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FEMA Region IX California Coastal Analysis and Mapping Project / Open Pacific Coast Study Intermediate Data Submittal #3: Nearshore Hydraulics Humboldt County, California

BakerAECOM

FEMA Region IX

California Coastal Analysis and Mapping Project / Open Pacific Coast Study

Intermediate Data Submittal #3:

Nearshore Hydraulics

Humboldt County, California

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1 Introduction

The Federal Emergency Management Agency (FEMA) in Region IX is conducting a coastal engineering study of the Open Pacific Coast (OPC) of California, which includes detailed modeling and analyses of coastal hazards, as part of the California Coastal Analysis and Mapping Project (CCAMP). Phase 1 was initiated in 2010 and includes the northern California counties of Del Norte, Humboldt, Mendocino, Sonoma, and Marin, as well as the central California counties of San Francisco, San Mateo, Santa Cruz, Monterey, and San Luis Obispo. Phase 2 includes the southern California counties of Santa Barbara, Ventura, Los Angeles, Orange, and San Diego and was initiated in late 2011. The results from this study will be used to re-map the coastal flood hazards in all coastal California communities. Analyses and mapping are being conducted in accordance with the Final Draft *Guidelines for Coastal Flood Hazard Analysis and Mapping for the Pacific Coast of the United States* (FEMA, 2005), hereafter referred to as the Pacific Guidelines, as applicable; any deviations from this guidance require approval by the FEMA Study Manager prior to implementation. Final deliverables will include revised Flood Insurance Rate Map (FIRM) panels, to be issued as a Physical Map Revision (PMR), and an updated Flood Insurance Study (FIS) report.

The CCAMP OPC Study is being documented in four Intermediate Data Submittal (IDS) reports. IDS #1 – Scoping and Data Review described the study area, data sources, methodology for analysis, field reconnaissance investigations, and transect layout map (BakerAECOM, 2012a). IDS #2 – Offshore Water Levels and Waves described the primary analyses of water level and wave conditions to be applied during the detailed analysis in the nearshore hydraulics phase (BakerAECOM, 2013a). This report is IDS #3 – Nearshore Hydraulics for Humboldt County. It documents the one-dimensional transect-based analyses conducted to develop the base flood conditions at the shoreline that will inform the mapping. A thorough discussion of the methods employed and the results of the analyses of coastal flood hazards to the open Pacific coastline of Humboldt County is included. An account of the methods and results of the analysis of coastal flood hazards to the sheltered waters within Humboldt-Arcata Bay is also included. IDS #4 – Hazard Mapping will describe the use of the coastal analysis results to identify and delineate flood hazard zones.

Humboldt County (Figure 1) has approximately 110 miles of coastline exposed to the Pacific Ocean, from the border with Mendocino County at the southern end of the study area to the border with Del Norte County in the north.

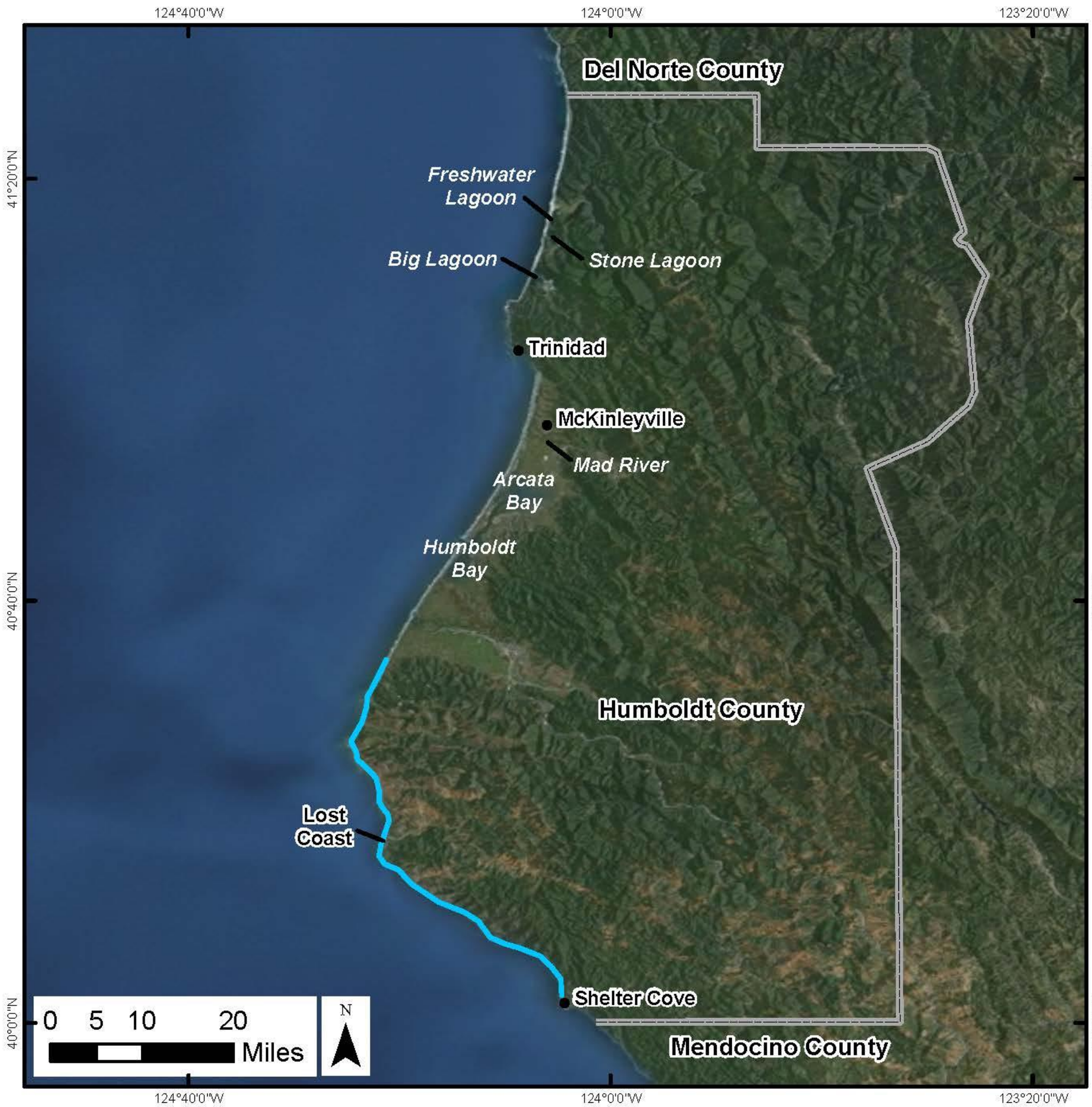

Figure 1: Humboldt County Overview Map

2 Overview of Technical Approach

The detailed nearshore hydraulics coastal analysis conducted along the Humboldt County coastline included the following general tasks:

- Construction of elevation profiles along one-dimensional wave analysis transects placed with regard to coastal topography and bathymetry, shoreline orientation and exposure, land use and development, and incident wave conditions

- Adjustment of beach profiles to model episodic beach and dune erosion
- Evaluation of coastal structures and their effects on coastal erosion and flooding
- Calculation of wave setup and runup along wave analysis transects to create a response-based 50-year hindcast of total water levels (TWLs) along the California coast
- Extreme value analysis of the annual maxima of computed TWLs to determine the Base (1-percent-annual-chance) Flood Elevation (BFE) and the 50-percent-, 20-percent-, 10-percent-, 4-percent-, 2-percent-, and 0.2-percent-annual-chance flood elevations
- Calculation of wave overtopping of natural and engineered backshore features
- Wave analysis in sheltered embayments, where applicable
- Backshore analyses for overland wave propagation, where applicable
- Identification of the Primary Frontal Dune (PFD), where applicable
- Calculation of coastal flood hazard zones and BFEs

These tasks represent the surf zone processes presented in the summary flowchart in Figure 2, modified from the Pacific Guidelines for the current study process. The study approach and results for the offshore zone and shoaling zone processes were presented previously in IDS #1 and IDS #2, respectively. It should be noted that it is beyond the scope of this study to conduct combined riverine and coastal flood probability analysis.

Because no single discrete mechanism is responsible for the 1-percent-annual-chance storm on the Pacific coast, an event-based analysis, such as that used on the Atlantic and Gulf coasts to analyze hurricane impacts, is likely to produce an oversimplified assessment of the coastal hazards. Instead, a response-based approach is being used to assess the combined effects of all of the physical processes and to statistically determine the 1- percent-annual-chance TWLs. This approach considers the effects of distant swell events, locally generated storm waves, nearshore tidal variations, and elevated water levels due to El Niño effects. The flood hazard response at the shoreline to each occurrence of these simultaneous physical processes is recreated from wave and tide hindcasts at hourly intervals from 1960 through 2009. The entire 50-year time series of wave, tide, and shoreline interactions is analyzed and the 1- percent-annual-chance TWL is calculated statistically from those results to establish BFEs and inform the mapping.

It is important to note that in the sheltered embayments in which wave hazards were studied, an event-based approach was used to evaluate wave runup and overland wave propagation because the 50-year hindcast information does not extend into the embayments (Section 4.7).

A more detailed discussion of the technical approach is presented in Section 4.

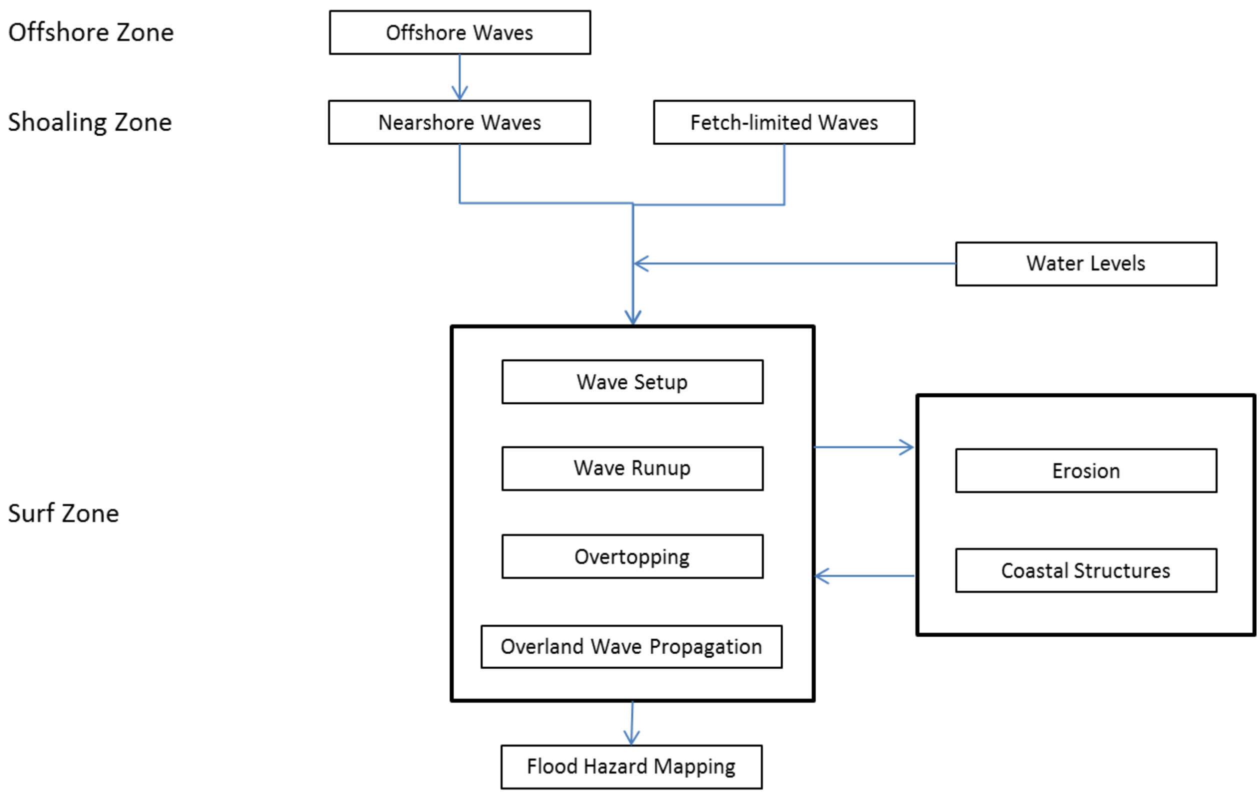

**Figure 2: Summary of Technical Approach
Adapted from the Pacific Guidelines (FEMA, 2005)**

Coastal flooding hazards were evaluated using one-dimensional transect-based methods. Wave setup, runup, and overtopping; episodic erosion; and overland wave propagation were analyzed, as appropriate, for each transect along the Humboldt County coastline (Appendix A). Shore-perpendicular transects were placed with consideration of variations in topography, shoreline type, development density, land use, and incident wave conditions. Wave parameters used for input conditions to the transect-based analysis were obtained from the Scripps Institution of Oceanography (SIO) linear spectral propagation model, which incorporates the refraction and shoaling of offshore waves to the nearshore environment. However, the SIO model was not designed to transform the waves at the discretization necessary to resolve surf zone dynamics, including wave breaking and the generation of wave setup, so the one-dimensional transects were utilized to transform the waves through the surf zone. The SIO SHELF model input points assigned to each one-dimensional transect are provided in Table 1.

Wave runup was calculated along transects using one of three methods, depending on shoreline characteristics. As recommended in FEMA's Pacific Guidelines, the Direct Integration Method (DIM) was used to calculate runup for transects with natural, gently sloping ($m < 0.125$) profiles. The Technical

Advisory Committee for Water Retaining Structures (TAW) (van der Meer 2002) method was used for shorelines with shore protection structures and steeply sloping ($m \geq 0.125$) natural shorelines. The U.S. Army Corps of Engineers Shore Protection Manual (SPM) method (USACE 1984) was used to calculate wave runup on vertical walls. The total runup elevation is also referred to as the TWL. Profiles for transects traversing sandy dunes were adjusted to account for episodic erosion. Transects bisecting engineered coastal structures were also assessed and modified as necessary for analysis (Section 4.5). Annual TWL maxima were selected from the 50 years (1960-2009, inclusive) of hindcast data, and the generalized extreme value (GEV) distribution was employed to determine the 1-percent-annual-chance TWL from the annual maxima at each transect. Wave overtopping was evaluated for transects where the runup elevation exceeded the structure or bluff crest. The wave hazards evaluated and the methods used for each transect are presented in Table 1.

Table 1: Wave Hazard Analyses by Transect

Transect Number[1]	Shore Type	SIO SHELF Input	Runup Method	Erosion	Sheltered	Overtopping	WHAFIS[2]
1	Sandy Beach / Dune / Bluff	HU968	DIM	YES	-	-	-
2	Sandy Beach / Dune / Bluff	HU942	DIM	YES	-	-	-
3	Bluff	HU919	DIM/TAW	-	-	-	-
4	Dune	HU895	DIM	YES	-	-	-
5	Sandy Beach with Roadway	HU888	DIM/TAW	YES	-	-	-
6	Dune	HU874	DIM	YES	-	-	-
7	Bluff	HU865	DIM	-	-	-	-
8	Dune	HU838	DIM	YES	-	-	-
9	Bluff	HU821	DIM	-	-	-	-
10	Bluff	HU817	DIM	-	-	-	-
11	Bluff	HU785	DIM/TAW	-	-	-	-
12	Bluff	HU756	DIM/TAW	-	-	-	-
13	Bluff	HU747	DIM/TAW	-	-	-	-
14	Bluff	HU728	DIM/TAW	-	-	-	-
15	Bluff	HU728	TAW	-	-	-	-
16	Bluff	HU708	TAW	-	-	-	-
17	Dune	HU694	DIM	YES	-	-	-
18	Dune / Bluff	HU679	DIM	YES	-	-	-
19	Sandy Beach / Bluff	HU670	DIM/TAW	-	-	-	-

Transect Number[1]	Shore Type	SIO SHELF Input	Runup Method	Erosion	Sheltered	Overtopping	WHAFIS[2]
20	Sandy Beach	HU660	DIM	-	-	YES	-
21	Dune	HU640	DIM/TAW	YES	-	-	-
22	Dune	HU601	DIM	YES	-	-	-
23	Dune	HU580	DIM/TAW	YES	-	-	-
24	Dune	HU557	DIM/TAW	YES	-	YES	-
25	Dune	HU520	DIM	YES	-	-	-
26	Dune	HU507	DIM	YES	-	-	-
27	Dune	HU480	DIM	YES	-	-	-
28	Sandy Beach	HU457	DIM/TAW	-	-	YES	-
29	Dune	HU439	DIM	YES	-	-	-
30	Dune	HU428	DIM	YES	-	-	-
31	Bluff	HU414	TAW	-	-	-	-
32	Bluff	HU400	DIM	-	-	-	-
33	Bluff	HU369	DIM/TAW	-	-	-	-
34	Bluff	HU359	DIM	-	-	-	-
35	Bluff	HU351	DIM	-	-	-	-
36	Bluff	HU340	DIM/TAW	-	-	-	-
37	Bluff	HU330	DIM	-	-	-	-
38	Revetment	HU319	TAW	-	-	YES	-
39	Dune/Bluff	HU289	DIM	YES	-	-	-
40	Bluff	HU272	DIM	-	-	-	-
41	Bluff	HU254	DIM	-	-	-	-
42	Dune	HU246	DIM	YES	-	-	-
43	Bluff	HU230	DIM	-	-	-	-
44	Bluff	HU220	DIM	-	-	-	-
45	Bluff	HU198	DIM/TAW	-	-	-	-
46	Bluff	HU160	DIM	-	-	-	-
47	Bluff	HU136	DIM	-	-	-	-
48	Bluff	HU118	DIM/TAW	-	-	-	-
49	Bluff	HU104	DIM/TAW	-	-	-	-
50	Bluff	HU095	DIM/TAW	-	-	-	-

