area with ocean depths of about 180 feet. 63052 is the closest WIS station to Boston Light on Little Brewster Island, at the entrance to Boston Harbor. Rose plots showing the occurrence of wave height and periods by compass sector is shown in Figure 12. From this plot it is seen that the most commonly occurring wave direction is the east sector, from where wave come from 26.6% of the record. 72.6% of wave heights in the record have a H_s significant wave height that is less than 3 feet. In 43.7% of the span of the record, wave periods are between 6.5 and 9.5 seconds.

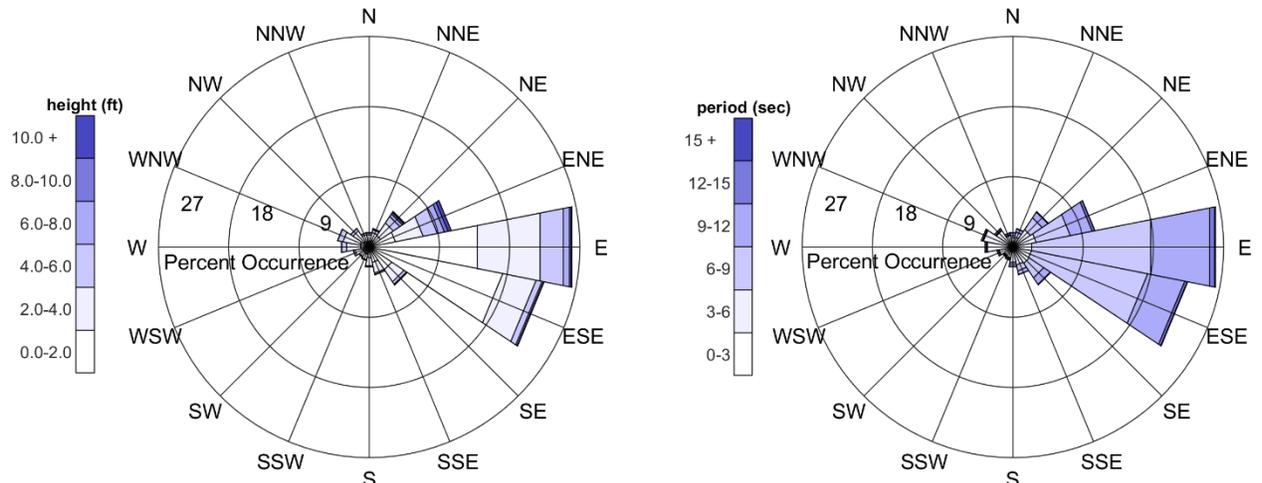


Figure 12. Rose plots of significant wave height (H_s , left) and peak wave period (T_p , right), for the WIS hindcast record at station 63052. Grey-tone segments indicate the percentage of time wave weights and periods in the record are within the indicated ranges for each compass sector.

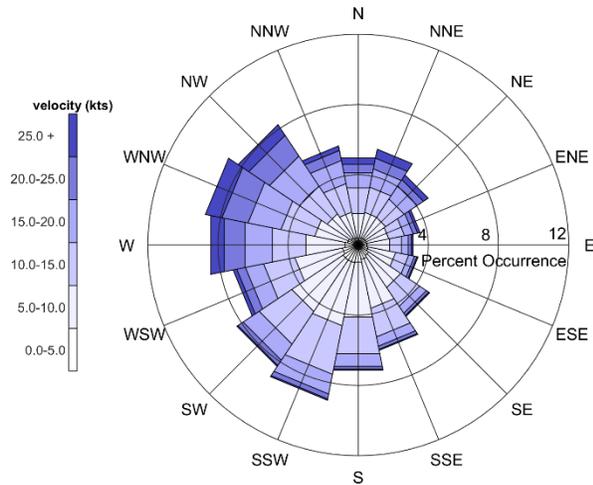


Figure 13. Rose plots of wind speed (knots) for the WIS hindcast record at station 63052. Grey-tone segments indicate the percentage of time winds in the record blow within the indicated speed range from the indicated compass sector.

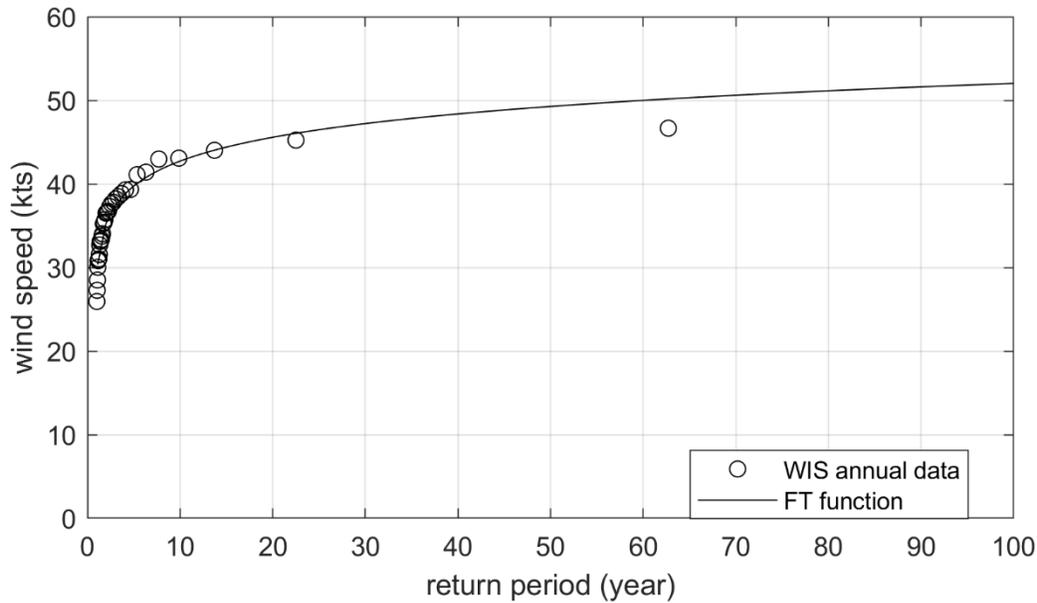


Figure 14. Plot of return period wind speeds for the north compass sector, using the WIS hindcast record (1980 through 2020) at station 63052. Sorted annual maximum windspeeds are indicated by the circle markers, and the Fischer-Tippet (FT) fit of the data is shown as the solid line. R^2 correlation of the FT PDF is 0.98, with an RMS error of 1.1 knots.

Water Level Data. Water elevation data used in this analysis include tide data available from the NOAA tide station in Boston Harbor, and extremal return period water levels available from the FEMA Flood Insurance Study (FIS) for Plymouth County (2021). NOAA Boston tide data recorded during the recent December 23, 2022 northeast storm (Figure 15) that impacted the region were downloaded from the NOAA Tides and Currents website (<https://tidesandcurrents.noaa.gov>). FEMA publishes return period still water elevation (SWEL) data for several transects along the shoreline of Plymouth County, including a transect in Hingham Harbor that is next to Barnes Wharf. At this transect (Plymouth County FIS Transect 10) the reported 10-year SWEL is 8.4 feet NAVD, and the 100-year SWEL is 9.7 feet NAVD. The maximum water level recorded during the Dec 23, 2023 northeast storm is 8.4 feet, equal to the 10-year SWEL Hingham Harbor. A tide time series for the 100-year return period event was created by scaling the surge component of the total water level (which is the combination of the astronomical tide + surge) so that the maximum water level reached the 100-year SWEL.

