SAN FRANCISCO WATERFRONT COASTAL FLOOD STUDY, CA

APPENDIX B.1.3 – WAVE OVERTOPPING SENSITIVITY [DRAFT]

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USACE TULSA DISTRICT | THE PORT OF SAN FRANCISCO

US Army Corps of Engineers

PORT OF SAN FRANCISCO

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Acronyms and Abbreviations

Acronym	Definition
AEP	Annual Exceedance Probability
β	angle of wave attack relative to normal on structure
Bay	San Francisco Bay
EVA	extreme value analysis
d	water depth
$\xi_{m-1,0}$	breaker parameter
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
g	acceleration due to gravity
H_0	initial wave height
H_1	attenuated wave height due to vegetation
h*	impulsive waves for vertical wall
H_{m0}	significant wave height
k, k_i	exponential decay rate constant
l/s/m	liters per second per meter
$L_{m,1-0}$	spectral wavelength in deep water
m3/s/m	cubic meters per second per meter
MLW	Mean Low Water
NAVD88	North American Vertical Datum of 1988
NNBF	Natural or Nature-Based Feature
OPC	California Ocean Protection Council

PED	Preconstruction Engineering and Design
PDT	Project Delivery Team
q	mean overtopping discharge
Rc	Freeboard
RP	Return Period
SFWCFS	San Francisco Waterfront Coastal Flood Study
SLC	Sea Level Change
SWEL	Stillwater Elevation
SWL	Stillwater Level
TWL	Total Water Level
USACE	U.S. Army Corps of Engineers
γ_β	influence factor for a berm
$\gamma(vegetation)$	influence factor for vegetation
z_0	starting elevation of vegetation on slope
z_1	stillwater elevation (for wave attenuation due to vegetation)

Section B.1.3-1. Introduction

Understanding local wave conditions is a crucial part of Coastal Storm Risk Management, both with respect to infrastructure design (including coastal defense structures) and understanding residual risk. The Port of San Francisco and the U.S. Army Corps of Engineers (USACE) chose not to model wave runup on the shoreline structures proposed in the San Francisco Waterfront Coastal Flood Study (SFWCFS) alternatives (*Appendix A: Plan Formulation*), as this would require a level of detail design more appropriate for the Preconstruction Engineering and Design (PED) phase. Instead of performing detailed wave modeling, the Project Delivery Team (PDT) chose to use a 2-foot wave proxy. The intent of the proxy is to inform the basis of design and cost estimates, under the assumption that the future detailed design of the measure(s) can achieve sufficient wave energy dissipation to limit the wave runup elevation (i.e., total water level [TWL] elevation) to 2 feet above the 1% annual exceedance probability (AEP) stillwater elevation (SWEL).

Based on a review of the wave heights along the shoreline (*Sub-Appendix B.1.1 Coastal Storms Report*), the wave runup elevations on the existing Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRMs) (FEMA, 2021), and the scientific literature, a 2-foot wave proxy was considered reasonable. Wave dissipation features, either gray, green or hybrid, will be required along much of the shoreline to reduce the potential for wave runup to meet this proxy.

For wave conditions in the San Francisco Bay, where 1% AEP wave heights are on the order of 2 to 4+ feet (*Sub-Appendix B.1.1 Coastal Extreme Water Levels and High Tide Flooding*), moderate wave overtopping may be allowable in some conditions and is considered in the wave overtopping sensitivity assessment.

The objectives of this wave overtopping assessment are to:

1. Provide background on the local wave climate conditions, including availability of water level and wave data to support wave runup and overtopping analysis at select shoreline locations under existing conditions and with future sea level rise.

2. Estimate shoreline crest height above the 1% stillwater elevation in response to a range of design features (e.g., slope, armoring, and vegetation) to limit potential wave overtopping from:
   - Exceeding hazardous conditions for pedestrians (more stringent condition)
   - Damaging structures (less stringent condition)

3. Validate wave runup and overtopping reduction through design features to achieve a total water level elevation that satisfies the 2-foot wave proxy assumption.

The sensitivity analyses do not seek to set design conditions or elevations to meet specific USACE or FEMA wave runup or overtopping criteria and are not a replacement for detailed wave analysis during the PED phase. This assessment includes wave runup sensitivity analysis to support the selection of the 2-foot wave proxy. The sensitivity analyses (1) evaluate potential wave runup and overtopping at select locations under existing conditions and with future sea level rise, and (2) examine how wave runup and overtopping can be reduced through design features to achieve a TWL elevation that satisfies the 2-foot wave proxy assumption. For San Francisco Bay (Bay) wave conditions, where 1% AEP wave heights are on the order of 2 to 4+ feet (*Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding*), moderate wave overtopping may be allowable in some conditions. The sensitivity analysis looked at two potential wave overtopping conditions. The more stringent condition limits wave overtopping to prevent hazardous conditions for pedestrians. The other condition limits wave overtopping to prevent damage to structures.

The sensitivity analyses are not a replacement for detailed wave analysis during the PED phase. This assessment does not consider the full range of potential shoreline types, or hydrodynamic and wave conditions, along the San Francisco shoreline. The sensitivity analyses are hypothetical in nature, and not representative of actual conditions or proposed green-gray solutions to dissipate wave energy. As such, the feasibility of each shoreline type is not considered beyond its impact on wave runup and overtopping, and no consideration is given to the space requirements and broader impacts of that shoreline for this assessment.

B.1.3-1.1 Key Findings

The wave overtopping sensitivity assessment showed that the 2-foot wave proxy assumption (above the 1% AEP water level) is reasonable for evaluation of the SFWCFS alternatives across different waterfront locations with varying wave climatology, given further design and engineering of the shoreline slope and flood protection structure characteristics. Further design and engineering would be required at locations where wave runup and overtopping exceeds 2 feet above the 1% AEP, while some locations especially in sheltered creeks or locations where flood protection is setback from the current shoreline may not need 2 feet above the 1% AEP. Note that the findings from the assessment do not consider the coastal hydrodynamics inside sheltered creeks (Mission Creek and Islais Creek), where hazardous wave conditions may be further attenuated due to the physical or geographical characteristics of the sheltered creeks.

The key findings from the wave overtopping sensitivity assessment are presented below including the required freeboard above the 1% AEP water level for typical shoreline types without optimizing for the required shoreline and structure slope, landward footprint, and flood protection structure characteristics required to converge on a 2-foot freeboard to limit hazardous wave runup and overtopping conditions.

- Steep slopes (3H:1V) and vertical slopes likely require shoreline armoring or wave dissipation features to satisfy the 2-foot wave proxy and reduce the potential of hazardous conditions on structures. Reducing the likelihood of hazardous conditions for pedestrians during extreme conditions may be more challenging at some locations.

- Shoreline slope is a primary factor in calculating the TWL elevation, or minimum shoreline crest elevation to minimize or prevent wave overtopping. Holding wave conditions constant, shallower sloping shorelines have lower TWLs than steeper shorelines.
- Shoreline armoring can reduce the TWL elevation, and therefore the minimum shoreline crest elevations. The sensitivity analyses found that up to a 50% reduction in wave energy could be achieved, but the amount of reduction is dependent on shoreline slope.
- For vertical walls, rock mound armoring features could reduce foreshore water depth, potentially triggering impulsive wave conditions and higher TWLs. The PED phase should consider water depth relative to incident wave.
- Natural or nature-based features (NNBFs) in combination with shallow sloping shorelines can significantly dissipate wave energy and wave heights, subsequently minimizing wave runup and required shoreline crest elevations.
- TWL elevation increased linearly with sea level rise for most conditions. However, this assessment did not account for additional factors that would change with sea level rise, potentially resulting in higher TWL elevations. Refinements to this assessment are needed to better assess the effects of a warming climate on wave runup elevations.
- Wave runup analysis summary for limiting hazardous overtopping conditions for structures:
    - Steep slopes (3H:1V) may require a shoreline crest elevation 2 to 5 feet above the 1% AEP SWEL, exceeding the 2-foot wave proxy in some locations. Shoreline armoring and wave dissipation features can reduce shoreline crest elevation and satisfy the 2-foot wave proxy.
    - Vertical slopes may require a shoreline crest elevation 1 to 3 feet above the 1% AEP SWEL. Shoreline armoring and wave dissipation features can reduce shoreline crest elevation and satisfy the 2-foot wave proxy.
    - Shallow vegetated slopes (20H:1V) can satisfy this condition even without additional wave dissipation features.
- Wave runup analysis summary for limiting hazardous overtopping conditions for pedestrians:
    - Steep slopes (3H:1V) may require a shoreline crest elevation 4 to 7 feet above the 1% AEP SWEL, exceeding the 2-foot wave proxy. Shoreline armoring and wave dissipation features can reduce shoreline crest elevation and satisfy the 2-foot wave proxy in some locations, but the 2-foot wave proxy may be insufficient at some locations (i.e., shoreline crest elevation that are higher than those proposed in the alternatives may be

required to satisfy this criterion at some locations). This analysis does not consider marginal wharfs which may eliminate this overtopping hazard.

- Vertical slopes may require a shoreline crest elevation 2 to 4 feet above the 1% AEP SWEL, exceeding the 2-foot wave proxy at some locations. This analysis does not consider marginal wharfs which may eliminate this overtopping hazard.
- Shallow vegetated slopes (20H:1V) can satisfy this condition even without additional wave dissipation features.

Section B.1.3-2. Wave Proxy Selection

Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding provides a comprehensive overview of the Bay hydrodynamics and wave dynamics, including wave characteristics along the San Francisco shoreline. Waves that travel toward and perpendicular to the shoreline can runup shoreline structures including flood defenses (e.g., seawalls and levees), and, if the elevation of the wave runup exceeds the shoreline elevation, wave overtopping can occur. The height of the wave runup depends on many factors, including the Bay water level depth and height, shoreline slope, shoreline roughness, and the presence or absence of wave energy dissipation features. In the Bay, wind-driven waves are the dominant wave hazard along the shoreline, and the varied wind conditions generate waves with 1% AEP wave heights of 2 feet to 4+ feet along the shoreline (*Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding*). The 1% AEP wave height does not occur concurrently with the 1% AEP SWEL.

All Future with Project flood protection measures described in the feasibility report, whether engineered, natural, or hybrid green-grey shorelines, have a limited level of design detail. The PDT chose not to model the wave runup reduction potential of all green, gray, or hybrid measures as part of the feasibility study because this would require a level of detail design more appropriate for the PED phase. Instead of performing detailed wave modeling, the PDT chose to use a 2-foot wave proxy. The intent of the proxy is to inform the basis of design and cost estimates, under the assumption that future detailed design of the measure(s) can achieve sufficient wave energy dissipation to limit the wave runup elevation to 2 feet above the 1% AEP SWEL. This 2-foot wave proxy was developed through review of the wave heights along the shoreline (*Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding*) and the wave runup elevations on the existing FEMA FIRMs (FEMA, 2021).

As an approximation, if the 1% AEP SWEL is 10 feet North American Vertical Datum of 1988 (NAVD88), the addition of the 2-foot wave proxy would lead to an existing design elevation (i.e., shoreline crest elevation) for coastal flood defenses of 12 feet NAVD88. The San Francisco FEMA FIRMs suggest a maximum TWL elevation (i.e., existing wave runup elevation) of 15 feet NAVD88, with most of the shoreline having a TWL elevation of 13 feet NAVD88 or less (FEMA, 2021). During the PED phase, green, gray, or hybrid shoreline flood risk reduction measures should be designed to reduce wave runup elevations by about 3 feet in high wave energy areas (e.g., 15 feet NAVD88 – 3 feet = 12 feet NAVD88), with wave runup reductions of 1 to 2 feet required for most of the shoreline to satisfy the 2-foot wave proxy design assumption.

This wave overtopping sensitivity assessment is intended to further support the selection of the 2-foot wave proxy and show the feasibility of accomplishing the needed wave runup reductions with features that add surface roughness or otherwise dissipate wave energy.

Section B.1.3-3. Analysis Locations

Three locations that capture a range of existing shoreline and wave conditions were selected to inform the wave overtopping sensitivity analysis (Figure B.1.3-1), including two locations along the Northern Waterfront, and one location along the Southern Waterfront. The analyses use transects and shoreline profiles developed for the FEMA San Francisco Bay Area Coastal Study (BakerAECOM, 2013). Using the existing FEMA transects provides a basis for comparison with the FEMA wave runup analysis results. The transect number used in the FEMA analysis is presented on Figure B.1.3-1, and the FEMA 1% AEP TWL for each transect is noted below (BakerAECOM, 2013):

- FEMA Transect 18 – Ferry Building, 1% AEP TWL = 11.2 feet NAVD88
- FEMA Transect 20 – Brannan Street, 1% AEP TWL = 13.6 feet NAVD88
- FEMA Transect 23 – Bayfront Park, 1% AEP TWL = 12.1 feet NAVD88

B.1.3-3.1 Shoreline Profiles

Figure B.1.3-2 to Figure B.1.3-4 show the shore perpendicular profile for Transects 18, 20, and 23. The transect profiles are taken directly from the FEMA analysis (BakerAECOM, 2013). The transect profiles represent bare-earth conditions (i.e., engineered structures are not represented). Shoreline toe and crest locations identified in the FEMA analysis are also shown in the respective figures.

Figure B.1.3-1: Transect Locations for Wave Overtopping Analysis (FEMA Analysis Transect Locations)

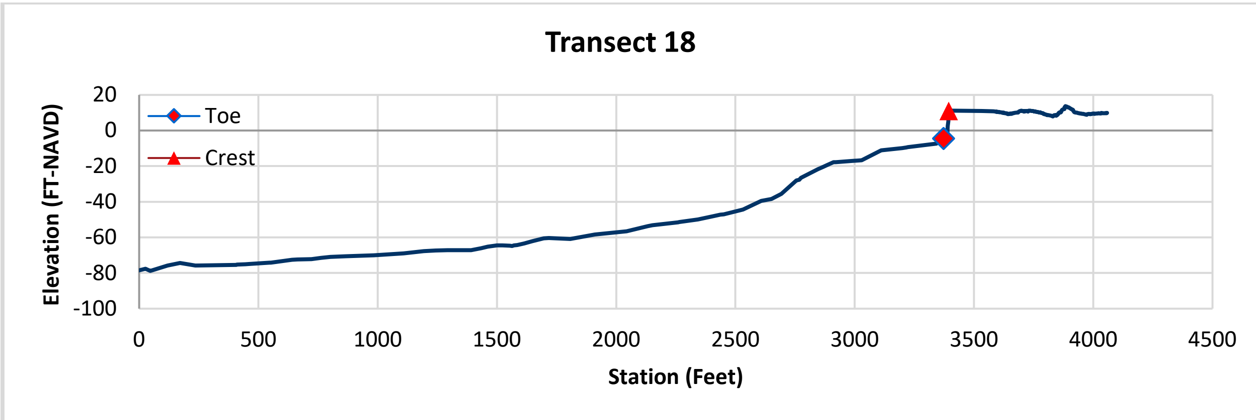


Figure B.1.3-2: Existing Shoreline Profile – Transect 18 (near Ferry Building)

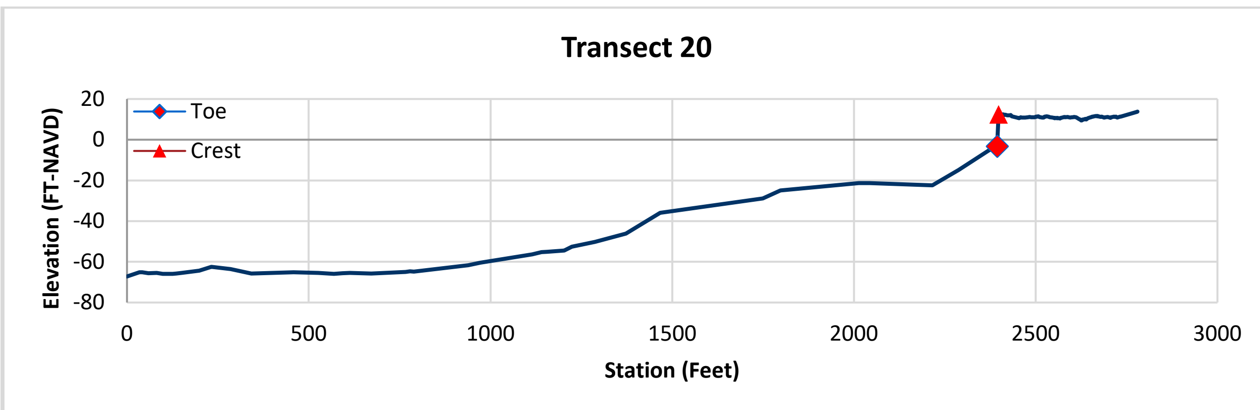


Figure B.1.3-3: Existing Shoreline Profile – Transect 20 (near Brannan Street)

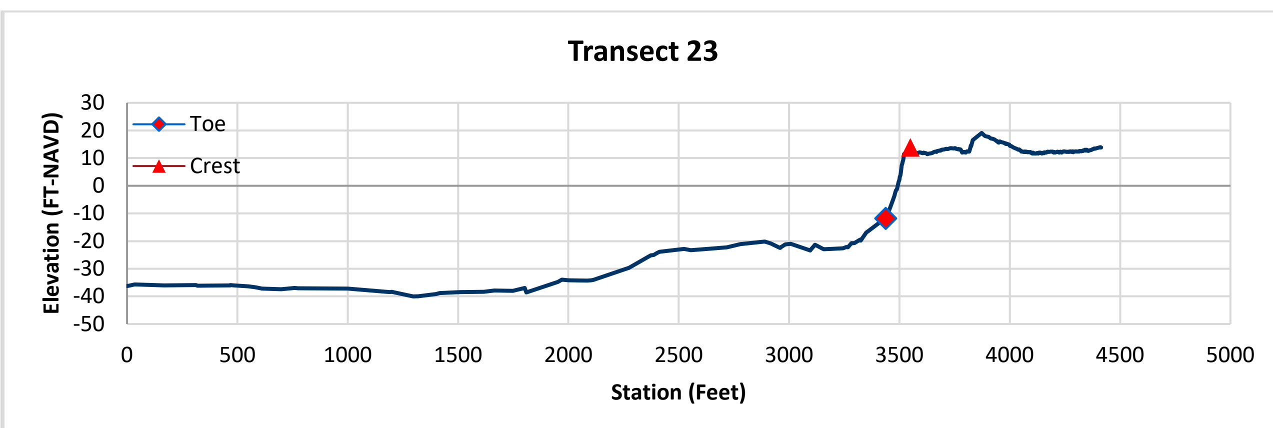


Figure B.1.3-4: Existing Shoreline Profile – Transect 23 (near Bayfront Park)

Section B.1.3-4. Coastal Hydrodynamic Inputs

The water levels and wave heights in the Bay are driven by a multitude of factors, including local bathymetry and shoreline orientation, astronomical tides, and other multiscale ocean and atmospheric processes bringing winds, swell waves, storm surge, and other influences (e.g., El Niño). The complexity of the Bay dynamics results in no single storm event producing both the highest water elevation and the highest wave heights, therefore evaluating coastal flood hazards in the Bay requires evaluating combinations of water levels and wave heights to understand the conditions that result in hazardous flooding; robust long-term water level and wave data is needed to adequately evaluate flood hazards.