Transect Number[1]	Shore Type	SIO SHELF Input	Runup Method	Erosion	Sheltered	Overtopping	WHAFIS[2]
51	Bluff	HU083	DIM	-	-	-	-
52	Bluff	HU155	DIM	-	-	-	-
53	Bluff	HU047	DIM	-	-	-	-
54	Bluff	HU043	DIM	-	-	-	-
55	Bluff	HU041	TAW	-	-	-	-
56	Bluff	HU036	DIM/TAW	-	-	-	-
57	Bluff	HU031	TAW	-	-	YES	-
58	Bluff	HU027	DIM/TAW	-	-	-	-
59	Bluff	HU026	TAW	-	-	-	-
60	Bluff	HU014	DIM	-	-	-	-
61	Bluff	HU007	DIM/TAW	-	-	-	-
62	Beach	-	-	-	YES	-	YES
63	Mudflats	-	TAW	-	YES	-	-
64	Marsh / Earthen Bern	-	-	-	YES	-	YES
65	Beach / Earthen Berm	-	TAW	-	YES	YES	-
66	Revetment	-	TAW	-	YES	-	-
67	Mudflat	-	DIM	-	YES	YES	-
68	Earthen Berm	-	TAW	-	YES	-	-
69	Marsh	-	DIM	-	YES	-	-
70	Bluff	-	TAW	-	YES	YES	-
71	Revetment	-	TAW	-	YES	-	-
72	Vertical Wall	-	SPM	-	YES	YES	-
73	Revetment	-	TAW	-	YES	YES	-
74	Beach	-	TAW	-	YES	YES	-
75	Revetment	-	TAW	-	YES	YES	-
76	Revetment	-	TAW	-	YES	-	YES
77	Dune	-	DIM/TAW	-	YES	-	-
78	Beach	-	-	-	YES	-	YES
79	Beach / Rip Rap	-	TAW	-	YES	YES	-
80	Mudflat / Earthen Berm	-	-	-	YES	-	YES

Transect Number[1]	Shore Type	SIO SHELF Input	Runup Method	Erosion	Sheltered	Overtopping	WHAFIS[2]
81	Mudflat / Bluff	-	TAW	-	YES	-	-

[1]IDS#1 describes methods used to classify shore types.
[2]WHAFIS is Wave Height Analysis for Flood Insurance Studies and is used to analyze overland wave propagation

Overland wave propagation modeling, using the FEMA Wave Height Analysis for Flood Insurance Studies (WHAFIS) model, Version 4 (FEMA, 1988; Divoky, 2007), was performed for transects with gently sloping profiles where the prevailing ground is inundated by the stillwater elevation (SWEL) and setup only. WHAFIS solves the wave action conservation equation and incorporates wind-generated wave growth and dissipation by marsh grasses. Rigid blockages to wave growth, such as buildings or rigid vegetation, are also included within the formulations.

Humboldt-Arcata Bay is the only sheltered embayment on the coast of Humboldt County for which coastal wave hazards were evaluated in this study. This water body is considered a sheltered embayment for the purposes of this study because it is largely protected from exposure to Pacific Ocean swell waves. Despite being sheltered from the open coast, it may still have wave-driven flood hazards as a result of wave growth driven by local winds. Flood hazards, including those from locally generated wind waves, were evaluated for Humboldt-Arcata Bay.

A one-dimensional, event-based approach based upon the Guidance for Coastal Flood Hazard Analyses and Mapping in Sheltered Waters (FEMA, 2008) was used to evaluate coastal flood hazards within the sheltered water bodies. For each transect, a representative wave condition was paired with the 1-percent-annual-chance water level. Starting wave conditions were generated using parametric wave growth equations and a representative wind speed. Static wave setup was calculated using the DIM. Wave energy propagating through the entrance channels into these sheltered embayments was considered for the areas inside the inlet, notably the community of King Salmon. Wave runup, overtopping, and overland wave propagation were calculated as appropriate.

3 Data Sources

Data sources were thoroughly described in IDS #1. For ease of reference, a brief summary of the data used for the nearshore analyses is included in this section.

3.1 Terrain Data

A digital ground elevation terrain was built from the best available sources of topographic and bathymetric data. The topographic datasets are remotely sensed, high-resolution elevation data acquired by an airborne collection platform using Light Detection and Ranging (LiDAR) and were obtained from the California Ocean Protection Council (COPC) and the U.S. Geological Survey (USGS). All of the

topographic data were collected between 2009 and 2011. Bathymetric datasets were obtained from the National Oceanic and Atmospheric Administration (NOAA). BakerAECOM merged these datasets (BakerAECOM, 2012b) to produce a seamless surface for use in the analysis of coastal hazards. The various datasets, complete metadata, and documentation have been submitted to FEMA as part of the topographic data development deliverable and meet FEMA standards for leveraged data. Details regarding data merging and prioritization can be found in the IDS #1 report.

The USACE San Francisco District Coastal Structures Program's Hydrographic and Aerial Surveys of the Humboldt Jetties and Entrance Channel Survey, dated September 2009, was used to supplement the terrain at the entrance to Humboldt-Arcata Bay. This dataset was used to verify elevations within the Humboldt Bay Entrance Channel used in the swell propagation analysis through the Humboldt jetties.

3.2 Wave Hindcast Data

To determine coastal hazards within the study area, impacts are computed as functions of both waves and water elevations. Wave impacts on the open coast of California are especially important due to the high-energy winter storms and the historical damage they have caused. Measured wave data are both geographically and temporally sparse, and numerical modeling is a useful way to fill both temporal and spatial gaps in data coverage. For the open coast of California, the most important waves for a hazard assessment are long-period swells. These waves are typically generated by storm systems far offshore and would not be captured by a simple wave model that only considered local wind conditions. For this project, a suite of numerical models was used to model wave growth and propagation at varying scales, eventually bringing them into the nearshore environment. This section provides a brief background regarding the development of the offshore and nearshore wave hindcast data that was used to analyze the coastal hazards for this study.

3.2.1 Deepwater Wave Modeling

The offshore deepwater wave climate modeling effort conducted by Oceanweather, Inc. (OWI) developed a 50-year hindcast from January 1, 1960, through December 31, 2009, that provided boundary wave spectra to drive the shelf-scale wave transformation model developed by the University of California SIO.

OWI utilizes a version of the WAve prediction Model called UNIWAVE to model wave conditions throughout the world. This effort used the Global Reanalysis of Ocean Waves (GROW) model, which incorporates numerous wind sources and has been extensively tested. OWI also used a more finely resolved regional GROWFine: NorthEast Pacific (NEPAC) model. A third project-specific coastal scale grid (COASTAL) was developed for this study to provide increased resolution in the wave climate in the northeastern Pacific basin. All models were nested, such that boundary spectra were passed from GROW to NEPAC to COASTAL. OWI used lists of historically significant storm events for an extensive reanalysis of the wind fields and testing of the wave output. These results were compared against data from National Buoy Data Center (NBDC) and Coastal Data Information Program buoys located along the

northern California coastline for model validation. Details of the modeling effort, process, and results can be found in the report, California Pacific Coastal Studies (Northern and Central Coastal Counties): High Resolution Deep Water Wave Climate Forcing Development, (Oceanweather, Inc., 2012) which is presented as part of IDS #2.

3.2.2 Nearshore Wave Transformation

The nearshore transformation modeling was conducted by SIO to provide boundary conditions for the surf zone transformations to determine runup, overtopping, and overland wave propagation, as appropriate. SIO utilized a modified version of the MOnitoring and Prediction modeling system, called the SHELF model, that has been validated using a network of buoys off the California coast for this study. The information from these buoys was used to drive a high spatial resolution linear spectral propagation model, which incorporates refraction, shoaling, and island blocking. SHELF was modified to accept inputs extracted from the OWI project-specific model along the coastline.

The modified SHELF hindcast produces hourly spectra and bulk parameters at each of the SHELF data transfer points. Bulk parameters include the significant wave height, H_{m0}, peak period, T_p, spectral moments, and spectral shape parameters. Details of the model development, implementation, and validation can be found in the SIO report, "*Northern and Central California Coastal Hindcast Methodology*" which was submitted as part of IDS #2.

Nearshore data extraction points are located at a depth of approximately 15 meters (49.2 feet), which is assumed to be seaward of the surf zone under most conditions. The SHELF data transfer point which provided the wave inputs to each transect is noted in Table 1 and in the results in Section 5.1.

3.3 Tidal Elevation Data

BakerAECOM conducted the offshore water levels analysis to provide continuous 50-year hourly hindcasts of stillwater[1] levels (SWL) for the period of January 1, 1960 through December 31, 2009, at each of the long-term tide stations along the California coastline. These were used for input into the response-based one-dimensional transect-based wave hazard analysis. A regional frequency analysis of annual maxima recorded at the long-term tide stations was also performed to determine various statistical return period SWELs. This section provides an overview of the methodologies applied and results of these analyses, including the determination of SWELs in sheltered embayments. Detailed documentation of the methodologies applied for this effort is provided in the IDS #2 report.

[1] In this report, the stillwater level (SWL) is the time-varying offshore water level in the absence of wave effects and should not be confused with the stillwater elevation (SWEL) which refers to a statistically determined flood elevation, such as the 1-percent-annual-chance SWEL.

3.3.1 Stillwater Level Hindcast Reconstruction

Tidal elevation data for tide stations along the California coast were obtained from the NOAA National Ocean Service (NOS). However, the existing tide gauge records along the coast provide an incomplete record, both spatially and temporally. The gaps needed to be filled to obtain reasonable and continuous SWLs as inputs for response-based onshore analyses. Temporal gaps in the records were filled using an approach that applied the relationships of observed tidal residuals between neighboring gauges to estimate residual components at stations with missing data. Using these correlations and an understanding of the spatial variability of regional storms, the gaps in gauge records were empirically filled to provide a continuous hourly time series of SWLs for the desired period of record at each tide gauge. Details of this process and data sources were submitted as part of IDS #2, which detailed offshore water levels and waves.

Once the hourly SWL hindcast was reconstructed for each tide gauge, each tide gauge was assigned the coastal reach for which it was calculated to be most representative of the SWLs. The water level record from the representative tide gauge was used for each one-dimensional transect within the spatially defined reach. The Humboldt County tide gauge assignments for spatial application of SWL hindcast results are presented in Table 2 and shown in Figure 3.

Table 2: Spatial Application of Tide Gauge Analyses Results

Coastal Reach	Tide Gauge Station	Length of Record
Del Norte County border to Patrick's Point	Crescent City	1970-present
Patrick's Point to Punta Gorda	Humboldt Bay, North Spit	1977-present
Punta Gorda to Mendocino County border	Shelter Cove with residuals transferred from Arena Cove	None*

*No observed data obtained; only tide predictions were used

Note that the Crescent City tide gauge is located in Del Norte County so is not visible in Figure 3.

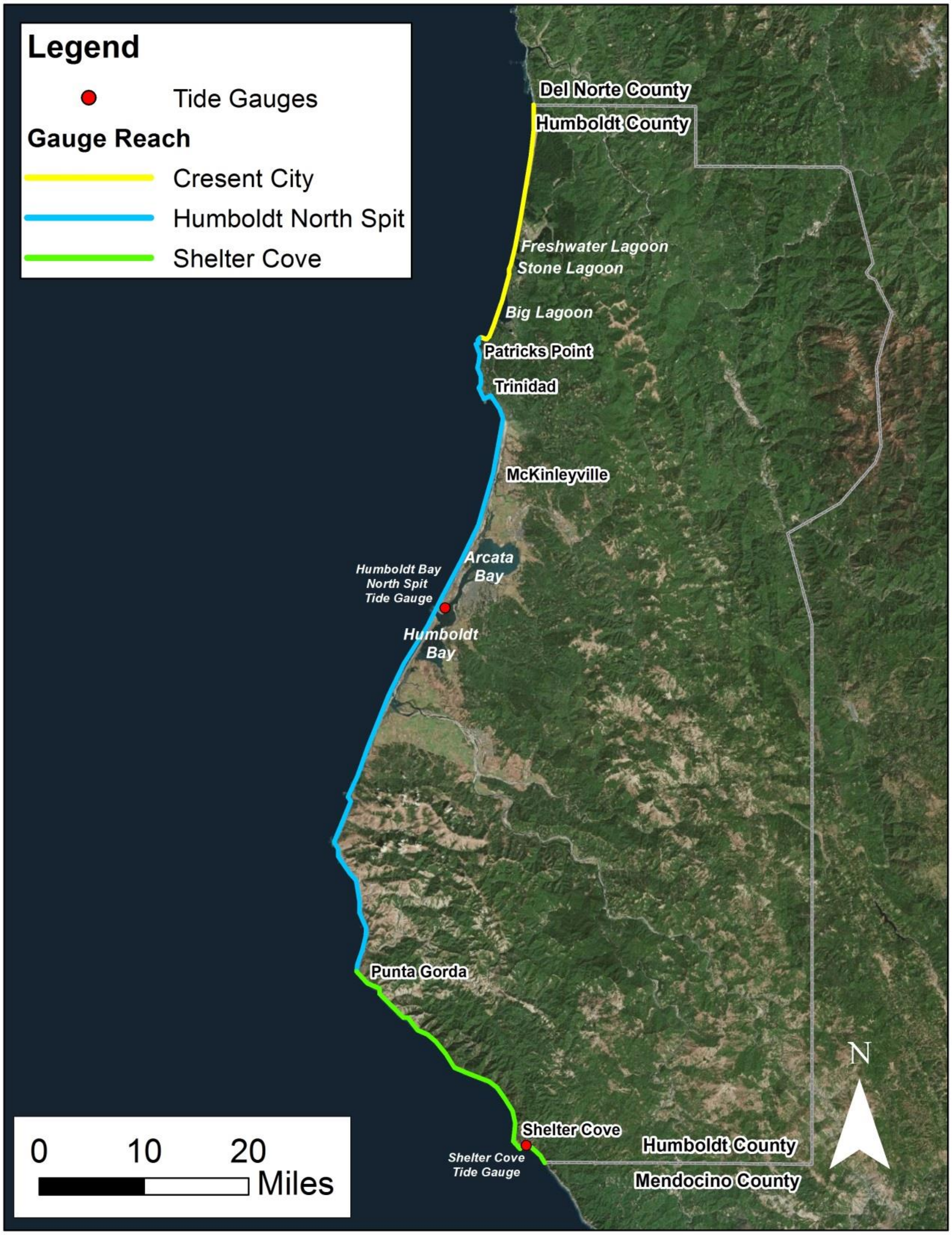
Legend
Tide Gauges
Gauge Reach
Cresent City
Humboldt North Spit
Shelter Cove
Del Norte County
Humboldt County
Freshwater Lagoon
Stone Lagoon
Big Lagoon
Patricks Point
Trinidad
McKinleyville
Arcata Bay
Humboldt Bay North Spit Tide Gauge
Humboldt Bay
Punta Gorda
Shelter Cove
Shelter Cove Tide Gauge
Humboldt County
Mendocino County
N
0 10 20
Miles

Figure 3: Coastal Reach Assignment for Spatial Application of Tide Gauge Analyses Results

3.3.2 Tide Frequency Analyses

Annual maxima from tide gauges along the California coastline that had sufficient lengths of observed records were used in the determination of statistical SWELs. The regional L-Moments method was employed to conduct the frequency analysis of the tide gauge data. This approach involved fitting various frequency distributions using the method of L-Moments (Hosking, 1996; Hosking and Wallis, 1997) and using statistical tests to determine "homogenous" regions as well as the best frequency distribution to fit the tide gauge data. This approach assumes that the environmental response variable of interest within a homogenous region is produced by common climatological or meteorological forcing functions, such as El Niños and coastal storms, each having the same regional probability distribution. Details of this process and data sources were submitted as part of IDS #2, which detailed the offshore water levels and waves. Results of the SWEL frequency analysis are presented in Table 3 and apply to the reaches described in Table 2 for the open coast.

The 50-year hindcast from the Humboldt Bay North Spit tide gauge was used to conduct the SWEL frequency analyses for Humboldt-Arcata Bay. Although several subordinate stations with short periods of record are present within Humboldt-Arcata Bay, the North Spit gauge data was considered to be more accurate than the limited data available at the subordinate stations. Details of the methods used to calculate the SWELs for sheltered embayments and results were described in IDS #2. The SWELs for Humboldt-Arcata Bay are included in Table 3.