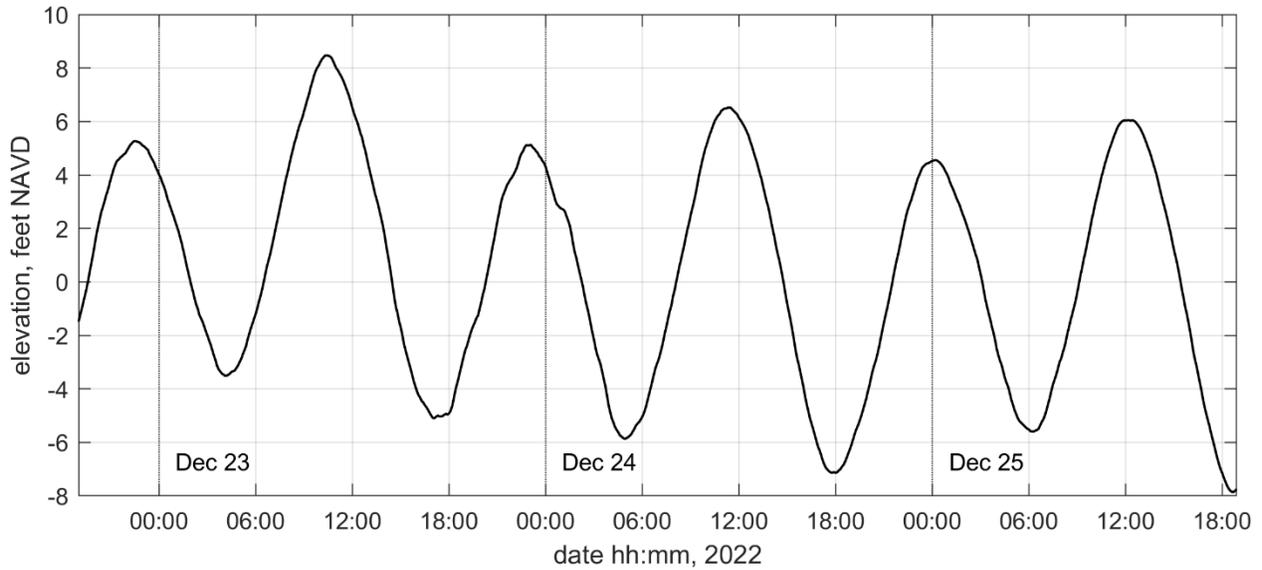


Figure 15. NOAA measured tides in Boston Harbor during the Dec 2022 northeast storm, when peak water levels reached 8.4 feet NAVD.

Sediment Data. For coastal analysis transects in this study that are for beach areas along the Hingham waterfront, sediment grain size data was taken from an existing construction specification for Bathing Beach. In this specification, a minimum and maximum acceptable grain size distribution is provided (Figure 16). Median (D50) sediment grain sizes from these two distributions are 0.50 mm for the minimum, and 1.18 mm for the maximum.

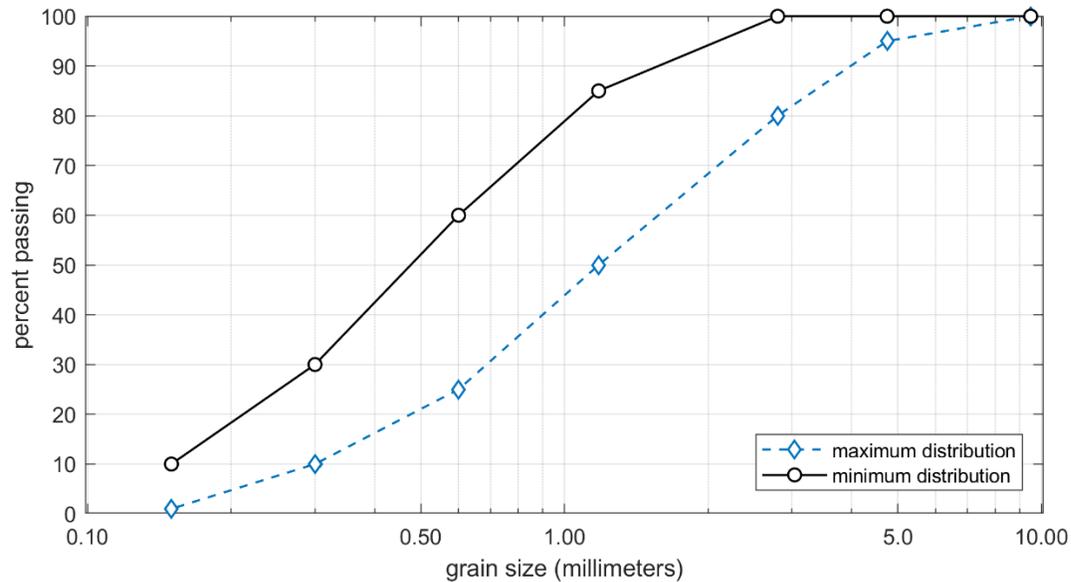


Figure 16. Bathing Beach specified grain size distribution curves that designate the minimum (solid black line) and maximum (dashed blue line) allowable percent passing for indicated sand grain sizes. The D50 grain size resulting from these two distributions is 0.50 and 1.18 mm for the minimum and maximum specified distributions, respectively.

Design Wave and Overtopping Analysis

As part of this analysis, design wave conditions were computed for the waterfront area of the Hingham Harbor using the SWAN 2D wave model (Booij, 1999). Model inputs included wind speeds and wave parameters developed from the extremal analysis of the USACE WIS wave hindcast record (Table 4). Wave model outputs were used to determine seawall height elevations that would limit wave overtopping rates in the waterfront area to levels that would not cause damage to structures or paved surfaces. SWAN wave model output was also used as inputs to 2D cross-shore profile models of two transects at Bathing Beach. These cross-shore models were used to determine dune elevation and crest widths that would be required to capably withstand extreme storm conditions.

Table 4. Storm wind and wave characteristics (1% return frequency) used in the runs of the Hingham SWAN wave model.

Storm parameter	Compass sector						
	WNW	NW	NNW	N	NNE	NE	ENE
Sustained wind speed (kts)	42.7	44.9	48.7	52.0	50.6	48.4	48.1
Offshore wave height (ft)	8.8	9.2	9.5	11.5	15.2	21.7	23.3
Offshore wave mean period (sec)	6.8	6.9	7.0	7.4	8.3	9.9	10.3
Still water level (ft, NAVD)	9.7	9.7	9.7	9.7	9.7	9.7	9.7

SWAN Model Development. Development of the Hingham Harbor SWAN model proceeded by first creating the numerical grid, using available topography and bathymetry elevation data. Storm conditions run with the model were developed from the extremal analysis of winds and waves from the WIS hindcast record in Massachusetts Bay, at station 63052. The SWAN model for Hingham Harbor consists of three cartesian grid meshes. They range from a coarse mesh with a 131-foot (40-meter) mesh that covers all of Boston Harbor and its entrance to Massachusetts Bay, a 49-foot (15-meter) intermediate mesh that covers Hingham Bay, and finally a 2.2-foot (2-meter) fine-scale mesh in the area of the Hingham Harbor waterfront. The bathymetry and extents of these three grids is shown in Figure 17. Boundary conditions for each of the finer scale grids is extracted from the next-courser grid, which allows for a high level of grid refinement in the particular area of interest, while allowing for a larger grid mesh in areas where fine detail is not needed. In this case the Hingham Harbor grid is nested within the Hingham Bay grid, which in turn is nested within the Boston Harbor/Massachusetts Bay grid.

