APPENDIX D1
COASTAL ENGINEERING EVALUATION

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1 INTRODUCTION

A coastal engineering evaluation was conducted to provide predictions of extreme wave conditions along the shoreline at Port Gamble Bay ("Site") to inform the remedial design for the Port Gamble Bay Cleanup. Long-term wind data from a nearby wind gauge was used to estimate the 2-, 10-, 20-, 50-, and 100-year storm events for the area from a variety of wind directions. These extreme wind speeds, fetch lengths, and average depths were then used to estimate the storm waves that will impact the intertidal zone of Port Gamble. Predicted wave heights were used to estimate stable rock sizes and the extent of armoring required for the proposed intertidal cap areas. The impacts of predicted sea level rise (SLR) for the years 2050 and 2100 (NRC 2012) on predicted wave heights and proposed stable rock sizes for remedial actions are also discussed in this appendix.

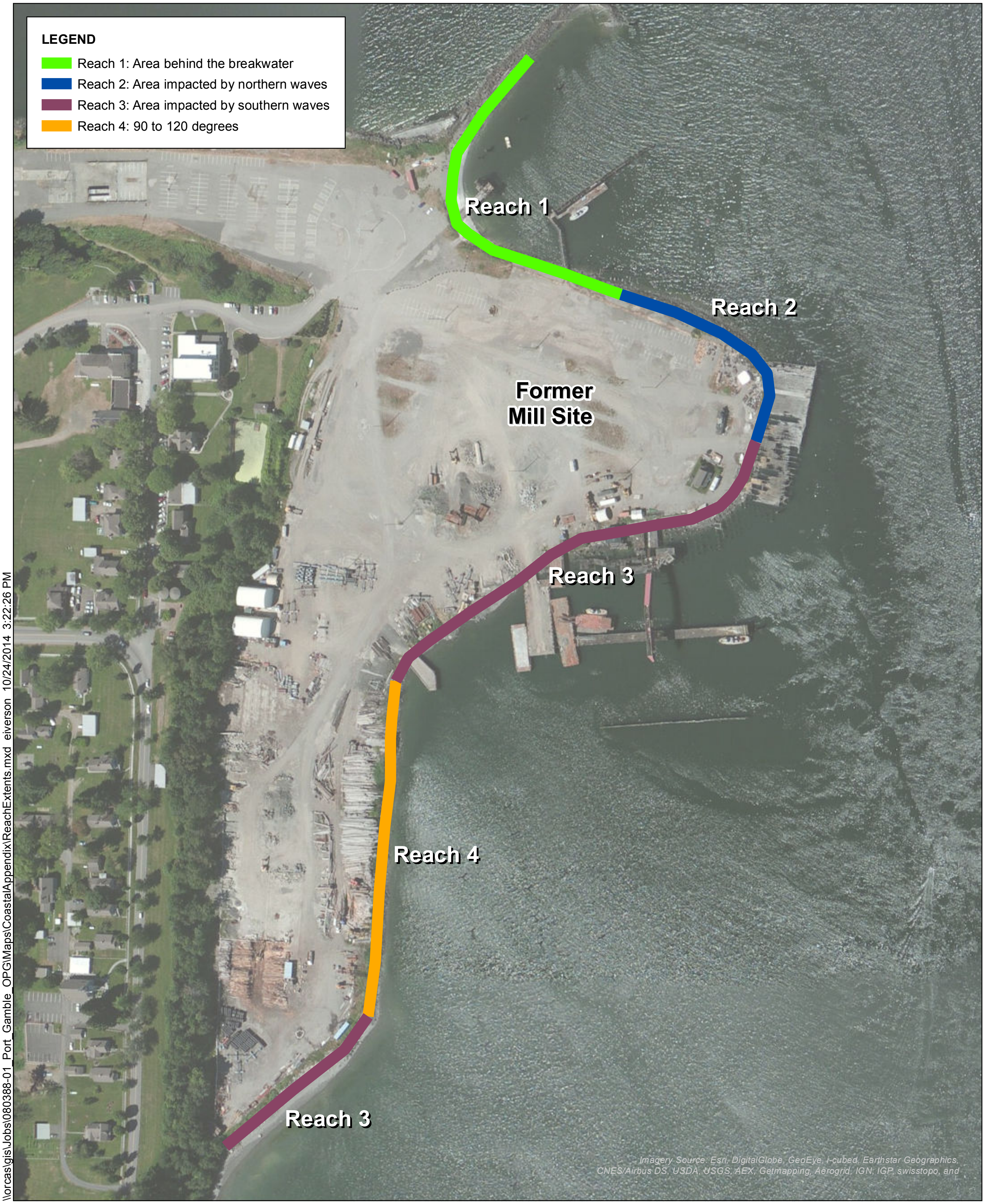
Possible nearshore/shoreline restoration options (i.e., coordinated with but separate from the cleanup project) were also considered as part of the overall coastal engineering evaluation. The results of this coastal evaluation were used to help inform conceptual design of proposed restoration options, as described in Appendix M of this EDR.