Table 3: Summary of Regional SWELs at Tide Gauges

Tide Gauge/ Embayment	Regional 50-percent (feet, NAVD)	Regional 20-percent (feet, NAVD)	Regional 10-percent (feet, NAVD)	Regional 4-percent (feet, NAVD)	Regional 2-percent (feet, NAVD)	Regional 1-percent (feet, NAVD)	Regional 0.2-percent (feet, NAVD)
Crescent City	8.8	9.2	9.5	9.9	10.2	10.5	11.3
Humboldt Bay, North Spit*	8.5	8.9	9.2	9.6	9.96	10.2	11.0
Shelter Cove	7.8	8.2	8.4	8.8	9.0	9.3	10.1

*The regional frequency analyses at Humboldt Bay, North Spit were applied for the 50-, 20-, 10-, 4-, 2-, 1-, and 0.2-percent-annual-chance SWELs within Humboldt-Arcata Bay. Subordinate tide gauges within Humboldt-Arcata Bay were considered to be less accurate than direct use of the North Spit tide gauge throughout the Bay. Additional details are provided in IDS #2

3.3.3 Tidal Datums

The tidal datums used in the Humboldt County study area are included in Table 4 for reference. These values are reported directly from the NOAA data inventory (http://tidesandcurrents.noaa.gov). Mean Sea

Level (MSL) is used as the baseline for overland wave analysis, and Mean High Water (MHW) and Mean Low Water (MLW) are used in the erosion analysis, as described later in this report.

Table 4: Tidal Datums for the Study Area

Gauge Name	Station ID	Latitude (deg N)	Longitude (deg W)	Tidal Datums referenced to NAVD88 (feet)				
				MHHW	MHW	MSL	MLW	MLLW
Crescent City	9419750	41.7462	121.1833	6.49	5.85	3.33	0.86	-0.38
Humboldt Bay, North Spit	9418767	40.7667	124.2167	6.52	5.81	3.37	0.92	-0.33
Shelter Cove (Arena Cove)*	9418024 (9416841)	38.9133	123.7067	5.74	5.07	3.01	1.03	-0.13

*residuals transferred from Arena Cove to Shelter Cove

3.4 Wind Data

Wind data input is required for overland wave propagation, fetch-limited wave growth analyses in embayments, and splash overtopping analyses. Wind data were acquired from several sources, including the NOAA National Climate Data Center (NCDC), the NOAA NDBC, and the Weather Underground (WU). The NOAA NCDC provides 2-minute averaged wind speed and direction with gusts at 5-minute intervals, available as hourly observations for stations owned and maintained by NCDC, and it provides meteorological data at various sampling frequencies for stations owned and maintained by the United States Air Force (USAF). The NOAA NDBC provides offshore wind speeds and directions averaged over 8-minute intervals, available as hourly observations for stations owned and maintained by the NDBC, and it provides offshore wind speeds and directions averaged over 2-minute intervals, available as 6-minute observations for stations owned and maintained by NOS. The Weather Underground (WU) provides meteorological data at various sampling frequencies for airports and weather observation stations across the United States. Wind stations specific to Humboldt County are compiled in Table 5 and were reviewed to develop an appropriate wind speed to use in the coastal analyses for this study. Stations included are NCDC Station 725848, offshore of Shelter Cove; NDBC Station 46022, 18 miles west of Humboldt Bay; Station KACV located at the Arcata Airport; Station KEKA at Murray Field Airport in Eureka; NCDC Station 725947 on the western coast of the north spit; and NOS Station 9418767, also referred to as Station HBYC1, located on the north spit near Humboldt-Arcata Bay (Figure 4). Details regarding the selection of a representative wind speed are documented in Section 4.7.1 of this report.

Table 5: County-specific Wind Station Information

Station ID	Station Name	Lat.	Lon.	Elevation (feet)	Period of Record (years)	Source/ Owned By
KACV	Arcata/Eureka	40.983	-124.100	217	63	WU
KEKA	Eureka/Murray	40.800	-124.167	59	63	WU

Station ID	Station Name	Lat.	Lon.	Elevation (feet)	Period of Record (years)	Source/ Owned By
46022	Eureka	40.724	-124.578	16.4	31	NDBC
46030	Blunts Reef	40.423	-124.525	16.4	17	NDBC
9418767/ HBYC1	North Spit	40.767	-124.217	6.6	3	NDBC/NOS
725947	Samoa/Humboldt Bay	40.767	-124.233	6.6	21	NCDC/USAF
725848	Shelter Cove	40.017	-124.067	68.9	34	NCDC/USAF

Figure 4: Wind Station Location for Humboldt County Analysis

3.5 Land Use Data

Land use in the sheltered embayments, where overland wave analysis was conducted, was determined through visual inspection of detailed aerial imagery and from data and photos collected on field visits to the study site. Areas vulnerable to flooding could be assessed with relative ease to confirm land use determinations based on aerial photography. ESRI's World Imagery data layer was used as the primary aerial imagery source. This data layer is updated twice per year and provides a seamless, color mosaic

NASA Blue Marble: Next Generation 500m resolution imagery at small scales (above 1:1,000,000). Orthoimagery collected as part of the terrain data collection effort in 2010 to 2011, as well as imagery available through the California Coastal Records Project at www.californiacoastline.org (Adelman and Adelman, 2010), was used to supplement the ESRI imagery to assign land use classifications where necessary for analysis.

4 Coastal Flood Hazard Analysis

The technical methodology employed in the coastal engineering analyses to assess coastal flood hazards to the county was developed in accordance with the Pacific Guidelines and applicable procedure memoranda issued by FEMA. The following sections outline the methods used to conduct coastal flood hazard analyses along the coast of Humboldt County.

To adequately capture the flood hazards and resolve transitions between flood hazard zones in the mapping phase, transects were placed with consideration of variations in topography, bathymetry, shoreline type, shoreline exposure, density of development and proximity of structures to the flooding source, land use, and incident wave conditions. Specifics regarding transect placement were provided in IDS #1.

Ground elevations along each transect were derived from the bathymetric and topographic terrain dataset used in this study. Nearshore and structure slopes were selected for use in the engineering analyses from these profiles. Structure geometry, including crest and toe locations and elevations, were also obtained from these profiles. Structure characteristics were determined from field observations and aerial imagery, where appropriate. The final transect layout for Humboldt County is included as GIS shapefiles with this submittal and graphics are included in Appendix A.

Once transect elevation profiles were extracted from the terrains, SWLs were calculated, and waves were characterized, the one-dimensional onshore coastal analysis could be conducted along each transect. For each hour over the 50-year wave and water level hindcasts, SWLs were coupled with wave conditions provided by the SHELF (provided that the waves were propagating onshore within 80 degrees of the transect orientation), and these waves were transformed through the shoaling zone to determine the response at the shoreline for each set of input conditions. The methodology employed in the surf zone and backshore analysis was consistent with the Pacific Guidelines and included the following general steps: calculation of setup, runup, erosion of the beach profile, overtopping, and, where applicable, overland wave propagation.

4.1 Transect Profile Analysis

Transect profiles were extracted from the GIS terrain; station and elevation pairs were plotted, and specific points of interest (such as toe, crest, etc.) were identified for input to the wave setup, runup,

erosion, and overtopping analyses. Profiles were inspected at a 1:1 to 3:1 data aspect ratio, and point locations were identified in conjunction with the visual inspection of the GIS terrain and aerial imagery. For all transects, the crest, toe, face edge, and a pair of overtopping points were selected. The crest was defined as the peak of a dune, bluff, or structure and was used to determine if overtopping might occur at a given transect. The toe location was defined based on a break in profile slope at the beach/dune junction, at the beach/bluff junction, or at the toe of a structure. The toe location was used with the face edge point to define the face slope and to determine the depth at the toe for depth-limited wave breaking. The face edge point was selected at the top edge of the structure, bluff, or dune, and was used with the toe location to define the face slope used in the runup and wave overtopping analyses. Two overtopping points were also selected to represent the slope landward of the crest in the event that wave overtopping occurred. On profiles where dunes were identified, the dune toe location was set equal to the toe location and used in the erosion analysis.

4.2 Wave Setup and Runup

Wave setup is an elevation of the water level due to the effects of wave momentum being transferred to the surf zone. In wave systems composed of more than one wave component, as occurs in the Pacific Ocean, the setup oscillates and comprises a static and a dynamic component. Wave runup is the culmination of the wave breaking process, whereby the wave surges up the beach, bluff, or structure face along the shoreline. Overtopping occurs when the wave runup exceeds the profile crest elevation, which can result in flooding landward of the crest. Runup is a function of several key parameters. These include the wave height, H, the wave period, T, the wave length, L, the profile slope, m, and the surf similarity parameter (or Iribarren number), ξ, defined as $m/\sqrt{H/L}$. The TWL is defined as the sum of the total runup and the SWL, referenced to an established vertical datum. The total runup, R, is composed of three main components:

- Static wave setup, $\bar{\eta}$;
- Dynamic wave setup, η_{rms}; and
- Incident wave runup, R_{inc} .

The results for this study are referenced to the North American Vertical Datum of 1988 (NAVD88) vertical datum.

Wave setup and runup were computed at each hourly time step in the 50-year wave and water level hindcast time series. Current policy for the National Flood Insurance Program is to define the wave runup elevation as the value exceeded by 2 percent of the runup events. The 2-percent value was chosen during the development of the Pacific Guidelines and is a standard definition of runup, commonly denoted as $R_{2\%}$.

Wave setup and runup were combined with coincident water level values at each transect to develop the TWL values. Annual maxima TWLs were recorded, and a statistical Generalized Extreme Value (GEV) analysis was performed on these values to determine the 1-percent-annual-chance TWL. The overtopping rate, as well as the inland limit of overtopping, was calculated for instances where the TWL exceeded the dune or barrier crest and overtopping occurred. Each step used to evaluate hazards is described in detail in the following subsections.

4.2.1 Wave Setup

Both static and dynamic components of wave setup were calculated using the parametric DIM as recommended in the Pacific Guidelines. The DIM approach calculates wave setup and runup using a parameterized set of equations that consider wave and bathymetric characteristics, specifically the shape of the wave energy spectrum and the nearshore slope (m_{DIM}). The wave setup equations include factors for wave height (F_H and G_H), wave period (F_T and G_T), JONSWAP spectral narrowness factor (F_{Gamma} and G_{Gamma}), and nearshore slope (F_{Slope} and G_{Slope}).

Static wave setup, $\bar{\eta}$, is calculated as:

$$\bar{\eta} = 4.0 F_H F_T F_{Gamma} F_{Slope} = 4.0 \left(\frac{H_0'}{26.2}\right)^{0.8} \left(\frac{T_p}{20.0}\right)^{0.4} (1.0) \left(\frac{mDIM}{0.01}\right)^{0.2} \qquad \text{(Equation 1)}$$

Dynamic wave setup, η_{rms}, is calculated as:

$$\eta_{rms} = 2.7 G_H G_T G_{Gamma} G_{Slope} = 2.7 \left(\frac{H_0'}{26.2}\right)^{0.8} \left(\frac{T_p}{20.0}\right)^{0.4} (Gamma)^{0.16} \left(\frac{mDIM}{0.01}\right)^{0.2} \qquad \text{(Equation 2)}$$

The wave parameters required as input for DIM are the deepwater equivalent significant wave height, in feet, (H_0') and the spectral peak wave period (T_p), as well as a measure of the spectral shape (*Gamma*). The spectral peakedness parameter, *Gamma*, was computed via a polynomial fit between the spectral width parameter, ν, and *Gamma* (Equation 3).

$$Gamma = 20471\nu^4 - 30830\nu^3 + 17828\nu^2 - 4769.9\nu + 507.1 \qquad \text{(Equation 3)}$$

Values of ν are computed directly from the spectral moments (m_0, m_1, and m_2) based on the Longuet-Higgins definition of the spectral narrowness:

$$\nu = \left[\frac{m_0 m_2}{m_1} - 1\right]^{1/2} \qquad \text{(Equation 4)}$$

Gamma values are limited from 1 to 38, based on the range of data used to relate the spectral narrowness, ν, to the peakedness parameter, and are indicative of the limitations of the polynomial fit at low values of ν (Figure 5).

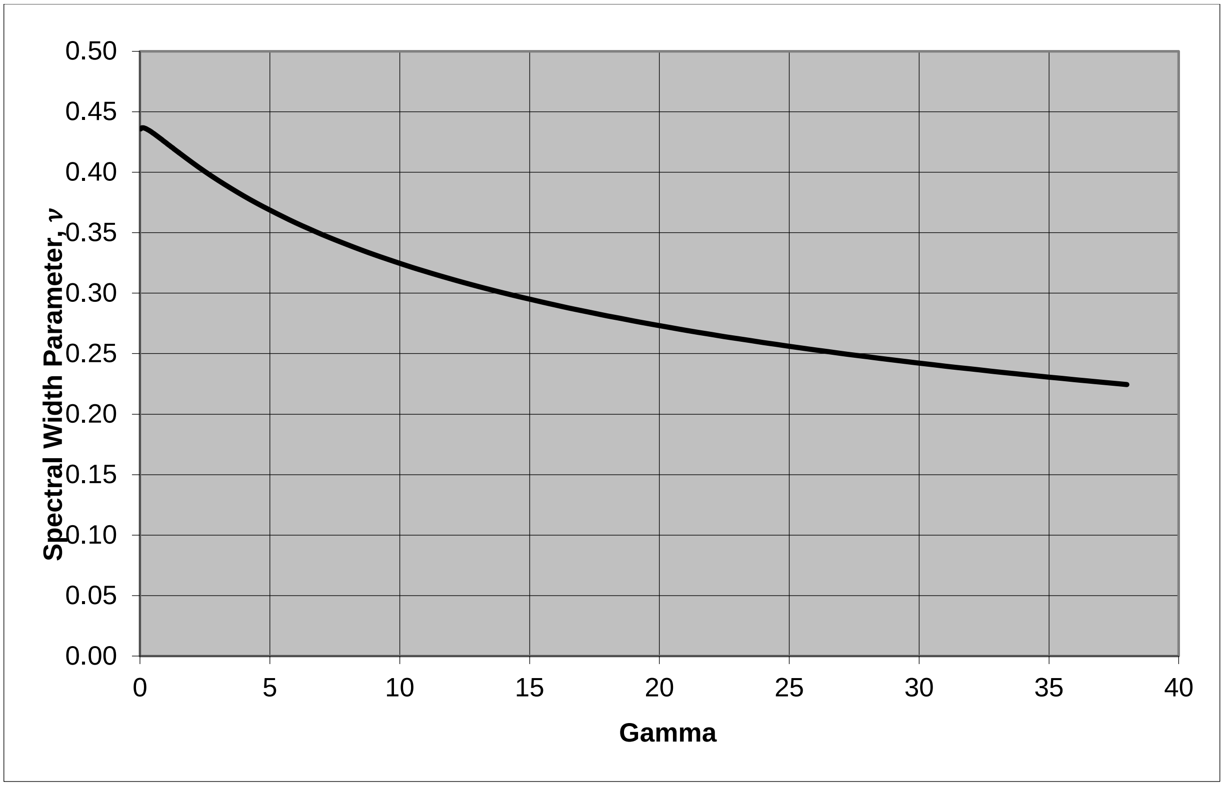

Figure 5: Spectral Width Parameter, ν, versus Gamma (Corrected from Figure D.4.5-5 (FEMA, 2005))

The deepwater equivalent significant wave height, H_0', and the peak period, T_p, were provided as output from the SHELF modeling and were input directly into Equations 1 and 2. The nearshore slope, m_{DIM}, is taken as the average slope between the landward limit of wave runup and the location offshore where the water depth is two times the depth at which the deepwater significant wave height would be subject to depth-limited breaking (van der Meer, 2002; FEMA, 2005). The landward limit of wave runup was calculated iteratively, with the initial approximation being the SWL.

4.2.2 Runup

Wave runup was calculated using one of three methods, depending upon the dynamic water level relative to the profile toe and the shoreline slope, m_{TAW}, calculated iteratively across the surf zone between the stillwater line minus 1.5 times the spectral significant wave height at the toe, H_{m0}, or the toe of the structure or bluff, whichever is higher, and the runup limit. As recommended in the Pacific Guidelines, the DIM was used to calculate runup for transects with natural, gently sloping (m_{DIM} < 0.125) profiles. The TAW method (van der Meer 2002) was used for shorelines with shore protection structures and steeply sloping (m_{TAW} ≥0.125) natural shorelines where the dynamic water level exceeded the toe of the structure or bluff. If, on these shorelines, the dynamic water level did not reach the toe of the structure or

bluff face, the DIM was used for gently sloping profiles while a modified TAW approach was implemented on steeper shorelines. In these cases, the Iribarren number, ξ_0, was compared to a critical Iribarren number, ξ_{0c}, to determine whether to use DIM or the modified TAW approach. The Shore Protection Manual (SPM) method (USACE, 1984) was used to calculate wave runup on vertical walls. The total runup, including wave setup and incident wave runup, was added to the SWL to determine the TWL (Figure 6). Each of these methods is described in detail in the following subsections.