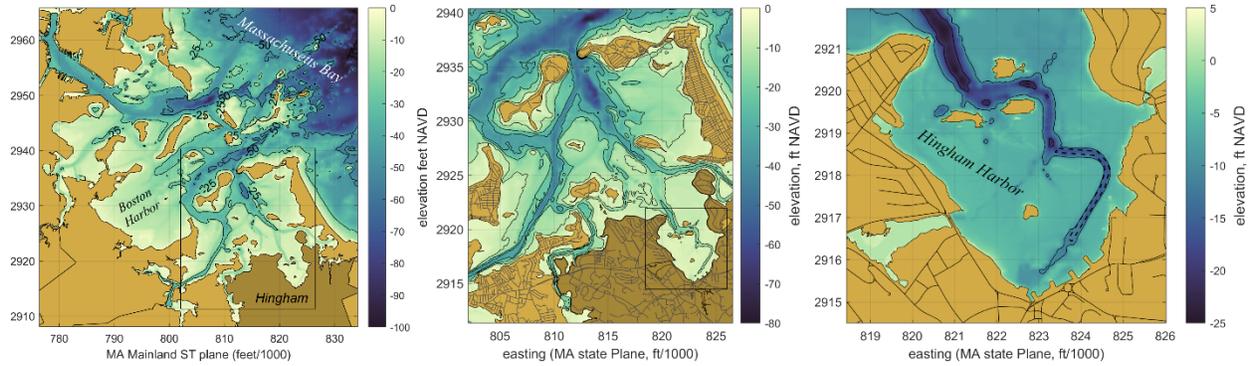


Figure 17. Contour plots of bathymetry used in the wave model coarse 40-meter grid of Boston Harbor, the intermediate 15-meter grid of Hingham Bay, and the fine nested 2-meter grid of Hingham Harbor, including the shoreline of the project area.

Winds blowing from the north compass sector generate the largest waves in Hingham Harbor. For this wave case, significant H_s wave heights range between 2.5 and 2.8 feet between Bathing Beach and Barnes Wharf. Peak wave periods range between 2.7 and 3.3 seconds. In addition to the 1% storm with present mean sea level, the same wave model cases were run for expected 2050 and 2070 mean sea levels. Wave heights in the harbor do increase slightly for these projected future conditions, but by only about a maximum of 5% even for 2070 water levels.

Shaded contour plots of wave heights in Boston Harbor, Hingham Bay, and Hingham Harbor are presented in Figure 18 for 100-year storm conditions with winds blowing from the north, and with present mean sea level.

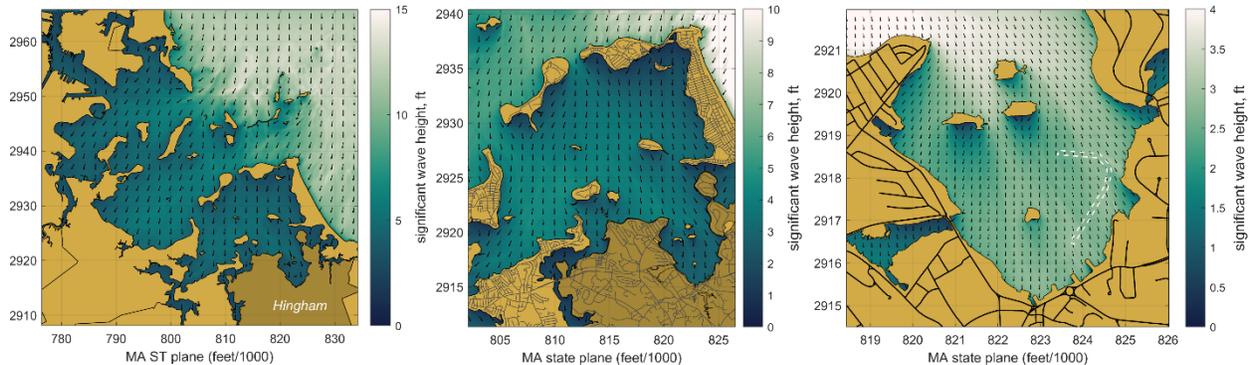


Figure 18. Contour plots of wave height (H_s) and direction (arrows) for the modeled 100-year storm conditions with winds blowing from the north, for the Boston Harbor grid (left), Hingham Bay grid (center), and the Hingham Harbor fine grid (right).

Wave Overtopping of Structures. For the eight total analysis transects where coastal structure improvements are being considered, wall crest elevations were determined that limit the amount of wave overtopping flows to acceptable rates. Basically, given wave conditions and water levels that occur at each analysis transect, wall crest elevations were iterated to reduce overtopping rates to 50 liters/second per meter length of wall (0.54 cfs/foot), which according to Table VI-5-6 of the USACE Coastal Engineering Manual (2011) is the upper limit where paved surfaces will resist damage.

Wall crest height determinations were performed using methods available from the EurOtop Manual (2018), for vertical walls (transects 4, 5, and 7 through 10) and for sloped revetment (transect 6).

Vertical Wall Transects. From the EurOtop Manual, the overtopping rate (q) on a vertical wall is found using the equation:

$$\frac{q}{\sqrt{g \cdot H_{mo}^3}} = 0.054 \exp \left[- \left(2.12 \frac{R_c}{H_{mo}} \right)^{1.3} \right]$$

where R_c is the structure freeboard

H_{mo} is the offshore significant wave height

Revetted Transects. The discharge of water from waves over the crest of a structure is referred to as wave overtopping. Methods presented in the EurOtop manual (2018) were used to determine overtopping rates for

From the EurOtop Manual, the overtopping rate (q) on a simple slope is found using the equation:

$$\frac{q}{\sqrt{g \cdot H_{mo}^3}} = \frac{0.026}{\sqrt{\tan \alpha}} \cdot \xi_{m-1.0} \cdot \exp \left[- \left(2.5 \frac{R_c}{\xi_{m-1.0} \cdot H_{mo}} \right)^{1.3} \right]$$

where R_c is the structure freeboard

H_{mo} is the offshore significant wave height

α is the structure slope angle

ξ is the surf similarity parameter, as before

and h_{wall} as the crown wall height above the revetment.

For the two revetment transects, wall crest elevations were determined for slopes of 1:1.5 (v:h) and 1:2.0 (v:h). 1:1.5 is generally accepted as the maximum slope for stone revetments. Flatter slopes generally reduce wave runoff elevations and overtopping volumes for walls with the same crest elevation.

Overtopping Analysis Results. Wall crest elevation determined using the EurOtop methodology are presented in Table 5 for the vertical wall transects and Table 6 for revetment sections.

Table 5. Vertical wall crest elevations, in feet NAVD88, to prevent damage to paved surfaces from wave overtopping, for present, 2050, and 2070 sea level scenarios.

transect	Sea level scenario		
	present	2050	2070
4	11.0	12.1	12.9
5	11.0	12.0	12.9
7	10.9	12.0	12.9
8	10.9	12.0	12.8
9	10.9	12.0	12.8
10	11.0	12.1	12.9

Table 6. Revetment slope crest elevations, in feet NAVD88, to prevent damage to paved surfaces from wave overtopping, for structures with 1:1.5 and 1:2.0 slopes, and for present, 2050, and 2070 sea level scenarios.

transect	Sea level scenario					
	present		2050		2070	
	1.5	2.0	1.5	2.0	1.5	2.0
6	13.0	11.9	14.0	12.9	14.8	13.7

Beach Transects. For the two beach transects at Bathing Beach (transects 1 and 2), a cross-shore morphological model was employed to determine the dimensions of a dune which would serve as an erodible barrier to ocean surges up to the 1% (100-year) still water level (SWEL), for the same three different MSL scenarios used in the overtopping analysis. The cross-shore transport model XBeach-X (Roelvink, *et al.*, 2015) was used to determine a dune fill elevation and crest width which would withstand a major storm event with some remaining flood protection capacity. 1-percent wave conditions applied to the model open boundary were derived from the SWAN wave model of Boston and Hingham Harbors, by applying 1-percent winds from the north (52.0 kts). The Boston tide record from the December 23, 2022 northeast storm was used as the source of input water levels during the XBeach simulation. The storm surge component of the recorded Boston tide was scaled up so that the peak total water level would reach the present FEMA-designated 1-percent SWEL (9.7 feet NAVD for present MSL conditions). Circa 2016 topography/bathymetry elevation data from the USGS CoNED DEM was interpolated to the Bathing Beach Point XBeach model transects.

