Wind Wave Attenuation and Runup Analysis for the Natural Shoreline Infrastructure Project Area

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Introduction and Purpose

The Natural Shoreline Infrastructure (NSI) project (Project) proposes to construct a natural shoreline along a vulnerable segment of the Highway 101 transportation corridor within the Project area. The natural shoreline design elements include (1) enhancing existing tidal wetlands; (2) constructing new tidal wetlands to an approximate elevation of 2.16 m (7.07 ft); (3) constructing a sand/gravel beach along the westerly edge of the tidal wetland to allow short-term marsh development following construction and provide long-term erosion protection of the wetland edge; and (4) an upland transition zone to the structure crest at an elevation of 3.5 m (11.5 ft). A goal of the proposed natural shoreline is to provide wind-wave attenuation and reduce flood risk to the transportation corridor and provide a more natural and resilient system for sea-level rise adaptation.

The purpose of this analysis is to determine wind-wave attenuation and wave runup at the Project area for existing and proposed design conditions that considers wind setup, wave generation and dissipation across North Bay, wind-wave attenuation at the shoreline, and wave run. This analysis was conducted in SI units (e.g. wave height in meters (m), wind speed as meters per second (mps)), and only final tabulated results will be presented in English units.

Methods and Intermediate Results

The methodology for conducting the wind-wave analysis incorporates a numerical wave model and numerous analytical approaches for determining specific wind and wave components for existing and design conditions. The following provides a general overview of specific components of the wind-wave analysis in the order they were estimated:

- Extreme water levels and tidal datum elevations,
- Extreme and typical wind speeds and direction,
- Fetch cross-section configuration, length and average water depth,
- Wind setup,
- Wave generation and dissipation along the cross-section (SWAN model), and
- Wave runup.

This section provides the methodology and intermediate results for each of the above components that are ultimately integrated and combined into the final wind-wave attenuation and wave runup analysis.

Extreme Water Levels and Tidal Datum Levels

The coastal still water levels for this analysis came from the 2D model developed as part of the Humboldt Bay sea-level rise modeling and inundation vulnerability mapping project (NHE, 2015). Estimates of average tidal datum levels (e.g. mean higher high water (MHHW)) and extreme high-water levels were extracted from five (5) grid cells adjacent to the Project area (Figure 1), and the average of the five grid cells was used to represent existing water levels adjacent to the Project Area (Table 1). For this analysis, water levels were referenced to Year 2012 in NAVD88.

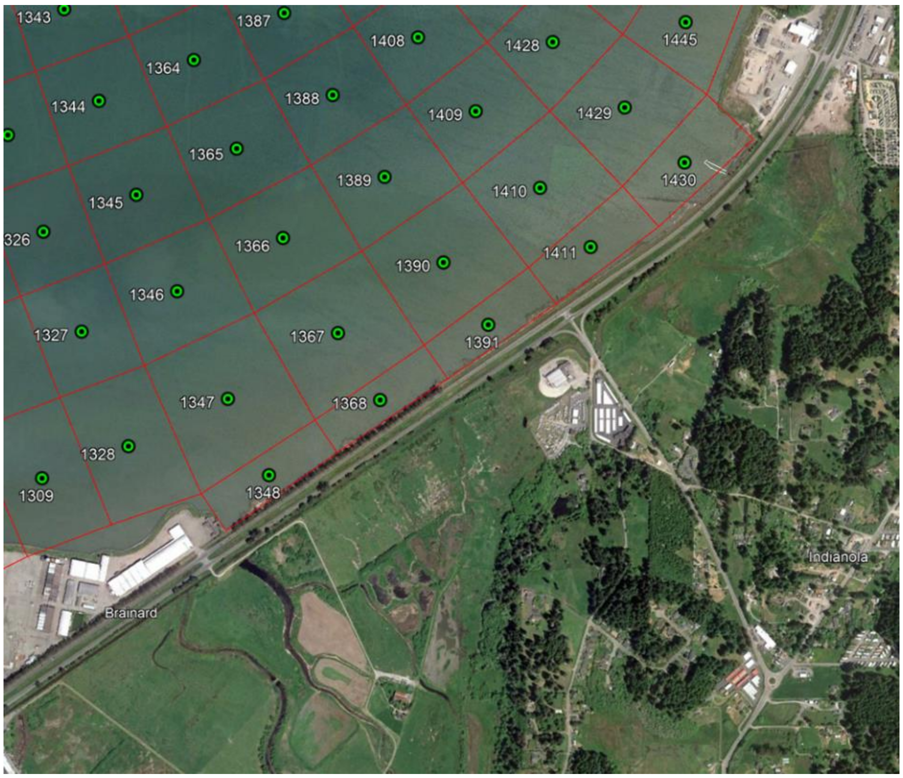

Figure 1 Location of five (5) grid cells extracted from the NHE (2015) 2D model to provide tidal datum and extreme high-water levels for the NSI Project area.

Table 1 Summary of average tidal datum and extreme high-water levels used for the NSI Project area wind-wave analysis extracted from the NHE (2015) 2D model. Water levels are referenced to Year 2012.

Tidal Datum or Return Level (yr)[1)	Annual Exceedance Probability (%)	Water Level	
		(m, NAVD88)	(ft, NAVD88)
MHHW		2.155	7.07
MMMW		2.562	8.41
2	50	2.858	9.38
10	10	3.050	10.01
100	100	3.251	10.67

1) MHHW = mean higher high water; and MMMW = mean monthly maximum water.

Extreme Wind Speed and Direction

The extreme 2-min average wind speeds (Table 2) used for this wind-wave analysis were obtained from the extreme wind speed analysis conducted for the Arcata/Eureka Airport as part of the NSI Project (see appropriate Appendix). Results of the extreme wind analysis also demonstrated that the fastest peak wind speeds tend to be from the northwest (315°) and southeast (135°) directions. For the NSI shoreline, the wind-wave analysis will focus on winds from the northwest (315°), or wind blowing towards the southeast. This wind direction represents winds blowing near perpendicular to the Project shoreline which provides the most conservative condition for wind-wave analysis at the shoreline.

Table 2 Summary of extreme 2-min average wind speeds used for the NSI Project area wind-wave analysis from the extreme wind analysis conducted for the Project.