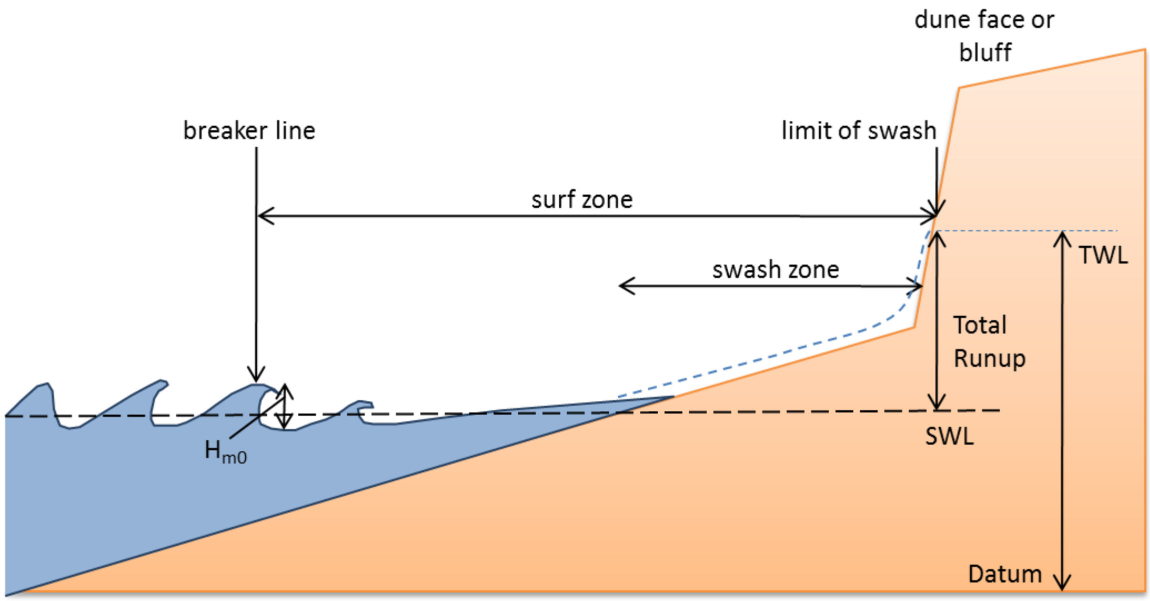

Figure 6: Conceptual Model Showing the Components of Wave Runup Associated with Incident Waves (Modified from Pacific Guidelines (FEMA, 2005))

4.2.3 DIM Runup Calculations

Runup on gently sloping, natural shorelines, and beaches seaward of a structure or bluff toe, was calculated using the DIM. The runup calculation is based on the standard deviations of the oscillating wave setup and the incident wave runup components, and is a continuation of the DIM approach for wave setup.

The dynamic setup, η_{rms}, is defined as the standard deviation of setup fluctuations. This is approximated with the parametric DIM model given in Equation 2. The standard deviation of the incident wave oscillations (wave runup), σ_2, on natural beaches is given in the Pacific Guidelines as:

$$\sigma_2 = 0.3\xi_0 H_0' \qquad \text{(Equation 5)}$$

Where:

- H_0' = deepwater equivalent significant wave height

- ξ_0 = Iribarren number ($m_{DIM}/\sqrt{H_0'/L_0}$)
- m_{DIM} = nearshore slope
- L_0 = deepwater wave length ($gT_p^2/2\pi$)
- T_p = peak wave period

The total oscillating component to the total wave runup, $\hat{\eta}_T$, is determined as the combination of the two standard deviations of the fluctuating components:

$$\hat{\eta}_T = 2.0\sqrt{\eta_{rms}{}^2 + \sigma_2{}^2} \quad \text{(Equation 6)}$$

Combining the results from Equations 1 and 6 yields the total wave runup, and when combined with the SWL, results in the TWL:

$$TWL = \bar{\eta} + \hat{\eta}_T + SWL \quad \text{(Equation 7)}$$

4.2.4 TAW Runup Calculations

Runup on barriers, including steep (m_{TAW} > 0.125) dune features, bluffs, and coastal armoring structures such as revetments, was calculated using the TAW method (van der Meer, 2002). Wave runup on barriers is a function of the geometry and roughness of the structure, as well as the height and steepness of the incident wave. The TAW method provides a mechanism for calculating wave runup with adjustments made through various reduction factors to account for surface roughness and the effects associated with the angle of wave approach.

The TAW methodology is based on wave tank measurements in which wave setup due to breaking at the structure is inherently included in the wave runup heights recorded in the study. As a result, the wave setup component of the TWL in this work is calculated at the toe of the structure, and wave setup landward of the toe of the structure is not included. Wave setup seaward of the toe of the structure was computed with the DIM, using the nearshore slope, m_{DIM}. Wave setup was not included for cases where waves would not have broken prior to reaching the toe of the structure.

The reference water level at the toe of the structure for runup calculations using the TAW method is defined as the 2-percent Dynamic Water Level (DWL2%). The dynamic water level is the sum of the measured SWL, the static wave setup, and the dynamic wave setup. The Pacific Guidelines suggest applying a reduction in the dynamic wave setup to account for the dynamic wave setup present during the laboratory experiments that generated the wave runup methodology. The intent of this reduction is to avoid double counting a portion of the dynamic wave setup when combining the dynamic wave setup from DIM with the wave runup from TAW. However, it was noted that there is little cross-shore variation in the magnitude of the dynamic setup and that no reduction to the dynamic setup is needed

(BakerAECOM, 2013b). Instead, because DIM provides the static setup at the shoreline and not the barrier toe, and the magnitude of static wave setup varies with depth across the surf zone, from a maximum at the shoreline to approximately zero seaward of the breaking point, a reduction to the static setup component was applied for cases where the barrier toe elevation is inundated by the SWL and the TAW method is used for computing wave runup (BakerAECOM, 2013c).

This procedure involves computing the static wave setup at the shoreline and at the toe location to determine a static setup reduction factor to be applied to the static wave setup calculated using DIM. The wave setup at the shoreline and toe location and subsequent reduction factor are based on the root mean square of the breaking significant wave height, $(H_b)_{rms}$, and the depth at the toe of the barrier relative to SWL, h. $(H_b)_{rms}$ was determined using the deepwater equivalent significant wave height (H_0') and the peak wave period (T_p) as:

$$(H_b)_{rms} = \left(\frac{\kappa}{g}\right)^{1/5}\left(\frac{H_0'^2 C_0}{2}\right)^{2/5}/1.4 \qquad \text{(Equation 8)}$$

where κ is the breaker criterion equal to 0.78 and C_0 is the deepwater wave celerity ($C_0 = L_0/\, T_p$). The static wave setup at the SWL shoreline is:

$$\overline{\eta_0} = 0.189(H_b)_{rms} \qquad \text{(Equation 9)}$$

and the static wave setup at the toe of the barrier is:

$$\bar{\eta}(h) = 0.189(H_b)_{rms} - 0.186h \qquad \text{(Equation 10)}$$

The static wave setup reduction factor, γ_η, is then a ratio of the static wave setup at the toe to the static wave setup at the SWL shoreline:

$$\gamma_\eta = {}^{\bar{\eta}(h)}\!/_{\overline{\eta_0}} \qquad \text{(Equation 11)}$$

This reduction factor is then applied to the DIM static wave setup to compute a depth-adjusted static wave setup at the toe of the barrier, $\bar{\eta}'$:

$$\bar{\eta}' = \gamma_\eta \bar{\eta} \qquad \text{(Equation 12)}$$

DWL2% is therefore:

$$DWL2\% = \bar{\eta}' + 2\eta_{rms} + SWL \qquad \text{(Equation 13)}$$

With the DWL2% calculated, the wave height at the toe of the barrier and wave runup were computed next. H_{m0} is the spectral significant wave height at the toe of the structure and is determined by shoaling

and refracting the wave height provided at the SHELF point to the structure toe. If the DWL2% depth at the structure toe is found to be too shallow to support the calculated wave height, the wave was assumed to be depth-limited and the incident wave height was calculated using a breaker index of 0.78 ($H_{m0} = 0.78 \times h_{toe}$).

The average slope for use in the TAW methodology, m_{TAW}, was calculated iteratively between the stillwater line minus $1.5H_{m0}$ (or the toe of the structure or bluff, whichever is higher) and the runup limit. The lower slope point cannot be lower than the toe of the structure or bluff. Since the runup limit is initially unknown, the stillwater line $+1.5H_{m0}$ was chosen as a first estimate (Figure 7). If the runup limit exceeded the selected face point, the runup limit was set as the face point.

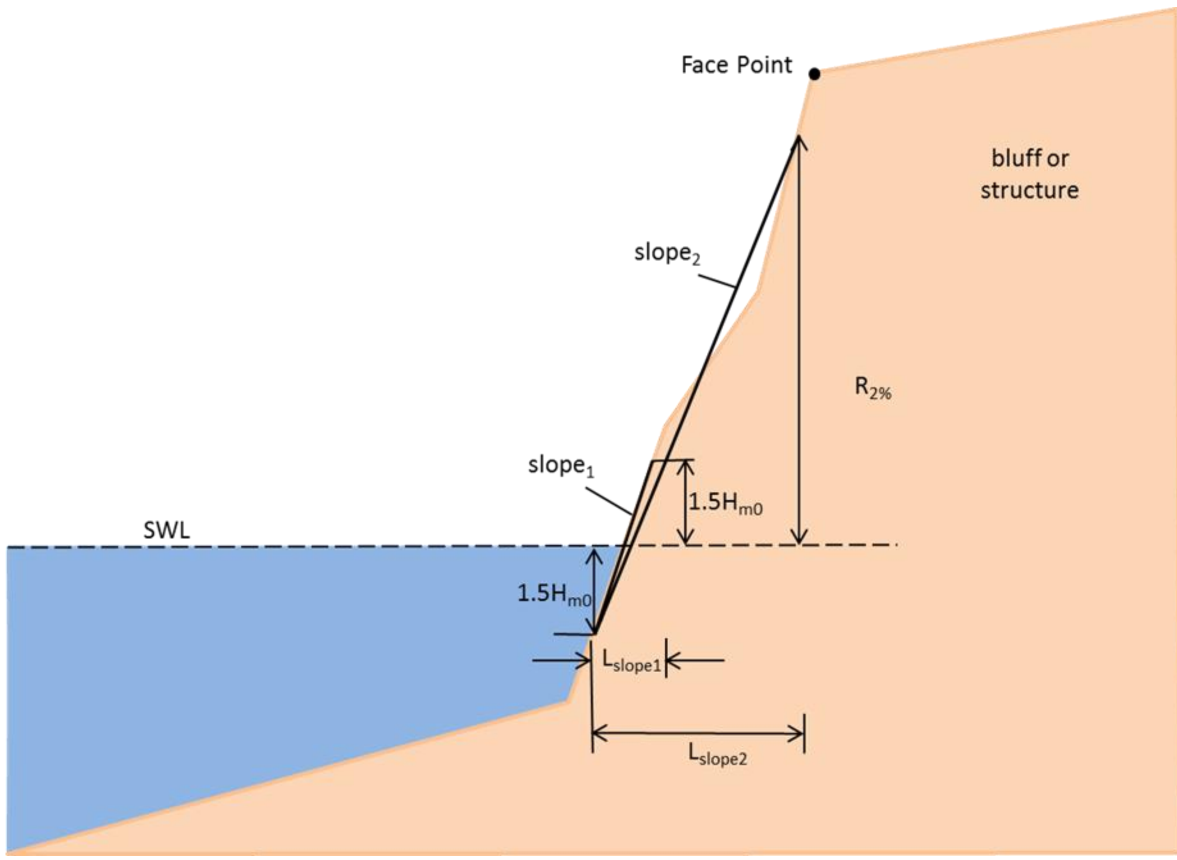

Figure 7: Determination of an Average Slope Based on an Iterative Approach (Modified from van der Meer, 2002)

The general formula of TAW for calculating the 2-percent wave runup on barriers is:

$$R_{2\%} = H_{m0}\begin{Bmatrix} 1.77\gamma_r\gamma_b\gamma_\beta\xi_{0m}, & 0.5 \le \gamma_b\xi_{0m} < 1.8 \\ \gamma_r\gamma_b\gamma_\beta\left(4.3 - \frac{1.6}{\sqrt{\xi_{0m}}}\right), & 1.8 \le \gamma_b\xi_{0m} \end{Bmatrix} \qquad \text{(Equation 14)}$$

Where:

- $R_{2\%} = 2\,\sigma_2$, wave runup height exceeded by 2 percent of the incoming waves
- H_{m0} = spectral significant wave height at the structure toe
- γ_r = influence factor for roughness element of slope
- γ_b = influence factor for a berm
- γ_β = influence factor for oblique wave attack
- γ_p = influence factor for porosity
- ξ_{0m} = Iribarren number $(m_{TAW}/(\frac{H_{m0}}{L_{m-1.0}})^{0.5})$
- m_{TAW} = TAW slope
- $L_{m-1.0}$ = the deepwater spectral wave length $(gT_{m-1.0}^2/2\pi)$
- $T_{m-1.0}$ = spectral wave period

The spectral wave period, $T_{m-1.0}$, and its associated wavelength, $L_{m-1.0}$, were calculated from the zero-eth and first negative deepwater spectral moments provided as output from the SHELF wave model.

Influence factors for roughness, the presence of a berm, and oblique wave attack were selected according to Table D.4.5-3 in the Pacific Guidelines. The roughness reduction factors used to account for the surface roughness of structures are summarized in Table 6.

Table 6: Roughness Reduction Factor Values

Surface Type	Value of γ_r
Smooth concrete, asphalt	1.0
Natural shoreline and grass	1.0
Rocky bluff	0.8
Armor stone revetment	0.6-1.0

The influence factor for oblique wave attack was calculated at each time step, relating the direction of wave propagation to the transect orientation. The spectral significant wave height, H_{m0}, was shoaled and refracted from the SHELF point to the structure toe. The wave direction at the toe was compared to the transect orientation, perpendicular to the shoreline, to determine the angle of wave attack. For cases in which waves broke seaward of the structure toe, the wave direction was taken from the point of breaking; the incident wave height at the toe was depth-limited and calculated using a breaker index of 0.78

($H_{m0} = 0.78 \times h_{toe}$). The porosity reduction factor was taken as unity for all shorelines. This conservative assumption was based on the uncertainty related to the permeability of structure cores in the study area. The influence factor for the presence of berms was also conservatively set to 1.0 for all transects.

Incident wave runup ($R_{2\%} = 2\,\sigma_2$) was then statistically combined with the reduced dynamic wave setup (η_{rms}), as with the application of DIM, and added to SWL and static wave setup to yield the TWL:

$$TWL = SWL + \overline{\eta}' + 2.0\sqrt{\eta_{rms}{}^2 + (\frac{R_{2\%}}{2})^2} \qquad \text{(Equation 15)}$$

For non-vertical structures with slopes greater than 1:1, the TAW manual (van der Meer, 2002) suggests using the TAW method with an additional reduction factor to account for runup on very steep (but not vertical) slopes. With steep slopes, the Iribarren number, ξ_{0m}, becomes large which means that the waves will not break. To keep the relationship between the type of breaking and the Iribarren number, the vertical wall must be schematized as a 1:1 slope. Therefore, the barrier slope was set to 1:1 for the Iribarren number calculation, and a reduction factor for steep slopes was applied:

$$\gamma_v = 1.35 - 0.0078 \tan^{-1} m_{TAW} \qquad \text{(Equation 16)}$$

While this approach was based on work done for vertical walls atop dikes, sensitivity testing showed that it compared well with the TAW method and the SPM method for vertical walls as an intermediate approach to calculating runup on steep slopes. The use of this reduction factor accounts for wave reflection expected on slopes greater than 45 degrees, and this approach generates results that fall between those for a 45-degree slope and those for a vertical wall.

4.2.5 Modified TAW Runup Approach

In cases where the DWL2% did not reach the structure toe, the DIM was used for gently sloping profiles, while a modified TAW approach was implemented on steeper shorelines. To determine which runup method to use, the Iribarren number, ξ_0, was compared to a critical Iribarren number:

$$\xi_{0c} = 1.340\,\frac{0.78^{1/8}}{m_{DIM}^{1/4}} \qquad \text{(Equation 17)}$$

If $\xi_0 \leq \xi_{0c}$, runup was calculated using DIM, as presented in Section 4.4.4.1. Where $\xi_0 > \xi_{0c}$, runup was computed using a modified TAW approach, where the TAW equation for $1.8 \leq \gamma_b \xi_{0m}$ (Equation 14) was rewritten in terms of deepwater wave parameters and the nearshore slope, m_{DIM} (McDougal, 2013):

$$\sigma_2 = 0.1198 H_0{}' \left(\frac{\xi_0}{m_{DIM}}\right)^{1/5} \left(4.3 - \frac{1.345}{m_{DIM}{}^{1/10} \xi_0{}^{2/5}}\right) \qquad \text{(Equation 18)}$$

This modified TAW runup component was then combined with the dynamic setup to determine the total oscillating component to the total wave runup, $\hat{\eta}_T$, and the TWL, as in Equations 6 and 7, respectively.