Long-term water level and wave data is available from a FEMA MIKE21 hydrodynamic model developed for the FEMA San Francisco Bay Area Coastal Study, with a hindcast period from 1973 through 2003 (see *Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding* for additional detail on the FEMA MIKE21 model and development) (DHI, 2011). The FEMA MIKE21 model provides hourly water level and wave model outputs along the San Francisco shoreline. These model outputs are used as inputs for this wave overtopping sensitivity assessment.

An overview of the tidal datums at each transect location is presented in Table B.1.3-1, which shows an increasing tidal range as you move from Transect 18 to Transect 23. Table B.1.3-2 presents the 1% AEP SWELs for each transect location. The 1% AEP SWELs are statistically derived using the extreme value analysis (EVA) methods described in *Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding*. Table B.1.3-2 presents both a best estimate and an upper and lower bound confidence interval, which are used in Section B.1.3-6 Results to calculate the freeboard above the 1% AEP SWEL to account for potential wave runup and overtopping. Extreme stillwater levels (SWLs) represent a temporary, short-term (hours to months) increase in sea level above the predicted astronomical tide and may include storm surge, El Niño and/or Pacific Decadal Oscillation cycles, local wind setup, freshwater inflows, or a combination of these factors.

Table B.1.3-1: Tidal Datums at each Transect Location (feet NAVD88)

Transect	18	20	23
MHHW	6.20	6.27	6.35
MHW	5.62	5.68	5.75
MTL	3.31	3.31	3.31
MSL	3.21	3.20	3.23
MLW	1.00	0.93	0.86
MLLW	-0.12	-0.20	-0.28

MHHW = Mean Higher High Water
MHW = Mean High Water
MLW = Mean Low Water
MLLW = Mean Lower Low Water
MSL = Mean Sea Level
MTL = Mean Tide Level

Table B.1.3-2: Reference 1% AEP Stillwater Elevation (feet NAVD88)

Scenario	Statistic	18	20	23
Historical	Best Estimate	9.62	9.64	9.74
	Bounds	*9.53 to 9.82*	*9.59 to 9.91*	*9.67 to 9.95*
OPC Likely 2040	Best Estimate	10.42	10.44	10.54
	Bounds	*10.33 to 10.62*	*10.39 to 10.71*	*10.47 to 10.75*
OPC Likely 2090	Best Estimate	10.71	10.73	10.83
	Bounds	*10.62 to 10.91*	*10.68 to 11.00*	*10.76 to 11.04*
OPC Likely 2140	Best Estimate	12.52	12.54	12.64
	Bounds	*12.43 to 12.72*	*12.49 to 12.81*	*12.57 to 12.85*
USACE High 2040	Best Estimate	13.75	13.77	13.87
	Bounds	*13.66 to 13.95*	*13.72 to 14.04*	*13.80 to 14.08*
USACE High 2090	Best Estimate	14.92	14.94	15.04
	Bounds	*14.83 to 15.12*	*14.89 to 15.21*	*14.97 to 15.25*
USACE High 2140	Best Estimate	18.64	18.66	18.76
	Bounds	*18.55 to 18.84*	*18.61 to 18.93*	*18.69 to 18.97*

OPC = California Ocean Protection Council

B.1.3-4.1 Wind-driven Waves

The modeled incident significant wave heights and coincident SWELs for the three transects are presented in scatter plots on Figure B.1.3-5 through Figure B.1.3-7. The plots show two distinct wave climates. At Transect 18, the bulk of the significant wave heights at the shoreline are below 1 foot and no significant wave heights above 1.5 feet were observed concurrently with SWELs above 8 feet. At Transect 20 and Transect 23, stronger winds lead to wave heights of approximately 2 feet occurring concurrently with SWELs above 8 feet. Wave heights that are not propagating onshore within 90 degrees of the shoreline have been removed from the distribution.

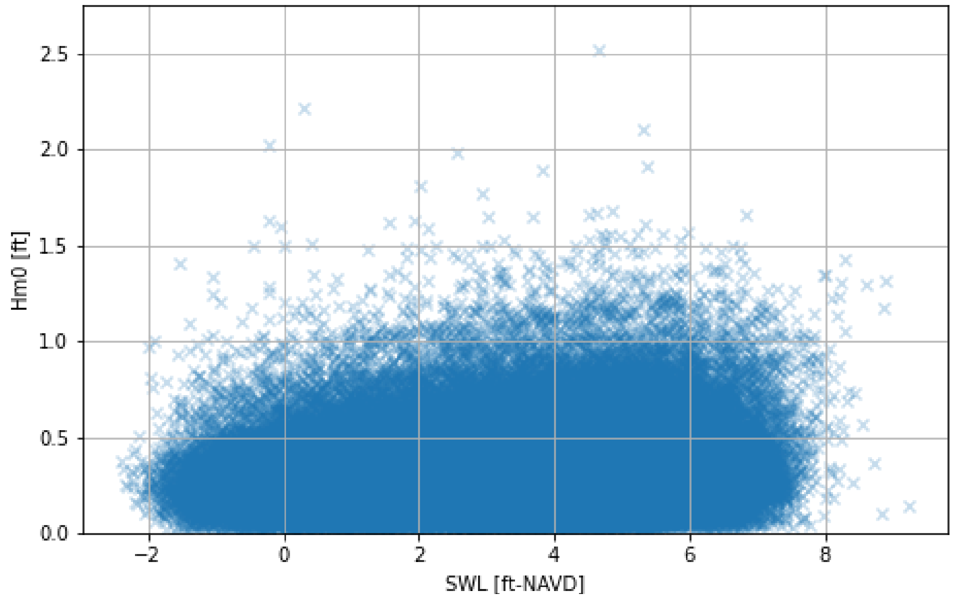


Figure B.1.3-5: Hourly Water Level and Wave Heights (1973-2003) – Transect 18

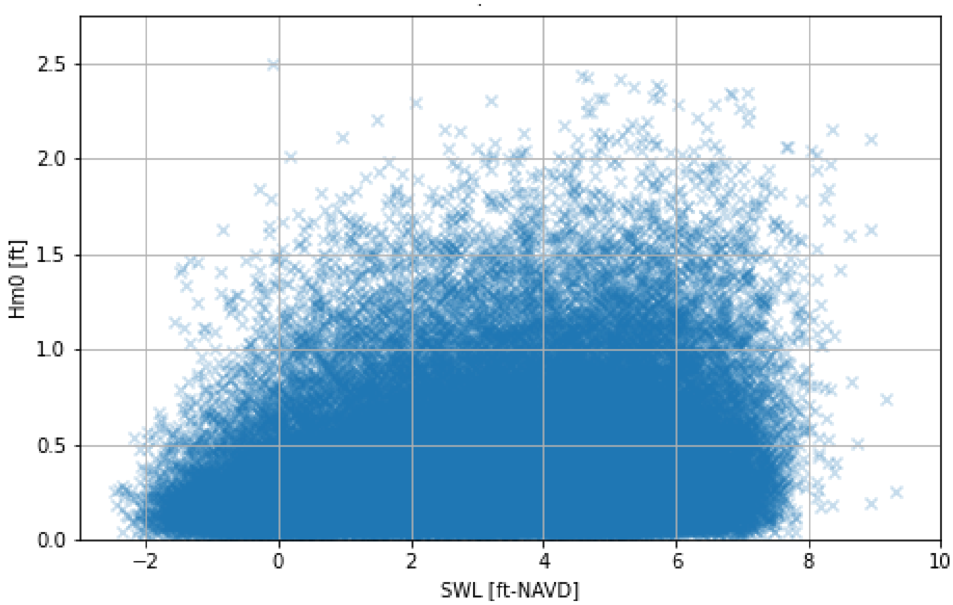


Figure B.1.3-6: Hourly Water Level and Wave Heights (1973-2003) – Transect 20

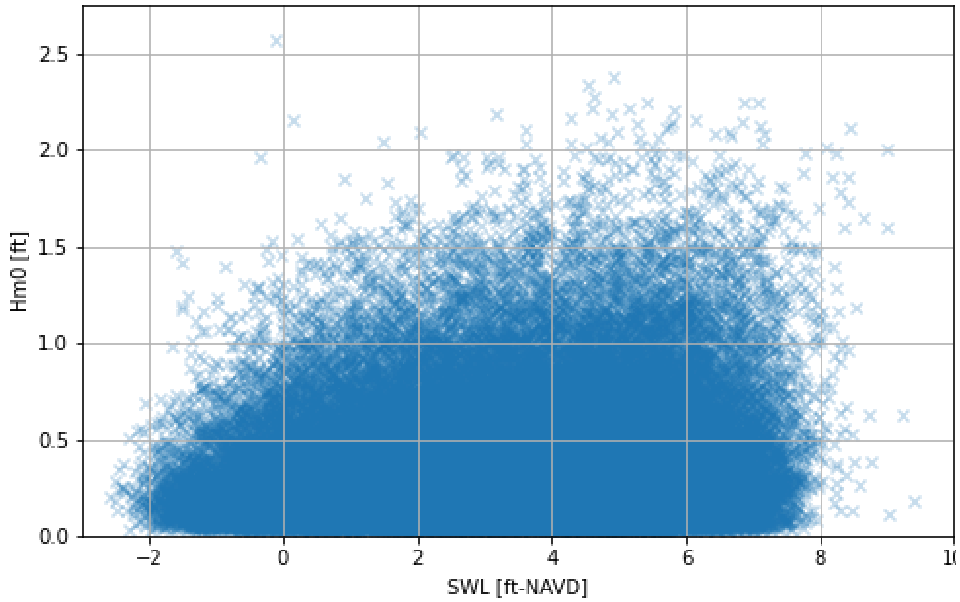


Figure B.1.3-7: Hourly Water Level and Wave Heights (1973-2003) – Transect 23

Figure B.1.3-8 through Figure B.1.3-10 show the peak hourly significant wave height distribution by year and month within the 1973 to 2003 FEMA model hindcast period. The highest wave heights occur during the winter months (October through March) where extratropical cyclones produce extreme weather conditions including coastal storm surge and high winds (see *Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding* for more detail on Bay Area storm systems affecting local extreme water levels and wave heights). Figure B.1.3-11 through Figure B.1.3-13 shows the average hourly significant wave height distribution by year and month within the 1973 to 2003 FEMA model hindcast period. There is an increasing trend in average wave heights during the winter months into the 1990s. However, the hindcast period ends in 2003, and additional modeled wave heights are needed to provide additional insights into potential climate-driven trends in wave heights over time.

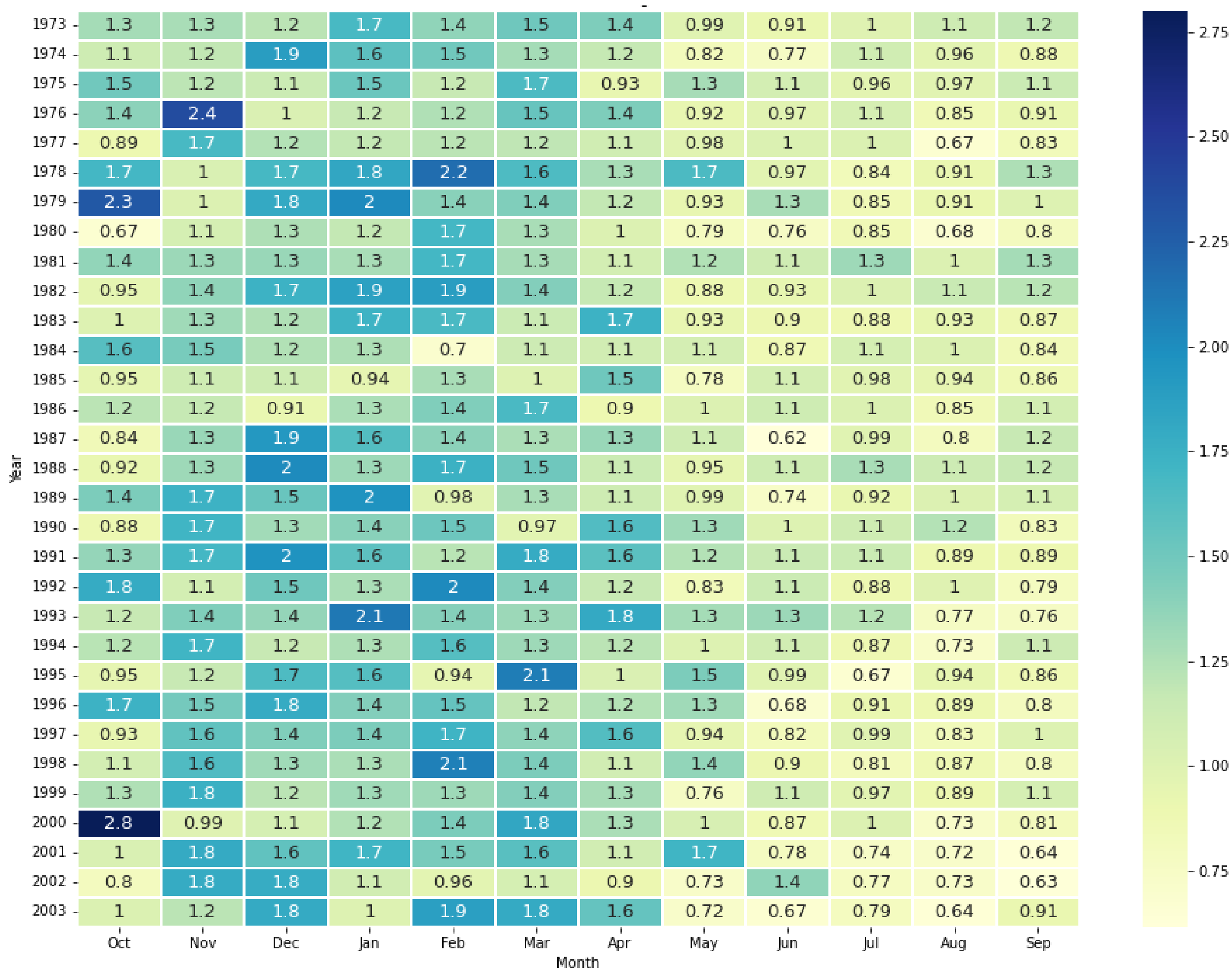


Figure B.1.3-8: Peak Hourly Wave Height by Month and Year (1973-2003) – Transect 18

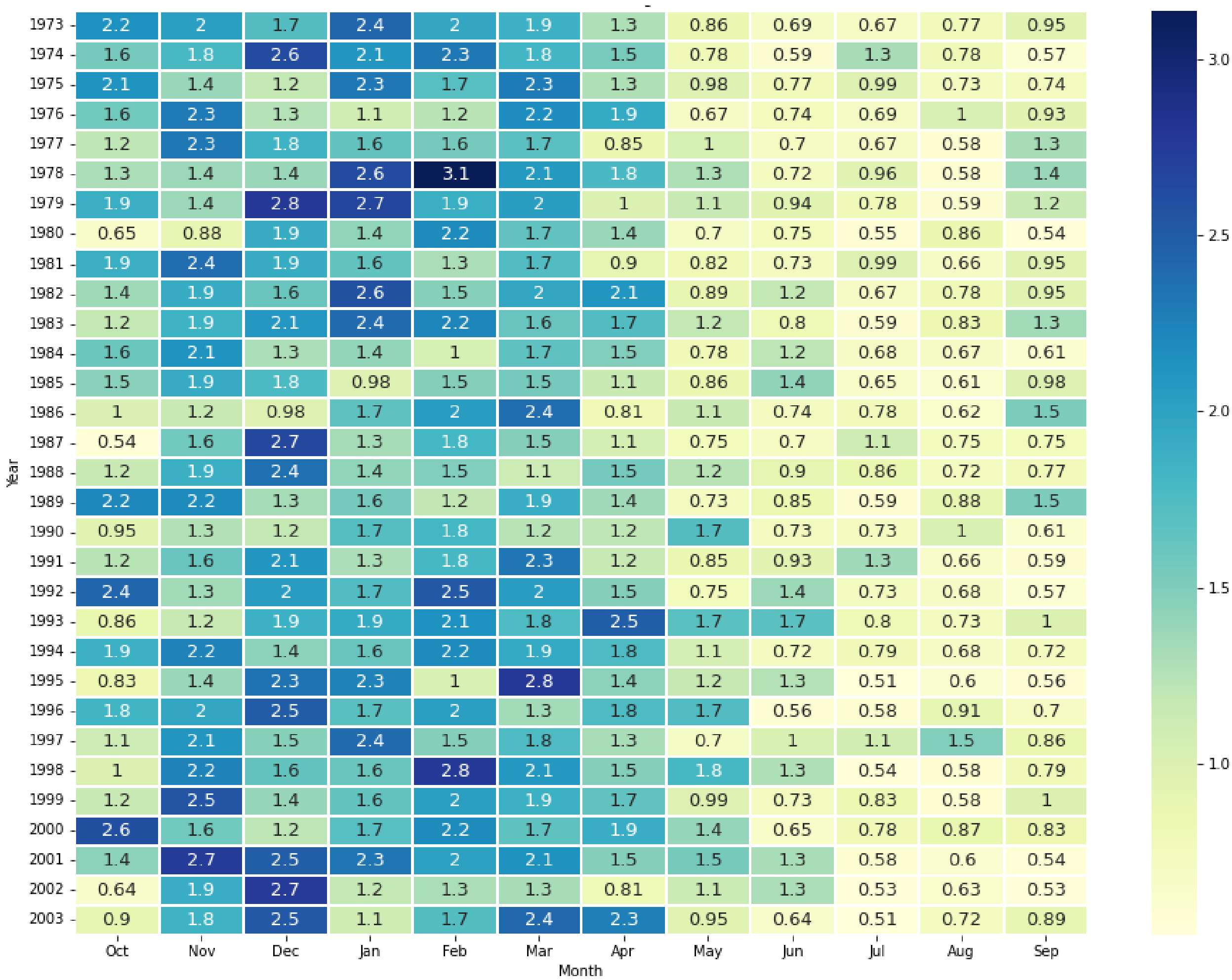


Figure B.1.3-9: Peak Hourly Wave Height by Month and Year (1973-2003) – Transect 20