2 SITE DESCRIPTION

The Port Gamble project Site is located in the northwest portion of Puget Sound at the northern end of Hood Canal. The north shoreline faces Puget Sound (Hood Canal) and the east and south shorelines face Port Gamble Bay. For the purposes of the coastal evaluation, the project shoreline was divided into four reaches, as shown in Figure D1-1, based on the adjacent waterbody and shoreline orientation.

Reach 1 (shown in green in the figure) includes the shoreline reach adjacent to Puget Sound that is sheltered by the existing breakwater. This reach is exposed somewhat to waves from the north. The Reach 2 shoreline (shown in blue in the figure) is also adjacent to Puget Sound, and is partially sheltered from waves from the northwest by the existing breakwater, but is exposed to waves from the north. Reaches 3 (shown in purple in the figure) and 4 (shown in orange in the figure) include shorelines adjacent to Port Gamble Bay. Reach 3, which consists of two segments, is located behind the existing wooden breakwater[1] and is exposed to waves from the south and southeast. Reach 4 is located south of the existing wooden breakwater and is exposed to waves from the east, and to a lesser extent the southeast.

[1] Although there is an existing wooden breakwater south of the Mill Site that currently protects the Reach 3 shoreline, this breakwater is scheduled for demolition and will not be replaced as part of the cleanup project.



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Coordinates are in NAD83 US State Plane Washington North, U.S. Survey Feet.

Feet
0 200 400

ANCHOR QEA

Figure D1-1
Shoreline Reaches
Appendix D1: Coastal Engineering Evaluation
Port Gamble Bay Cleanup Project

3 WATER LEVELS

National Oceanic and Atmospheric Administration (NOAA) Station 9445016 at Foulweather Bluff, located 5.75 miles north of Port Gamble, was used to estimate tides at the project Site. Table D1-1 shows the tide levels for the gage.

Table D1-1
Tide Levels for NOAA Station 9445016

Tide Type	Tide Level (feet MLLW)
Mean Higher High Water (MHHW)	10.2
Mean High Water (MHW)	9.3
Mean Tide Level (MTL)	6.0
Mean Sea Level (MSL)	6.0
Mean Low Water (MLW)	2.8
Mean Lower Low Water (MLLW)	0.0

4 WIND WAVE HINDCAST

The wave conditions near Port Gamble were estimated by applying wind wave growth formulas to wind data from NOAA station WPOW1 in West Point, Washington[2]. The wind data encompassed hourly wind speeds (2-minute averages) for the years of 1984 to 2009. Figure D1-2 shows a wind rose (frequency of occurrence based on wind speed and wind direction) for the wind data over the 25-year period of record. The wind data were used to predict extreme wind speed values for 2-, 10-, 20-, 50-, and 100-year return period storm events. The extreme wind speeds were evaluated for each 30-degree wind direction bin from true north (e.g., 0 to 30 degrees, 30 to 60 degrees, etc.). The Raleigh distribution was used to develop the extreme wind speeds with R^2 values equal to or greater than 0.91 for all direction bins.