Return Level (yr)	Annual Exceedance Probability (%)	Extreme 2-min Average Wind Speeds	
		(mps)	(mph)
1.053	95	16.94	37.9
2	50	17.94	40.1
10	10	19.82	44.3
100	100	21.43	47.9

Fetch Configuration, Length, and Average Water Depth

The fetch configuration for the wind-wave analysis generally followed a northwest (315°) to southeast (135°) transect (Figure 2) consistent with the peak northwest wind direction affecting the Project shoreline. The actual fetch cross section was adjusted slightly to maximize fetch length, avoid the small island in the middle of North Bay, and cross the Project shoreline more-or-less perpendicular. This resulted in the cross section having an actual orientation of 324° towards the northwest and 144° to the southeast. The extreme winds will be applied along this fetch direction.

Figure 2 Fetch cross-section location in North Bay for the NSI Project area.

The fetch cross-section topography (Figure 3) was extracted at a 1 m resolution from the 2020 USGS CoNED Topobathy DEM: Northern California (downloaded from: https://coast.noaa.gov/dataviewer).

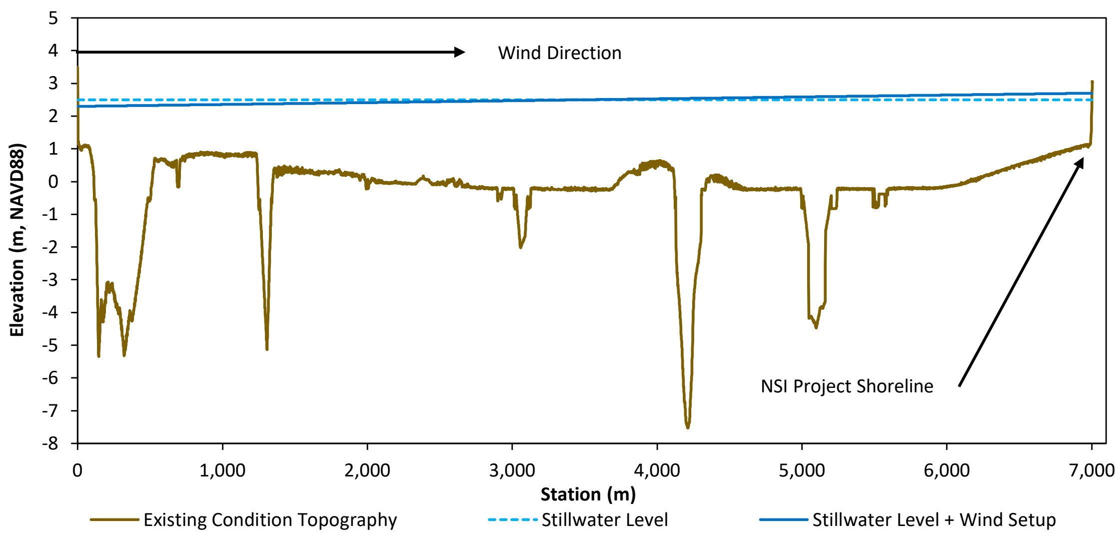

Figure 3 Fetch cross-section topography for existing condition showing the horizontal stillwater level at 2.5 m and the stillwater plus wind setup for a 21.4 mps wind speed.

For this analysis two design conditions were assessed each with modifications to the exiting Project shoreline (Figure 4). Design_Condition_RSP has the shoreline and rock slope protection (RSP) extending to a crest elevation 3.50 m (11.5 ft) with no natural shoreline features. Design_Condition_NS has the proposed natural shoreline design to the crest elevation with no RSP.

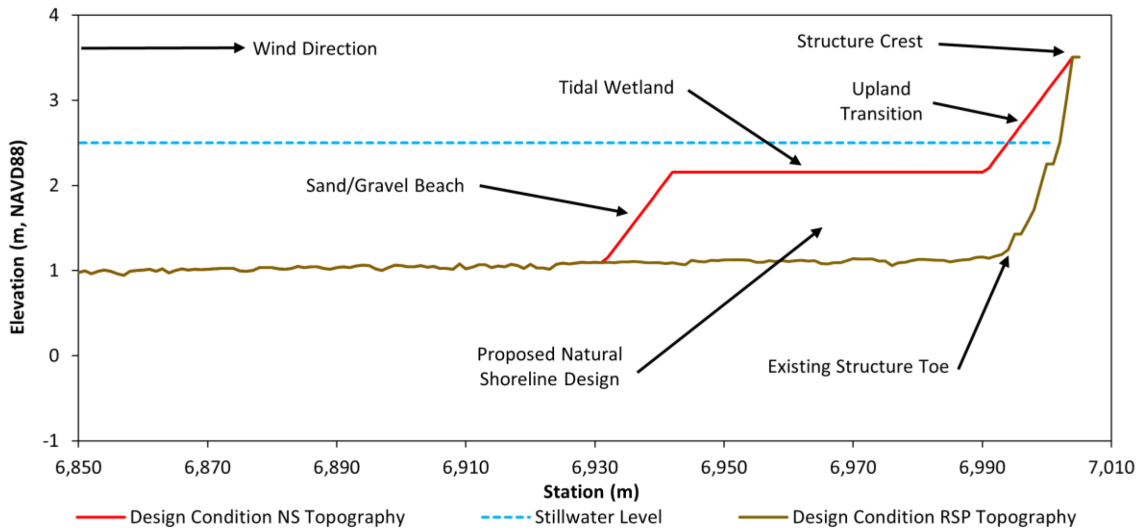

Figure 4 Fetch cross-section topography enlarged to the NSI Project area showing Design_Condition_RSP and Design_Condition_NS topography and project elements at the Project shoreline; horizontal stillwater level at 2.5 m.

Based on the cross-section configuration and bathymetry/topography, the fetch length and average water depth along the fetch were estimated for each water level (Table 3). For this analysis fetch length and average water depth were assumed the same for both design conditions, and will be used to support later analysis.

Table 3 Summary of fetch length and average fetch water depth for the water levels used in the NSI Project area wind-wave analysis. Water levels are referenced to Year 2012.