4.2.6 Runup on Vertical Walls

Since the Pacific Guidelines do not provide guidance for calculating runup on vertical structures, runup on vertical structures such as seawalls was calculated using the methodology listed in the SPM (USACE, 1984) and FEMA's *Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update* (FEMA, 2007), hereafter referenced as the Atlantic and Gulf Guidelines. This method, illustrated in Figure 8, determines a mean runup value by specifying the mean of the deepwater equivalent significant wave height (H_0') and the mean wave period (T), as well as the water depth at the structure (d_s).

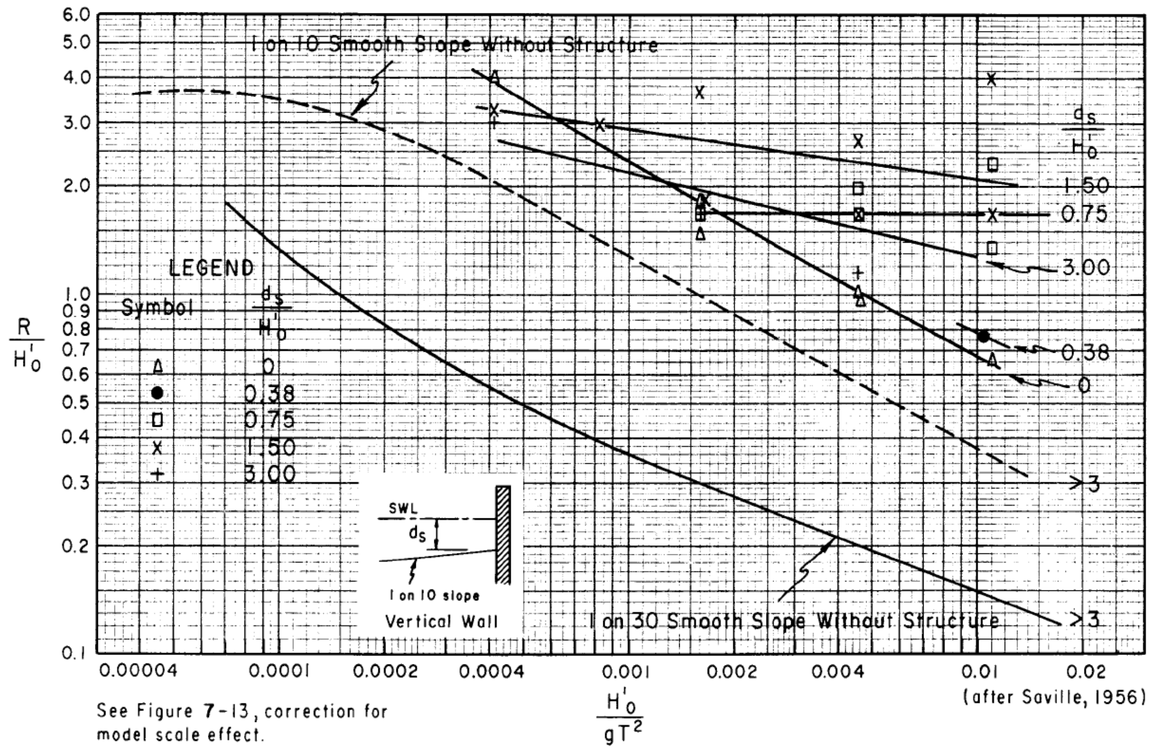

Figure 8: Wave Runup on an Impermeable Vertical Wall (From USACE, 1984)

In accordance with the guidance provided in the Atlantic and Gulf Guidelines, the mean wave height is approximated as 0.626 times the significant wave height. The mean wave period was taken directly from the SHELF output. The calculated mean runup was multiplied by 2.2 to determine the 2-percent runup height (FEMA, 2007). This runup value was then added directly to the SWL, without wave setup, to obtain the TWL.

Note that Figure 8 is consistent with the original figure published in the SPM (USACE, 1984). A labeling error on the top curve was found in the Atlantic and Gulf Guidelines version of this figure. The error has since been corrected through FEMA Procedure Memorandum 60 (FEMA, 2011).

4.3 Statistical Analysis

The preferred approach for determining the 1-percent-annual-chance (base) flood elevation (BFE) involves utilizing a reasonably long observational (or continuous model) record to establish a probability distribution that can be used to evaluate the flood elevation for any frequency. A general rule of thumb is that a historical record at least one-third the length of the return period of interest is the minimum record needed to produce statistically reliable results. The extremal probability distribution can be used to establish any flood elevation frequency, but the levels of confidence in the values decrease with the length of record. In this case, a modeled continuous record of 50 years of offshore and nearshore wave conditions will be used to derive a hindcast of TWLs. This hindcast is long enough that an extreme value distribution can be applied to it, in order to estimate the TWL elevation for a 1-percent-annual-chance condition. The Pacific Guidelines recommend using an annual maxima/Generalized Extreme Value (GEV) fit in the extreme value analysis.

The cumulative distribution function of the GEV family of distributions is given as:

$$F(z) = \exp\left\{-\left[1+\xi\left(\frac{z-\mu}{\sigma}\right)\right]^{-1/\xi}\right\} \quad \text{(Equation 19)}$$

The model has three parameters: μ is the mode of the extreme value distribution (also known as the location parameter), σ is the dispersion (also known as the scale parameter), and ξ, not to be confused with the Iribarren number in wave runup equations, is a shape parameter that determines the type of extreme value distribution. These parameters were determined using routines for GEV statistical analysis within the Wave Analysis for Fatigue and Oceanography, Version 2.1.1 (WAFO) toolbox for Matlab, which contains tools for fatigue analysis, sea state modeling, statistics, and numerics (WAFO-group, 2000). The three parameters, $\mu, \sigma,$ and $\xi,$ of the cumulative distribution function were evaluated for the maximum likelihood solutions.

TWL annual maxima were selected for each of the 50 years (1960-2009) of wave runup hindcast results and statistically analyzed using the GEV approach as defined in the Pacific Guidelines. The water year for the Pacific coast was defined to extend from July 1 through June 30 to keep the stormy winter season intact. This prevents single storm events, which often last for several days and thus may extend across calendar years, from defining two annual maxima in two consecutive years, rather than being captured as a single maximum. This does, however, leave 1960 and 2009 as incomplete water years. A review of buoy data for the period from July 1, 1959, through December 31, 1960, the first half of the 1960 water year, showed no extreme events to rival the maximum wave event occurring in 1960, thereby confirming the annual maximum event for the 1960 water year, despite the curtailed dataset. This resulted in a hindcast time series from January 1, 1960, through June 30, 2009, that was analyzed for this study.

Computed TWL statistical elevations include BFEs, which will inform the mapping on the FIRM, as well as the 50-percent-, 20-percent-, 10-percent-, 4-percent-, 2-percent-, and 0.2-percent-annual-chance flood elevations, which will be included in the FIS report. Goodness-of-fit measures, including z-values (Hosking and Wallis, 2007), and frequency curves were prepared and analyzed. A review of the goodness-of-fit measures and frequency curves validated the use of GEV to compute the statistical TWLs.

4.4 Erosion

Beaches along the Pacific Coast undergo seasonal changes in profile as well as changes in response to storm events. As a result, both seasonal and episodic erosion must be accounted for on sandy beaches within the study area. Winter conditions typically result in narrower beach faces with steeper foreshore slopes. Therefore, it is important to first estimate the beach profile conditions that existed just before the occurrence of an episodic winter storm. This initial beach profile, defined as the Most Likely Winter Profile (MLWP), represents the likely winter profile conditions for a particular coastal setting. Once the MLWP has been defined, a profile recession due to the elevated water levels and high surf that occur during storm events can be determined.

Event-based erosion of sandy beaches backed by low sand berms or high dunes was conducted according to the MK&A (Komar et al., 2002; McDougal and MacArthur, 2004) geometric method for beach profile erosion and dune recession, which was suggested for use in the Pacific Guidelines. For this study, the geometric dune erosion method was used only to erode the dune and not to erode or lower the beach profile. The geometric model assumes the initial beach profile originates as the MLWP. Therefore, the extracted transect profile must first be eroded using average winter conditions to generate the MLWP. The MLWP then erodes further in response to the elevated TWL to account for storm-induced erosion.

A primary parameter necessary for the construction of eroded profiles using the MK&A method is the TWL. For erodible profiles, wave runup and TWLs were calculated on the transect profiles without adjustment to inform the configuration of the MLWP. TWLs were then recalculated on the MLWP to determine initial 1-percent-annual chance statistical TWLs. This initial 1-percent TWL event was used in estimating beach profile change in response to storm events to determine the final eroded profile. The initial 0.2-percent TWL event was used to estimate beach profile change in response to the 0.2-percent-annual-chance storm event to determine the 500-year eroded profile. Wave runup and statistical TWLs were recomputed for the final eroded profile.

4.4.1 Estimating the Most Likely Winter Profile

Ideally, the MLWP should be determined from profile data for the period immediately following a significant storm or series of winter storms. However, given a lack of available data regarding these conditions, the MLWP can be determined from typical winter wave and water level conditions. Therefore, the MLWP was estimated from the extracted cross-shore profile along dune-backed transects and typical winter wave conditions. TWL annual maxima were selected from the 50-year time series.

These values were averaged and used to represent the annual storm event and the dune-beach juncture ($E_{j,MLWP}$) corresponding to the MLWP (Figure 9).

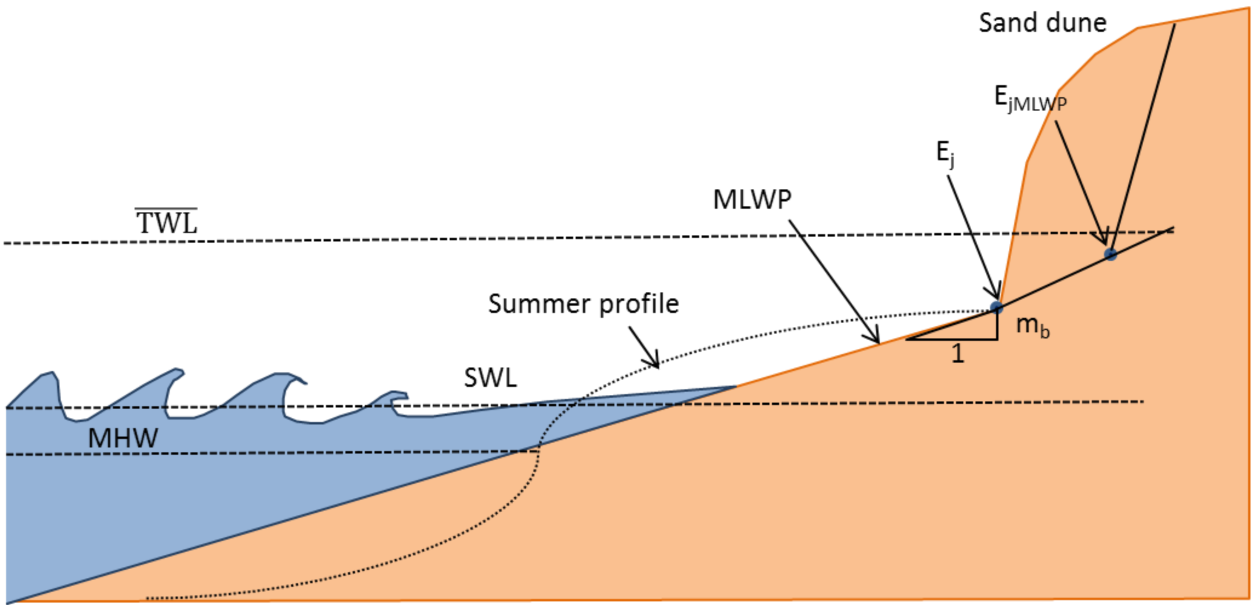

Figure 9: Definition Sketch of the MLWP for the MK&A Geometric Model (Modified from Pacific Guidelines (FEMA, 2005))

The deep water equivalent wave height (H_o') and peak spectral wave period (T_p) corresponding to each annual maximum event were also extracted and averaged to approximate typical winter wave conditions.

The geometric model provides an estimate of the maximum potential cross-shore displacement of the profile. Wave and water levels must persist long enough to achieve this maximum. However, the typical storm may often have a shorter duration than is required to achieve the maximum potential cross-shore recession. Therefore, an adjustment to the cross-shore recession distance based on the time dependency of profile response was developed by Kriebel and Dean (1993) and described in the Pacific Guidelines. This method determines the ratio, α, of duration-limited recession (R_m) to the maximum potential recession ($R_{\infty Storm}$), by relating the time scale for the beach profile to the duration of the storm. The following calculations were made determine the beach profile time scale and the ratio α.

The averaged wave parameters were first used to calculate the depth of wave breaking (h_b):

$$h_b = \frac{1}{g^{1/5}\gamma^{4/5}}\left(\frac{H_0'^2 C_0}{2}\right)^{2/5} \qquad \text{(Equation 20)}$$

where γ is the breaker criterion of 0.78 and C_0 is the deep water wave celerity. The breaking wave height (H_b) is calculated from h_b using the breaker criterion for use in the erosion time scale (T_S) equation:

$$T_S = 320 \frac{H_b^{3/2}}{g^{1/2} A^3} \left(\frac{1}{1 + \frac{h_b}{B} + x_b (tan\beta_f)/h_b} \right) \quad \text{(Equation 21)}$$

where B is the berm elevation, $tan\beta_f$ is the foreshore slope measured between MLW and MHW, and x_b is the cross shore location of the breaking wave $((h_b/A)^{3/2})$. The sand at each dune-backed transect in Humboldt County was characterized as medium grain and the corresponding beach profile parameter, A, in Table 7 was selected for use in Equation 19.

Table 7: Profile Shape Parameters, A, from Binned Grain Size

Grain Size Classification	Median Grain Size (mm)	A ($m^{1/3}$)
Fine	d < 0.25	0.08
Medium	0.25 < d < 0.5	0.12
Coarse	d > 0.5	0.19

The storm duration (T_D), defined as the time the TWL exceeds MHW, was determined for each annual maximum TWL event. The average storm duration over the 50-year annual maxima was used with the time scale to define β, the ratio of the erosion time scale to the storm duration, used in the transcendental equation for ξ, the phase parameter related to the timing of the maximum erosion response, t_m:

$$exp\left(-\frac{2\xi}{\beta}\right) - cos(2\xi) + \frac{1}{\beta} sin(2\xi) = 0 \quad \text{(Equation 22)}$$

where $\beta = 2\pi \frac{T_S}{T_D}$ and $\xi = \frac{t_m \pi}{T_D}$.

The ratio of duration limited recession (R_m) to the maximum potential recession ($R_{\infty Storm}$), α, can be calculated from:

$$\alpha = \frac{1}{2}[1 - cos(2\xi)] \quad \text{(Equation 23)}$$

The configuration of the MLWP was established using the geometric model with backshore slope m_b (measured between the initial dune toe elevation, E_j, and Mean High Water), E_j, and the average annual maximum TWL (Figure 9). Preserving the backshore slope, $E_{j,MLWP}$ was then adjusted from the maximum potential recession location to the duration-limited recession location based on the calculated α ratio.

The angle of the profile extending from $E_{j,MLWP}$ inland toward the dune crest is taken as the angle of repose for dry sand, 33 degrees.

4.4.2 Estimating Beach Profile Change for Storm Events

Once the MLWP has been established, it can then be modified according to the amount of erosion that occurs during a specified storm event as the result of increased water levels and wave action associated with episodic storm events. This erosion procedure follows the MK&A geometric method used in determining the MLWP, but uses 1-percent conditions in place of typical winter storm conditions.

Runup and TWLs were recomputed on the MLWP and an initial 1-percent-annual-chance TWL was determined from the time series results. Erosion of the MLWP for the 1-percent TWL event was calculated by applying the MK&A geometric model with the 1-percent TWL, $E_{j,MLWP}$, and the same backshore slope, m_b, to get a new maximum recession distance. The maximum recession distance ($R_{\infty Storm}$) is estimated from the storm's TWL (E_{jStorm}) and the geometry of the MLWP:

$$R_{\infty Storm} = \frac{E_{jStorm} - E_{jMLWP}}{m_b} \qquad \text{(Equation 24)}$$

The ratio α and duration-limited recession (R_m) are recomputed using the same time scale as was used to construct the MLWP, but a revised storm duration, the average T_D from the annual maxima plus one standard deviation of the storm durations. The addition of one standard deviation accounts for uncertainty in the duration selection method and the potential to underestimate the longer storm duration associated with the 1-percent-annual-chance event. The duration limited recession for each storm, R_{storm}, will be:

$$R_{storm} = \alpha R_{\infty Storm} \qquad \text{(Equation 25)}$$

The final eroded profile will be eroded from $E_{j,MLWP}$ a distance R_{storm} inland, preserving the backshore slope (Figure 10). Wave runup and TWLs will be recomputed on the final eroded profile determined based on the profile's duration limited recession.