The original wind data were found to have some apparent outliers, which represent significantly higher sustained wind speeds than nearby gage locations for the same time period. These data tend to skew estimates of extreme wind speeds for 150 to 210 degree directions, resulting in higher wind speeds than may be realistic. However, the original data were not altered (outliers were not removed) for this analysis, in order to be conservative.

[2] NOAA station WPOW1 was selected for the wave hindcast due to its long record of data.

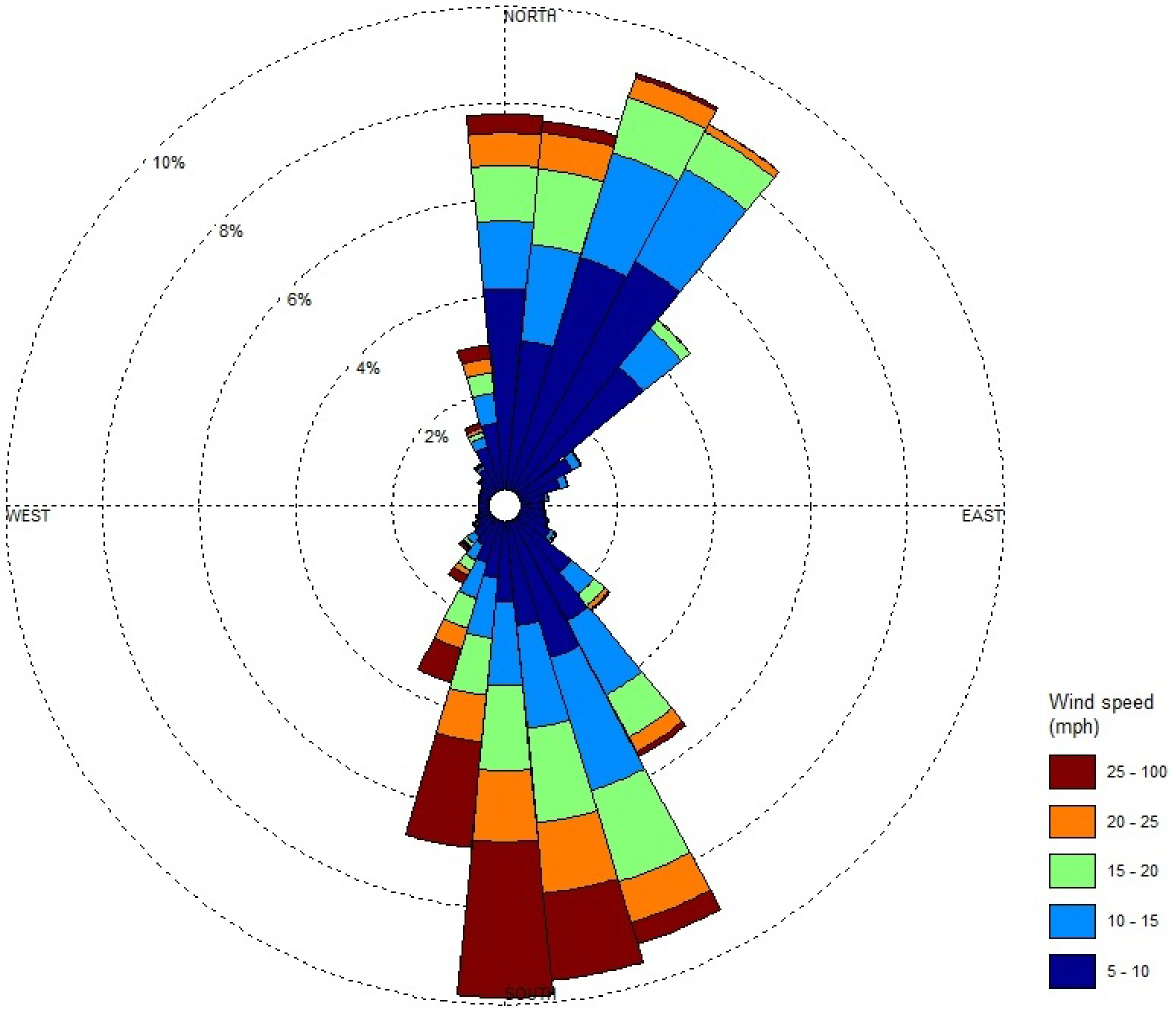

Figure D1-2
West Point, Washington, Wind Speed Distribution (1984 to 2009)