Constructed dune crest width and elevation were iterated with the goal finding a configuration which would have some portion of the dune remain at its original height after the duration of the storm. For present sea levels, the existing dune at Bathing Beach is able to withstand the 1-percent storm (Figure 19). The present dune has a foreshore and backside slope of approximately 1:4, a dune crest of 10.7 feet NAVD, and a crest width of about 22 feet. The dune toe (start of the foreshore slope on the beach) is at an elevation of about +7 feet NAVD. For 2050 MSL conditions, the dune is able to withstand the 1-percent storm if the crest elevation is increased to +11 feet NAVD. For 2070 MSL conditions, the dune crest elevation would need to be increased to +12 feet NAVD to withstand the 1-percent storm (Figure 20).

At Transect 2 (Figure 21), there is presently no dune in place, and the beach berm has a crest elevation of +8.5 feet NAVD. A dune with similar dimensions to the existing dune at Transect 1 was added to the profile of Transect 2 (+11 feet NAVD crest, 22-foot crest width, with 1:4 foreshore and backside slopes). Similar to Transect 1, this dune is adequate for present and 2050 projected MSL with 1-percent storm conditions. For projected 2070 MSL, the dune crest must be raised to +12 feet NAVD in order to withstand the 1-percent storm (Figure 22), similar to Transect 1. A summary of dune design requirements to withstand coastal erosion and wave overtopping during 100-year storm events is provided in Table 7.

For Transect 3 (Figure 9), located between Town Wharf and the boat ramp, incorporation of a dune with similar dimensions to Transects 1 and 2 would be required to provide upland flood protection. However, this dune feature would not be effective as coastal flood mitigation if the storm tide pathway through the boat ramp is not addressed. Additional engineering analyses will be required to ensure that potential incorporation of a dune/berm west of Town Wharf is incorporated into flood mitigation improvements at the boat ramp.

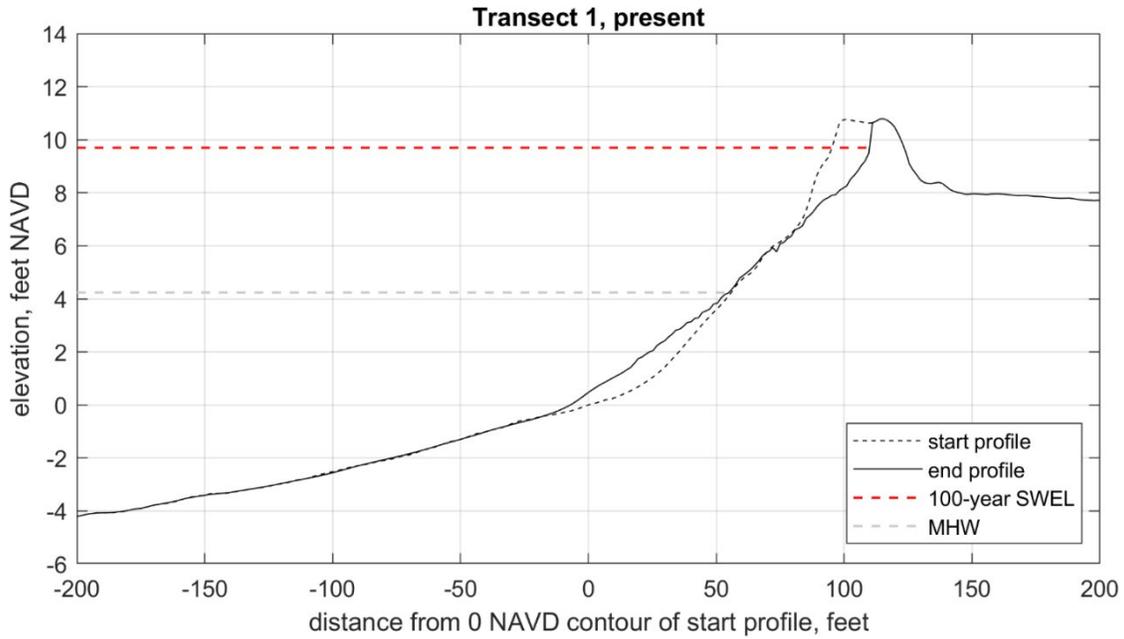


Figure 19. Xbeach model output for the modeled 1-percent (100-year) storm for Transect 1 at Bathing Beach, with existing topography, for present MSL. The start profile is indicated by the dashed black line, and the shoreline at the end of the simulation is indicated by the solid black line. Present MHW and the present 1-percent SWEL are also indicated.

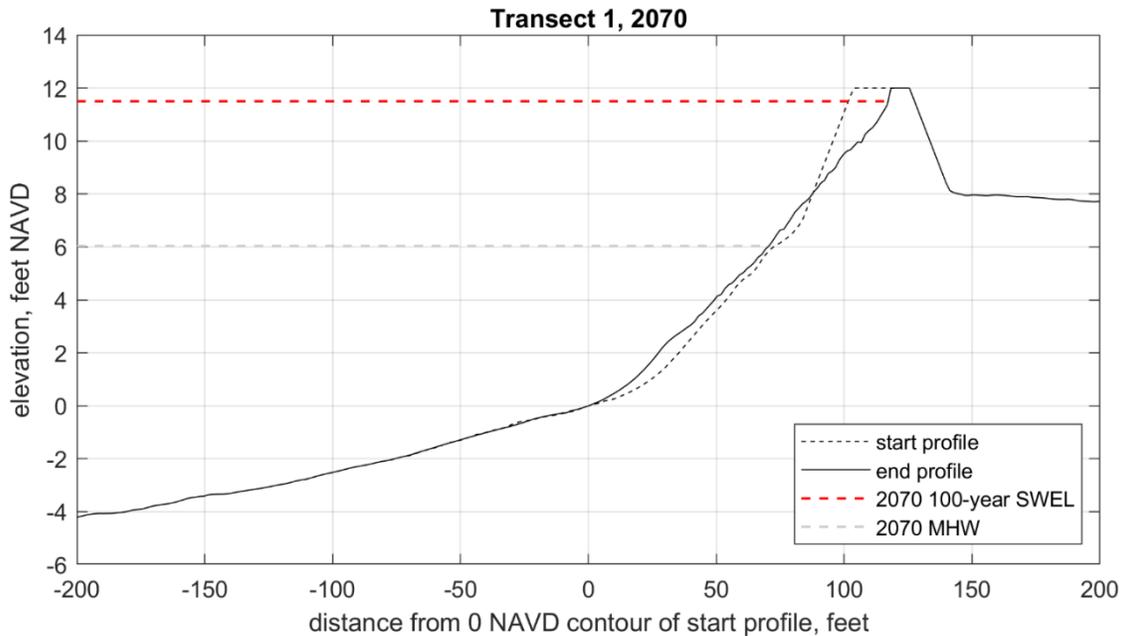


Figure 20. Xbeach model output for the modeled 1-percent (100-year) storm for Transect 1 at Bathing Beach, with existing topography, for projected 2070 MSL. The start profile is indicated by the dashed black line, and the shoreline at the end of the simulation is indicated by the solid black line. 2070 MHW and the 2070 1-percent SWEL are also indicated.