The 500-year eroded profile was eroded in the same way using the 0.2-percent TWL, the same time scale used to construct the MLWP, and a storm duration equal to the average T_D from the annual maxima plus two standard deviations of the storm durations.

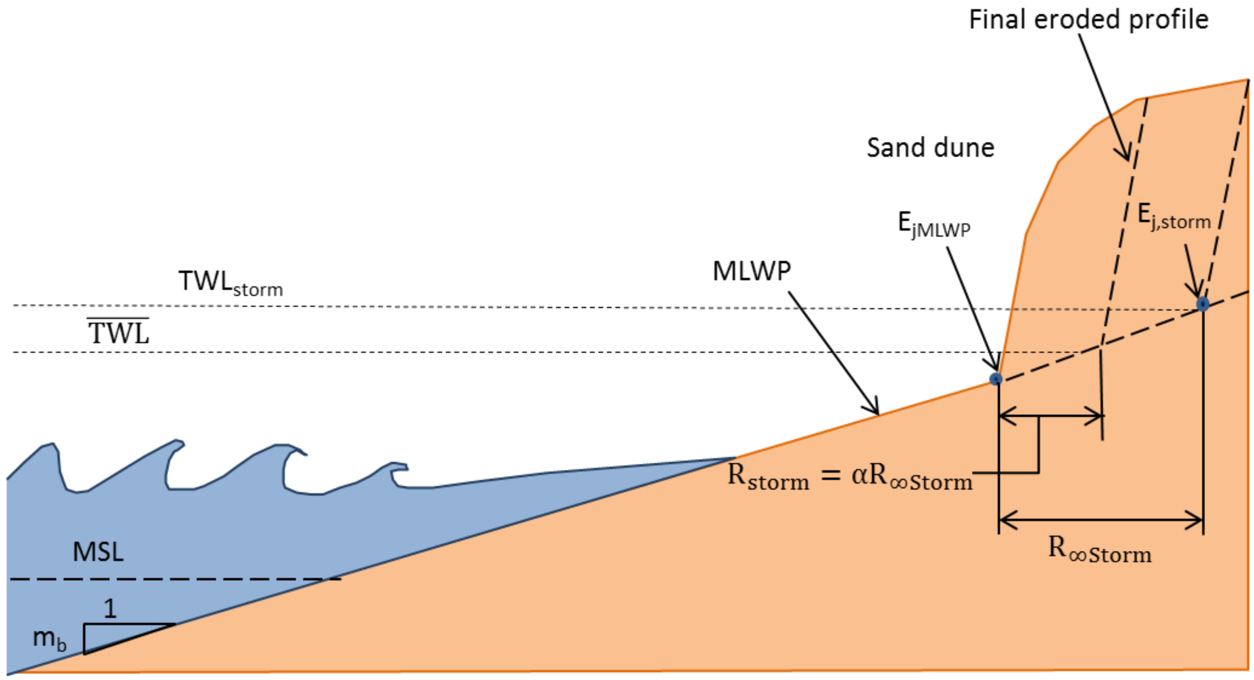

Figure 10: Definition Sketch of the MK&A Geometric Erosion Model (Modified from Pacific Guidelines (FEMA, 2005))

The erosion model was applied to sand beaches backed by low berms or dunes in the study area.

Episodic erosion of erodible coastal bluffs was not implemented for this study. A thorough review of bluff erosion approaches and analysis requirements as presented in the Pacific Guidelines and a geographical impact assessment of the study area were presented to FEMA (BakerAECOM, 2012c).

4.5 Structures

Coastal structures affect local topography, as well as waves and flood hazards. Therefore, structures along the coast were evaluated to determine whether they would survive the 1-percent-annual-chance coastal flood event and provide protection to upland areas.

Criteria for conducting a detailed engineering evaluation of the stability and performance of coastal armoring structures, such as seawalls and revetments, for FIS purposes are well-developed and are based on USACE CERC TR-89-15 (Walton et al., 1989). However, since this kind of effort is beyond the scope of this study, FEMA recommends using engineering judgment and readily available information to assess structures. If the available information does not clearly point to the survival or failure of a coastal structure, FEMA advises erosion and wave analyses for both the intact and failed scenarios; in the case of revetments, FEMA advised that partially failed structure cases be analyzed, and the flood hazards

associated with the more hazardous case be mapped. If the structure is likely to fail, it is removed completely from the onshore analysis. Specifics pertaining to complete removal are described below.

Coastal structures were identified in the data acquisition phase, as documented in IDS #1. The majority of these structures were visited during the field reconnaissance mission. Historical performance, readily available data, and engineering judgment were used to determine whether each structure analyzed along the coastline is likely to survive the 1-percent-annual-chance coastal flood conditions. For each structure, the following characteristics, where available, were evaluated:

- Structure type, condition, engineering, and materials
- Site-specific historical performance of structure
- Global historical performance of structure type in study area (e.g., whether this type of structure in this region sustained damage during extreme coastal events)
- Scale and extent of structure
- Ownership
- Maintenance plan and/or inspections including records of repairs to the structure
- Backshore geometry and type
- California Coastal Commission or USACE input, if available

The Pacific Guidelines, provide essentially three options for the treatment of structures within the study area:

- Option 1. Fail and fully remove the structure from the topographic profile. This option was applied to all uncertified seawalls.

In the case of a vertical free-standing structure, the structure is removed from the profile and the analysis is run on the remaining bare earth profile. This option was also applied to structures that have little or no impact on the coastal analysis, such as single layers of rip rap or rubble installed for erosion protection. Removing these structures would not result in any significant change to the profiles, so the bare-earth topography was used “as is” along transects containing these structures, but no roughness reduction factor was applied, thereby giving no credit to the structure.

For seawalls that are free-standing, removal merely requires the deletion of the crest elevation point, if present in the terrain, from the profile. The analysis is then conducted on the remaining bare earth profile.

In the case of vertical or near-vertical structures with fill behind them, the scour at the toe of the structure will be approximately equal to the depth-limited wave height at the structure. The structure is assumed to fail and fall into a rough, porous slope at a 1:1.5 slope. This 1:1.5 slope is extended from the depth of scour at the structure toe landward to the point where it intersects the existing grade, as shown in Figure 11.

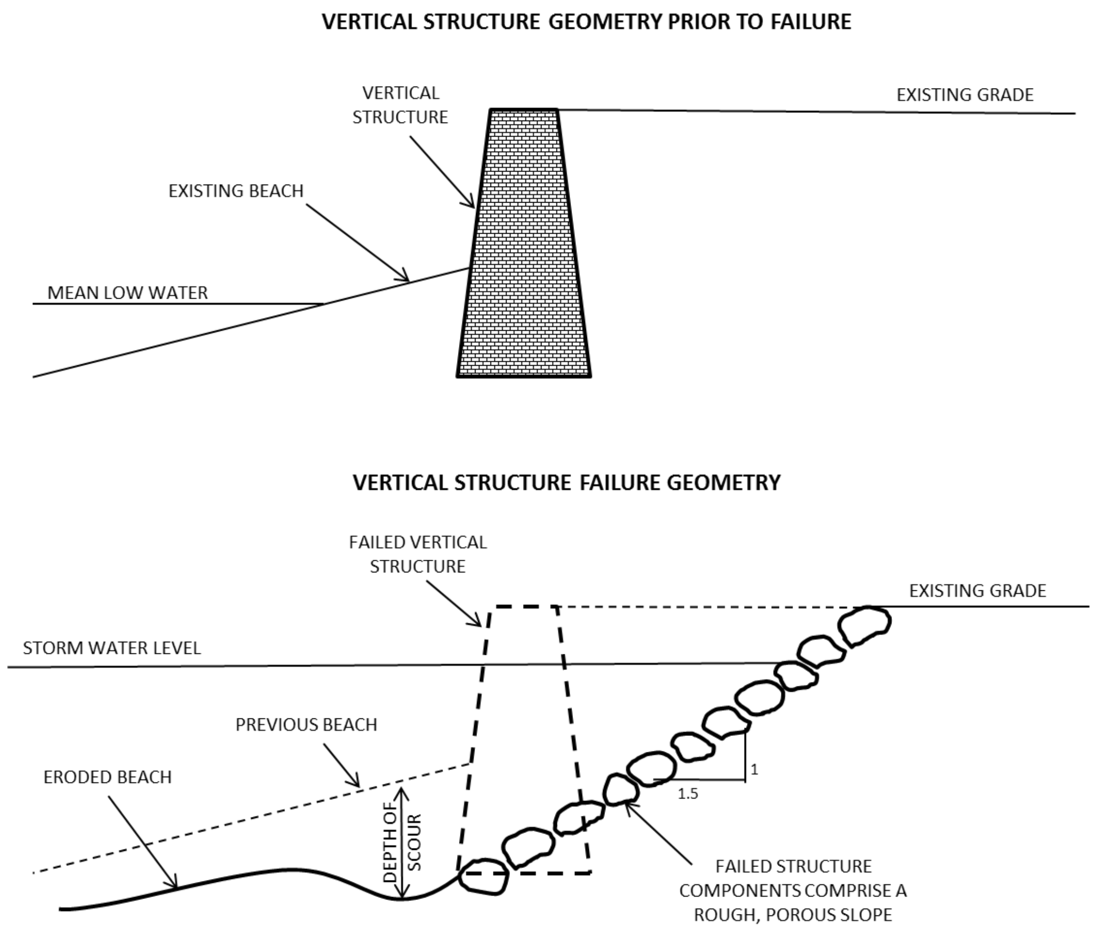

Figure 11: Failure of a Vertical or Near-Vertical Structure Adapted from Pacific Guidelines (FEMA, 2005)

- Option 2. Partially fail a structure. This option was applied to uncertified but substantial seawalls and revetments that are engineered and maintained by a city, county, State, or Federal agency.

In the case of revetments, this assumes the structure collapses in place into a triangular section throughout the structure footprint, with side slopes equal to the original structure slope. One layer of rock equal to the diameter of the outer rock is removed, modifying the geometry and depth at the toe of the structure. The landward side of the failed configuration is assumed to be half exposed and half buried, with a1:1.5 slope landward from the failed structure, as depicted in Figure 12.

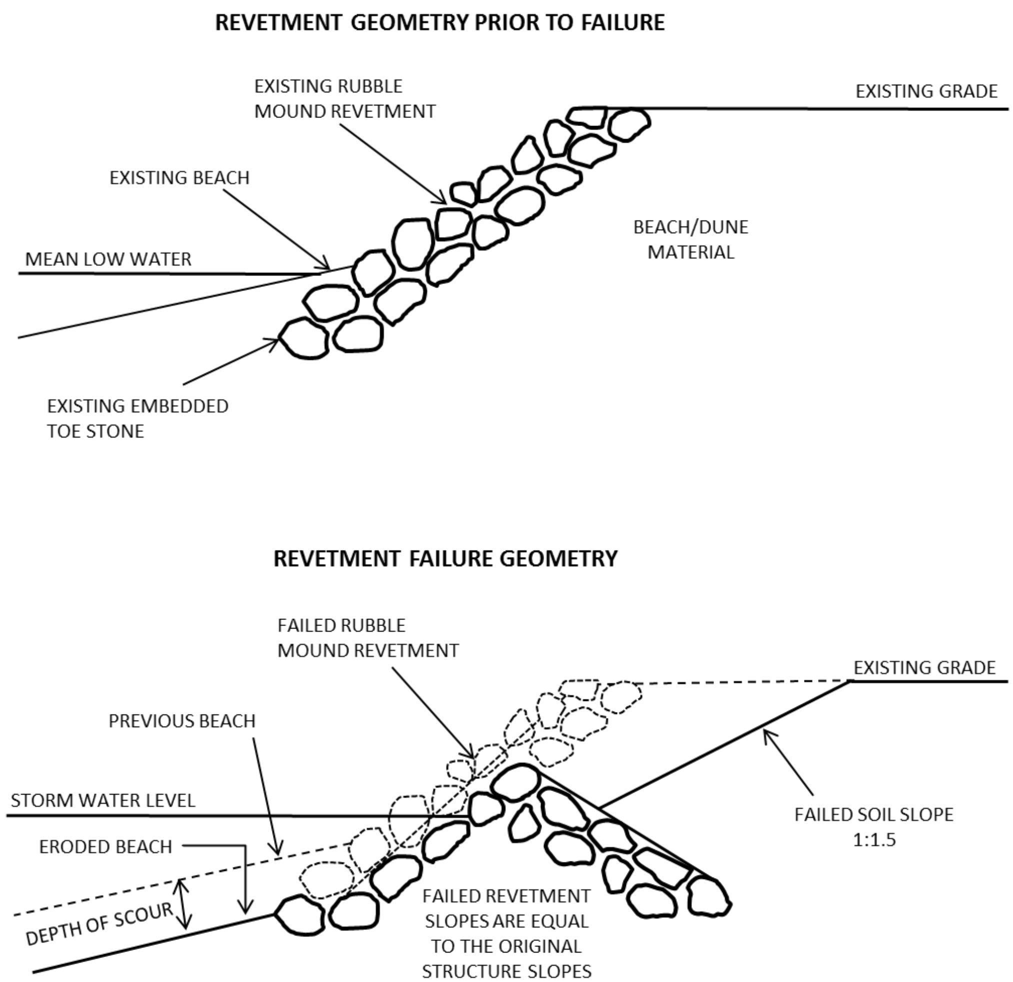

**Figure 12: Partial Failure of a Dune-backed Revetment
Adapted from Pacific Guidelines (FEMA, 2005)**

Since erosion of bluffs is not considered in this study, partial failure of revetments fronting bluffs allowed for the top layer of rock equal to the diameter of the outer rock to be removed, modifying the geometry and depth at the toe of the structure as shown in Figure 13. The roughness reduction factor for partially failed structures remains equal to that of the intact structure since these configurations still provide some protection from the waves.

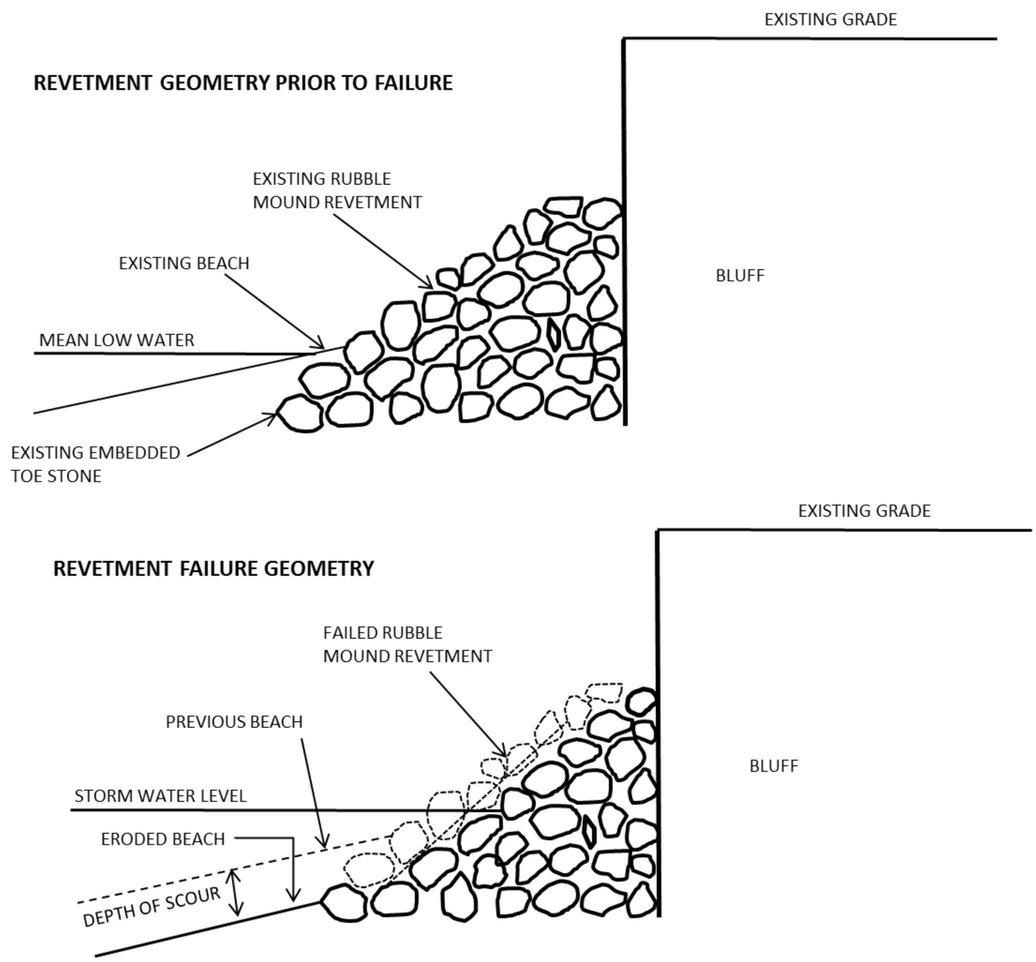

Figure 13: Partial Failure of a Bluff-backed Revetment

- Option 3. Leave the structure intact. This option was applied to structures that are certified and maintained by a city, county, State, or Federal agency. Per FEMA's *Guidance for Coastal Flood Hazard Analyses and Mapping in Sheltered Waters* (FEMA, 2008), causeways, roads, railroads, industrial facilities, tank farms, containment berms, perimeter roads, and related structures within the sheltered embayments were left in place as captured in the topographic data.