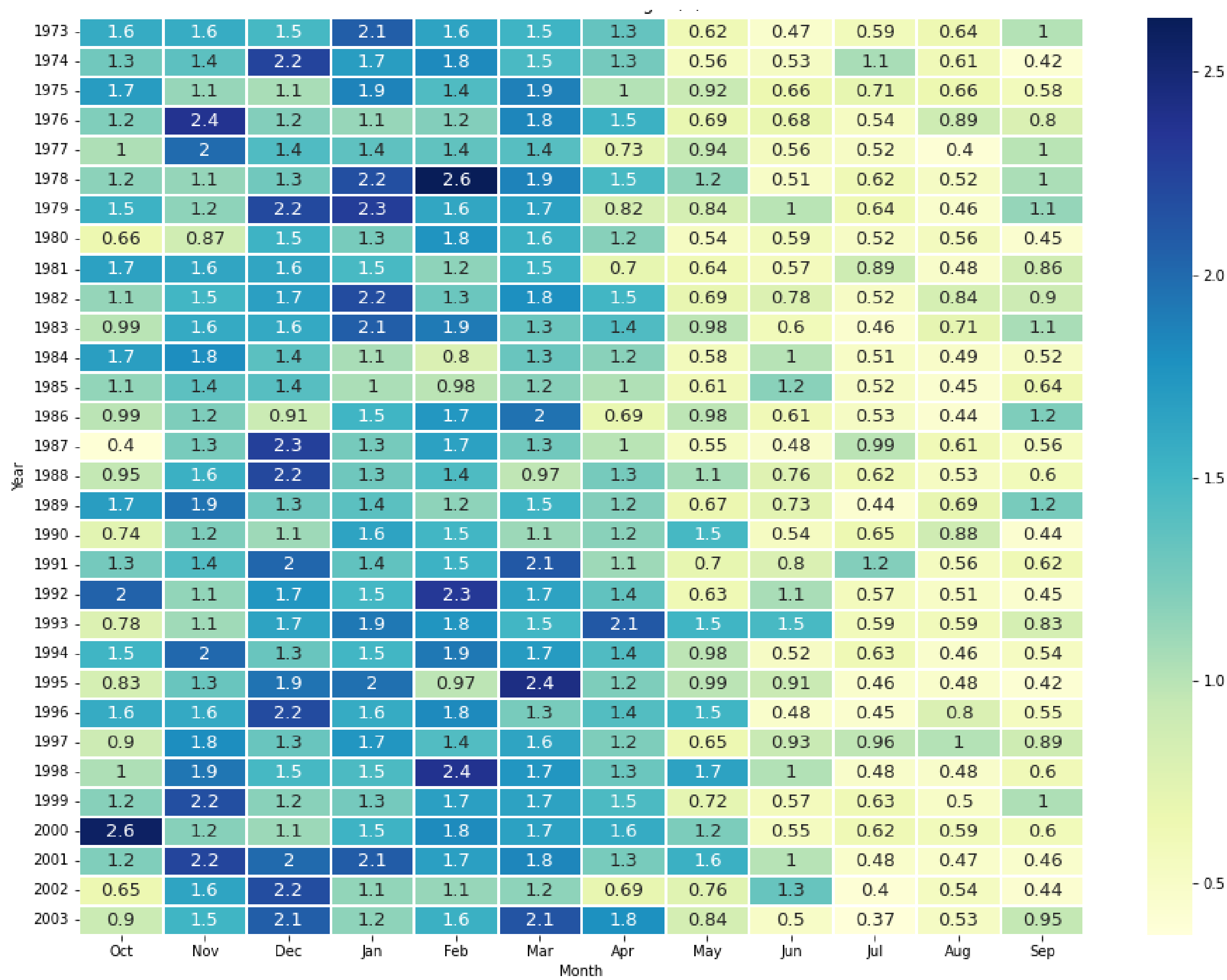


Figure B.1.3-10: Peak Hourly Significant Wave Height by Month and Year (1973-2003) – Transect 23

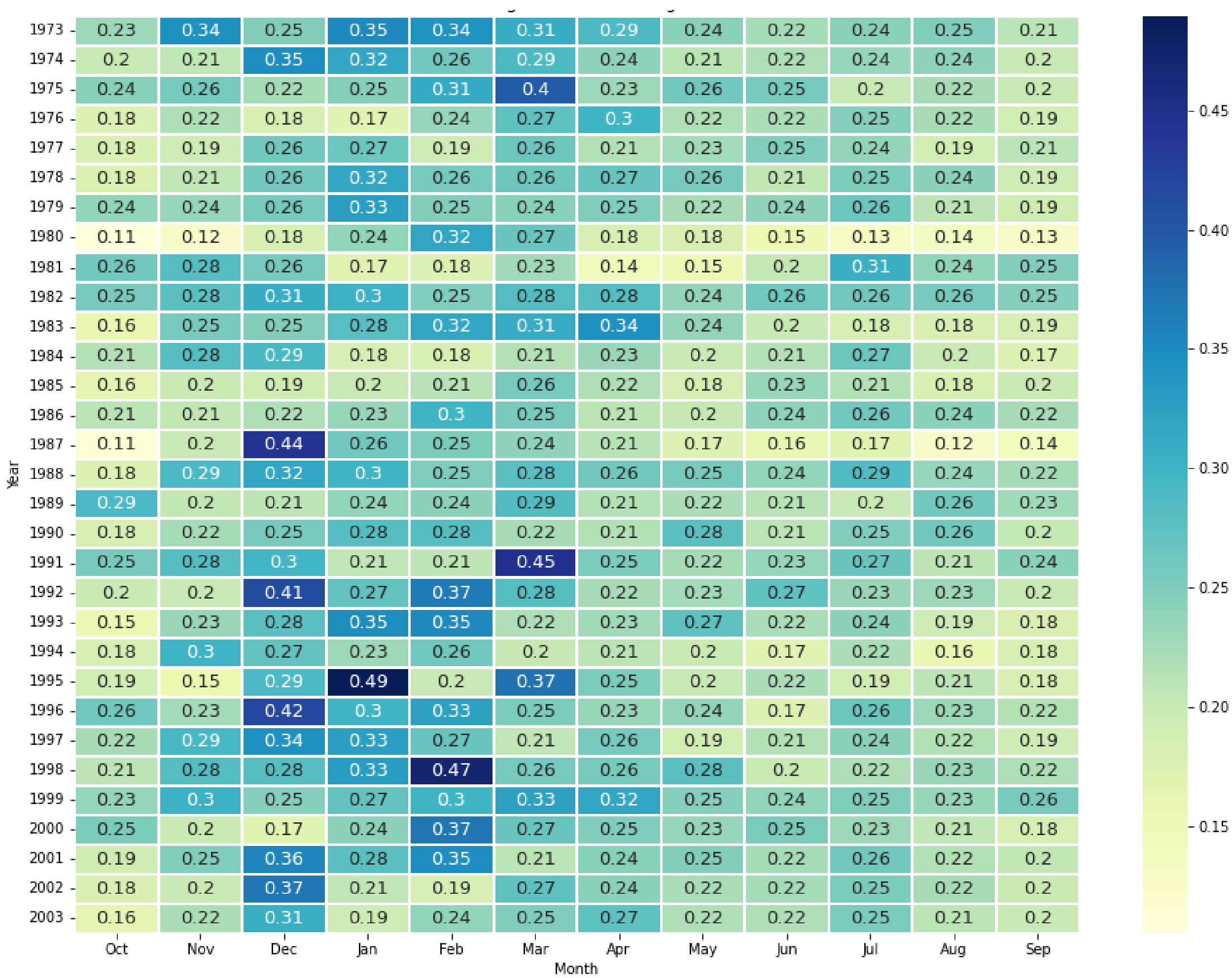


Figure B.1.3-11: Average Hourly Wave Height by Month and Year (1973-2003) – Transect 18

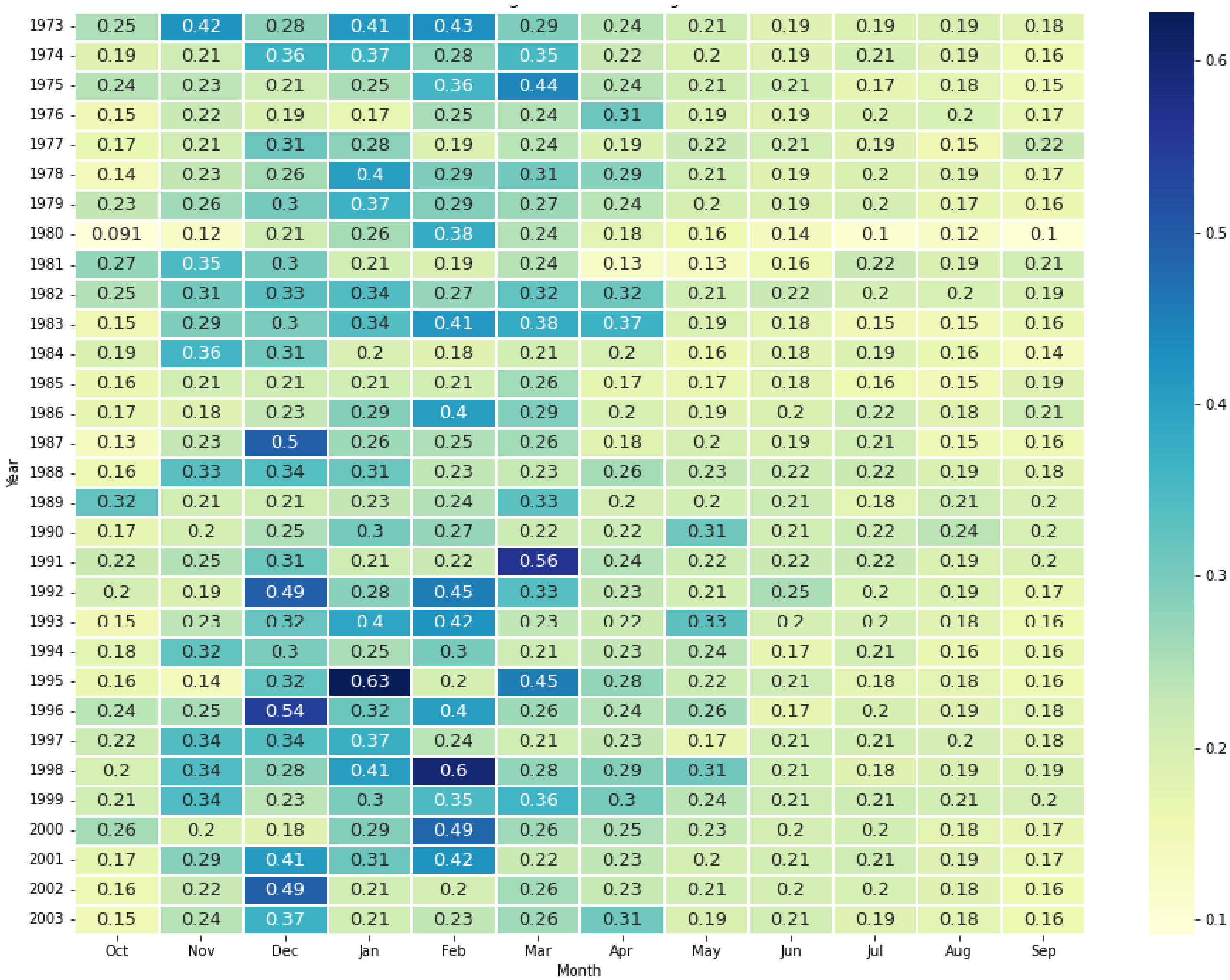


Figure B.1.3-12: Average Hourly Wave Height by Month and Year (1973-2003) – Transect 20

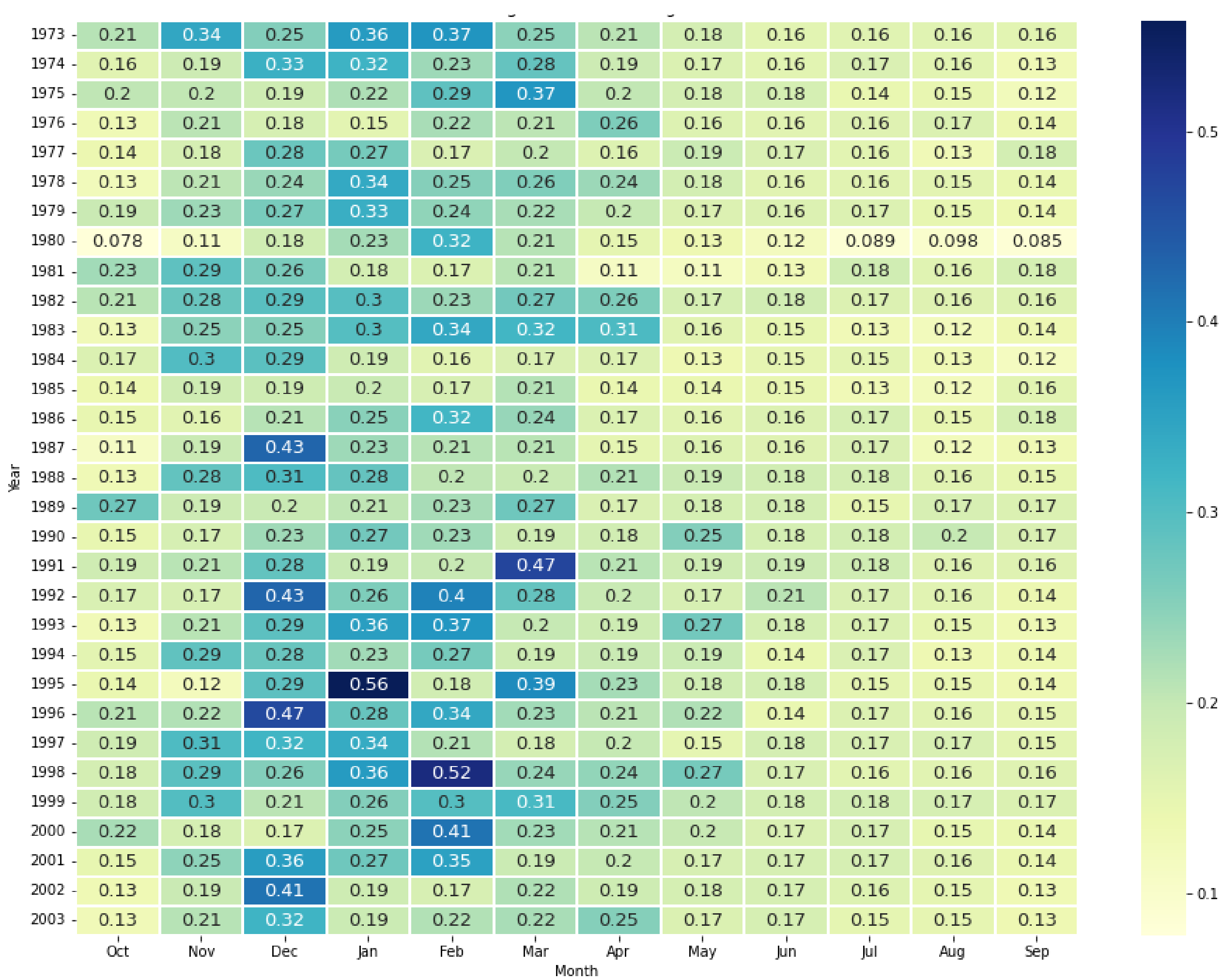


Figure B.1.3-13: Average Hourly Significant Wave Height by Month and Year (1973-2003) – Transect 23

Figure B.1.3-14 shows the distribution of the offshore wave direction at the FEMA model output locations for Transect 18, 20, and 23 for the full 31-year hindcast period. The dominant wave direction is from the north, with higher wave heights propagating from the south-east direction. Near Transect 23, there is a significant duration of waves from the west direction, but the wave heights from this direction are small (0 to 0.5 foot). Transect 18 has a north-east orientation and Transect 20 and 23 have a south-east orientation (Figure B.1.3-1), which aligns with the direction of highest incident wave heights.

Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding provides additional details relative to Bay hydrodynamics, wave dynamics, and storm climatology.