There is also a Coast Guard station closer to the Site at Point No Point Lighthouse (USAF #742065) with recorded wind data. Figure D1-3 shows the wind speed distribution for the station at Point No Point, for reference to compare to the West Point station wind distribution in Figure D1-2. A notable difference between the Point No Point Lighthouse and the West Point data is the prominent northwest wind measured at the Point No Point station. Although these data are different between the two stations, winds from the northwest direction will not generate waves that impact Reaches 1 through 4. Waves generated by winds from the northwest will impact the north shore of Port Gamble and the north side of the rock jetty. The Point No Point station only has a 5-year record of data.

Thus, considering the 25-year record of data from the West Point station, and considering that the distribution of winds (with the exception of the northwest direction) is similar between both stations, the West Point station was considered more appropriate for design purposes because of the greater statistically reliability associated with making predictions based on the longer wind record.

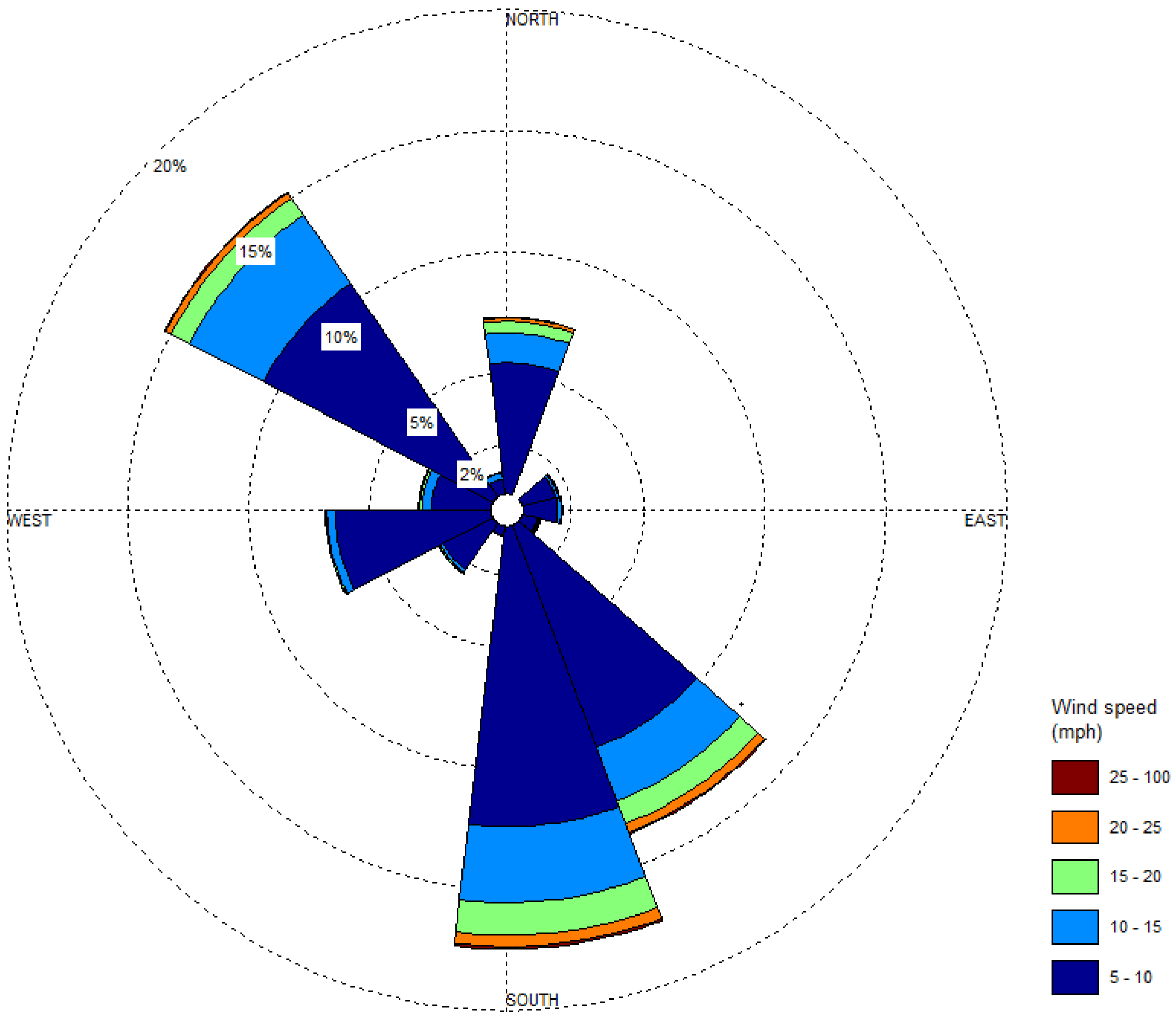

Figure D1-3
Point No Point Lighthouse Wind Speed Distribution (1985 to 1990)

Predicted values of extreme wind speeds from station WPOW1 were used as input into the Automated Coastal Engineering System (ACES) using the Windspeed Adjustment and Wave Growth module (fetch limited) to predict significant wave heights and peak wave periods generated by the extreme winds (USACE 1992). Results of the wave growth analysis are shown in Table D1-2. The highest winds and waves are from the north and south (as shown in Figure D1-2 and Table D1-2). During a 100-year storm from