Table 8 lists the structures identified along the coastline of Humboldt County for consideration in this study along with the method for treatment of each structure in the current flood hazard analysis. Suggested treatments provided for each structure are evaluated on a case-by-case basis and incorporate historical performance, ownership and maintenance, exposure to wave attack, and engineering judgment. Results from all analyzed options were archived for comparison. Treatment option 1 is referenced as

"Removed," option 2 is "Failed," and option 3 is "Intact." The more hazardous condition is reported in the results and will be mapped.

Table 8: Structures in Humboldt County

Location			Transect	Structure	Notes	Treatment Option
Description	Latitude	Longitude				
Shelter Cove Boat Launch	40.023	-124.067	NA	Revetment (Jetty)	The revetment at the Shelter Cove boat launch protects the road descending from the bluff to the beach and is sheltered from wave attack due to its southeastern orientation and the presence of buffer rocks offshore; it is unlikely that this structure would be damaged during the 1-percent-annual-chance storm.	3
Mattole Road Revetment north	40.410	-124.393	38	Revetments	The revetments at Mattole Road are massive but are subject to some rock displacement during winter storms and should be investigated as partially failed and intact; Caltrans maintains these shore protection structures. Average rock size of 3 ft.	2, 3
Mattole Road Seawalls	40.408 40.411 40.413	-124.392 -124.393 -124.394	NA	Seawalls	The concrete seawall installed to protect Mattole Road is uncertified and should be investigated as both intact and failed. The seawalls were not intersected by the transect and were not analyzed directly.	1, 3
Mattole Road Revetment south	40.395	-124.375	NA	Revetments	The revetments at Mattole Road are massive but are subject to some rock displacement during winter storms and should be investigated as partially failed and intact; Caltrans maintains these shore protection structures. Approximate armor diameter of 3 feet.	2, 3
South Spit Jetty	40.755	-124.234	N/A	Jetty	The North and South Spit jetties at the entrance to Humboldt Bay are massive and maintained by the USACE so should be modeled as intact in the analyses.	3
King Salmon Jetties	40.742	-124.218	NA	Jetties	The King Salmon jetties were built by the USACE in response to the 1982-83 El Nino storms and were designed to withstand the 1-percent-annual-chance storm conditions and shall be analyzed intact.	3
Humboldt-Arcata Bay			62-81	Throughout the Bay	All structures remain in the profiles and are analyzed as intact in sheltered waters unless there is evidence to indicate that the structure will fail and should be removed.	3
North Spit Jetty	40.761	-124.229	N/A	Jetty	The North and South Spit jetties at the entrance to Humboldt Bay are massive and maintained by the USACE so should be modeled as intact in the analyses.	3
Mad River Caltrans Revetment	40.977	-124.118	NA	Revetment	The Caltrans revetment has been buried since the Mad River began its migration southward and should remain intact, provided it remains buried and unthreatened by the river.	3
Trinidad	41.055	-124.147	NA	Revetment	The revetment at Trinidad is located within a sheltered southeast-facing harbor and should remain intact.	3

NA – Not Analyzed directly; no transect bisected this structure. The treatment option is the recommended method to apply to these structures should direct evaluation be required for future studies or investigations

4.6 Overtopping

Wave overtopping occurs when a potential runup elevation exceeds a profile crest elevation. When wave runup is shown to exceed the bluff or barrier crest in a flood hazard study, wave overtopping is evaluated to determine the depth of overtopping, the extent of high-velocity overtopping, and the inland extent of overtopping flow. The Pacific Guidelines recommend using the Cox-Machemehl (1986) method (C-M method) to determine these values for splash and bore overtopping.

Depending on the height of the potential runup, measured with respect to the DWL2% and the barrier crest, overtopping will occur as either bore overtopping or splash overtopping. Hazards associated with wave overtopping can be linked to several parameters:

- Mean overtopping discharge, q;
- Overtopping flow depth, h, at distance, y, landward of the crest; and
- Landward extent of bore and splash overtopping, $y_{G\ outer}$

4.6.1 1-Percent-Annual-Chance Overtopping Conditions

The required input parameters for the C-M method are the TWL, the wave period, and the DWL2%. Overtopping depths and extents are closely related to the TWL. The 1-percent TWL is a direct product of the wave runup and subsequent extreme value analysis and is readily available for use in calculating overtopping. For transects where all events used in the statistical analysis result in overtopping, the C-M method can be applied for all events to directly determine the 1-percent overtopping limits. However, if not all events in the statistical analysis result in overtopping, the C-M method is more difficult to implement because the 1-percent TWL is a statistical value and is not associated with a specific wave period or DWL2%. Therefore, the mean T_p and the maximum DWL2% associated with the 50 annual maximum TWLs were chosen in these instances for use with the 1-percent TWL to estimate the 1-percent overtopping hazard. Overtopping rates were also calculated to help quantify the 1-percent-annual-chance overtopping condition.

4.6.2 Overtopping Calculations

The ratio of the runup height above the DWL2% to the freeboard, R'/z_c', dictates whether overtopping is classified as bore overtopping or splash overtopping. Bore overtopping represents the special case where the volume of water associated with an overtopping event is so great that a mass of water is expected to propagate landward of the barrier for some distance before dissipating. Splash overtopping is the more common occurrence of water passing landward of the barrier, without the presence of coherent overland flow. Bore overtopping occurs for values of R'/z_c' greater than or equal to 2, while splash overtopping occurs when this ratio is less than 2. Figure 14 illustrates splash overtopping, with bore propagation landward of splashdown. The variables involved in determining the limits of overtopping and the hazard zones landward of the barrier crest are also shown.

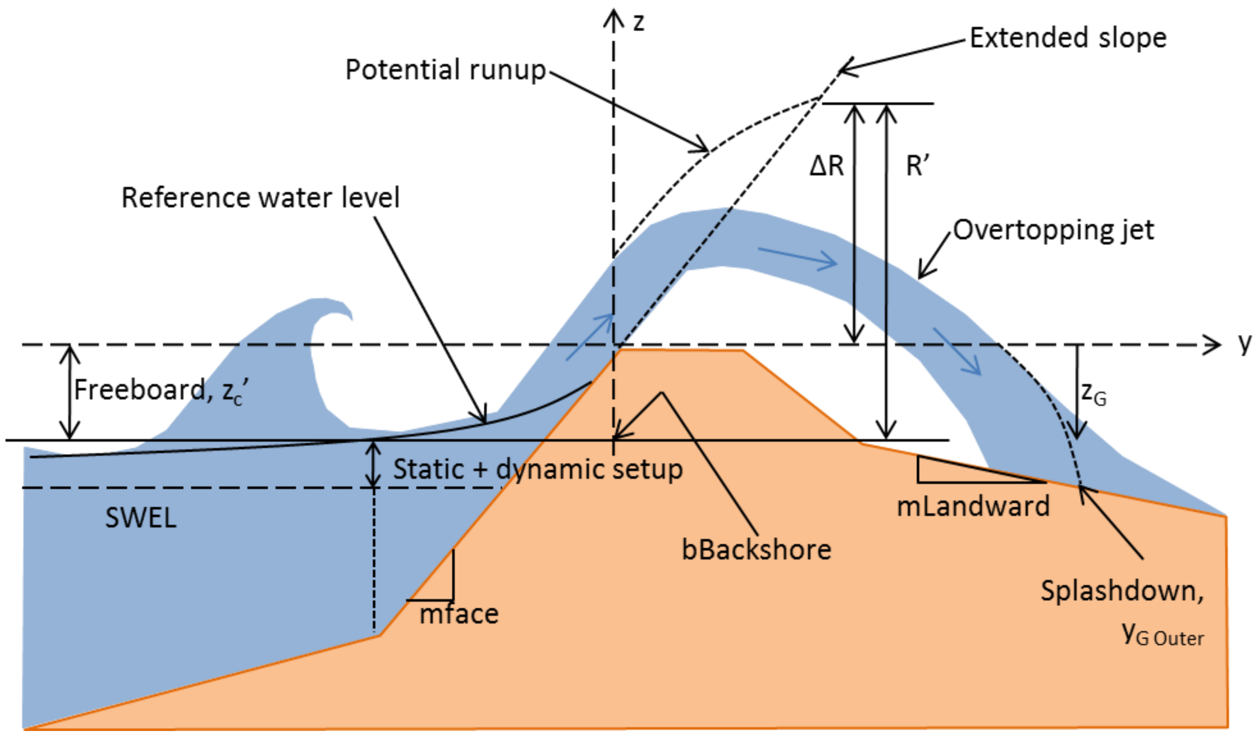

Figure 14: Illustration of Splash Overtopping and Associated Variables (Modified from Pacific Guidelines (FEMA, 2005))

The landward limit of the VE zone, defined as $hV^2 = 200\ ft^3/sec^2$ where h is the water depth and V is a uniform velocity, was computed for splash and bore overtopping following the guidance in the Pacific Guidelines. One correction was made to the coefficient used in computing the initial splashdown depth, h_0. A coefficient of 0.38 was used in place of 0.19 to be consistent with the use of a Froude number of 1.8 and the initial depth calculation made for bore overtopping. The following algorithm was derived from Figure D.4.5-15 in the Pacific Guidelines to allow for automation of the calculation of the outer limit of the splash region, $y_{G\ outer}$:

$$y_{G\ outer} = \frac{(V_c cos\alpha_{face})'}{g} * (V_c sin\alpha_{face} - mLandward * (V_c cos\alpha_{face})') * \left\{1 + \sqrt{1 - \frac{(2 * g * bBackshore)}{(V_c sin\alpha_{face} - mLandward * (V_c cos\alpha_{face})')^2}}\right\}$$

and $$z_G = bBackshore + mLandward * y_{G\ outer} \qquad \text{(Equation 26)}$$

where V_c is the water velocity at the crest, *bBackshore* is the intercept for the landward slope adjacent to the barrier crest, *mLandward* is the landward slope, and α_{face} is the face slope in degrees ($\alpha_{face} =$

$\tan^{-1} m_{face}$) (Oregon Department of Geology and Mineral Industries, 2010). In cases of splash overtopping, the onshore wind speed of 44 ft/sec listed in the Pacific Guidelines was used to calculate an enhanced onshore water velocity. The C-M method was used to determine the landward limit of overtopping hazard areas for both bore and splash overtopping. Given the initial water depth and velocity, h_0 and V_0, the bore depth decays with distance as:

$$h(y) = \left[\sqrt{h_0} - \frac{5(y-y_0)}{A_m\sqrt{g{T_p}^2}}\right]^2 \qquad \text{(Equation 27)}$$

where y_0 is the horizontal location of the barrier crest. For flat landward slopes, A_m=1. For non-zero landward slopes, $A_m = 1 - 2.0 \times mLandward$, but is limited to the range 0.5 to 2.0.

Results from the C-M method and bore propagation distances will also be checked for reasonableness against the calculated overtopping rates, q, calculated from the equations for wave overtopping in the Pacific Guidelines Table D.4.5-7. The appropriate equation was selected for each instance of overtopping based on the structure slope and wave conditions at the site.

4.7 Sheltered Waters

Humboldt-Arcata Bay is the only embayment for which coastal wave hazards were evaluated in Humboldt County. This water body is considered a sheltered embayment for the purposes of this study because it is largely protected from exposure to Pacific Ocean swell waves by the sand spits and east-west jetties stabilizing the inlet. Despite being largely sheltered from the open coast, this embayment has sufficient fetches such that locally generated wind waves may contribute to flood hazards within the bay. In addition to locally-generated wind waves, swell propagating through the inlet directly impact the community of King Salmon and its protective jetties. Flood hazards, including those from swell propagating through the inlet and locally generated wind waves, were evaluated within Humboldt-Arcata Bay.

Humboldt-Arcata Bay is approximately 14 miles long and 4.5 miles wide and occupies an area of 24 square miles at high tide and 11 square miles at low tide (Barnhart et al., 1992). The harbor opens to the sea through a narrow and dynamic passage, which was historically fronted by shifting sand bars. The bay mouth was stabilized by 6,000-foot-long jetties completed in 1899. Storms occasionally damage the jetties, and portions were rebuilt and fortified in 1971 and 1984 by the USACE, which maintains the jetties (Tuttle, 2007). The channel has been dredged since 1881 and is currently maintained annually to -48 feet MLLW (Humboldt Bay Harbor Recreation and Conservation District, 2007).

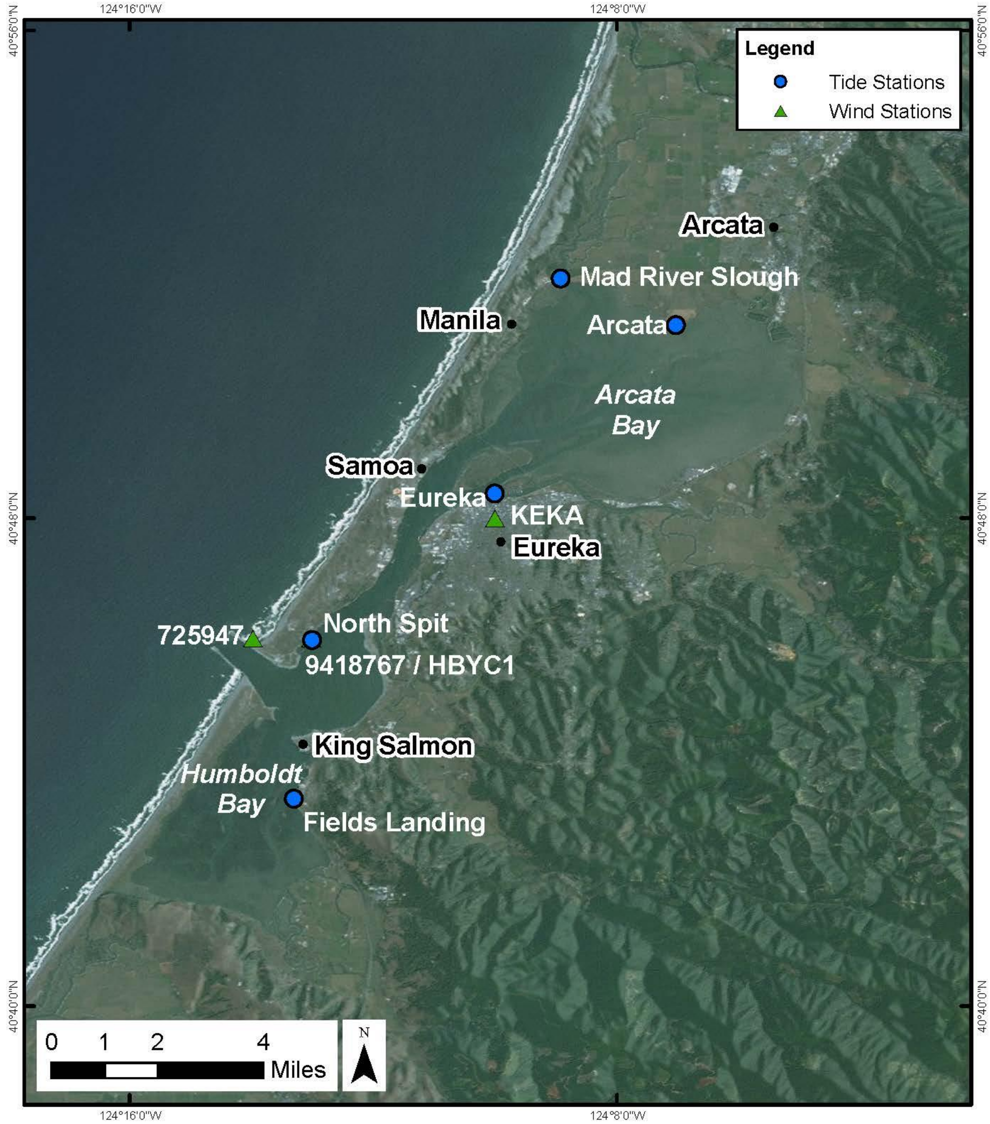

Figure 15: Humboldt-Arcata Bay

4.7.1 Locally Generated Wind Wave Hazard Analysis

Due to the nature of this embayment and the dearth of water-level and wind or wave data representative of Humboldt-Arcata Bay, a response-based analysis is not feasible for this portion of the study. A one-dimensional, event-based approach based upon the Guidance for Coastal Flood Hazard Analyses and Mapping in Sheltered Waters (FEMA, 2008) was used to evaluate coastal flood hazards within the Harbor. Static wave setup from locally generated waves was calculated using the DIM. Wave runup and overtopping were calculated using methods similar to those used on the open coast.