Tidal Datum or Return Level (yr)	Annual Exceedance Probability (%)	Water Level (m, NAVD88)	Fetch Length (m)	Average Water Depth (m)
MHHW	--	2.155	6,992	2.452
MMMW	--	2.562	6,996	2.857
2	50	2.858	6,999	3.152
10	10	3.050	7,000	3.343
100	100	3.251	7,002	3.543

Wind Speed Adjustment for Duration and Fetch Length

In larger water bodies the wave generation process producing maximum fetch-limited wave heights responds to average wind speeds over relatively long durations, for example a 30-min duration or longer (CEM 2015). The wind duration that generates fetch-limited conditions is a function of wind speed and fetch length, and it is often necessary to adjust extreme wind speeds to a fetch-limited duration wind speed. For this analysis, the procedure outlined in CEM (2015) was used to adjust extreme wind speeds to fetch-limited conditions.

The extreme 2-min average wind speeds (Table 2) are adjusted to the appropriate duration using the following two equations (CEM 2015):

$$U_t/U_{3600} = [1.277 + 0.296 \tanh(0.9 \log(45/t))] \qquad \text{for } t < 3{,}600 \text{ s} \qquad (1)$$

$$U_t/U_{3600} = 1.5334 - 0.15\, log(t) \qquad \text{for } 3{,}600 < t < 36{,}000 \text{ s} \qquad (2)$$

with wind speed duration t, U_t the wind speed at specified duration, and U_{3600} the 1-hour (3,600 s) wind speed.

The time for waves to reach fetch-limited conditions for a given fetch length and wind speed is given by (CEM 2015)

$$t_{x,u} = 77.23 \frac{x^{0.67}}{u^{0.34}\, g^{0.33}} \qquad (3)$$

where $t_{x,u}$ is the required for fetch-limited condition (s), x is the fetch length (m), u is the wind speed (mps), and g is the gravitational constant 9.81 m/s^2.

For wind speeds measured overland, the CEM (2015) procedure recommends increasing the wind speed by a factor of 1.2 to represent overwater wind speeds for fetch lengths less than 16 km. Since the NSI fetch length is ~ 7 km, the overland wind speed estimates were increased by the 1.2 factor.

Equations (1), (2) and (3) were iteratively used to adjust each extreme 2-min average wind speed to the appropriate fetch-limited conditions and then factored by 1.2 to overwater conditions (Table 4). Although fetch length is a function of water level (Table 3), the actual fetch lengths did not significantly change the reported final adjusted wind speeds. The final adjusted wind speeds listed in Table 4 are the wind speeds used in the wave setup, SWAN model and wave runup analysis.

Table 4 Summary of adjusted extreme wind speeds by wind speed and duration, and fetch length.

Return Level (yr)	Annual Exceedance Probability (%)	Extreme 2-min Average Wind Speed (mps)	Adjusted Wind Duration to Fetch (min)	Adjusted Wind Speed to Duration (mps)	Final Adjusted Wind Speed [1.2 Factor] (mps)	Final Adjusted Wind Speed [1.2 Factor] (mph)
1.053	95	16.94	87.2	14.14	16.96	38.0
2	50	17.94	85.5	15.00	18.00	40.3
10	10	19.82	82.6	16.61	19.93	44.6
100	100	21.43	80.4	17.98	21.58	48.3

Wind Setup

Wind blowing over water exerts a shear stress on the water surface, which can alter water levels and circulation patterns. Over large distances the change in water levels along the fetch can be significant (known as wind setup), with increases in water level at the leeward end and decreases at the windward end. For this analysis wind setup was estimated using the following equation (COE 1956, van Rinsum 2015):

$$h = 0.5k \frac{u_{10}^2}{gd} F \qquad (4)$$

where h is the wind setup (m), u_{10} is the wind speed (mps) at 10 m elevation, g is the gravitational constant 9.81 m/s^2, k is a friction constant (assumed 3.4 x 10^{-6} for this analysis), d is the average water depth (m) across the fetch, and F is the fetch length (m). The average water depth and fetch lengths used in the wave setup equation (4) for each water level are the values listed in Table 3, and the wind speeds are the final duration and fetch adjusted overwater wind speeds (Table 4).

For each water level and wind speed, maximum wind setup (Table 5) was estimated at the leeward end (at the Project shoreline) with equation (4), and then linearly interpolated across the fetch cross-section. Figure 3 shows an example of the horizontal stillwater level and the stillwater level plus the interpolated wind setup. The stillwater level plus interpolated wind setup is used as water level input for the wave analysis (SWAN model) described in a later section.

Table 5 Summary of wind setup estimates at the NSI Project shoreline based on water levels (Table 3) and adjusted overwater wind speeds (Table 5).

Adjusted Overwater Wind Speed Return Level (yr) and Estimate (mps)		Water Level Return Level and Estimate (m)				
		MHHW	**MMMW**	**2-yr**	**10-yr**	**100-yr**
		2.452	**2.857**	**3.152**	**3.343**	**3.543**
		Wave Setup at NSI Project Shoreline (cm)				
1.053-yr	**16.96**	14.2	12.2	11.1	10.4	9.9
2-yr	**18.00**	16.0	13.8	12.5	11.8	11.1
10-yr	**19.93**	19.6	16.9	15.3	14.4	13.6
100-yr	**21.58**	23.0	19.8	17.9	16.9	16.0

Wave Generation and Dissipation (SWAN Model)

The SWAN (Simulating WAves Nearshore) model was used to assess wave generation and dissipation of wind-waves across North Bay and adjacent to the Project shoreline for both design conditions (Design_Condition_RSP and Design_Condition_NS). SWAN is a third-generation wave action model that accounts for wave generation, propagation, and dissipation from wind in coastal areas, lakes and estuaries (The SWAN team 2021). The SWAN model can be configured as a 1D or 2D model and can be used for stationary and nonstationary simulations. For more details on the SWAN model reference can be made to Booij et al. (1999) and The SWAN team (2021).

Model Configuration

For this analysis SWAN was used in a 1D configuration and simulations were conducted as a stationary model. The SWAN model was configured to generate short-crested waves from wind; account for wave propagation from refraction, diffraction, and shoaling; and wave dissipation from whitecapping, nonlinear wave-wave interactions, depth-induced breaking, bottom friction, and vegetation (The SWAN team 2021). For this analysis, wave-induced setup was also estimated with SWAN.