the north, waves are estimated to be 4.7 feet (impacting the rock jetty on the north side of shoreline Reach 1). The south side of the point (shoreline Reach 3) is estimated to have waves of 3.4 feet high during a 100-year storm. The east and west directions experience far less occurrence of high wind speeds (see Figure D1-2); however, waves from the east would be able to impact shoreline areas along Reaches 2 and 4. From the east, a 100 year storm is estimated to produce waves of 0.4 foot.

Table D1-2
Wind and Wave Data

Direction (degrees)	Average Depth (ft)	Fetch (miles)	2-year			10-year			20-year			50-year			100-year		
			Wind (mph)	Height (ft)	Period(s)	Wind (mph)	Height (ft)	Period(s)	Wind (mph)	Height (ft)	Period(s)	Wind (mph)	Height (ft)	Period(s)	Wind (mph)	Height (ft)	Period(s)
0-30	5	1.2	34	1.03	1.89	42	1.29	2.11	44	1.34	2.15	47	1.45	2.24	49	1.52	2.29
31-60	5	0.5	24	0.48	1.28	29	0.60	1.41	31	0.65	1.46	33	0.70	1.51	35	0.75	1.56
61-90	20	0.2	17	0.21	0.86	20	0.26	0.94	22	0.29	0.99	23	0.30	1.01	24	0.32	1.03
91-120	10	0.3	15	0.22	0.89	21	0.33	1.06	22	0.35	1.09	25	0.41	1.17	26	0.43	1.21
121-150	5	0.6	33	0.76	1.58	41	0.98	1.78	44	1.06	1.84	47	1.15	1.91	49	1.20	1.96
151-180	8	2.2	52	2.23	2.80	66	2.85	3.18	71	3.06	3.30	76	3.27	3.42	80	3.44	3.52
181-210	8	1.3	56	2.05	2.59	67	2.5	2.86	70	2.62	2.93	74	2.79	3.02	76	2.87	2.06
271-300	50	2.5	13	0.54	1.46	18	0.79	1.75	20	0.90	1.85	22	1.01	1.95	23	1.07	2.00
301-330	40	2.4	24	1.10	2.02	36	1.82	2.53	40	2.08	2.68	44	2.35	2.83	47	2.55	2.94
331-360	60	5.0	38	2.80	3.18	48	3.78	3.63	51	4.09	3.76	55	4.50	3.92	57	4.71	4.00

Notes:
Although these data were found to have high-velocity outliers in the 150 to 210 degree directions, these outliers were not removed from the data set for this analysis as a conservative assumption.