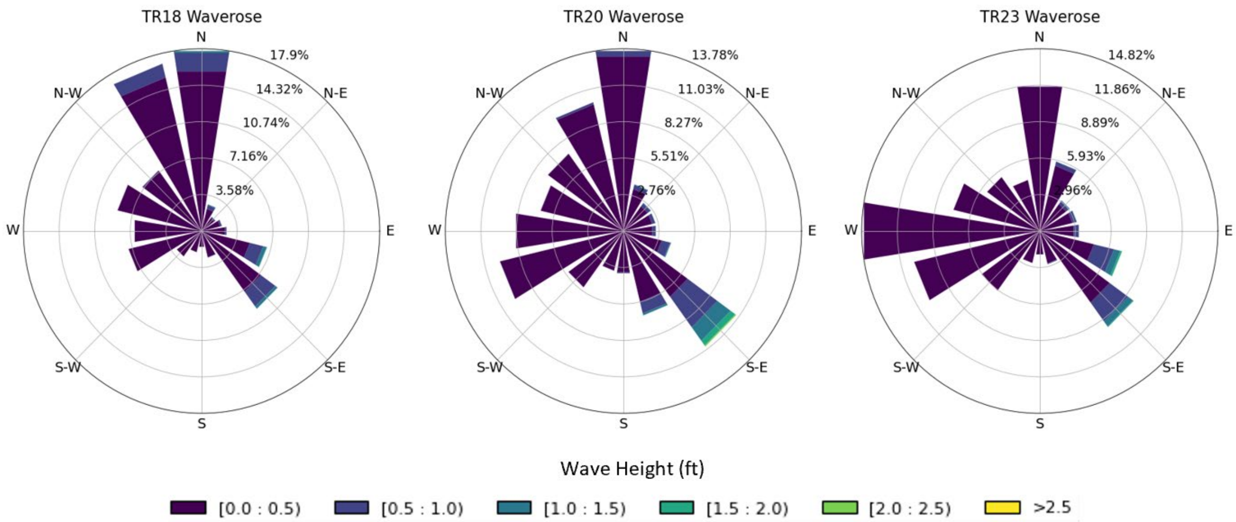


Figure B.1.3-14: Peak Hourly Significant Wave Height (Percentage of Time and Direction)

B.1.3-4.2 Nearshore Wave Transformation

B.1.3-4.2.1 Water Levels and Waves

Hourly water levels (SWELs) are retrieved from an offshore model output “passpoint” and used without additional transformation.

Hourly wave heights are retrieved from the same offshore passpoint and transformed to toe of the shoreline slope as the spectral significant wave height (H_{m0}) required for the EurOtop overtopping equations. Prior to the wave transformation, an initial filtering step removes waves with a mean wave direction beyond 90 degrees from a shore perpendicular orientation.

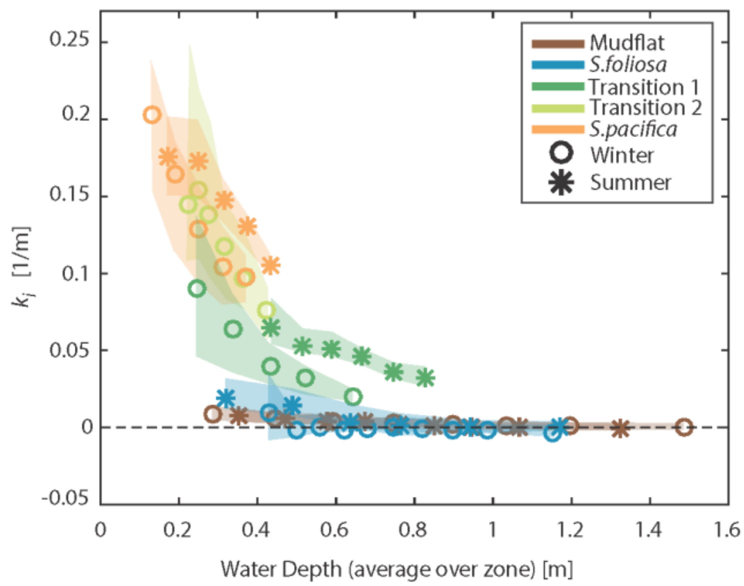
At every timestep, the spectral significant wave height (H_{m0}) at the shoreline slope is calculated by shoaling and refracting the wave from the offshore passpoint location to the shoreline toe.

If wind-driven waves and swell waves are coincident within 90 degrees, the wave heights and waves periods were combined following the approach outlined in the FEMA San Francisco coastal analysis (BakerAECOM, 2013) and Guidance for Flood Hazard Analyses in Sheltered Waters (FEMA, 2008). In general, swell waves have minimal contribution to the total wave height at the three locations.

B.1.3-4.2.2 Wave Dissipation from Vegetation

NNBFs (including coastal wetlands and ecotone levees) provide wave energy dissipation benefits (*Appendix I: Engineering with Nature*). However, the wave dissipation benefits of the NNBFs within each alternative were not quantified during the feasibility study; detailed wave modeling was deferred to the PED phase. This assessment presents potential wave height and runup reduction provided by tidal marsh vegetation on shallow shoreline slopes.

For shallow slopes (20H:1V), wave attenuation across tidal marsh is captured in the transformation of waves from the offshore passpoint. The degree of wave attenuation is primarily a function of vegetation type and hydrodynamic conditions (Foster-Martinez et al., 2018), and can be represented by applying exponential decay constants to the incident wave height affected by water depth and marsh vegetation type. Foster-Martinez et. al (2018) conducted field measurements of marsh vegetation and wave conditions in San Pablo Bay in northern San Francisco Bay to approximate changes in wave height due to vegetation and water depth. Two dominant marsh vegetation types were surveyed, including Salicornia pacifica (pickleweed) and Spartina foliosa (Pacific cordgrass). A limitation to this study is that only two vegetation surveys were conducted (one winter survey in January 2015, and one summer survey in June 2016). Foster-Martinez et. al (2018) developed decay constants (k) representative of each marsh species and their transition zones, binned by water depth (Figure B.1.3-15).



Source: Adapted from Foster-Martinez et al., 2018

Figure B.1.3-15: Wave Height Exponential Decay Constant (k) Binned by Water Depth

The following exponential equation can be used to approximate the change in wave height due to marsh vegetation, with *k* representing the decay constant specific to vegetation type and water depth, and *x* representing a horizontal distance over which the wave is attenuated (Foster-Martinez et al., 2018).

$$\frac{\mathrm{H}}{\mathrm{H_0}} = \exp(-k_i * \mathrm{x})$$

This equation assumes a constant depth and a single vegetation type (i.e., a fixed decay rate constant, *k*), which means it cannot be directly applied to an entire slope, but the relation holds for sufficiently small horizontal distances. By taking the differential of the above equation and then integrating across the entire slope, a modified exponential decay equation is produced as noted below. $\gamma(vegetation)$ is estimated from this equation as the total attenuation from the elevation where vegetation begins, z_0, up to the SWEL, z_1, and incorporated into the influence factors for the EurOtop equation described in Section B.1.3-5.4.

$$\gamma(vegetation) \cong \frac{\mathrm{H_1}}{\mathrm{H_0}} = \exp\left[\int_{\mathrm{z_0}}^{z_1} -\frac{\mathrm{k(z)}}{\mathrm{m}}\mathrm{dz}\right]$$

The vegetated shallow slope (20H:1V+veg) profile used in this assessment assumes that the vegetation bands relative to the mean low water (MLW) tidal datum are identical to those observed in San Pablo Bay (Foster-Martinez et al., 2018), but applied relative to the local MLW at each transect (Table B.1.3-1):

- Spartina Foliosa: 0.4 to 1.1 meters above MLW
- Transition Zone: 0.7 to 1.3 meters above MLW
- Salicornia Pacifica: >1.3 meters above MLW

A key simplifying assumption in the wave attenuation across vegetation types is that the marsh continues tracking with sea level rise (i.e., the vegetation bands and their height relative to MLW is preserved even with sea level rise, meaning the marsh keeps pace with sea level rise). Additionally, it is assumed that Salicornia pacifica extends far enough up the shoreline such that the SWEL never reaches an unvegetated portion of the profile. These simplifying assumptions should be refined in subsequent design phases.

B.1.3-4.2.3 Sea Level Change

For this sensitivity assessment, three future time horizons for two sea level change (SLC) scenarios were evaluated to better understand how wave overtopping will respond to sea level rise (Table B.1.3-3).

Table B.1.3-3: USACE High and OPC Likely Sea Level Change Projections

Year	OPC Likely (feet)	USACE High (feet)
2040	0.8	1.09
2090	2.9	4.13
2140	5.3	9.02

Source: OPC & CNRA 2018; USACE 2019
OPC = California Ocean Protection Council

To simplify the wave overtopping assessment, the sea level rise amounts in Table B.1.3-3 were added to each timestep of water levels for the entire hindcast period. The wave overtopping analysis was conducted for each of the six combinations of time horizon and projected SLC.

Section B.1.3-5. Methods for Overtopping Analysis

B.1.3-5.1 Shoreline Profiles, Modified

Figure B.1.3-16 to Figure B.1.3-18 show the current shore perpendicular profile for Transect 18 with modifications to represent a vertical wall, steep sloped shoreline (3H:1V), and shallow sloped shoreline (20H:1V) with a crest height of 15.5 feet NAVD88. These are theoretical modifications for illustrative purposes. The wave overtopping sensitivity assessment considers an infinite range of shoreline crest elevations, governed by the wave runup elevations and overtopping thresholds described in Section B.1.3-5.3.

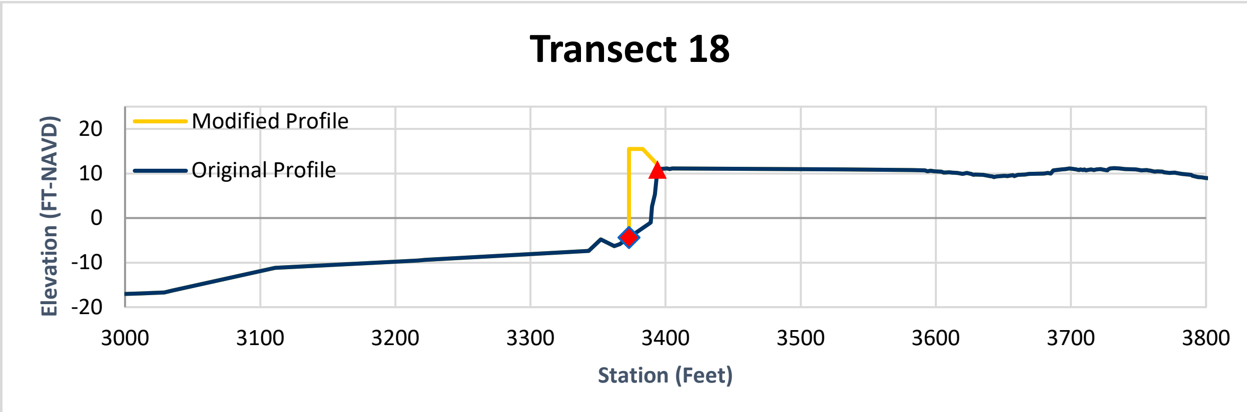


Figure B.1.3-16: Modified Transect 18 Profile Illustrating a Vertical Structure

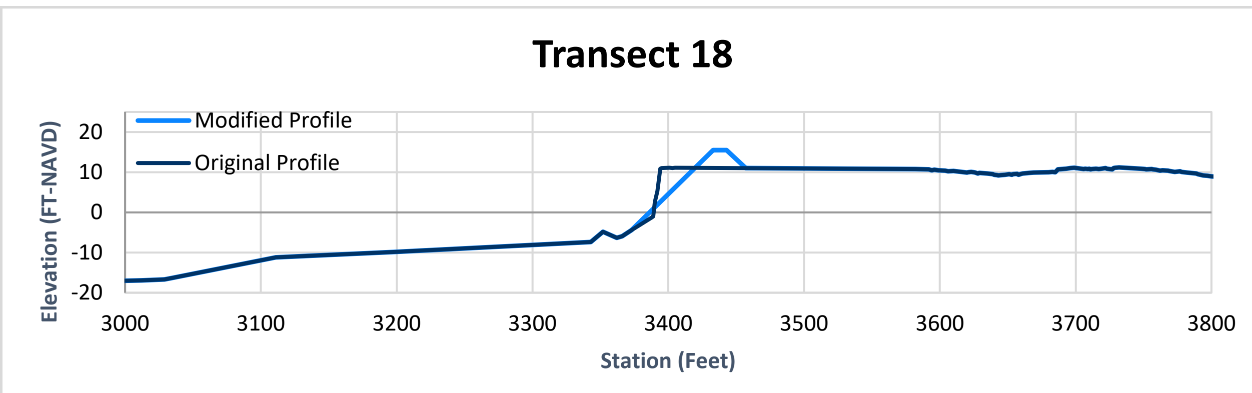


Figure B.1.3-17: Modified Transect 18 Profile Illustrating a Shoreline with a Steep 3H:1V Slope

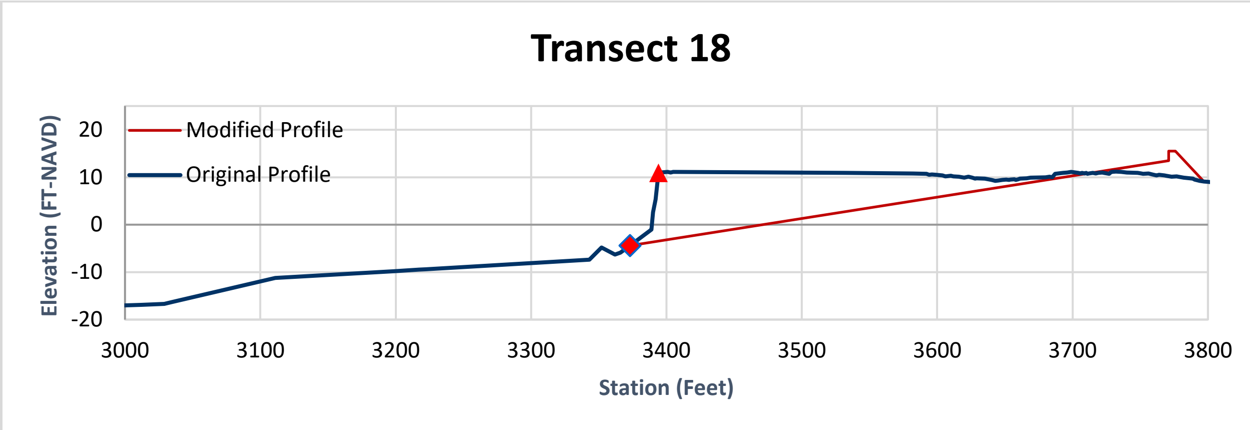


Figure B.1.3-18: Modified Transect 18 Profile Illustrating a Shoreline with a Shallow 20H:1V Slope

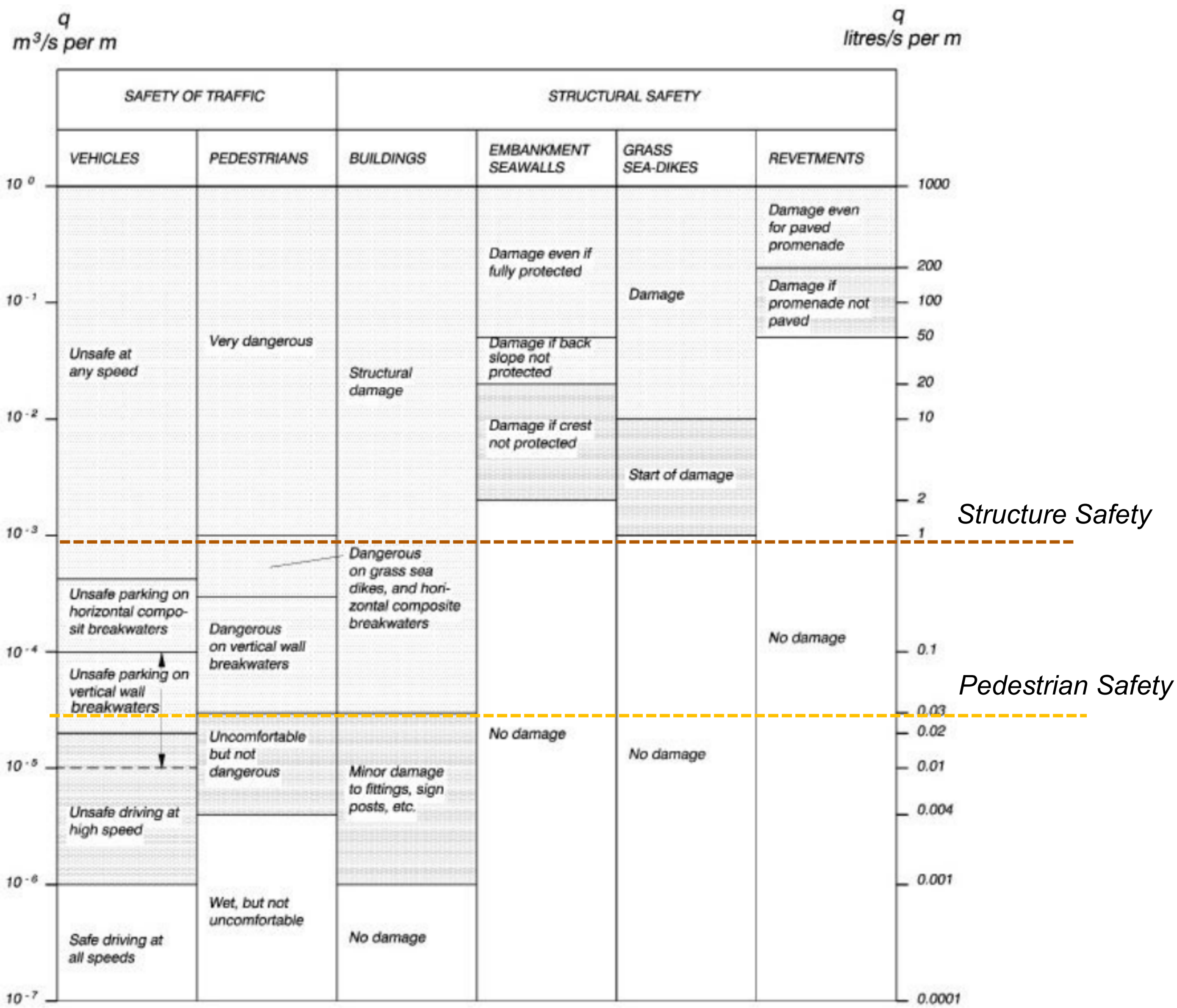
B.1.3-5.2 Overtopping Calculations

The overtopping equations are derived from the EurOtop Manual (Second Edition) on wave overtopping of sea defenses and related structures (EurOtop, 2018). The overtopping equations are empirically derived through university research and government agencies to assist engineers perform wave runup and overtopping volume/rate calculations. While EurOtop was primarily developed for European regions, the methodology was intended for global applications. EurOtop equations are commonly used as best practice in the coastal engineering industry and many of the EurOtop formulas are referenced in the USACE Coastal Engineering Manual (USACE, 2011).