The 1D Cartesian grid consisted of 1-m spaced nodes aligned with the 1-m fetch cross-sections for both design conditions (Figures 3 and 4). Since wave data are not available at the site, calibration and validation of the SWAN model was not possible, so the default SWAN values were used for most model parameters. A few of the key SWAN parameters are listed below:

- Whitecapping based on Komen formulation,
- Depth-induced breaking index = 0.73, and

- Bottom friction based on the JONSWAP formulation, with a friction coefficient = 0.019 m^2/s to account for the smoother mudflat bed.

For Design_Condition_RSP wave dissipation from vegetation was not used. For Design_Condition_NS, vegetation dissipation was implemented across the tidal wetland and upslope transition (Figure 4), with the following parameters used to represent tidal wetland vegetation measured on the Elk River (Caltrout et al. 2019):

- Plant height = 1.219 m,
- Plant diameter = 0.007 m,
- Number of stems = 365 stems/m^2, and
- Drag coefficient = 0.5.

The design condition SWAN models were run for each water level (Table 1) and final extreme wind speed (Table 4), resulting in 40 total simulations for both conditions. Prior to each simulation, the estimated water level plus the interpolated wind setup were input into SWAN. For this analysis, the current velocity was set to zero.

Wave Runup

The final component of the wave analysis was estimating wave runup at the Project shoreline. Wave runup is the surge of water up a beach or structure face from a breaking wave. When the wave runup exceeds the crest height of a structure, for example, overtopping can occur.

The 2% total wave runup (R) consists of three components (FEMA 2005): (1) static wave setup $(\bar{\eta})$, (2) dynamic wave setup $(\hat{\eta})$, and (3) the incident wave runup (R_{inc}), so that

$$R = \bar{\eta} + \hat{\eta} + R_{inc} \tag{5}$$

The two oscillating terms $(\hat{\eta} + R_{inc})$ are typically combined statistically, for example as $\left(2.0\sqrt{\hat{\eta}^2 + \left(\frac{R_{inc}}{2}\right)^2}\right)$.

The total wave runup is added to the estimated water level components described earlier (e.g. stillwater level) to provide an estimate of the total water level (TWL).

The incident wave runup (R_{inc}) was determined using the Technical Advisory Committee for Water Retaining Structures (TAW) method (van der Meer 2002) as modified in FEMA (2005):

$$R_{2\%} = H_{mo} 1.77 \gamma_r \gamma_b \gamma_\mathrm{B} \gamma_P \xi_{om} \qquad 0.5 \le \gamma_b \xi_{om} \le 1.8 \tag{6a}$$

$$R_{2\%} = H_{mo} \gamma_r \gamma_b \gamma_\mathrm{B} \gamma_P \left(4.3 - \frac{1.6}{\sqrt{\xi_{om}}}\right) \qquad 1.8 \le \gamma_b \xi_{om} \tag{6b}$$

where

$R_{2\%}$ is the 2% runup = $2\sigma_2$,
H_{mo} = the spectral significant wave height at the structure toe,
γ_r = reduction factor for influence of surface roughness (assumed 0.6 for this analysis),
γ_b = reduction factor for influence of berm (assumed 1.0 for this analysis),
γ_B = reduction factor for influence of angled wave attack (assumed 1.0 for this analysis),
γ_P = reduction factor for influence of structure permeability (assumed 1.0 for this analysis), and
ξ_{om} is a modified Iribarren number defined by equation (7).

The modified Iribarren number is defined as follows:

$$\xi_{om} = \frac{m_{TAW}}{\sqrt{H_{mo}/L_{m-1.0}}} \tag{7}$$

where

m_{TAW} = the slope of the structure determined iteratively per FEMA (2005, 2015),
$L_{m-1.0}$ = the deepwater wave length $(gT_{m-1.0}^2/2\pi)$,
$T_{m-1.0}$ = spectral wave period $(T_p/1.1)$, and
T_p = peak wave period at the structure toe.

Consistent with the approach used by FEMA (2015) to determine wave runup and total water levels in Humboldt Bay, wave runup was typically estimated along the existing shoreline where the shoreline is primarily composed of a natural shoreline (without fringing tidal wetland) or shoreline structures. For shorelines with a fringing tidal wetland, wave runup was not estimated as it was assumed the wetlands would dissipate the incident waves. A similar approach was adopted for this analysis, where wave runup was only estimated for Design_Condition_RSP along the armored shoreline. The Design_Condition_NS SWAN modeling demonstrated that waves are dissipated across the natural shoreline tidal wetland feature so wave runup does not occur. For Design_Condition_RSP simulations, the required wave information for the TAW equation (6) was taken at the toe of the existing rock shoreline which is approximately shown in Figure 4.

As noted in FEMA (2005, 2015), the TAW equation (6) is based on wave tank measurements which accounts for wave setup landward of the structure toe. Consequently, FEMA (2005) recommends reducing the dynamic setup to account for this. Also, if the incident waves have not broken prior to reaching the structure toe, then wave setup is not included in the total runup. For all cases simulated in this analysis, the water depth at the toe of the RSP structure is greater than 0.78 times the wave height from the SWAN results, indicating that waves have not broken prior to reaching the structure toe and dynamic wave setup $(\hat{\eta})$ was assumed zero. This is also consistent with the approach used by FEMA (2015) for determining wave runup estimates in Humboldt Bay.

For this analysis, the final total water level (TWL) was estimated as

$$TWL = stillwater + wind\ setup + wave\ height\ (SWAN\ model) + R_{2\%} \qquad (8)$$

It should be noted that the wave height determined by SWAN includes a small amount of static wave setup $(\bar{\eta})$, which could be positive for wave setup or negative for wave setdown depending on the simulation condition.

Final Results and Discussion

The wind-wave attenuation and wave runup analysis consisted of 40 simulated cases total that created a large amount of output data and potential results. Since the simulation results were used to support other analysis for this study, the results were compiled into an interactive spreadsheet for dissemination to the Project team. The remainder of this section provides summary results for all simulations, and graphical results for a couple simulations to demonstrate wave effects for both design conditions.