5 INTERTIDAL CAP ARMOR SIZE

The U.S. Army Corps of Engineers (USACE) ACES Rubble Mound Revetment Design module was used to estimate cap armor stone sizes, thicknesses, and gradation characteristics required, as well as runup estimates (USACE 1992). Table D1-3 provides the median (D_{50}) rock size that would be stable (limited to no damage) for the maximum given wave conditions in Table D1-2 for each shoreline reach (Figure D1-1) for several assumed different slopes (2H:1V, 3H:1V, and 6H:1V). The waves were assumed to impact the slope head-on. Reach 4 can be impacted head-on by waves from the east; however, waves from that direction are much smaller than waves from the southeast. Therefore, stabile rock sizes for Reach 4 based on waves from the southeast were also included in Table D1-3 to be conservative. Table D1-3 also provides the vertical runup height for each slope and shoreline reach. The vertical runup represents the expected maximum runup found using the Ahrens and Heimbaugh method (USACE 1992).

Table D1-3

Stable Armor Rock Size and Projected Vertical Wave Runup in Feet

Location	2H:1V Slope		3H:1V Slope		6H:1V Slope	
	D_{50}	Runup	D_{50}	Runup	D_{50}	Runup
20-year Storm						
Reaches 1 and 2[a]	0.2	1.1	0.2	0.8	0.1	0.5
Reach 3 and Reach 4[b]	0.9	4.9	0.7	3.8	0.5	2.3
Reach 4[c]	0.1	0.6	0.1	0.4	0.1	0.3
50-year Storm						
Reaches 1 and 2[a]	0.2	1.1	0.2	0.9	0.1	0.5
Reach 3 and Reach 4[b]	0.9	5.3	0.8	4.1	0.5	2.4
Reach 4[c]	0.2	0.7	0.1	0.5	0.1	0.3
100-year Storm						
Reaches 1 and 2[a]	0.2	1.2	0.2	0.9	0.1	0.5
Reach 3 and Reach 4[b]	1.0	5.6	0.8	4.3	0.6	2.5
Reach 4[c]	0.1	0.7	0.1	0.5	0.1	0.3

Notes:

a. Maximum wave direction from due north; calculated for head-on waves.

b. Maximum wave direction from the southeast; calculated for head-on waves. The large waves from the southeast impact Reach 4 at oblique angles to the shoreline. However, the effect of oblique wave approach on armor layer stability has not been quantified. Existing studies suggest that there is not significant impact for waves up to a 60 degree angle of approach (Allsop 1995).

c. Maximum wave direction from the east; calculated for head-on waves.

6 EXTENT OF INTERTIDAL CAP ARMOR

The intertidal cap armor should extend upslope to the vertical extent of wave runup based on the water level elevation at mean higher high water (MHHW) and downslope to a depth that is no longer impacted by the breaking waves at mean lower low water (MLLW). The highest runup elevation is found by adding the runup height (shown in Table D1-3) with the elevation of MHHW at the Site (shown in Table D1-1, 10.2 feet). The lower bound elevation of the armor is found by multiplying the significant wave height by 1.5 and subtracting that number for the MLLW elevation (USACE 2002). The upper and lower bounds of the armor are shown in Table D1-4.

The extent of intertidal armor in Table D1-4 refers to requirements to stabilize intertidal areas within the active surf zone due to breaking waves.