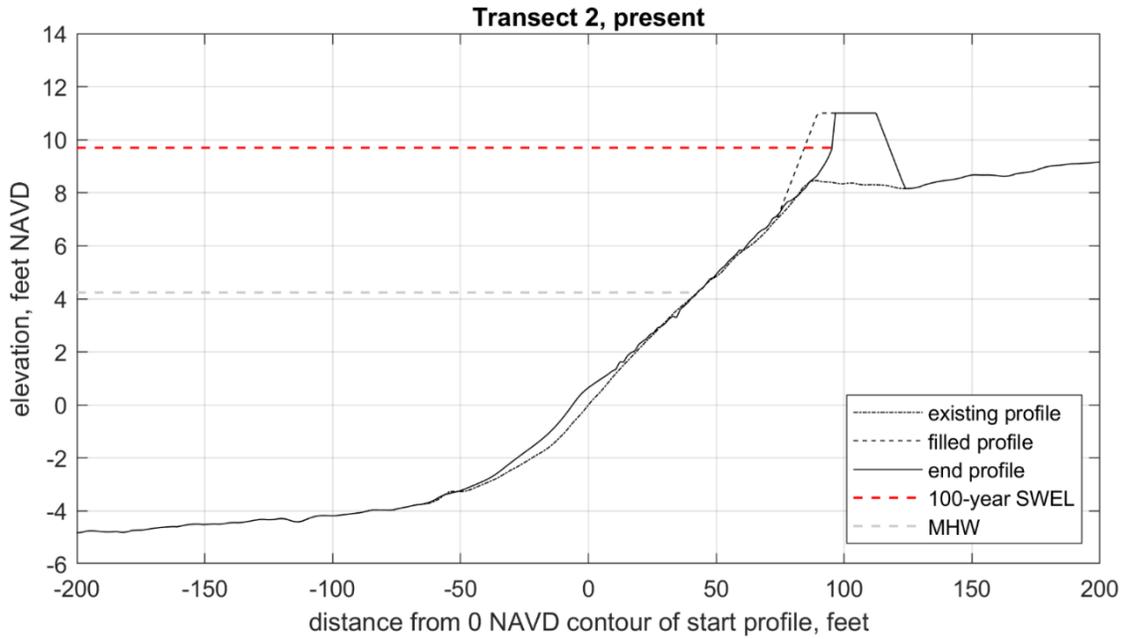


Figure 21. Xbeach model output for the modeled 1-percent (100-year) storm for Transect 1 at Bathing Beach, with existing topography, for present MSL. The start profile is indicated by the dashed black line, and the shoreline at the end of the simulation is indicated by the solid black line. Present MHW and the present 1-percent SWEL are also indicated.

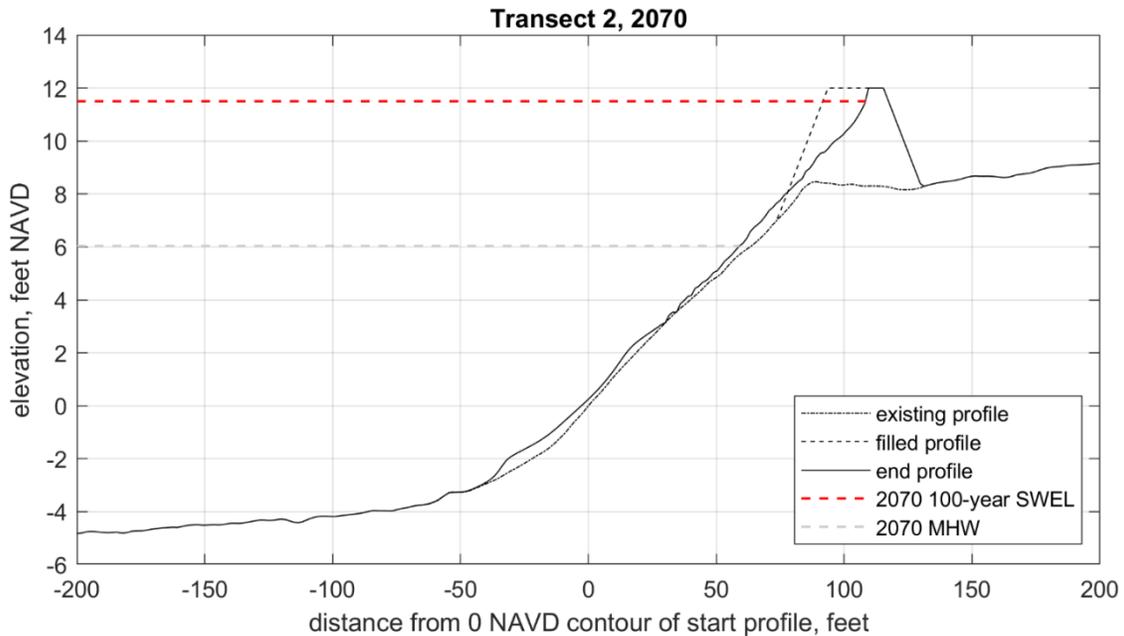


Figure 22. Xbeach model output for the modeled 1-percent (100-year) storm for Transect 1 at Bathing Beach, with existing topography, for projected 2070 MSL. The start profile is indicated by the dashed black line, and the shoreline at the end of the simulation is indicated by the solid black line. 2070 MHW and the 2070 1-percent SWEL are also indicated.

Table 7. Dune crest elevations and crest widths, in feet NAVD88, to prevent wave overtopping and erosion during 100-year storm event; for present, 2050, and 2070 sea level scenarios.

	Sea level scenario					
	present		2050		2070	
	elevation	crest width (ft)	elevation	crest width (ft)	elevation	crest width (ft)
Transect 1	10.7	22	11.0	22	12.0	22
Transect 2	10.7	22	11.0	22	12.0	22

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Attachment 10
Home Meadows Watershed and Upland Flooding Analysis

A. Watershed analysis and extremal rainfall

An extremal rainfall analysis was performed for Home Meadows, in order to determine how tides in the marsh should be managed in order to provide adequate reserve storage capacity for extreme rain events. This analysis is based on the 1% (100-year), 24-hour return period rainfall event for the region, and was developed using extreme precipitation data available from the Northeast Regional Climate Center (NRCC) via a website hosted by Cornell University (<http://precip.eas.cornell.edu>). The available data include total extreme rainfall amounts for a variety of events with different return periods and durations. Also available are distribution curves of rainfall rates as a function of time for selected return periods (e.g., 10-, 25-, 50- and 100-year return periods). The data available from this NRCC project is meant as an update to the regional rainfall climatology that had not been changed since published by the United States Department of Agriculture Soil Conservation Service (USDA SCS) in Technical Paper 40 (TP-40) in 1961 (Hershfield, 1961). Generally, the new NRCC data increase the intensity of rainfall for all return periods, compared to the older TP-40 climatology. The plot of rainfall intensity as a function of time developed using the data from the NRCC website for the Home Meadows region is presented in Figure A1. The total rainfall amount for this event is 8.61 inches.