The SWAN model results for wave height are not wave crest elevations, but the height of the wave from trough to crest. Linear wave theory assumes that half of the wave height is above the stillwater level and half is below for symmetric waves (Dean and Darlrymple 1991). Under this assumption, the wave crest elevation would be determined by adding one-half of the wave height to the stillwater elevation. However, for this analysis it was assumed that the waves where asymmetrical with 70% of the wave height above the stillwater level, which is the assumption recommended by FEMA (2019); and the wave crest elevation was determined as the stillwater level (includes wind setup) plus 0.7 times the wave height.

Wave Attenuation for Both Design Conditions

An example of the simulated wave generation and attenuation process across the entire cross-section for Design_Condition_RSP is shown in Figure 5 for a water level of 2.562 m (MMMW) and wind speed of 19.82 mps (10% extreme wind). This figure shows wave generation from the windward end (Station 0) with the wave crest elevation increasing across the entire fetch length to approximately Station 6,300 where the maximum wave height occurs. At approximately Station 6,300 the mudflat begins to increase in elevation towards the Project shoreline (Station 7,000). The wave height and resulting wave crest elevation begins to decrease from Station 6,300 towards shore as the wave energy dissipates across the shallower mudflat.

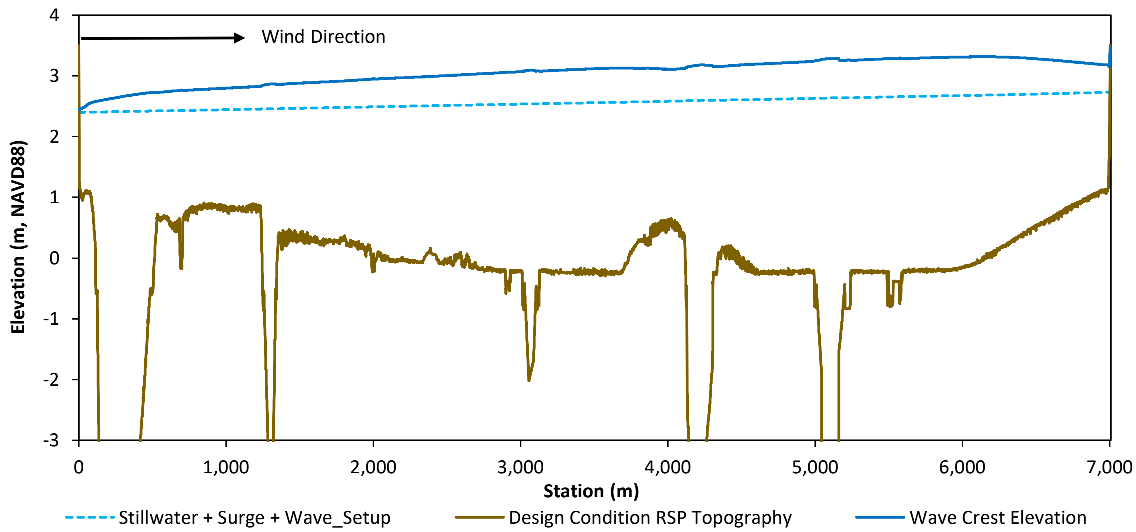

Figure 5 Wave generation and dissipation across entire fetch cross-section for Design_Condition_RSP with a water level of 2.562 m (MMMW) and wind speed of 19.82 mps (10% extreme wind). Vertical elevation truncated at -3 m.

To provide insight into how each design condition responds to different wind-wave conditions, results for two simulations are provided. Figure 6 shows the stillwater level (including wind and wave setup) for Design_Condition_NS, and wave crest elevations for both design conditions for a mean monthly maximum water (MMMW) level (2.562 m) and the 10% extreme wind speed (19.82 mps). For this simulation condition, both wave crests are attenuated at the shoreline and do not overtop the structure crest. For Design_Condition_RSP the wave height does not dissipate much across the mudflat, and results in a breaking wave along the structure face with wave runup approaching the crest of the structure. The Design_Condition_NS wave height is significantly dissipated as the wave traverses the sand/gravel beach and then the tidal wetland, with wave crest elevations approaching the stillwater level at the upland transition.

Figure 7 shows the stillwater level for both design condition wave crest elevations at the 10% water level (3.343 m) and the 18.00 mps 2% extreme wind speed. Under this wind and water level condition, the wave height and resulting wave runup for Design_Condition_RSP overtops the structure crest. However, the beach and tidal wetland dissipate the wave energy for Design_Condition_NS reducing the wave height to the stillwater level at the upland transition.

For both simulation combinations, the proposed natural shoreline significantly reduces wave heights as the wave traverses the sand/gravel beach and tidal wetland. Any residual wave height can easily be dissipated along the upland transition, essentially preventing wave runup from occurring for the natural shoreline. However, for Design_Condition_RSP wave runup occurs at the structure RSP for all simulated conditions