Given the complexity of water levels and waves in the Bay (Section B.1.3-4.2.1), this wave overtopping sensitivity assessment uses an approach that evaluates wave overtopping on an event-by-event basis using water levels and waves from the 1973 to 2003 FEMA MIKE21 model hindcast period.

B.1.3-5.3 Overtopping Thresholds

The goal of this assessment is to support the selection of the 2-foot wave proxy as the basis of design to support plan formulation, design, and cost estimates at the feasibility stage. To evaluate potential wave overtopping, thresholds of tolerable overtopping were selected to calculate minimum shoreline crest elevations necessary to limit overtopping. Figure B.1.3-19 shows critical values of average overtopping discharges according to USACE Coastal Engineering Manual Volume VI Table VI-5-6 (USACE, 2011).



Source: Adapted from USACE 2011

Figure B.1.3-19: Permissible Wave Overtopping

Two overtopping thresholds were selected to capture a range of potential wave overtopping hazards (Figure B.1.3-19):

- Pedestrian Safety
  - Allowable overtopping (q) = 3e-5 (0.00003 m3/s; 0.03 l/s/m).
  - Aligns to the limit upper limit of the "*Uncomfortable, but not dangerous"* overtopping rate, and before conditions become "*Dangerous on vertical wall breakwaters."*

  - This threshold is stricter than the EurOtop tolerable overtopping rate to maintain pedestrian safety, which become less restrictive as wave height decreases. In a wave climate with small waves (e.g., below 2 meters) where the tolerable mean overtopping rate is relaxed to 1 l/s/m, which matches the more lenient structural safety threshold considered for this study. For waves closer to 1 meter in height, the tolerable overtopping rate is 10-20 l/s/m, which is within the "*Very dangerous*" zone for pedestrians on Figure B.1.3-19. While these allowable overtopping rates are orders of magnitude less conservative than those in the USACE threshold, it is important to note that overtopping scales exponentially with freeboard.
  - With this overtopping threshold, minimum crest elevations can be approximated during "zero overtopping" conditions, for significant wave heights of 1 meter or less. This condition is comparable to the 1% AEP TWL, which accounts for tides, storm surge, wave setup, and wave runup).
- Structural Safety
  - Allowable overtopping (q) = 1e-3 (0.001 m3/s; 1 l/s/m).
  - Relaxed overtopping threshold compared to pedestrian safety.
  - This threshold is similar to the EurOtop threshold for protecting property (building structure elements) behind shoreline structure (e.g., floodwall), where the significant wave height is between 1 and 3 meters.

B.1.3-5.4 EurOtop Equations

The general form of the EurOtop overtopping equation (5.9) is expressed by the following:

$$\frac{q}{\sqrt{g * H_{m0}^3}} = a * \exp\left[-\left(b * \frac{Rc}{\gamma H_{m0}}\right)^c\right]$$

  - q = overtopping discharge
  - g = 9.81 m/s2
  - H_{m0} = spectral significant wave height
  - a, b, c = fitted coefficients
  - R_c = freeboard
  - γ = influence factor(s)

For a given incident wave height (H_{m0}) and crest freeboard (R_c), this general form of the EurOtop overtopping equation can be solved to calculate the overtopping rate over the shoreline crest. However, for this wave overtopping sensitivity assessment, wave height

(H_{m0}) and discharge rate (q) are specified to solve for R_c, where the discharge rate (q) is either the maximum allowable overtopping rates for pedestrians or for structures).

For a known overtopping discharge rate, the EurOtop equation below can be solved for the required freeboard (R_c), and subsequently the shoreline crest elevation to limit overtopping at or below the defined threshold.

$$Rc = \frac{\gamma H_{m0}}{b} * \ln\left[a * \frac{\sqrt{gH_{m0}^3}}{q}\right]^{\frac{1}{c}}$$

Figure B.1.3-20 shows a simple conceptual illustration of Rc, which is the freeboard above the SWEL at the toe of a structure (e.g., vertical wall), which can limit the overtopping rate to a desired amount.

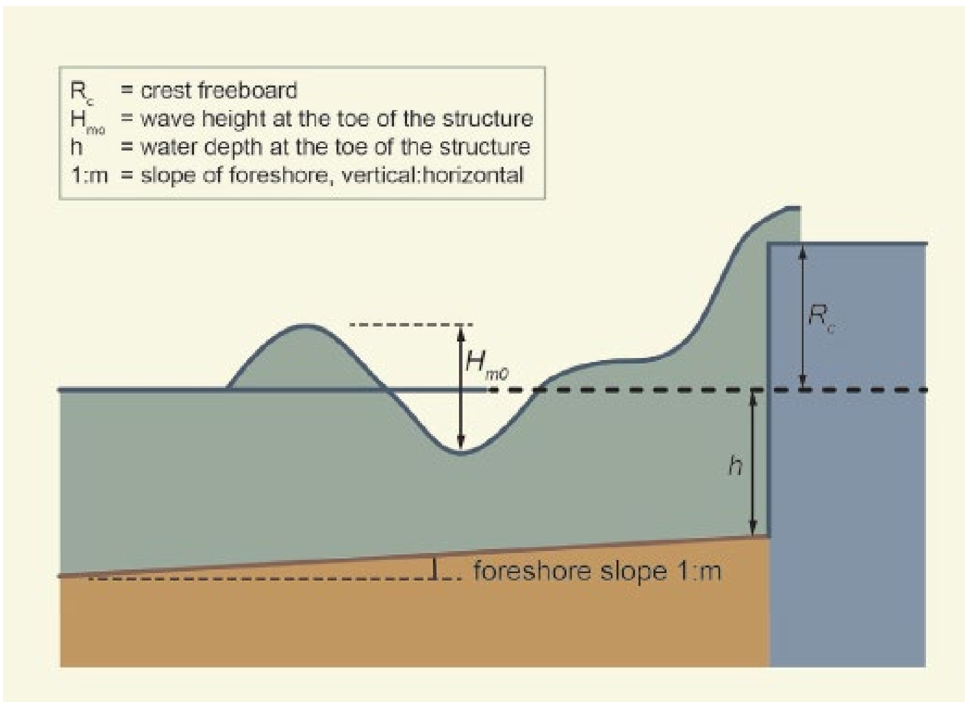


Figure B.1.3-20: Illustration of EurOtop Freeboard (R_c) Parameter Calculated at a Shoreline Structure

Source: EurOtop (2018)

At each analysis location, the required freeboard R_c to not exceed the allowable overtopping rate is calculated at each timestep of the 30-year hindcast with hourly water levels and wave heights. R_c is then added to the SWL, resulting in the shoreline crest elevation (feet NAVD88) to limit overtopping below the threshold.

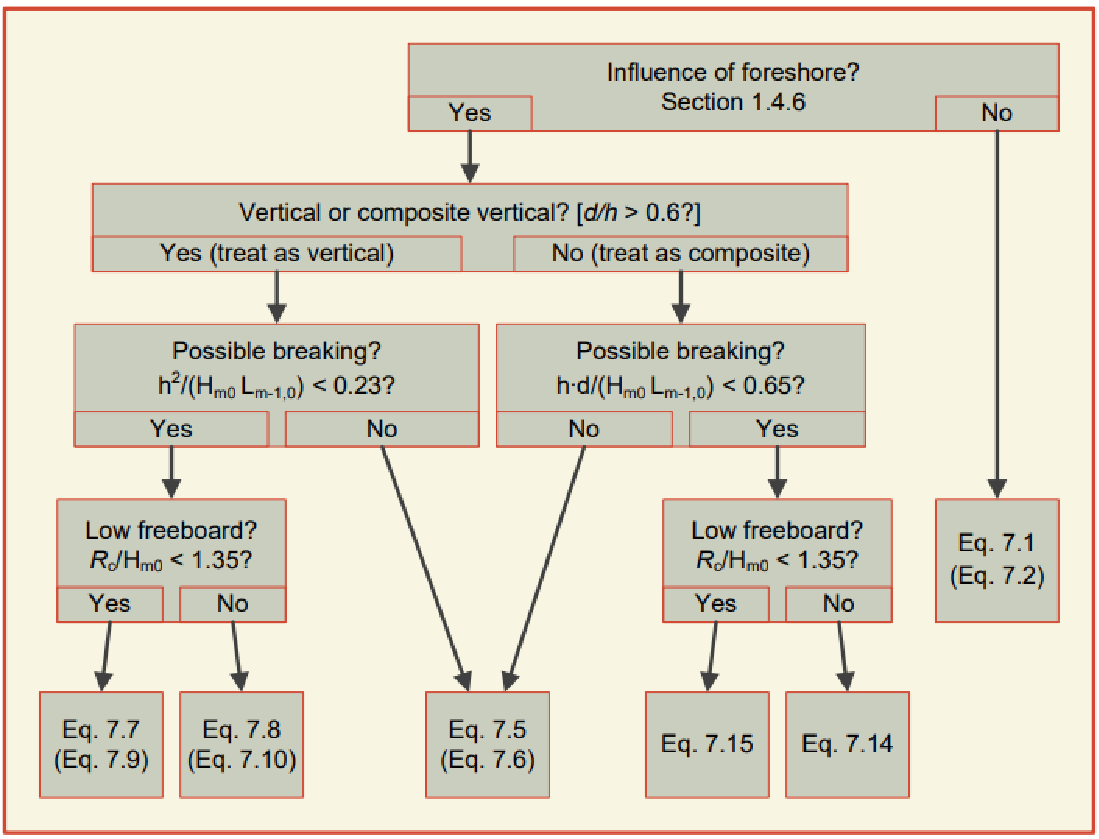
The general form of the EurOtop equation can be further refined to account for different shoreline slope conditions. For this wave overtopping sensitivity assessment, the shoreline types of interest are:

- Vertical
- Sloped (3H:1V)
- Shallow (20H:1V)

Sections B.1.3-5.4.1 to B.1.3-5.4.2 present the EurOtop equations used for the three shoreline types.

B.1.3-5.4.1 Vertical Slope

Figure B.1.3-21 shows the analysis process for calculating required freeboard for a vertical structure and checking for wave breaking conditions.



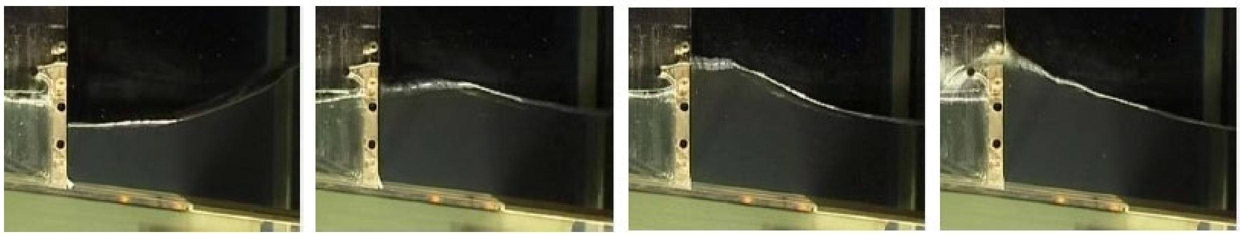
Source: EurOtop (2018)

Figure B.1.3-21: EurOtop Equation for Vertical Slope

B.1.3-5.4.1.1 Impulsive Wave Overtopping

Evaluating wave overtopping at steep (e.g., vertical) structures requires consideration of non-impulsive and impulsive wave conditions. The EurOtop equations used to calculate wave overtopping on vertical walls are dependent on whether non-impulsive or impulsive wave conditions exist. A check on non-impulsive and impulsive wave overtopping conditions was applied to every timestep of the analysis period.

Non-impulsive conditions occur when waves are small (relatively) compared to the water depth. Non-impulsive conditions are likely to occur when there is no foreshore present to influence the waves, or a deep foreshore is present, and waves can smoothly overtop the structure creating "green water" overtopping (Figure B.1.3-22).



Source: EurOtop (2018)

Figure B.1.3-22: Non-Impulsive Wave Conditions

Impulsive conditions occur on steep structures when waves are larger relative to the water depth, likely when a wave is shoaling over a foreshore or the structure's base and breaking at the structure. Waves will impact violently against the structure, generating short-term forces that are 10 to 40 times stronger than those experienced under non-impulsive conditions. In these conditions there is likely a violent jet of water rushing upwards, usually highly aerated in the process (Figure B.1.3-23).

Wave overtopping with impulsive wave conditions results in large increases in freeboard required to meet allowable overtopping thresholds.

Source: EurOtop (2018)

Figure B.1.3-23: Impulsive Wave Conditions

The EurOtop equation (7.4) below shows the check on non-impulsive versus impulsive wave conditions to determine the design value equations for overtopping on steep structures.

$$h^* = \frac{h^2}{H_{m0}L_{m,1-0}} > 0.23 \text{ [non-impulsive conditions]}$$

$$h^* = \frac{h^2}{H_{m0}L_{m,1-0}} \leq 0.23 \text{ [impulsive conditions]}$$

Or, for an armored rubble mound with significant influence (d/h < 0.6), EurOtop equation (7.13) is used instead.

$$h^* = \frac{dh}{H_{m0}L_{m,1-0}} > 0.65 \text{ [non-impulsive conditions]}$$

$$h^* = \frac{dh}{H_{m0}L_{m,1-0}} \leq 0.65 \text{ [impulsive conditions]}$$

Where:

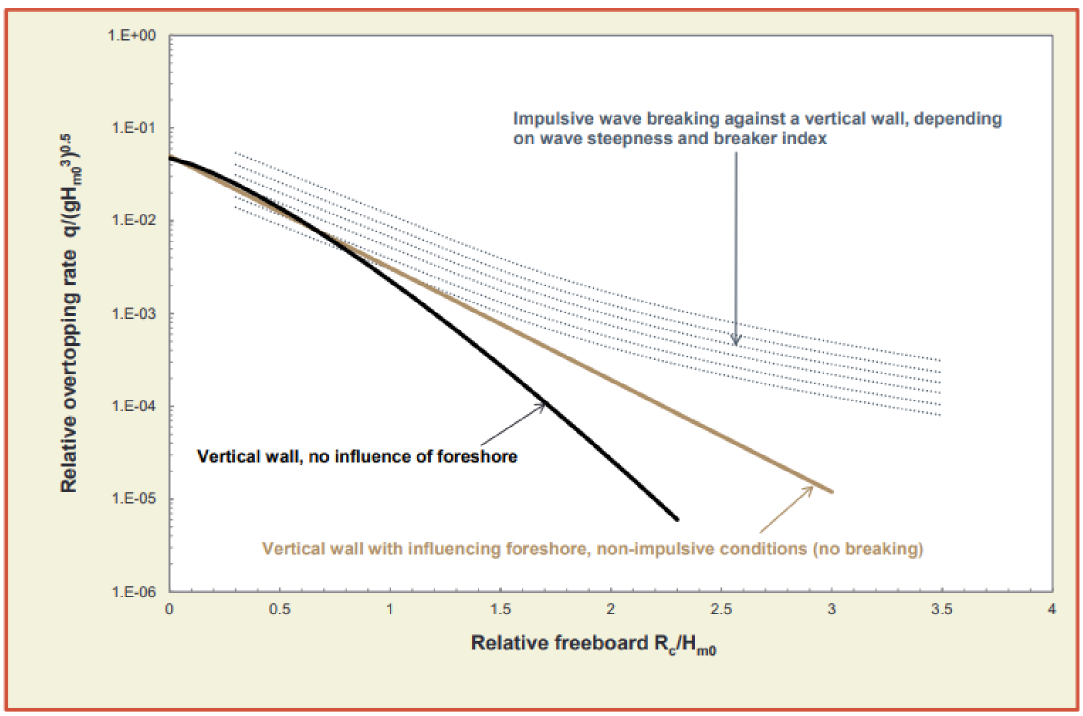
d = water depth to top of rubble mound

h = water depth at structure toe

H_{m0}= significant wave height

$L_{m,1-0}$ = deepwater wavelength (using wave period at structure toe)

Figure B.1.3-24 shows the EurOtop freeboard to significant wave height ratio and the relationship to overtopping amount for a vertical wall, for both impulsive and non-impulsive wave conditions. Where there are non-impulsive conditions (either due to no wave breaking toward the structure or where the foreshore water depth is sufficiently deep), the required freeboard is less compared to conditions where impulsive waves are breaking into the structure resulting in higher wave runup and overtopping.



Source: EurOtop, 2018

Figure B.1.3-24: EurOtop Relative Freeboard to Overtopping Relationship for Vertical Wall Structure

B.1.3-5.4.2 Steep (3H:1V) and Shallow (20H:1V) Slopes

For slopes that are less than 2H:1V, including the steep 3H:1V and shallow 20H:1V shoreline profiles, freeboard is calculated using the EurOtop equations (5.12, 5.13), below.

$$\frac{q}{\sqrt{g * H_{m0}^3}} = \frac{0.026}{\sqrt{\tan\alpha}}\xi_{m-1,0} * \exp\left[-\left(2.5 * \frac{Rc}{\gamma H_{m0}}\right)^{1.3}\right]$$

With a maximum bound of:

$$\frac{q}{\sqrt{g * H_{m0}^3}} = 0.1035 * \exp\left[-\left(1.35 * \frac{Rc}{\gamma H_{m0}}\right)^{1.3}\right]$$

B.1.3-5.4.3 Roughness/Slope Armoring

To account for armoring on the 3H:1V slope, a roughness factor of $\gamma_f = 0.6$ was applied to the shallow slope equations. This roughness factor corresponds to a single layer of rocks over an impermeable core, according to Table 6.2 of the EurOtop Manual, and is the lowest level of armoring presented (EurOtop, 2018). This roughness factor is also the same value recommended in the FEMA San Francisco Bay Area Coastal Study (BakerAECOM, 2013), providing additional confidence that this is a reasonable base estimate.