In watershed analysis of Home Meadows, the upland area that discharges to the marsh was delineated, separate from the area of the marsh plain itself. Hydrology tools available in the GIS program QGIS were utilized to delineate the watershed. Inputs to the analysis include NOAA Continuously Updated Digital Elevation Model (CUDEM) elevation data (Figure A2), which provides broad-coverage data on a regular 3-meter (9.8-foot) grid. The resulting watershed delineation for Home Meadows is presented in Figure A3.

Runoff discharges from the upland watershed were determined using the Natural Resources Conservation Service (NRCS) dimensionless unit hydrograph procedure (NRCS, 1972). The dimensionless hydrograph (Figure A4) provided by NRCS is used to determine the time-dependent distribution of runoff from a watershed. The time-to-peak (t_p) and peak discharge rate (q_p) determined for the upland watershed are needed to create the dimensional hydrograph.

The peak discharge rate (q_p) is calculated using the relationship

$$q_p = \frac{484AQ}{\frac{D}{2} + T_{lag}}$$

where A is the watershed area, Q is the total rainfall for the event (8.61 inches, from NRCC), D is the unit duration and T_{lag} is the lag time. The unit duration D is in turn determined as

$$D = 0.133T_c$$

where T_c is the time of concentration defined as

$$T_c = T_{lag}/0.6$$

and T_{lag} calculated using

$$T_{lag} = \frac{L^{0.8}(S + 1)^{0.7}}{1900(\%Slope)^{0.5}}$$

where L is the length of the longest drainage path in the watershed, S is the potential maximum retention after runoff begins and %Slope is the average watershed slope. From the GIS analysis of elevation data from the NOAA CUDEM, the average slope in the upland watershed is 12.5%. The value of L used for both upland watersheds is 0.34 miles. S is calculated as

$$S = \left(\frac{1000}{CN} \right) - 10$$

where CN is the curve number for the watershed. A value of 75 was used for CN, which is appropriate for developed residential areas with sandy loam soils. The time-to-peak (t_p) needed to fix the time base of the unit hydrograph is determined using the equation

$$t_p = \frac{D}{2} + T_{lag}$$

with D and T_{lag} as described above. The final determined values of t_p and q_p for the watershed were 0.18 hours (11 minutes) and 4,843 cubic feet per second. The cumulative volume of rain from the watershed that is discharged to Home Meadows is 97.0 acre-feet.

The total volume of rainfall during the 100-year event is the sum of the discharge from the watershed and the direct rainfall to the marsh. The direct rainfall component is calculated to be 64.6 acre-feet, which results in a total rainfall volume of 161.6 acre-feet, which is discharged to the marsh over the span of about an hour.

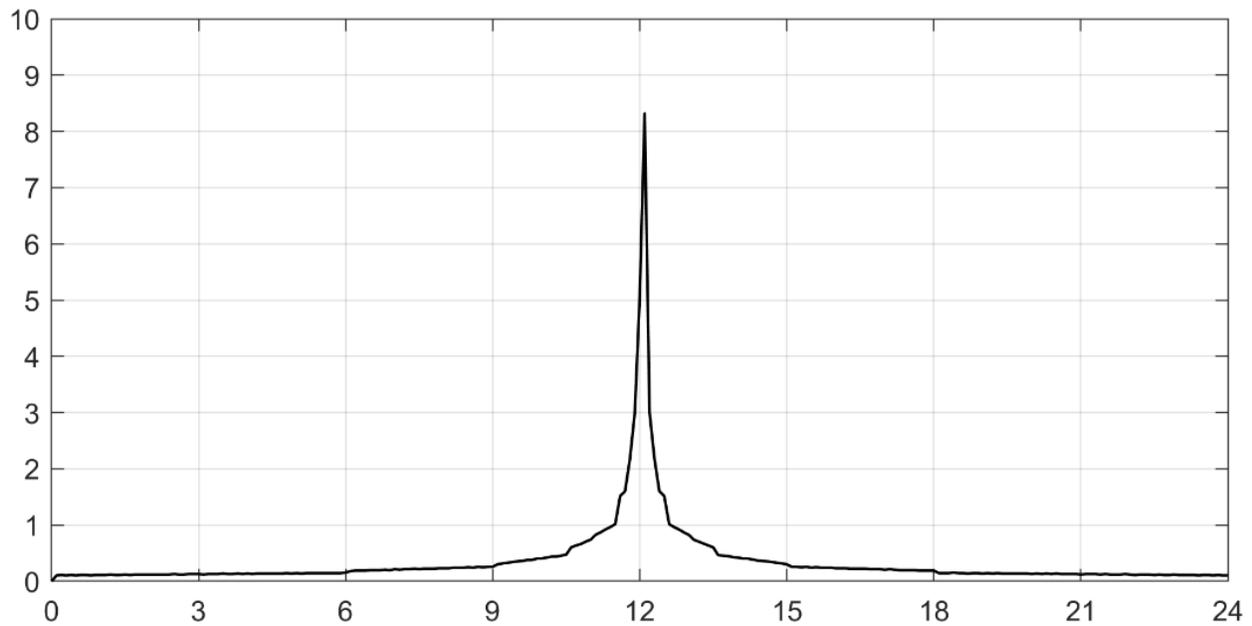


Figure A1. Rainfall intensity distribution from NRCC for the 100-year event for Home Meadows. Peak rate is 7.7 inches per hour. The cumulative rainfall amount for the 24-hour period is 8.6 inches.