Table D1-4
Extents of Intertidal Armor, Elevations in feet MLLW

Location	2H:1V Slope		3H:1V Slope		6H:1V Slope	
	Upper Elevation	Lower Elevation	Upper Elevation	Lower Elevation	Upper Elevation	Lower Elevation
20-year Storm						
Reaches 1 and 2[a]	11.3	-1.0	11.0	-1.0	10.7	-1.0
Reach 3 and Reach 4[b]	15.1	-4.6	14.0	-4.6	12.5	-4.6
Reach 4[c]	10.8	-0.5	10.6	-0.5	10.5	-0.5
50-year Storm						
Reaches 1 and 2[a]	11.3	-1.1	11.1	-1.1	10.7	-1.1
Reach 3 and Reach 4[b]	15.5	-4.9	14.3	-4.9	12.6	-4.9
Reach 4[c]	10.9	-0.6	10.7	-0.6	10.5	-0.6
100-year Storm						
Reaches 1 and 2[a]	10.4	-1.1	11.1	-1.1	10.7	-1.1
Reach 3 and Reach 4[b]	11.2	-5.2	14.5	-5.2	12.7	-5.2
Reach 4[c]	10.3	-0.6	10.7	-0.6	10.5	-0.6

Notes:
a. Maximum wave direction from due north; calculated for head-on waves.
b. Maximum wave direction from the southeast; calculated for head-on waves. The large waves from the southeast impact Reach 4 at oblique angles to the shoreline. This was not taken into account in estimates of vertical run-up to be conservative.
c. Maximum wave direction from the east; calculated for head-on waves.

7 ADDITIONAL CAP DESIGN CONSIDERATIONS

7.1 Optimization of Armor Rock Size for 100-year Storm Event

Storm wave heights (Section 4) and associated stable armor rock sizes (Section 5) estimated as part of this coastal engineering evaluation utilized conservative assumptions. These assumptions included using the estimates of deep water wave conditions to evaluate armor rock size along the shoreline without taking into account the reduction in wave energy as the waves propagate into shallower water.

As part of coordinated evaluations of possible nearshore/shoreline restoration options at the former Port Gamble mill site (see Appendix M of this EDR), Anchor QEA developed a more detailed wave model for the project area that was used to inform separate design of proposed restoration options (Anchor QEA 2015). This model was used to simulate wave transformation from deep water to shallow water for the 100-year storm events from the north, south, and southeast wind directions at the Port Gamble site. This was performed to optimize the armor size along the shoreline, while ensuring shoreline stability.

Based on the results of these model simulations, wave heights along the shoreline are predicted to be smaller than the deep water wave heights estimated using the wind-wave hindcast. Maximum model predictions of nearshore wave heights were approximately 2.7 feet (compared to 3.5 feet maximum deep water wave heights from the wind-wave hindcast). Therefore armor rock sizes summarized in Section 5 for the steeper slopes (2H:1V and 3H:1V) for the 100-year storm event can be reduced to 10 inches for the 2H:1V and 9 inches for the 3H:1V (see Table D1-3) and still ensure sufficient long-term shoreline protection.

7.2 Impacts due to Predicted Sea Level Rise

SLR estimates for the project area were taken from the National Resource Council Report *Sea Level Rise for the Coasts of California, Oregon, and Washington* (NRC 2012). Mid-range SLR estimates from this report are 17 cm (0.5 foot) by 2050 and 62 cm (2 feet) by 2100.

SLR estimates for 2050 would increase the MHHW elevation by approximately 0.5 foot compared to current conditions. This would have a negligible impact on predicted wave heights at the site; however, it could increase the vertical extent of wave run-up and amount of overtopping for the armored slope at the shoreline. The increase in wave run-up and overtopping based on the 0.5 foot of SLR is not anticipated to result in damage to the armored slope or changes to the slope design.

SLR estimates for 2100 would increase the MHHW elevation by approximately 2 feet compared to current conditions. A 2-foot increase in mean sea levels could result in slightly higher nearshore waves along the project shorelines. In addition, increases in wave run-up and overtopping could be significant, and may require the crest elevation of the armored slope to be increased to prevent overwash damage. As discussed in the *Operations, Maintenance, and Monitoring Plan* (Appendix F of this EDR), the performance of the armored slope, as well as revisions to SLR estimates, will be monitored periodically over the design life of the structure to ensure its stability.

8 REFERENCES

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