B.1.3-5.4.4 Wave Obliqueness

In addition to wave refraction accounted for in the shoaling process, freeboard estimates were modified to account for the obliqueness of waves once they reach the structure toe. An adjustment factor for wave obliqueness was applied, according to the EurOtop equations (5.29, 6.9, 7.17) for wave overtopping. These equations vary based on the type of shoreline, as shown below.

Sloped embankment:

$\gamma_\beta = 1 - 0.0033\beta$ for $\beta \leq 80^\circ$

$\gamma_\beta = 0.736$ for $\beta > 80^\circ$

Armored slope:

$\gamma_\beta = 1 - 0.0063\beta$ for $\beta \leq 80^\circ$

$\gamma_\beta = 0.496$ for $\beta > 80^\circ$

Vertical wall:

$\gamma_\beta = 1 - 0.0062\beta$ for $\beta < 45^\circ$

$\gamma_\beta = 0.72$ for $\beta \geq 45^\circ$

B.1.3-5.4.5 Extreme Value Analysis

An EVA is applied to the shoreline crest elevations calculated at every timestep in the 30-year hindcast, resulting in the crest elevation required to prevent hazardous overtopping associated with 1% AEP overtopping condition. To create a parameter analogous to the 2-foot wave proxy for comparison purposes, the 1% AEP SWEL (feet NAVD88) is subtracted from the 1% crest elevation (feet NAVD88). This parameter is called freeboard for the purposes of this assessment.

Freeboard (feet) = EVA(SWEL + R_C) - EVA(SWEL)

= 1% AEP Shoreline Crest Elevation - 1% AEP Stillwater Elevation

EVA = extreme value analysis

Rc = freeboard

SWEL = stillwater elevation

In using the EurOtop equations, wave runup (e.g., $R_{2\%}$) is not explicitly solved. The 1% AEP SWEL was calculated by applying an EVA to the SWEL timeseries, using a Peaks Over Threshold and Generalized Pareto fit. The statistical methods behind the EVA analysis are described in detail in *Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding*.

Section B.1.3-6. Results

Table B.1.3-4 to Table B.1.3-17 show the freeboard elevation required above the 1% AEP SWEL to define the minimum shoreline crest elevation for each location, for the three shoreline profiles with and without armoring or wave dissipation features. The shoreline profiles include:

- Vertical (no armoring)
- Vertical + rock mound armoring (2-foot rock mound)
- Steep Slope 3H:1V (no armoring)
- Steep Slope 3H:1V (armored)
- Shallow Slope 20H:1V (no vegetation)
- Shallow Slope 20H:1V (vegetated)

For the freeboard required above the 1% AEP SWEL, both the best estimate value and lower and upper bounds are provided. The lower and upper bounds are derived from the EVA analysis of the required shoreline crest elevations calculated for the entire hindcast period, where the lower bound represents the smallest freeboard estimate within an acceptable goodness of fit of the parameters used in the EVA analysis, and the upper bound represents the largest freeboard estimate within an acceptable goodness of fit. See *Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding* for detailed methodology of the EVA analysis, which uses the Generalized Pareto Distribution to fit maxima to a range of quantile bins used to define the number of maxima selected in the Peaks Over Threshold approach.

B.1.3-6.1 Freeboard (q = 0.001 m3/s/m; structure safety)

Table B.1.3-4 presents the best estimate of the freeboard required and the upper and lower bound for each shoreline profile.

For profiles with vertical walls, the best estimate of freeboard ranges from 1.0 to 2.5 feet if the profile is not armored. With armoring (shallow mound armoring), the best estimate of freeboard does not change if non-impulsive wave conditions dominate. Note that a higher mound will reduce the foreshore water depth and trigger impulsive wave conditions, resulting in additional height of shoreline needed to limit overtopping conditions.

Vertical wall estimates are highly sensitive to the presence or absence of impulsive wave breaking at the wall. Impulsive wave conditions are triggered from large waves combined with low enough water depths to trigger wave breaking at the structure. Analysis of the 31-year hindcast period found that extreme conditions were dominated by events with high SWELs and non-impulsive wave conditions, rather than lower SWEL and impulsive wave conditions. Under these conditions vertical walls perform quite well, with reduced freeboard requirement when compared with unarmored steep slopes (3H:1V), which will always trigger wave breaking at some point as waves progress up the slope of the structure. It should be noted, however, that vertical walls still pose a risk of performing drastically worse should they experience wave conditions beyond those considered in this study, such that impulsive overtopping conditions become statistically relevant to the EVA. While this stochastic transition was not well documented in this study, it was observed that individual events with impulsive conditions resulted in freeboard requirements up to two to three times larger than their non-impulsive counterparts.

For steeply sloping profiles, the best estimate of freeboard ranges from 2.2 to 4.5 feet if the profile is not armored. With armoring, the mean freeboard can be reduced to approximately 1.0 to 2.5 feet.

Steeper sloped shorelines (e.g., 3H:1V) without armoring require the largest freeboard to minimize wave overtopping conditions, however armoring on the structure slope can greatly reduce the amount of freeboard required.

For shallow profiles without vegetation, the best estimate of freeboard is 0.1 to 0.4 foot, and overtopping can be almost fully mitigated with sufficient vegetation to attenuate the incident wave heights.

Overall, shallow profile slopes with vegetation allow for the highest performance in reducing wave overtopping potential, however the reduction in wave overtopping is primarily attributed to the slope angle, where sufficient landward extent is required to allow the shoreline height to exceed most combinations of storm surge and wave conditions (while minimizing any wave runup).

Table B.1.3-4: Freeboard (q = 0.001 m3/s/m; structure safety)

Profile	Statistic	Transect 18	Transect 20	Transect 23
Vertical	Best Estimate	1.1	2.1	2.5
	Bounds	*0.64 to 1.20*	*1.80 to 2.41*	*1.87 to 2.74*
Vertical+mound	Best Estimate	1.1	2.1	2.5
	Bounds	*0.64 to 1.20*	*1.80 to 2.41*	*1.87 to 2.74*
3H:1V	Best Estimate	2.2	4.4	4.5
	Bounds	*1.49 to 2.58*	*3.98 to 4.71*	*4.12 to 4.70*
3H:1V+armor	Best Estimate	1.0	2.1	2.5
	Bounds	*0.64 to 1.12*	*1.73 to 2.27*	*1.86 to 2.59*
20H:1V	Best Estimate	0.3	0.4	0.3
	Bounds	*-0.25 to 0.38*	*0.08 to 0.53*	*0.03 to 0.52*
20H:1V+veg	Best Estimate	-0.1	0.0	0.0
	Bounds	*-0.29 to 0.29*	*-0.30 to 0.32*	*-0.27 to 0.28*

B.1.3-6.2 Freeboard (q = 0.00003 m3/s/m; pedestrian safety)

Additional freeboard height is required to limit overtopping below conditions dangerous to pedestrians, compared to that required for structures (Section B.1.3-6.1). Table B.1.3-5 presents the best estimate of the freeboard required and the upper and lower bound for each shoreline profile.

For vertical walls, the best estimate of freeboard ranges from 2.2 to 3.9 feet if the profile is not armored. With armoring (shallow mound armoring), the freeboard required does not change if non-impulsive wave conditions dominate. Note that a higher mound will reduce the foreshore water depth and trigger impulsive wave conditions, resulting in additional height of shoreline needed to limit overtopping conditions.

For steeply sloping profiles, the best estimate of freeboard ranges from 3.8 to 6.9 feet if the profile is not armored. With armoring, the freeboard required can be reduced to approximately 1.5 to 3.7 feet.

For shallow profiles without vegetation, the best estimate of freeboard ranges from 0.5 to 0.8 foot, and overtopping can be almost fully mitigated with sufficient vegetation to attenuate the incident wave heights.

Table B.1.3-5: Freeboard in Feet (q = 0.00003 m3/s/m; pedestrian safety)

Profile	Statistic	18	20	23
Vertical	Best Estimate	2.2	4.2	3.9
	Bounds	*1.66 to 2.78*	*3.74 to 4.39*	*3.65 to 4.26*
Vertical+mound	Best Estimate	2.2	4.2	3.9
	Bounds	*1.66 to 2.78*	*3.74 to 4.39*	*3.65 to 4.26*
3H:1V	Best Estimate	3.8	6.7	6.9
	Bounds	*3.48 to 4.16*	*6.41 to 7.32*	*6.44 to 7.17*
3H:1V+armor	Best Estimate	1.5	3.5	3.6
	Bounds	*1.02 to 1.84*	*3.06 to 3.60*	*3.39 to 4.03*
20H:1V	Best Estimate	0.6	0.8	0.8
	Bounds	*0.05 to 0.67*	*0.28 to 0.89*	*0.30 to 0.88*
20H:1V+vegetation	Best Estimate	-0.1	0.0	0.0
	Bounds	*-0.28 to 0.30*	*-0.30 to 0.32*	*-0.26 to 0.29*

B.1.3-6.3 Freeboard with Sea Level Rise

Table B.1.3-6 to Table B.1.3-11 show the freeboard required for each transect locations, considering sea level rise. The addition of sea level rise to the input water levels resulted in a generally linear response in the required freeboard for all shoreline profile types and all transect locations. This could be attributed to the incident wave heights in the hindcast being primarily non-depth limited; therefore, without increasing the magnitude of the incident wave heights coupled with increasing the water depths, nonlinear increases in wave runup as sea levels rise is not likely to occur. Increasing wave heights due to changes in wind speed that could occur as the climate changes was beyond the scope of this study but should be considered in a future update as increasing wave heights could trigger depth-limited breaking and a nonlinear response in wave runup with sea level rise. A nonlinear response on steep and vertical slopes has been observed in similar studies (FEMA, 2016).

Table B.1.3-6: Transect 18 – Freeboard in Feet (q = 0.001 m3/s/m; structure safety)

Profile	Statistic	Historical	OPC Likely 2040	OPC Likely 2090	OPC Likely 2140	USACE High 2040	USACE High 2090	USACE High 2140
Vertical	Best Estimate	1.1	1.1	1.1	1.3	1.1	1.2	1.5
	Bounds	*0.64 to 1.20*	*0.69 to 1.22*	*0.83 to 1.23*	*0.85 to 1.33*	*0.70 to 1.23*	*0.87 to 1.28*	*0.81 to 1.52*
Vertical+ mound	Best Estimate	1.1	1.1	1.1	1.3	1.1	1.2	1.5
	Bounds	*0.64 to 1.20*	*0.69 to 1.22*	*0.83 to 1.23*	*0.85 to 1.33*	*0.70 to 1.23*	*0.87 to 1.28*	*0.81 to 1.52*
3H:1V	Best Estimate	2.2	2.3	2.4	2.6	2.3	2.5	2.8
	Bounds	*1.49 to 2.58*	*1.54 to 2.65*	*1.67 to 2.83*	*1.76 to 3.02*	*1.58 to 2.68*	*1.72 to 2.96*	*1.95 to 3.14*
3H:1V+ armor	Best Estimate	1.0	1.0	1.0	1.1	1.0	1.1	1.2
	Bounds	*0.64 to 1.12*	*0.65 to 1.10*	*0.69 to 1.19*	*0.70 to 1.21*	*0.65 to 1.11*	*0.71 to 1.21*	*0.74 to 1.25*
20H:1V	Best Estimate	0.3	0.3	0.3	0.3	0.3	0.3	0.3
	Bounds	*-0.25 to 0.38*	*-0.24 to 0.37*	*-0.25 to 0.41*	*-0.22 to 0.40*	*-0.25 to 0.40*	*-0.21 to 0.41*	*-0.18 to 0.40*
20H:1V+ vegetation	Best Estimate	-0.1	-0.1	-0.1	-0.1	-0.1	-0.1	-0.1
	Bounds	*-0.29 to 0.29*	*-0.28 to 0.29*	*-0.28 to 0.29*	*-0.28 to 0.29*	*-0.28 to 0.29*	*-0.28 to 0.29*	*-0.28 to 0.29*

OPC = California Ocean Protection Council

Table B.1.3-7: Transect 18 – Freeboard in Feet (q = 0.00003 m3/s/m; pedestrian safety)

Profile	Statistic	Historical	OPC Likely 2040	OPC Likely 2090	OPC Likely 2140	USACE High 2040	USACE High 2090	USACE High 2140
Vertical	Best Estimate	2.2	2.3	2.2	2.4	2.3	2.3	2.7
	Bounds	*1.66 to 2.78*	*1.68 to 2.79*	*1.21 to 2.19*	*1.25 to 2.36*	*1.69 to 2.82*	*1.23 to 2.27*	*1.37 to 2.59*
Vertical+ mound	Best Estimate	2.2	2.3	2.2	2.4	2.3	2.3	2.7
	Bounds	*1.66 to 2.78*	*1.68 to 2.79*	*1.21 to 2.19*	*1.25 to 2.36*	*1.69 to 2.82*	*1.23 to 2.27*	*1.37 to 2.59*
3H:1V	Best Estimate	3.8	3.8	3.9	4.0	3.8	3.9	4.2
	Bounds	*3.48 to 4.16*	*3.53 to 4.23*	*3.62 to 4.41*	*3.72 to 4.60*	*3.55 to 4.26*	*3.69 to 4.53*	*3.97 to 4.95*
3H:1V+armor	Best Estimate	1.5	1.6	1.6	1.7	1.6	1.6	1.8
	Bounds	*1.02 to 1.84*	*1.03 to 1.81*	*1.06 to 1.93*	*1.05 to 1.93*	*1.04 to 1.82*	*1.06 to 1.97*	*1.11 to 2.03*
20H:1V	Best Estimate	0.6	0.5	0.5	0.5	0.5	0.5	0.5
	Bounds	*0.05 to 0.67*	*0.09 to 0.66*	*0.08 to 0.70*	*0.01 to 0.65*	*0.12 to 0.67*	*0.03 to 0.64*	*0.00 to 0.65*
20H:1V+ vegetation	Best Estimate	-0.1	-0.1	-0.1	-0.1	-0.1	-0.1	-0.1
	Bounds	*-0.28 to 0.30*	*-0.28 to 0.30*	*-0.28 to 0.30*	*-0.28 to 0.29*	*-0.28 to 0.30*	*-0.28 to 0.29*	*-0.28 to 0.29*

Table B.1.3-8: Transect 20 – Freeboard in Feet (q = 0.001 m3/s/m; structure safety)

Profile	Statistic	Historical	OPC Likely 2040	OPC Likely 2090	OPC Likely 2140	USACE High 2040	USACE High 2090	USACE High 2140
Vertical	Best Estimate	2.1	2.1	2.0	2.0	1.8	2.0	2.1
	Bounds	*1.80 to 2.41*	*1.79 to 2.41*	*1.69 to 2.35*	*1.74 to 2.37*	*1.54 to 2.36*	*1.72 to 2.35*	*1.81 to 2.41*
Vertical+ mound	Best Estimate	2.1	2.1	2.0	2.0	1.8	2.0	2.1
	Bounds	*1.80 to 2.41*	*1.79 to 2.41*	*1.69 to 2.35*	*1.74 to 2.37*	*1.54 to 2.36*	*1.72 to 2.35*	*1.81 to 2.41*
3H:1V	Best Estimate	4.4	4.4	4.4	4.5	4.4	4.5	4.6
	Bounds	*3.98 to 4.71*	*3.97 to 4.72*	*3.95 to 4.65*	*3.98 to 4.64*	*3.96 to 4.70*	*3.97 to 4.63*	*4.01 to 4.65*
3H:1V+armor	Best Estimate	2.1	2.1	2.1	2.0	2.0	2.0	2.0
	Bounds	*1.73 to 2.27*	*1.71 to 2.25*	*1.68 to 2.22*	*1.66 to 2.21*	*1.70 to 2.25*	*1.67 to 2.21*	*1.68 to 2.20*
20H:1V	Best Estimate	0.4	0.4	0.4	0.4	0.4	0.4	0.4
	Bounds	*0.08 to 0.53*	*0.08 to 0.53*	*0.07 to 0.53*	*0.07 to 0.53*	*0.07 to 0.53*	*0.07 to 0.53*	*0.07 to 0.53*
20H:1V+ vegetation	Best Estimate	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Bounds	*-0.30 to 0.32*	*-0.30 to 0.32*	*-0.30 to 0.32*	*-0.30 to 0.32*	*-0.30 to 0.32*	*-0.30 to 0.32*	*-0.30 to 0.32*

Table B.1.3-9: Transect 20 – Freeboard in Feet (q = 0.00003 m3/s/m; pedestrian safety)

Profile	Statistic	Historical	OPC Likely 2040	OPC Likely 2090	OPC Likely 2140	USACE High 2040	USACE High 2090	USACE High 2140
Vertical	Best Estimate	4.2	4.4	3.5	3.4	3.5	3.4	3.4
	Bounds	*3.74 to 4.39*	*3.79 to 4.65*	*3.26 to 3.77*	*3.23 to 3.87*	*3.08 to 3.79*	*3.21 to 3.84*	*3.29 to 3.95*
Vertical+ mound	Best Estimate	4.2	4.4	3.5	3.4	3.5	3.4	3.4
	Bounds	*3.74 to 4.39*	*3.79 to 4.65*	*3.26 to 3.77*	*3.23 to 3.87*	*3.08 to 3.79*	*3.21 to 3.84*	*3.29 to 3.95*
3H:1V	Best Estimate	6.7	6.7	6.7	6.7	6.6	6.7	6.8
	Bounds	*6.41 to 7.32*	*6.39 to 7.35*	*6.39 to 7.36*	*6.38 to 7.41*	*6.38 to 7.35*	*6.38 to 7.36*	*6.44 to 7.43*
3H:1V+ armor	Best Estimate	3.5	3.5	3.4	3.4	3.5	3.4	3.4
	Bounds	*3.06 to 3.60*	*3.04 to 3.58*	*2.99 to 3.54*	*2.97 to 3.50*	*3.03 to 3.58*	*2.98 to 3.52*	*2.96 to 3.52*
20H:1V	Best Estimate	0.8	0.8	0.8	0.8	0.8	0.8	0.8
	Bounds	*0.28 to 0.89*	*0.29 to 0.89*	*0.30 to 0.89*	*0.30 to 0.89*	*0.29 to 0.89*	*0.30 to 0.89*	*0.30 to 0.90*
20H:1V+ vegetation	Best Estimate	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Bounds	*-0.30 to 0.32*	*-0.30 to 0.32*	*-0.30 to 0.32*	*-0.30 to 0.32*	*-0.30 to 0.32*	*-0.30 to 0.32*	*-0.30 to 0.32*