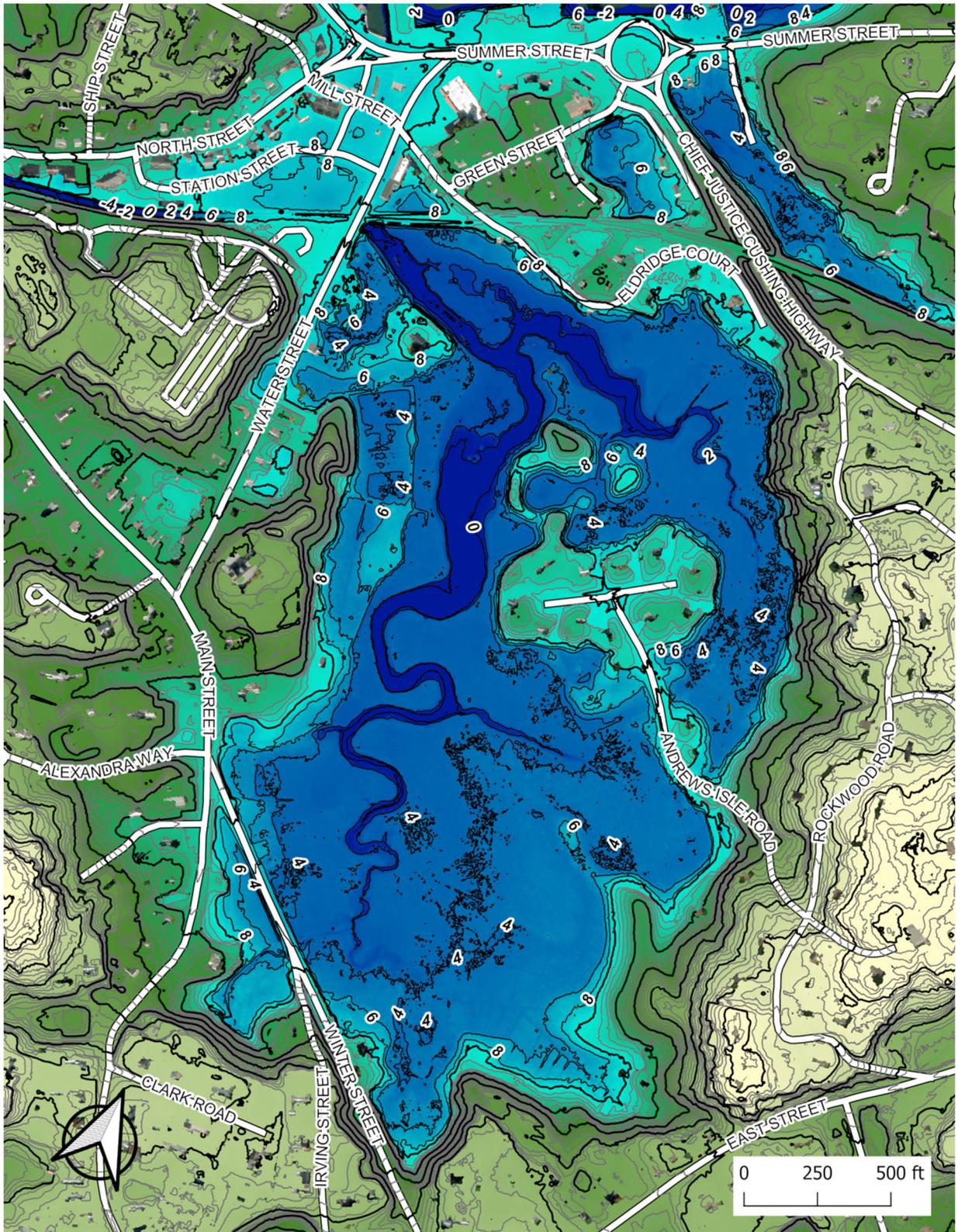


Figure A2. Contour map of Home Meadows vicinity. 2-foot contours lines are mapped with color shading of elevations (Feet NAVD). Contours at 10-foot intervals are shown as thick black lines.

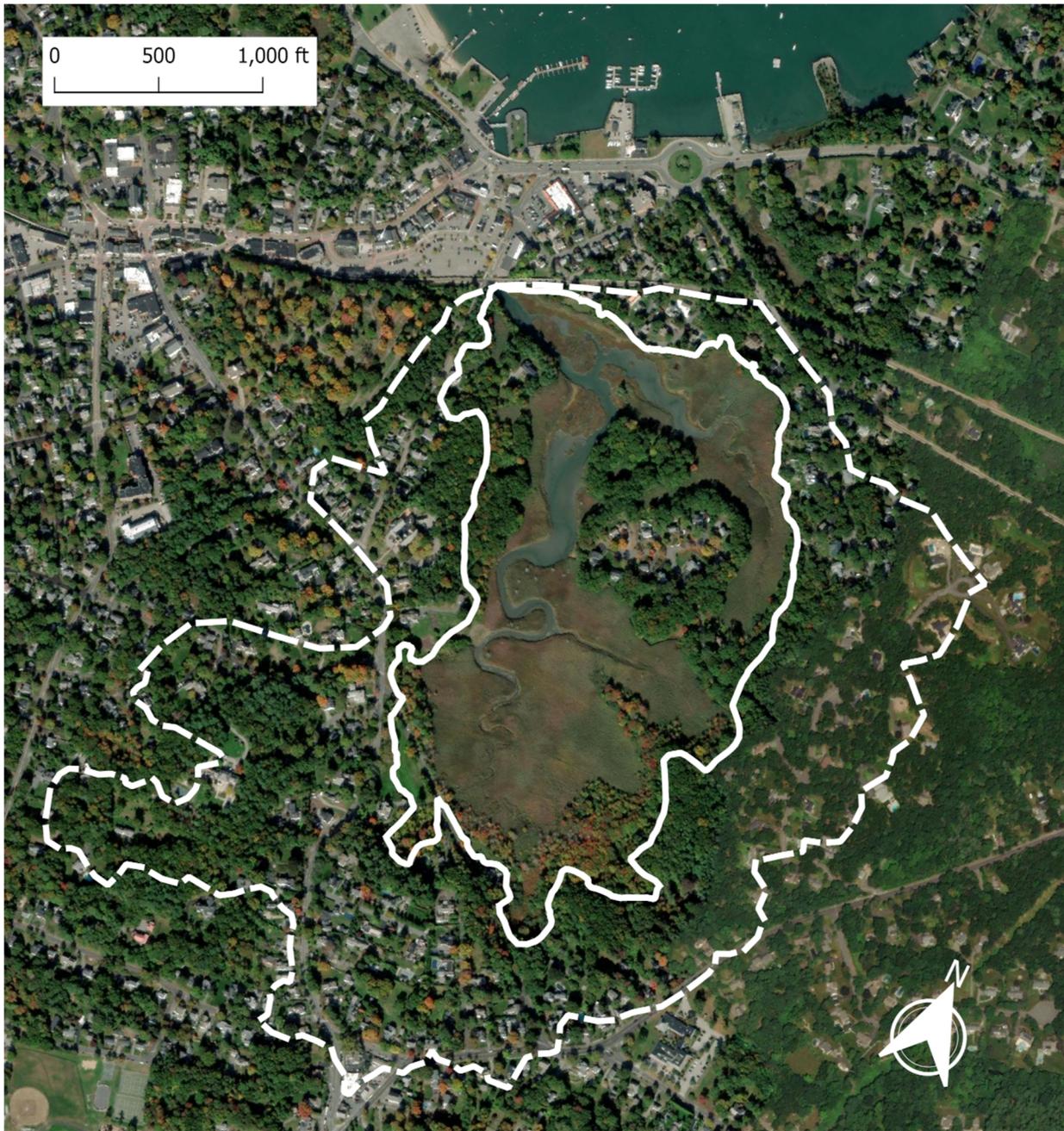


Figure A3. Home Meadows watershed. Area of direct rainfall to marsh surface is within the solid white line, while the upland limits of the entire estuary is indicated by the dashed white line.

B. Management of marsh water levels to prevent flooding

The results of the Home Meadows watershed analysis provide an estimate of the expected increase in water level in the marsh that would be expected from a 100-year, 24-hour extreme rainfall event. 161.6 acre-feet of rain water would be discharged to the marsh over the time span of about an hour, resulting in an increase in water level of about 1.8 feet. In order to ensure that there is adequate storage capacity within the marsh basin to deal with this volume of water, it would be necessary to limit ocean tide and surge elevations in the marsh so that the combination

of tide, surge and rainfall would not exceed an elevation of 8 feet NAVD 88. Above this elevation, low lying properties situated around the marsh would begin to be impacted by flooding waters.

To keep maximum water levels resulting from the 1%, 24-hour rainfall event from exceeding the elevation 8 feet, NAVD88, it would be required to limit the water level in the marsh prior to the addition of rainfall to 1.8 feet below this elevation, or 6.2 feet, NAVD. This elevation coincides, coincidentally, with annual astronomical high tide (AHT) elevation in Boston Harbor. The AHT is the highest tide level that occurs due to the astronomical component of the tide alone (not including storm surges).

An analysis of measured tides in Boston Harbor (6-minute water levels from the NOAA tide station) shows that water levels at that station were greater than 6.2 feet NAVD88 for a total of 70 hours over the course of the entire span of the year 2023. This indicates that if a tide gate was used to limit tides in the marsh to 6.2 feet NAVD88, the tide gate would be closed to tide flow from the Harbor for a maximum of 0.8 percent of any given year.