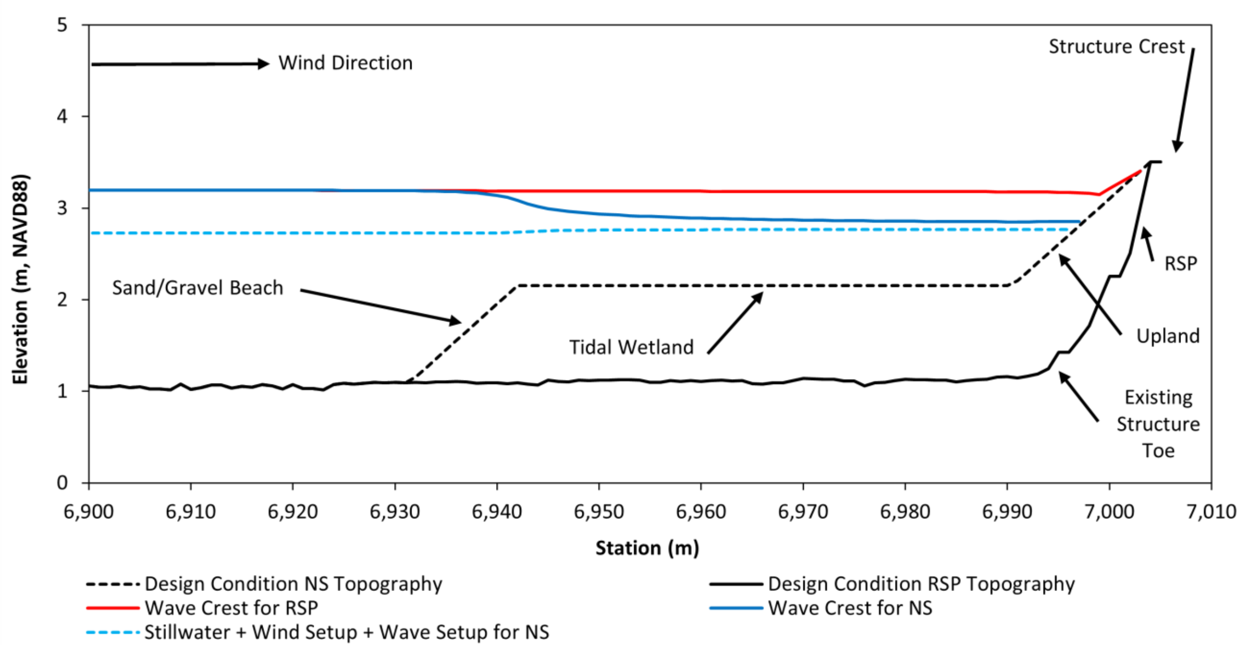

Figure 6 Wave generation and dissipation at Project shoreline for Design_Condition_RSP and Design_Condition_NS with a water level of 2.562 m (MMMW) and wind speed of 19.82 mps (10% extreme wind).

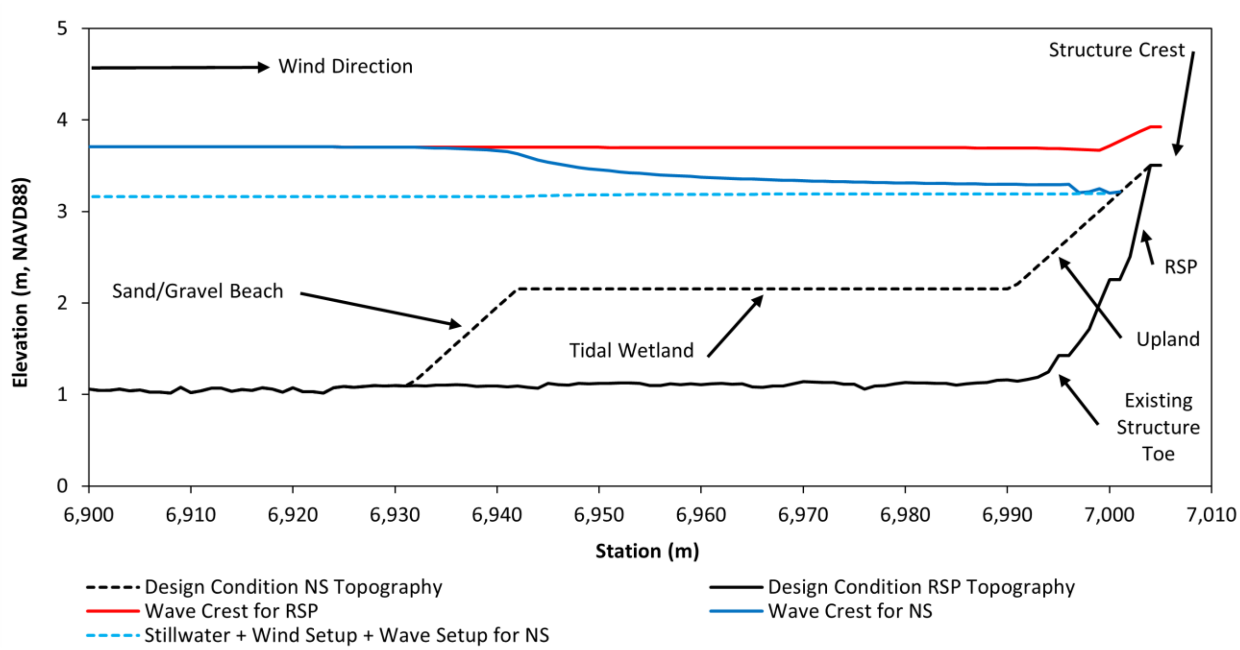

Figure 7 Wave generation and dissipation at Project shoreline for Design_Condition_RSP and Design_Condition_NS with a water level of 3.343 m (10% extreme water level) and wind speed of 18.00 mps (2% extreme wind).

Summary Results of Wave Crest and Wave Runup Elevations at the Project Shoreline for Both Design Conditions

Table 6 provides a summary of maximum reach wave heights, wave run up or wave crest height at the structure face, and the total water level at the bay edge of the structure face (either RSP or upland transition) for simulated conditions (Design_Condition_RSP and Design_Condition_NS). A “Y” or “N” flag is also provided if the total water level exceeds or does not exceed the structure crest elevation of 3.50 m (11.5 ft), respectively.

To provide a preliminary assessment of the resiliency of both design conditions to sea-level rise, a simple analysis was conducted using the simulated wind-wave results and a few basic assumptions. Assuming that the mudflat and/or tidal wetland keeps pace with sea-level rise, and the wave response across these features remains the same as existing conditions, then the sea-level rise estimate can just be added to the total water level and relate the result to the structure crest elevation. Table 6 also contains a “Y” or “N” flag indicating if the structure crest elevation is overtopped for 0.30 m (1 ft), 0.61 m (2 ft), and 0.91 m (3 ft) of sea level rise.

Tabulated results clearly indicate that for existing conditions the proposed natural shoreline design provides significant wind-wave attenuation and flood risk reduction at the Project shoreline compared to just extending the RSP (hardening) at the existing shoreline. The proposed natural shoreline prevents structure crest overtopping for all water levels and wind speeds analyzed. However, overtopping occurs for the hardened shoreline at the 2-yr and greater water levels and all extreme wind events.