Table B.1.3-10: Transect 23 – Freeboard in Feet (q = 0.001 m3/s/m; structure safety)

Profile	Statistic	Historical	OPC Likely 2040	OPC Likely 2090	OPC Likely 2140	USACE High 2040	USACE High 2090	USACE High 2140
Vertical	Best Estimate	2.5	2.5	2.6	2.6	2.5	2.6	2.7
	Bounds	*1.87 to 2.74*	*1.90 to 2.75*	*1.95 to 2.78*	*2.02 to 2.81*	*1.91 to 2.76*	*1.99 to 2.80*	*2.10 to 2.85*
Vertical+ mound	Best Estimate	2.5	2.5	2.6	2.6	2.5	2.6	2.7
	Bounds	*1.87 to 2.74*	*1.90 to 2.75*	*1.95 to 2.78*	*2.02 to 2.81*	*1.91 to 2.76*	*1.99 to 2.80*	*2.10 to 2.85*
3H:1V	Best Estimate	4.5	4.5	4.5	4.5	4.5	4.5	4.6
	Bounds	*4.12 to 4.70*	*4.12 to 4.70*	*4.14 to 4.72*	*4.16 to 4.73*	*4.13 to 4.70*	*4.15 to 4.73*	*4.18 to 4.77*
3H:1V+ armor	Best Estimate	2.5	2.5	2.5	2.5	2.5	2.5	2.6
	Bounds	*1.86 to 2.59*	*1.85 to 2.59*	*1.88 to 2.61*	*1.91 to 2.62*	*1.86 to 2.59*	*1.90 to 2.62*	*1.94 to 2.63*
20H:1V	Best Estimate	0.3	0.3	0.3	0.3	0.3	0.3	0.3
	Bounds	*0.03 to 0.52*	*0.03 to 0.52*	*0.04 to 0.53*	*0.04 to 0.53*	*0.03 to 0.52*	*0.04 to 0.53*	*0.04 to 0.53*
20H:1V+ vegetation	Best Estimate	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Bounds	*-0.27 to 0.28*	*-0.27 to 0.28*	*-0.27 to 0.28*	*-0.27 to 0.28*	*-0.27 to 0.28*	*-0.27 to 0.28*	*-0.27 to 0.28*

Table B.1.3-11: Transect 23 – Freeboard in Feet (q = 0.00003 m3/s/m; pedestrian safety)

Profile	Statistic	Historical	OPC Likely 2040	OPC Likely 2090	OPC Likely 2140	USACE High 2040	USACE High 2090	USACE High 2140
Vertical	Best Estimate	3.9	3.9	4.0	4.1	4.0	4.0	4.1
	Bounds	*3.65 to 4.26*	*3.67 to 4.28*	*3.72 to 4.34*	*3.76 to 4.40*	*3.68 to 4.29*	*3.73 to 4.37*	*3.83 to 4.44*
Vertical+ mound	Best Estimate	3.9	3.9	4.0	4.1	4.0	4.0	4.1
	Bounds	*3.65 to 4.26*	*3.67 to 4.28*	*3.72 to 4.34*	*3.76 to 4.40*	*3.68 to 4.29*	*3.73 to 4.37*	*3.83 to 4.44*
3H:1V	Best Estimate	6.9	6.9	7.0	7.1	6.9	7.1	7.3
	Bounds	*6.44 to 7.17*	*6.43 to 7.20*	*6.53 to 7.29*	*6.59 to 7.37*	*6.44 to 7.22*	*6.55 to 7.33*	*6.65 to 7.45*
3H:1V+ armor	Best Estimate	3.6	3.6	3.7	3.7	3.6	3.7	3.8
	Bounds	*3.39 to 4.03*	*3.41 to 4.03*	*3.42 to 4.04*	*3.43 to 4.05*	*3.41 to 4.03*	*3.42 to 4.04*	*3.45 to 4.09*
20H:1V	Best Estimate	0.8	0.8	0.8	0.8	0.8	0.8	0.8
	Bounds	*0.30 to 0.88*	*0.31 to 0.88*	*0.31 to 0.89*	*0.31 to 0.89*	*0.31 to 0.88*	*0.31 to 0.89*	*0.30 to 0.88*
20H:1V+ vegetation	Best Estimate	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Bounds	*-0.26 to 0.29*	*-0.26 to 0.29*	*-0.26 to 0.29*	*-0.26 to 0.29*	*-0.26 to 0.29*	*-0.26 to 0.29*	*-0.26 to 0.29*

B.1.3-6.3.1 EVA Model Fit and Extrapolation

The required freeboard results presented in Table B.1.3-4 to Table B.1.3-11 are relative to the 1% AEP water level derived using modeled hindcast data spanning 31 years. This means that the required freeboard estimates are larger than those required for any individual water level event in the 31 year hindcast. The use of an EVA model is necessary to bridge this gap between estimating the 1% AEP water level given its absence in the hindcast, and effectively extrapolate out a prediction of the 1% AEP water level. An example of the model fit and performance when estimating the 1% AEP water level is shown on Figure B.1.3-25 to Figure B.1.3-27 for a subset of the shoreline profiles. Figure B.1.3-25 to Figure B.1.3-27 show the results of the EVA model using the hindcast data compared to the empirical return period for the filtered extreme event (per the methodology described in *Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding*. This empirical return period is calculated as:

$$RP_{empirical} = \frac{Record\ Length\ (Years)}{Rank}$$

Where the rank is the index ranking of all extreme events from highest to lowest (i.e., 1 for the largest event, 2 for the second largest, etc.). Figure B.1.3-25 to Figure B.1.3-27 also shows the shoreline crest elevation (for three different shoreline types) required for the largest recorded event in the hindcast.

Figure B.1.3-25 to Figure B.1.3-27 show close alignment between the EVA model and corresponding empirical return periods, particularly in the moderate extremes of around 50% AEP (2-year return period) and below. This helps anchor the model for the higher return periods where data is much sparser and there is more uncertainty in the empirical return period. The largest three to four events in the hindcast tended to fall below the EVA model estimate, meaning these events were uncharacteristically large relative the trend established by the lower extremes and were slightly rarer, more extreme events than the statistical expectation for this period.

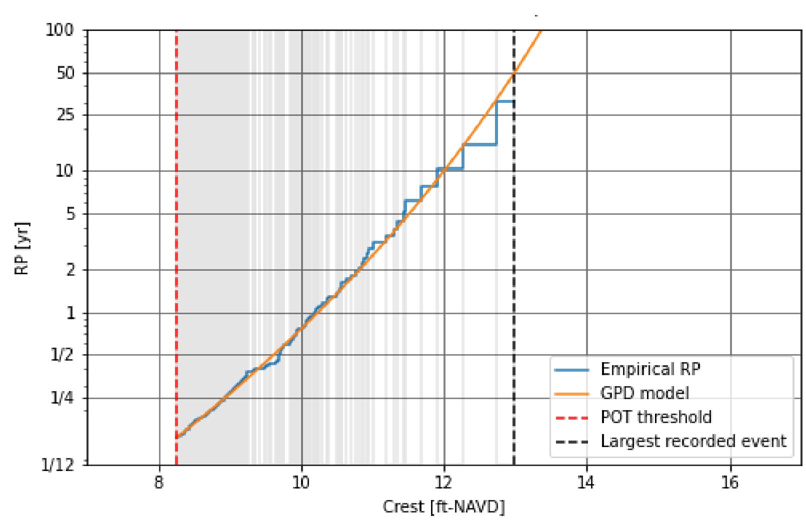


Figure B.1.3-25: EVA Model Fit, 3H:1V Steep Slope with Armoring, Transect 23, (q = 0.00003 m3/s/m)

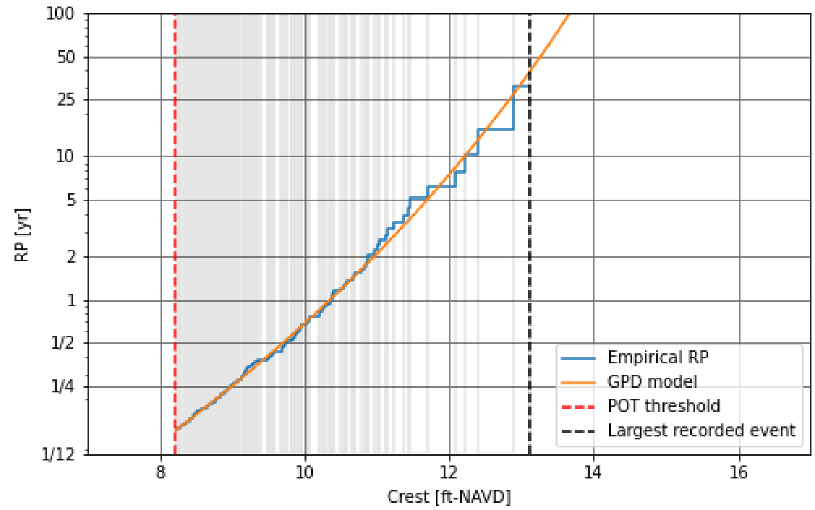


Figure B.1.3-26: EVA Model Fit, Vertical Wall with Armored Mound, Transect 23, (q = 0.00003 m3/s/m)

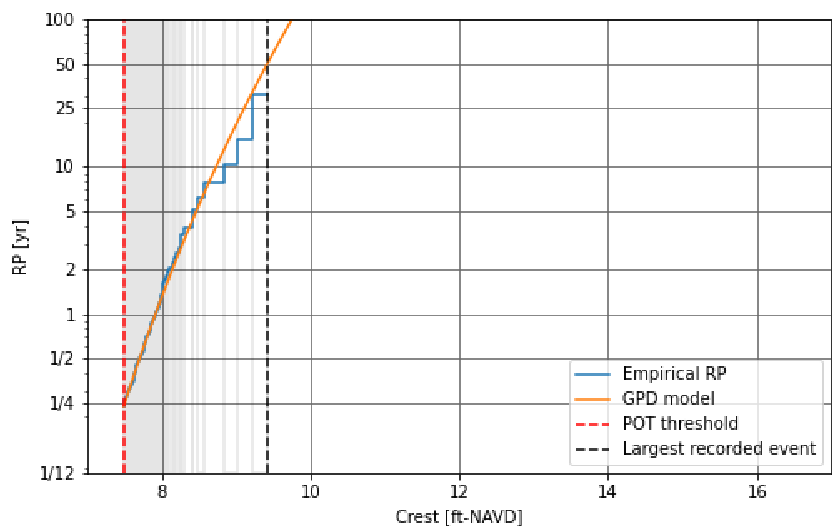


Figure B.1.3-27: EVA Model Fit, 20H:1V Shallow Slope with Vegetation, Transect 23, (q = 0.00003 m3/s/m)

B.1.3-6.3.2 Required Crest Elevation and Freeboard for Individual Events

Table B.1.3-12 to Table B.1.3-17 show the events during the 31-year hindcast resulting in the top ten highest shoreline crest elevations. The coincident water level, calculated freeboard requirement, wave height, wave angle and wave period that occurred is also presented.

These tables show that the peak events on record (roughly those with crest elevation return periods of 10 years or higher), were driven primarily by high stillwater (astronomical tides and storm surge) conditions of around 9 feet NAVD88, with a less extreme incident wave height of approximately 2 feet. However, for lesser extreme events (roughly those with crest return periods less than 10 years), show a larger proportion of wave dominant conditions.

The prevalence of wave dominant conditions in the moderate extrema scales directly with all the factors that resulted in larger estimates of the required freeboard. As conditions with stronger waves, or a higher sensitivity to waves, would naturally self-select a greater portion of large wave events for the extrema. This means that both Transect 20 and Transect 23 are more influenced by wave dominated events than Transect 18, and that the vertical wall and unarmored 3H:1V shorelines are more influenced by wave dominated events than the armored 3H:1V and shallow 20H:1V shorelines. Indeed, the influence of waves on the shallow 20H:1V slope is so weak that almost all the extreme events noted are SWL dominated. Lastly, the significance of wave dominated events is larger for the stricter pedestrian safety overtopping criteria of 0.00003 m3/s/m than the structural safety criteria of 0.001 m3/s/m. This occurs because the required crest elevation becomes more sensitive to wave height with the stricter overtopping threshold.

Table B.1.3-12: Transect 23 - 3H:1V Steep Slope with Armoring - (q = 0.001 m3/s/m; Structure Safety)

Date	Required Crest Elev. (feet NAVD)	Crest Return Period	SWL (feet NAVD)	SWL (Return Period)	Rc (feet)	Rc (Return Period)	Hm0-Toe (feet)	Hm0-Toe (Return Period)	Wave Angle (deg)	Wave Period (s)
1/26/1983	11.6	37.0	9.0	19.9	2.6	1.8	2.0	1.4	18.3	3.3
2/7/1998	11.3	21.6	8.4	4.7	2.8	4.1	2.1	2.4	15.5	3.4
1/11/2001	10.9	12.0	8.2	2.7	2.6	2.0	2.0	1.3	15.5	3.3
2/25/1983	10.7	8.8	8.4	4.3	2.3	0.7	1.9	0.8	17.2	3.1
2/19/1993	10.3	4.7	8.0	1.2	2.3	0.7	1.7	0.5	14.2	3.3
12/19/2002	10.2	4.3	7.8	0.6	2.4	1.1	1.9	0.9	23.5	3.5
3/9/1995	10.1	3.6	6.9	0.0	3.2	35.9	2.2	5.1	13.7	3.6
12/23/1979	9.9	3.0	7.1	0.0	2.8	4.1	2.1	2.5	22.9	3.6
1/16/1973	9.9	2.8	8.5	6.1	1.4	0.1	1.2	0.0	23.7	3.2
3/22/1995	9.9	2.6	7.4	0.1	2.5	1.2	1.8	0.7	12.1	3.2

Table B.1.3-13: Transect 23 - 3H:1V Steep Slope with Armoring - (q = 0.00003 m3/s/m; Pedestrian Safety)

Date	Required Crest Elev. (feet NAVD)	Crest Return Period	SWL (feet NAVD)	SWL (Return Period)	Rc (feet)	Rc (Return Period)	Hm0-Toe (feet)	Hm0-Toe (Return Period)	Wave Angle (deg)	Wave Period (s)
1/26/1983	13.0	49.3	9.0	19.9	4.0	1.9	2.0	1.4	18.3	3.3
2/7/1998	12.7	31.8	8.4	4.7	4.3	4.1	2.1	2.4	15.5	3.4
1/11/2001	12.3	15.0	8.2	2.7	4.0	2.0	2.0	1.3	15.5	3.3
2/25/1983	11.9	8.7	8.4	4.3	3.5	0.7	1.9	0.8	17.2	3.1
2/19/1993	11.7	6.3	6.9	0.0	4.8	31.5	2.2	5.1	13.7	3.6
12/19/2002	11.5	4.7	8.0	1.2	3.5	0.7	1.7	0.5	14.2	3.3
3/9/1995	11.4	4.5	7.8	0.6	3.7	1.0	1.9	0.9	23.5	3.5
12/23/1979	11.3	4.0	7.1	0.0	4.2	3.6	2.1	2.5	22.9	3.6
1/16/1973	11.3	3.7	7.1	0.0	4.2	3.3	2.2	5.2	8.8	3.1
3/22/1995	11.2	3.2	7.4	0.1	3.8	1.2	1.8	0.7	12.1	3.2

Table B.1.3-14: Transect 23 - Vertical Wall with Armored Mound - (q = 0.001 m3/s/m; Structure Safety)

Date	Required Crest Elev. (feet NAVD)	Crest Return Period	SWL (feet NAVD)	SWL (Return Period)	Rc (feet)	Rc (Return Period)	Hm0-Toe (feet)	Hm0-Toe (Return Period)	Wave Angle (deg)	Wave Period (s)
1/26/1983	11.6	37.2	9.0	19.9	2.6	1.0	2.0	1.4	18.3	3.3
2/7/1998	11.3	22.7	8.4	4.7	2.9	2.7	2.1	2.4	15.5	3.4
1/11/2001	10.9	12.4	8.2	2.7	2.6	1.1	2.0	1.3	15.5	3.3
2/25/1983	10.8	11.0	8.4	4.3	2.4	0.4	1.9	0.8	17.2	3.1
2/14/1992	10.3	5.3	7.1	0.0	3.2	11.4	2.2	5.2	8.8	3.1
2/19/1993	10.2	4.2	8.0	1.2	2.2	0.2	1.7	0.5	14.2	3.3
12/19/2002	10.1	3.8	7.8	0.6	2.3	0.3	1.9	0.9	23.5	3.5
3/9/1995	10.0	3.3	6.9	0.0	3.1	7.6	2.2	5.1	13.7	3.6
12/23/1979	9.8	2.7	7.1	0.0	2.7	1.6	2.1	2.5	22.9	3.6
1/16/1973	9.8	2.5	8.5	6.1	1.3	0.0	1.2	0.0	23.7	3.2

Table B.1.3-15: Transect 23 - Vertical Wall with Armored Mound - (q = 0.00003 m3/s/m; Pedestrian Safety)