Further analysis of historical extreme rainfall and maximum daily tide levels in Boston Harbor indicates that the largest 24-hour rain totals do not coincide with maximum water levels in the Harbor. For this determination, the hourly rainfall record from Boston Logan International Airport (BOS) was accessed using NOAA's Climate Data Online (CDO) web-based weather data archive. The data record from July 1979 to May 2024 were used for this analysis. This is the same time period available from NOAA's record of daily measured high and low tide elevations at their tide station on the Boston waterfront. 24-hour rainfall totals (for days that exceed 1.0 inch of rain) are plotted with the maximum recorded tide level in Boston Harbor for that same day in Figure B1. From this plot it is seen that the highest water levels (in excess of 8 feet NAVD 88) have associated rainfall totals that are 5 inches or less. The day with the greatest rainfall total (12.8 inches, on July 28, 2007) had a maximum water level of 4.9 feet NAVD88, which is lower than the typical spring tide in the Harbor. The rainfall amount for that day corresponds to the 500-year rainfall amount for the region, from the NRCC website.

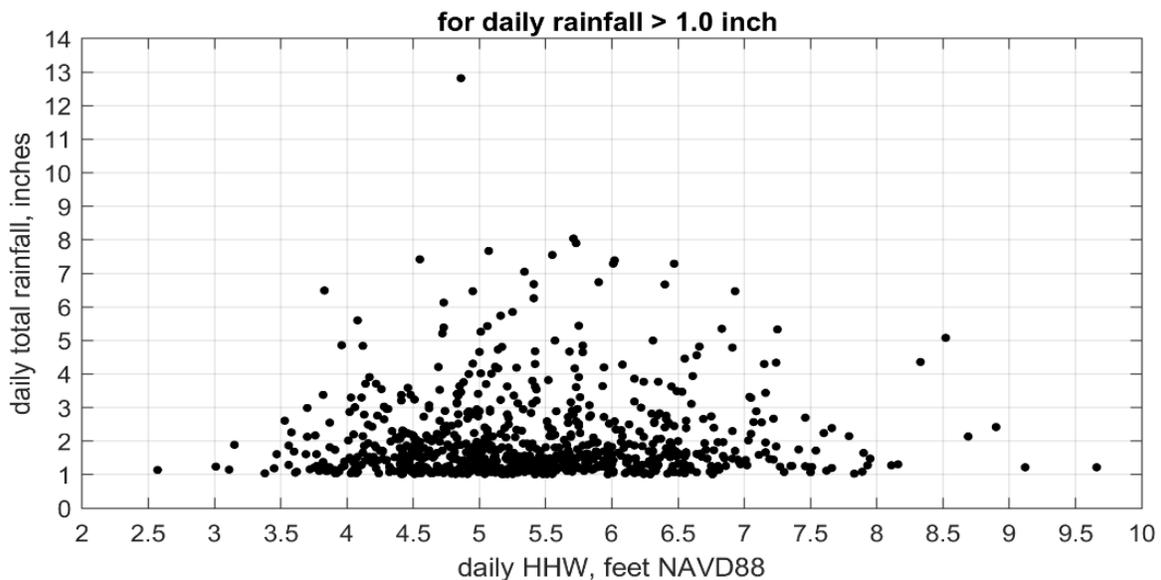


Figure B1. Daily higher high water (HHW) vs. daily total rainfall for Boston (July 1979 through May 2024), for days with rainfall greater than 1.0 inches.

A histogram plot of daily maximum water levels for days with 24-hour rain total of 1.0 inches or more is shown in Figure B2. This plot shows that high rainfall totals occur rarely when water levels in the Harbor are at their highest, which indicates that extreme rainfall events do not typically occur during conditions that cause elevated water levels in the Harbor (typically northeast storms).

The top ten 24-hour rainfall totals for BOS are listed in Table A, along with the corresponding maximum water level in Boston Harbor for that day. For each one of these events, the increase in water level in Home Meadows (from watershed runoff from the listed amount of rain) above the corresponding maximum water level does not exceed an elevation of 8.0 feet NAVD88. If the starting water level in the marsh is set at the 6.2 feet NAVD88 proposed maximum water level regulated by a tide gate in the culvert, the maximum water level in the marsh with the addition of rainfall discharge is still lower than 8.0 feet NAVD88 for all events except the top ranked event, which again corresponds to a 500-year 24-hour rainfall total. This shows that setting the tide gate to close when tides reach 6.2 feet NAVD88 would permit the gate to effectively manage extreme rainfall and storm surges in the marsh.

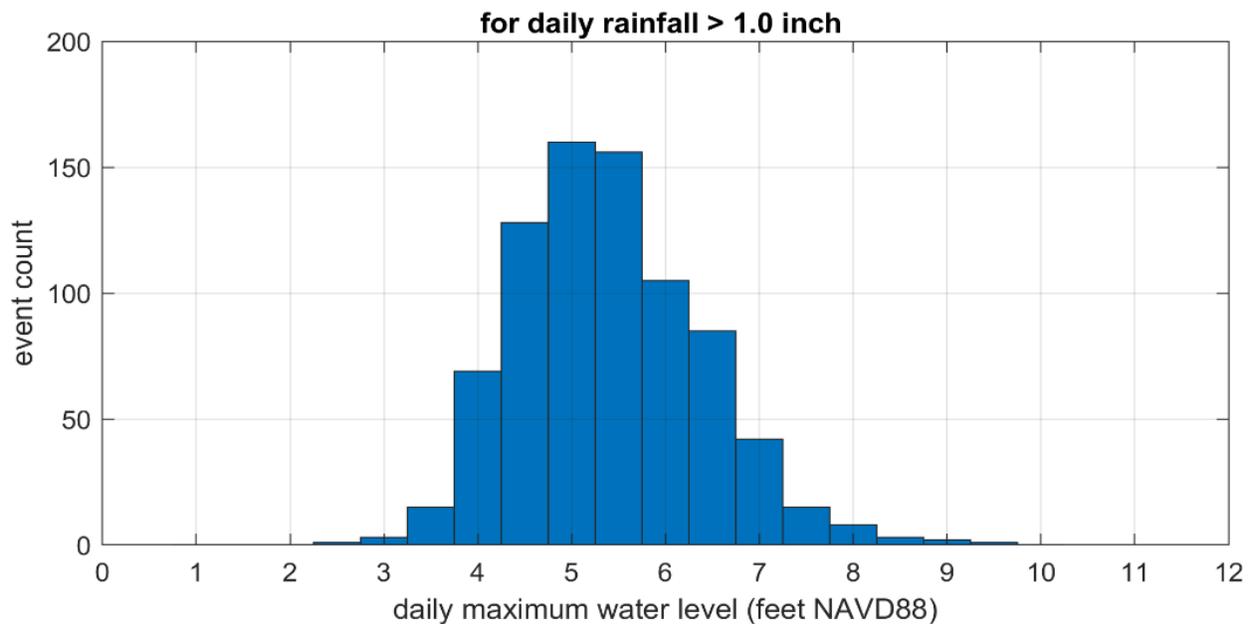


Figure B2. Histogram plot of daily maximum water level in Boston Harbor, for days with total rainfall was greater than 1.0 inches, from the data record that spans from July 1979 through May 2024.

Table A. List of top ten 24-hour rainfall events for Boston since July 1979, with corresponding maximum water level in Boston Harbor of that same day.

Event rank	Date	24-hr rainfall (inches)	Day high water (ft, NAVD)
1	28-Jul-2007	12.82	4.86
2	14-May-2006	8.04	5.71
3	18-Sep-2004	7.90	5.73
4	18-Jul-2012	7.67	5.07
5	13-May-2006	7.55	5.55
6	29-Jun-2019	7.42	4.55
7	10-Aug-2011	7.39	6.02
8	15-Oct-2005	7.29	6.47
9	12-Sep-2009	7.05	5.34
10	23-Aug-2020	6.74	5.90