For 0.30 m (1 ft) of sea-level rise, the proposed natural shoreline provides wave attenuation and flood risk reduction for the more frequent extreme events, but overtopping does occur for the less frequent events (e.g. 10-yr and 100-yr events). However, overtopping for the hardened shoreline occurs at the monthly water level (MMMW) and the more frequent extreme wind events. For 0.61 m (2 ft) of sea-level rise the natural shoreline provides overtopping protection for the monthly sea levels (MMMW) and any extreme wind condition, but overtopping occurs for all extreme water level and wind speed cases. The hardened shoreline only provides overtopping protection for daily water levels (MHHW) and the more frequent extreme wind speeds. By 0.91 m (3 ft) of sea-level rise the hardened shoreline is overtopped at daily levels and any extreme wind event, while the natural shoreline will provide protection at the daily water levels for all extreme wind events.

Based on sea-level rise projections (Figure 8) compiled by NHE (2018), 0.31 m (1 ft) of sea-level rise will likely occur around 2050 for either the RCP 2.6 or 8.5 emission scenario. Thus, the proposed natural shoreline should provide significant flood risk protection and sea-level rise resiliency for the Project shoreline to at least year 2050, and moderate risk protection to approximately 0.61 m (2 ft) of sea-level around year 2070.

Table 6 Summary of maximum reach wave heights, wave run up or wave crest height at structure face, total water level at structure face, structure crest overtopping, and sea-level effects at the Project shoreline for both design conditions (Design_Condition_RSP and Design_Condition_NS).

Return Level		Design Condition	Maximum Wave Height for Reach (m)	Stillwater Level + Wind Setup + Wave Setup (m, NAVD88)	Wave Condition at Structure Face				Overtop Structure Crest Elevation of 3.50 m (11.5 ft)			
Water Level	Wind Speed				Wave Runup (m)	0.7 x Wave Height (m)	Total Water Level (m, NAVD88)	Total Water Level (ft, NAVD88)	Existing Condition	0.30 m (1 ft) Sea Level Rise	0.61 m (2 ft) Sea Level Rise	0.91 m (3 ft) Sea Level Rise
MHHW	1.053-yr	RSP	0.739	2.299	0.489	--	2.788	9.15	N	N	N	Y
		NS		2.347	--	0.041	2.387	7.83	N	N	N	N
MHHW	2-yr	RSP	0.779	2.318	0.533	--	2.851	9.35	N	N	N	Y
		NS		2.364	--	0.047	2.411	7.91	N	N	N	N
MHHW	10-yr	RSP	0.835	2.354	0.578	--	2.932	9.62	N	N	Y	Y
		NS		2.400	--	0.059	2.458	8.07	N	N	N	N
MHHW	100-yr	RSP	0.882	2.389	0.616	--	3.005	9.86	N	N	Y	Y
		NS		2.432	--	0.067	2.499	8.20	N	N	N	N
MMMW	1.053-yr	RSP	0.784	2.683	0.654	--	3.336	10.95	N	Y	Y	Y
		NS		2.717	--	0.071	2.788	9.15	N	N	N	Y
MMMW	2-yr	RSP	0.836	2.699	0.684	--	3.383	11.10	N	Y	Y	Y
		NS		2.734	--	0.077	2.811	9.22	N	N	N	Y
MMMW	10-yr	RSP	0.927	2.731	0.734	--	3.465	11.37	N	Y	Y	Y
		NS		2.768	--	0.086	2.854	9.36	N	N	N	Y
MMMW	100-yr	RSP	0.990	2.761	0.786	--	3.547	11.64	Y	Y	Y	Y
		NS		2.805	--	0.013	2.818	9.24	N	N	N	Y
2-yr	1.053-yr	RSP	0.807	2.965	0.700	--	3.665	12.03	Y	Y	Y	Y
		NS		2.997	--	0.016	3.013	9.89	N	N	Y	Y
2-yr	2-yr	RSP	0.861	2.980	0.733	--	3.712	12.18	Y	Y	Y	Y
		NS		3.013	--	0.026	3.039	9.97	N	N	Y	Y
2-yr	10-yr	RSP	0.966	3.009	0.795	--	3.804	12.48	Y	Y	Y	Y
		NS		3.046	--	0.020	3.066	10.06	N	N	Y	Y
2-yr	100-yr	RSP	1.044	3.036	0.848	--	3.884	12.74	Y	Y	Y	Y
		NS		3.075	--	0.000	3.075	10.09	N	N	Y	Y

Table 6-Continued Summary of maximum reach wave heights, wave run up or wave crest height at structure face, total water level at structure face, structure crest overtopping, and sea-level effects at the Project shoreline for both design conditions (Design_Condition_RSP and Design_Condition_NS).

Return Level		Design Condition	Maximum Wave Height for Reach (m)	Stillwater Level + Wind Setup + Wave Setup (m, NAVD88)	Wave Condition at Structure Face				Overtop Structure Crest Elevation of 3.50 m (11.5 ft)			
Water Level	Wind Speed				Wave Runup (m)	0.7 x Wave Height (m)	Total Water Level (m, NAVD88)	Total Water Level (ft, NAVD88)	Existing Condition	0.30 m (1 ft) Sea Level Rise	0.61 m (2 ft) Sea Level Rise	0.91 m (3 ft) Sea Level Rise
10-yr	1.053-yr	RSP	0.818	3.151	0.727	--	3.878	12.72	Y	Y	Y	Y
		NS		3.179	--	0.006	3.185	10.45	N	N	Y	Y
10-yr	2-yr	RSP	0.874	3.164	0.762	--	3.926	12.88	Y	Y	Y	Y
		NS		3.195	--	0.006	3.201	10.50	N	Y	Y	Y
10-yr	10-yr	RSP	0.986	3.192	0.826	--	4.018	13.18	Y	Y	Y	Y
		NS		3.226	--	0.000	3.226	10.58	N	Y	Y	Y
10-yr	100-yr	RSP	1.072	3.217	0.882	--	4.099	13.45	Y	Y	Y	Y
		NS		3.254	--	0.011	3.265	10.71	N	Y	Y	Y
100-yr	1.053-yr	RSP	0.829	3.345	0.751	--	4.097	13.44	Y	Y	Y	Y
		NS		3.370	--	0.000	3.370	11.06	N	Y	Y	Y
100-yr	2-yr	RSP	0.884	3.357	0.790	--	4.147	13.61	Y	Y	Y	Y
		NS		3.385	--	0.004	3.390	11.12	N	Y	Y	Y
100-yr	10-yr	RSP	1.004	3.383	0.860	--	4.242	13.92	Y	Y	Y	Y
		NS		3.415	--	0.003	3.418	11.21	N	Y	Y	Y
100-yr	100-yr	RSP	1.094	3.407	0.914	--	4.322	14.18	Y	Y	Y	Y
		NS		3.442	--	0.004	3.446	11.30	N	Y	Y	Y