Date	Required Crest Elev. (feet NAVD)	Crest Return Period	SWL (feet NAVD)	SWL (Return Period)	Rc (feet)	Rc (Return Period)	Hm0-Toe (feet)	Hm0-Toe (Return Period)	Wave Angle (deg)	Wave Period (s)
1/26/1983	13.1	39.3	9.0	19.9	4.1	1.0	2.0	1.4	18.3	3.3
2/7/1998	12.9	27.8	8.4	4.7	4.5	2.6	2.1	2.4	15.5	3.4
1/11/2001	12.4	13.1	8.2	2.7	4.2	1.1	2.0	1.3	15.5	3.3
2/25/1983	12.2	10.1	8.4	4.3	3.8	0.4	1.9	0.8	17.2	3.1
2/14/1992	12.1	8.4	7.1	0.0	5.0	11.4	2.2	5.2	8.8	3.1
3/9/1995	11.7	5.0	6.9	0.0	4.8	7.3	2.2	5.1	13.7	3.6
2/19/1993	11.5	3.7	8.0	1.2	3.5	0.2	1.7	0.5	14.2	3.3
12/19/2002	11.4	3.6	7.8	0.6	3.7	0.3	1.9	0.9	23.5	3.5
12/23/1979	11.4	3.3	7.1	0.0	4.3	1.5	2.1	2.5	22.9	3.6
3/22/1995	11.2	2.8	7.4	0.1	3.8	0.4	1.8	0.7	12.1	3.2

Table B.1.3-16: Transect 23 - 20H:1V Shallow Slope with Vegetation - (q = 0.001 m3/s/m; Structure Safety)

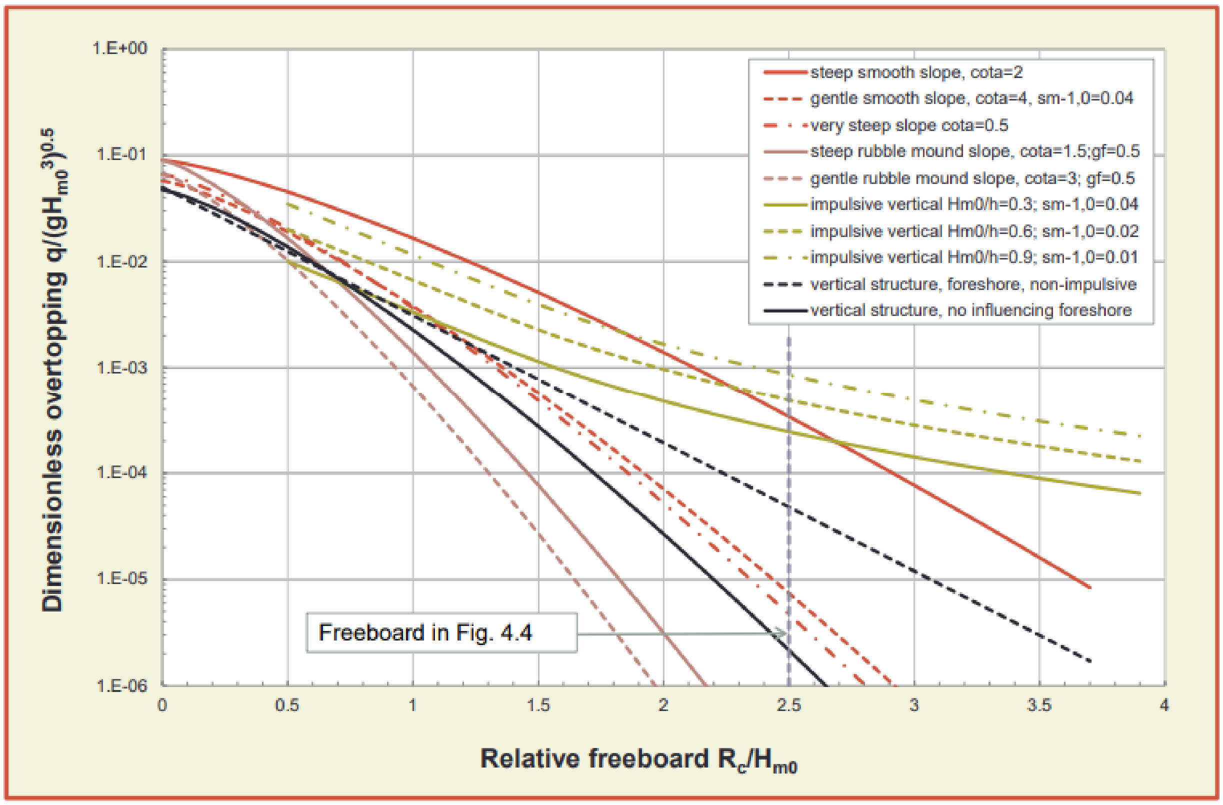
Date	Required Crest Elev. (feet NAVD)	Crest Return Period	SWL (feet NAVD)	SWL (Return Period)	Rc (feet)	Rc (Return Period)	Hm0-Toe (feet)	Hm0-Toe (Return Period)	Wave Angle (deg)	Wave Period (s)
1/27/1983	9.4	50.1	9.4	50.5	0.0	0.0	0.2	0.0	21.4	2.2
12/3/1983	9.2	32.9	9.2	33.2	0.0	0.0	0.6	0.0	82.2	1.5
2/6/1998	9.0	19.6	9.0	19.4	0.0	0.0	1.6	0.3	15.5	3.0
1/16/1973	8.8	13.3	8.8	13.4	0.0	0.0	0.0	0.0	-	0.7
11/30/1982	8.6	6.5	8.6	6.5	0.0	0.0	0.0	0.0	-	1.3
12/24/2003	8.5	5.1	8.5	5.1	0.0	0.0	0.1	0.0	79.9	0.9
2/25/1983	8.4	4.4	8.4	4.3	0.0	0.0	1.9	0.8	17.2	3.1
1/10/2001	8.4	4.4	8.4	4.3	0.0	0.0	1.7	0.5	20.8	3.1
12/11/1993	8.3	3.1	8.3	3.1	0.0	0.0	0.5	0.0	-11.9	1.9
1/7/1993	8.2	2.7	8.2	2.7	0.0	0.0	0.4	0.0	24.5	2.1

Table B.1.3-17: Transect 23 - 20H:1V Shallow Slope with Vegetation - (q = 0.00003 m3/s/m; Pedestrian Safety)

Date	Required Crest Elev. (feet NAVD)	Crest Return Period	SWL (feet NAVD)	SWL (Return Period)	Rc (feet)	Rc (Return Period)	Hm0-Toe (feet)	Hm0-Toe (Return Period)	Wave Angle (deg)	Wave Period (s)
1/27/1983	9.4	49.9	9.4	50.5	0.0	0.0	0.2	0.0	21.4	2.2
12/3/1983	9.2	32.8	9.2	33.2	0.0	0.0	0.6	0.0	82.2	1.5
2/6/1998	9.0	19.7	9.0	19.4	0.0	0.0	1.6	0.3	15.5	3.0
1/16/1973	8.8	13.2	8.8	13.4	0.0	0.0	0.0	0.0	-	0.7
11/30/1982	8.6	6.4	8.6	6.5	0.0	0.0	0.0	0.0	-	1.3
12/24/2003	8.5	5.0	8.5	5.1	0.0	0.0	0.1	0.0	79.9	0.9
2/25/1983	8.4	4.4	8.4	4.3	0.0	0.0	1.9	0.8	17.2	3.1
1/10/2001	8.4	4.4	8.4	4.3	0.0	0.0	1.7	0.5	20.8	3.1
12/11/1993	8.3	3.1	8.3	3.1	0.0	0.0	0.5	0.0	-11.9	1.9
1/7/1993	8.2	2.7	8.2	2.7	0.0	0.0	0.4	0.0	24.5	2.1

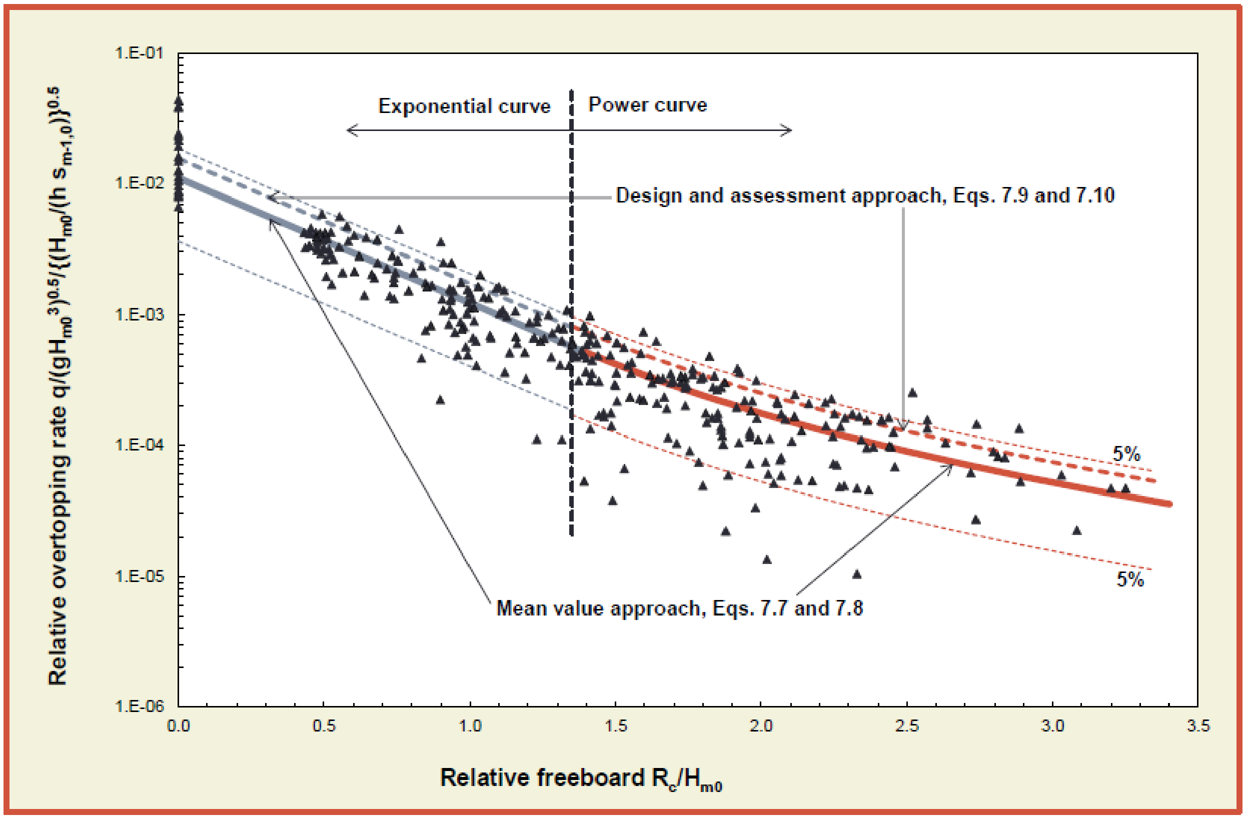
During this investigation, several events exceeded the relative freeboard to incident wave height ratio (R_c/H_{m0}) for vertical and steep structures. For example, if an incident wave height of 1 foot results in required freeboard of 8 feet, this is a relative freeboard ratio of 8.0, which is beyond the current EurOtop parameter space (see Figure B.1.3-26) for calculating overtopping (and physically unrealistic). For impulsive waves at vertical structures, overtopping is not well defined if the relative freeboard ratio exceeds 3.5; therefore, if the relative freeboard ratio for an individual extreme event exceeds 3.5, it was capped at 3.5. In the example of the 1 foot of incident wave height, the maximum freeboard is 3.5 feet. Implementation of this cap resulted in lower freeboard estimates for the vertical wall than their uncapped counterparts.

Figure B.1.3-28 shows freeboard to significant wave height ratio for various structures. Figure B.1.3-29 shows an example of the available measurements used to derive the relative freeboard curves on Figure B.1.3-28, where there are no available measures beyond a R_c/H_{m0} ratio close to 3.5.



Source: EurOtop (2018)

Figure B.1.3-28: Freeboard to Significant Wave Height Ratio (R_c/H_{m0}) for Various Structures



Source: EurOtop (2018)

Figure B.1.3-29: Empirical Eurotop Measurements to Derive Freeboard Relative to Significant Wave Height (R_c/H_{m0}) for Various Structures

Section B.1.3-7. Summary of Findings

Across all theoretical shoreline profile types, shallow slopes representative of natural shorelines with marsh vegetation can provide the most effective wave attenuation and reduction of additional shoreline crest height required above the 1% AEP SWEL.

Vertical structures require less additional height above the 1% AEP SWEL if minimal impulsive wave conditions persist during extreme events where the relative ratio of water depth to wave height is higher (see Section B.1.3-5.4.1.1). However, a rock mound with sufficient height relative to the water depth in front of a vertical wall may trigger impulsive wave conditions and higher wave runup even during extreme water level events.

Of the three shoreline types evaluated in this sensitivity assessment, steep shoreline slopes (e.g., 3H:1V) require the highest additional height above the 1% AEP SWEL. This could be attributed to a rapidly decreasing water depth closer to the crest of the structure, triggering wave breaking and higher runup conditions. For steep slopes (3H:1V), the presence of shoreline armoring can effectively reduce the required shoreline crest elevation to limit hazardous overtopping to pedestrians and structures.

For all analysis locations, armoring reduced the required wave proxy by approximately 50%.

For most shoreline configurations and locations, a linear response in wave runup and/or required freeboard was observed relative to sea level rise; however, the assessment did not capture all relevant processes of interest and the linear response should not be assumed to always hold true. In some cases (e.g., 3H:1V slope for Transect 18, limiting hazardous overtopping for structures) there was a nonlinear increase in freeboard required with sea level rise, with an additional 0.5 foot of height required by 2140 under the USACE High SLC curve. For the same slope, but limiting hazardous overtopping to pedestrians, the freeboard required increased by an additional 0.4 foot by 2140 under the USACE High SLC curve. This response should be considered, and evaluated in more detail, in the PED phase. This nonlinearity for steep slopes is likely occurring because of the high sensitivity of these slopes to waves. The nonlinear response is dampened with the addition of shoreline armoring.

In one instance, there is a decrease in freeboard required by 2140 for a vertical structure. This could occur if impulsive wave events resulting in higher overtopping (without sea level rise) begin to decrease in frequency in deeper water depths as sea levels rise. Because this sensitivity assessment considers many combinations of shoreline profile types, armoring conditions, locations, and sea level rise amounts within a generally low wave height regime resulting in extreme events, the results in Table B.1.3-4 to Table B.1.3-11 provide a best estimate of the freeboard required, with lower and upper uncertainty bounds, to account for variations from the statistical methods used to transform the extreme water levels in the 31 year hindcast into 1% AEP estimates.

These variations in linear or nonlinear response to sea level rise highlight the need for location and design specific shoreline profiles to further refine the required additional height of the crest elevation to account for wave runup and overtopping.

Section B.1.3-8. Caveats and Future Refinements

Several limitations and caveats are associated with this wave overtopping sensitivity assessment. There are limitations in the assessment due to constraints within the scope of the feasibility study as well as data gaps. Some limitations can be reduced during the PED phase.

- Individual water levels and wave height events were only available from the 1973 to 2003 FEMA hindcast period. Since 2004, there may have been combinations of water levels and wave heights that exceed those from the available hindcast. Several extreme events with high wave hazards occurred during the 2022-2023 winter storm season, resulting in structure damage along the San Francisco shoreline. These events are not reflected in the wave overtopping sensitivity assessment.
- This assessment does not consider changes in future storm conditions including more severe winter storms with larger storm surge, swell, or wind waves. Higher

wind speeds during storm events due to climate change could result in higher wave heights and larger wave runup events.

- The extreme wave heights and extreme water level conditions are not always concurrent. Statistical analysis of the combined crest elevation facilitated an understanding of how correlated wave and water level extrema were, but introduced some statistical variation in the EVA where the freeboard estimates have some degree of uncertainty. Both the best estimate and potential lower and upper ranges for freeboard estimates are provided.
- The current shoreline profiles from the FEMA San Francisco Bay Area Study were used as-is, bayward from the shoreline toe location. No change in the shoreline profile over time was considered with the sea level rise scenarios, either due to deposition or erosion of sediment or potential dredging activities. Sediment deposition in the foreshore would reduce the ratio of water depth to wave height, potentially triggering wave breaking or impulsive wave conditions leading to higher wave runup, and subsequently higher minimum required shoreline crest elevations.
- A simplifying assumption was made to assume marsh vegetation tracks with sea level rise on the shoreline profile (e.g., S. Pacifica tracks accordingly higher with the shift in the MLW tidal datum). This assumption was reasonable for this wave overtopping sensitivity analysis but should be further refined in subsequent design phases.
- This assessment does not consider other parameters relevant for evaluating overtopping hazards, including frequency of overtopping during extreme storm events, or the total volume of overtopping during a storm event.
- The freeboard heights presented in Table B.1.3-4 to Table B.1.3-11 represent the additional height above the 1% AEP SWEL needed to account for wave runup and limit hazardous overtopping. These freeboard heights do not apply to other return frequencies of other SWELs; however, the methods used in this assessment can support developing freeboard height estimates for additional return frequencies.

Future refinements to this wave overtopping sensitivity assessment could include:

- Refine shoreline profiles to better represent the flood protection measures, including wave dissipation features, in the PED phase.
- Evaluate additional shoreline locations and engineered shoreline slopes, including armoring configurations, to capture a wider range of foreshore and shoreline conditions for optimization of wave runup reduction benefits.
- Incorporate sensitivity assessment of larger storms occurring due to climate change, which may increase required minimum shoreline crest elevations.

- Evaluate additional overtopping thresholds and evaluate minimum crest elevations relative to 1% TWL and Maximum Wave Runup.
- Refine vegetation assumptions on natural shoreline slope (e.g., suitable vegetation types and zones relative to local tidal datums as they shift with sea level rise).
- Consider a broader range of NNBFs on a wider variety of shoreline types.

Section B.1.3-9. References

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