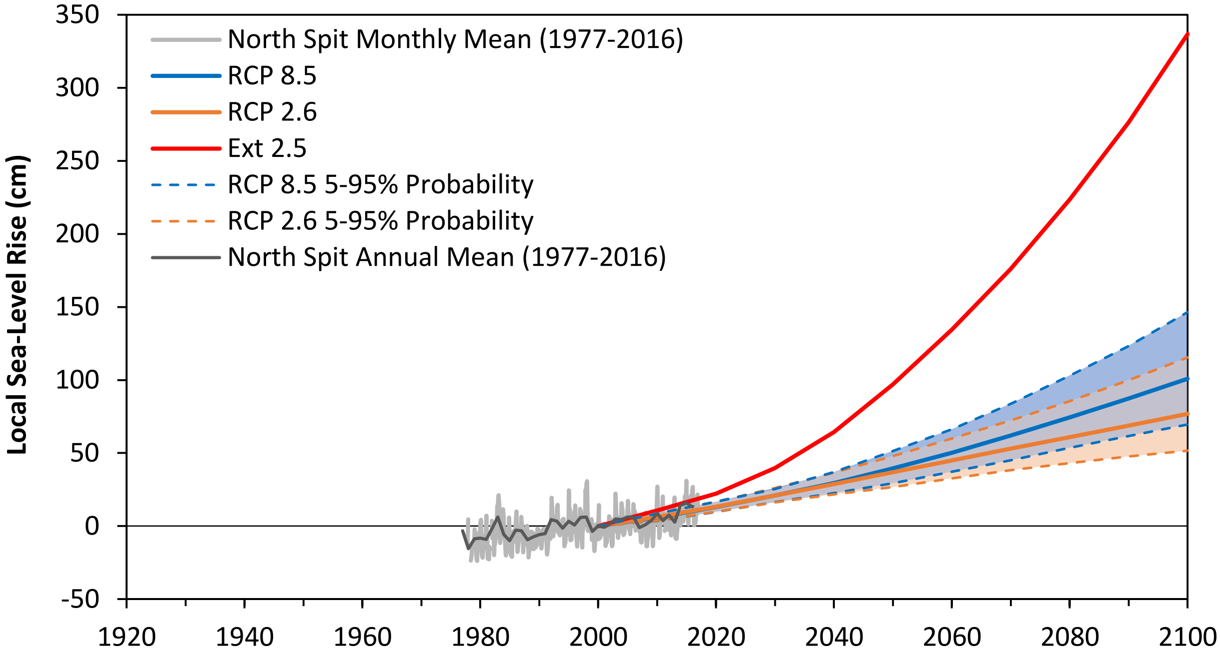

Figure 8 Local sea level rise projections at North Spit for RCP 8.5 and RCP 2.6 emission scenario (NHE 2018). The 5% and 95% probabilities are the shaded areas bounded by the dashed lines. The sea level observations are the annual and monthly mean sea levels for the 1977 to 2016 North Spit data (NOAA 9418767). All data referenced to year 2000 baseline.

References

Booij, N., R.C. Ris and L.H. Holthuijsen. 1999. A third-generation wave model for coastal regions, Part I, Model description and validation, J.Geoph.Research, 104, C4, 7649-7666.

California Trout, Stillwater Sciences, and Northern Hydrology & Engineering. 2019. Elk River Recovery Assessment: Recovery Framework. Prepared by California Trout, Arcata, CA; Stillwater Sciences, Arcata, CA; and Northern Hydrology & Engineering, McKinleyville, CA for North Coast Regional Water Quality Control Board, Santa Rosa, CA.

Coastal Engineering Manual (CEM). 2015. Chapter 2, Meteorology and Wave Climate. U.S. Army Corps of Engineers. Washington D.C. EM 1110-2-1100 (Part II).

Corps of Engineers (COE). 1956. Wind Set-Up and Waves in Shallow Water. Technical Memorandum No. 27. Beach Erosion Board, U.S. Corps of Engineers.

Dean, R.G. and R.A. Dalrymple. 1991. Water wave mechanics for engineers and scientists. World Scientific, New Jersey.

Federal Emergency Management Agency (FEMA), 2005. Final Draft Guidelines for Coastal Flood Hazard Analysis and Mapping for the Pacific Coast of the United States. Joint Project by FEMA Region IX, FEMA Region X, and FEMA Headquarters. FEMA Region IX, Oakland, CA.

Federal Emergency Management Agency (FEMA), 2014. Intermediate Data Submittal #3: Nearshore Hydraulics, Humboldt County, California. Prepared for FEMA Region IX. Prepared by Baker AECOM.

Federal Emergency Management Agency (FEMA), 2019. Guidance for Flood Risk Analysis and Mapping, Coastal Floodplain Mapping. FEMA, Guidance Document 39.

Northern Hydrology & Engineering (NHE). 2015. Humboldt Bay: Sea Level Rise, Hydrodynamic Modeling, and Inundation Vulnerability Mapping. Prepared for the State Coastal Conservancy, and Coastal Ecosystems Institute of Northern California. McKinleyville, CA. Report available at http://www.coastalecosystemsinstitute.org.

Northern Hydrology & Engineering (NHE), 2018. Sea-Level Rise in the Humboldt Bay Region, Update 2: December 2018. Partial funding for document preparation by the City of Arcata. NHE, McKinleyville, CA. Report available at http://www.coastalecosystemsinstitute.org.

The SWAN team. 2021. SWAN Cycle III Version 41.31 AB, User Manual, and SWAN Scientific and Technical Documentation. Delft University of Technology, Facility and Civil Engineering and Geosciences, Environmental Fluids Mechanics Section, The Netherlands.

Van Rinsum, G.P. 2015. Accuracy wind set-up for irregularly shaped lakes with a strong varying water depth. Thesis. University of Technology Delft, Civil Engineering and Geosciences.