4.7.1.1 Wind Analysis

An investigation of the coincidence of water levels and wind events in the sheltered waters of Humboldt-Arcata Bay was completed and confirmed that elevated water levels are decoupled from wind events. This investigation involved inspecting the timing of all wind events over 30 knots at the Eureka KEKA station and reviewing the SWL time series at the Humboldt Bay, North Spit gauge at these times. For all wind events, the SWL values ranged from -1.3 to 6.5 feet NAVD88. For reference, the 1-percent-annual-chance SWEL for the Humboldt Bay, North Spit gauge is 10.35 feet NAVD88. This investigation also involved inspecting maximum tidal events in the SWL time series and reviewing the associated wind speeds, which also showed no correlation. This supported the decision to pair the 1-percent-annual-chance SWEL with a "representative" wind speed determined from these analyses. For each transect, starting wave conditions were generated using parametric wave growth equations and this representative wind speed. The starting wave condition was then paired with the 1-percent-annual-chance water level and event-based TWLs were calculated for each transect.

Based on a review of the available wind station data identified in Table 5, it was determined that the Eureka KEKA station is most representative of wind conditions in Humboldt-Arcata Bay, given its location and relatively continuous record (Figure 15). The Eureka KEKA station has hourly wind direction and speed data from 1949 to present. Wind data from 1949 to 2012 was analyzed for use in developing wave conditions. Wind roses for the KEKA station and others in the Humboldt-Arcata Bay region are included in Appendix B.

In Humboldt-Arcata Bay, winds blow predominantly from the north to northwest for approximately three quarters of the year as the semi-permanent high pressure settles over the Pacific Ocean to the west of Eureka. During the winter, the winds are generally from the south. Based on this seasonal data and inspection of the directional distribution of wind events over 30 knots at the KEKA station, southerly and north-northwesterly winds were selected for use in generating starting wave conditions.

In addition to the wind data from the KEKA station, the May 2013 Pacific Gas and Electric Humboldt Bay Power Plant License Termination Plan, Chapter 8 Supplement to the Environmental Report (Pacific Gas and Electric Company, 2013) states that the 50-year return period for a 1-minute average wind speed is 58 mph. This wind speed was converted to a 3-hour-average wind speed of 43.4 mph using methods outlined in the Coastal Engineering Manual (USACE, 2006) to represent an appropriate wind speed for wind wave growth.

Engineering judgment was used in each situation where a wind input was required, to ensure the selected value was reasonable for the analysis requirements. Based on the criteria and data described in this section, 45 mph was chosen for the analysis.

4.7.1.2 Starting Wave Conditions

The Automated Coastal Engineering System (ACES) was utilized to estimate wind-generated wave growth for use as the starting wave conditions at each transect (Leenknecht et al., 1992). The wind adjustment and wave growth analysis for the shallow, restricted wind fetches module was used to calculate a weighted wave height and period for fetches where the predominant wind direction differs from the maximum fetch.

The input information for ACES, for a wind adjustment and wave growth analysis for shallow restricted wind fetches, includes wind speed, elevation and angle of the wind observation, fetch length, and average water depth along the fetch. At a point of interest, radial fetches were taken from various points along the shoreline of the embayment to each modeled transect at approximately 20 degree increments. The average water depth along a fetch was calculated using a baseline equal to the 1-percent-annual-chance SWEL. For Humboldt-Arcata Bay, winds approaching a point of interest from the north-northwest and south were tested. The wind direction that produced the greatest weighted wave height and period was used in the wave hazard analysis. A weighted wave height, period, and direction were calculated for each point of interest (Figure 16). The weighted wave direction calculated by ACES is a function of fetch length and fetch and wind direction. Inputs and results from the ACES analyses are presented in Appendix C.

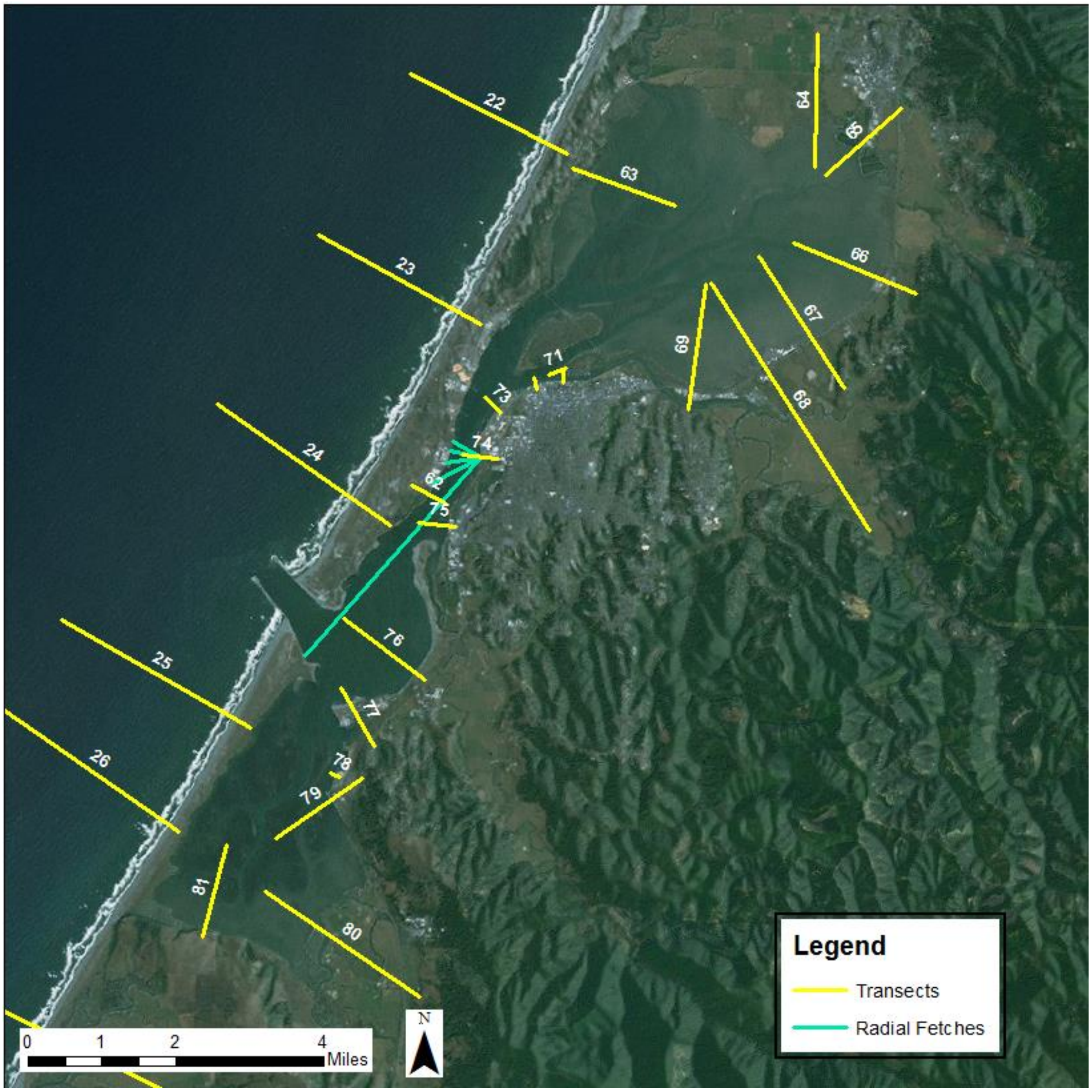

Figure 16: Example of Radial Fetches for ACES Analysis at Transect 74

4.7.2 Swell Propagation through Inlet

Wave energy propagating through the entrance channel was considered for the community of King Salmon, which is located directly across from the inlet, and the contiguous reaches. The orientation of the entrance channel relative to King Salmon makes it susceptible to both locally-generated wind waves and swell from the Pacific Ocean. Incident ocean waves often align (wave crest perpendicular) with the channel entrance, and the alignment of the jetties and the offshore bar focus the waves into the entrance bay on the southeast end of the entrance channel (Costa, 2002). Extensive erosion has occurred in the entrance bay and much of the shoreline in the vicinity of Transect 76 is now heavily armored. To account

for swell propagation through the Humboldt Bay entrance channel, wave height decay down the jettied entrance channel was described by an exponential decay (Dalrymple, 1992). Wave diffraction was then computed within the entrance bay to transition the waves from the jettied channel to King Salmon and the surrounding shorelines (Penney and Price, 1952).

For the case where wave length is long with respect to the channel width, as is the case at the Humboldt jetties, wave height decay down the channel can be described by an exponential decay,

$$H = H_0 e^{-\Gamma x} \qquad \text{(Equation 28)}$$

where H_0 is the significant wave height at the jetty entrance, x is a distance along the channel, and H is the wave height at distance x. $\Gamma = \gamma_r/2bk = \beta_r/2b$, where γ_r is the real part of a damping factor, $k = 2\pi/L$ is the wave number, $2b$ is the width of the channel, and β_r is the real part of the specific admittance of the jetties, a measure of the jetties' capacity to dampen wave energy. The equivalent admittance is defined as $sin\,\vartheta$ when β is real, where ϑ is the angle of the wave direction to the channel axis. Dalrymple (1992) successfully applied these equations to observations from a case study at Mission Bay, CA where waves typically approaching normal to the inlet with periods of 3 to 20 seconds, similar to those at the Humboldt Bay jetties, were observed. Values of ϑ in the case study were found to be between 8 and 11 degrees. Given this range of angles, a conservative value of 8 degrees was selected for ϑ in the Humboldt Bay entrance channel.

Starting wave conditions for H_0 were selected from SHELF point HU540 situated just outside the 1800 foot wide jetty entrance. The full 50 year hourly hindcast from HU540 was used for the swell propagation analysis. This wave data was paired with the matching hourly water level time series from the Humboldt Bay, North Spit tide gauge. The channel decay distance, x, extending the length of the jettied entrance channel in Figure 17 was 6300 feet.

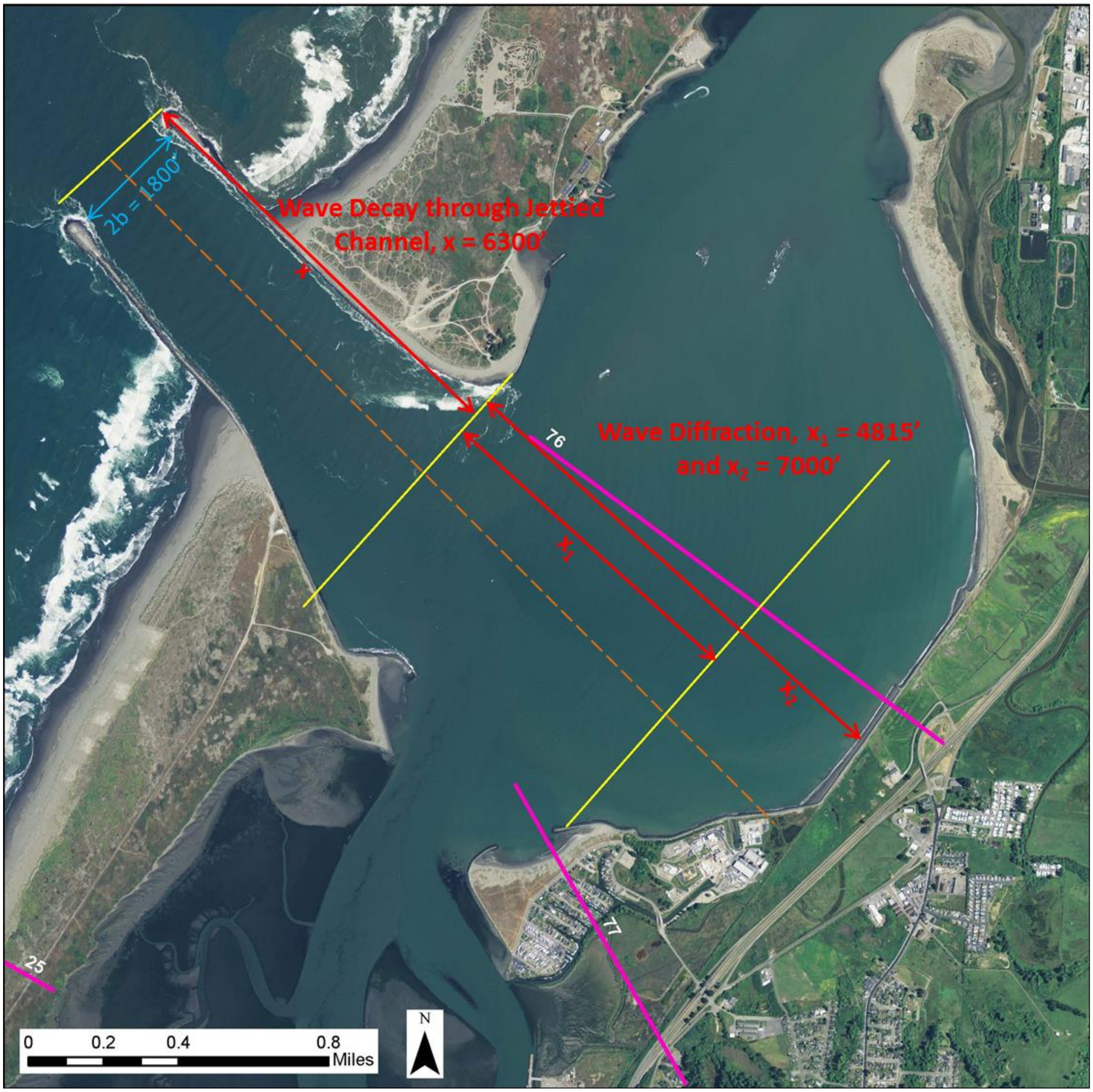

Figure 17: Definition of Parameters used in the Swell Propagation Analysis

As the channel constriction opens to the entrance bay, the propagating waves diffract. This situation was idealized as waves passing through a breakwater gap following Penny and Price (1952) where the diffraction coefficient, the ratio of the diffracted wave height to the incident wave height, *K'*, is defined as:

$$K' = \frac{2b}{\sqrt{Lx}}\left\{1 + \frac{\pi^2}{18L^2x^2}\left(\frac{1}{4}(2b)^2\right)\right\} \quad \text{(Equation 29)}$$

where *2b* is the width of the breakwater gap, *L* is the local wavelength, and *x* is the distance of wave propagation from the breakwater gap. For Transects 76 and 77, the wave propagation distances were 7000 and 4815 feet, respectively. The incident wave height, *H*, used in the diffraction analysis was the wave height at distance *x* from the wave height decay down the jettied channel in Equation 28. In addition to the wave decay and diffraction analyses, the input wave at HU540 was also shoaled to the end of the diffraction analysis. The resultant wave height was used as input to the sheltered waters wave runup analysis for Transects 76 and 77 in the vicinity of King Salmon and the revetted shoreline across from the entrance channel.

4.7.3 Wave Runup

As described in Section 4.1, potential runup is a function of SWEL, starting wave conditions and slopes along the transect. Deviations to the open coast runup methodology for sheltered embayments include the following: a) the nearshore slope, m_{DIM}, is taken as the average slope between the landward limit of SWEL plus static wave setup and the location offshore where the water depth is two times the depth at which the significant wave height would be subject to depth-limited breaking, b) wave height and period values were used directly in the runup calculations and no additional shoaling or refraction computations were completed, and c) the static setup reduction factor, γ_η, used with the TAW runup method was set to zero if waves had not broken seaward of the toe. Runup results are presented in Table 10.

4.7.4 Overland Wave Propagation

Overland wave propagation modeling was conducted using FEMA's Wave Height Analysis for Flood Insurance Studies (WHAFIS) model, Version 4 (FEMA, 1988; Divoky, 2007) for transects with gently sloping profiles where the prevailing ground is inundated by the SWEL flood level plus static wave setup. WHAFIS solves the wave action conservation equation and incorporates wind-generated wave growth and dissipation by marsh grasses. Rigid blockages to wave growth, such as buildings or rigid vegetation, are included within the formulations. The basic input information required by WHAFIS is SWEL plus static wave setup, wave and wind conditions, ground elevations, and land-use classifications with corresponding vegetation or building parameters.

In Humboldt-Arcata Bay, the predominant wind directions are north-northwest and south, seasonally. The maximum fetch for transects in Humboldt-Arcata Bay is not always aligned with the direction of the maximum winds. Therefore, for transects in sheltered waters modeled using WHAFIS, a weighted radial fetch approach that accounts for wind directions that differ from the fetch orientation was used in ACES to determine the wave height and associated period.

No vegetation parameters were used along any of the four WHAFIS transects, only inland fetch and above-surge parameters. The starting station of WHAFIS transects was taken to be the location where the

natural ground elevations equal the MSL. Tidal datums, including MSL, are given in Table 4. WHAFIS results are presented in Table 12.

5 Results

Section 4 summarized the coastal flood hazard analysis methodologies for runup, erosion, structure failure, and overtopping at open coast transects and for runup and overland wave propagation in sheltered waters. Here those results are compiled to provide a summary of the dominant flood hazard at each transect.

5.1 Open Coast Analysis

For each of the 61 open coast transects, the 50 maximum annual TWL events are provided in Appendix D. For transects with structure failure, results from the intact and failed profiles are given. For transects with erosion, results from the non-eroded profile or the MLWP are also included. The 1-percent-annual-chance TWLs for each transect, resulting from the TWL extreme value analysis, are summarized in Table 9. For all transects, the fit of the resulting cumulative distribution function to the annual maxima was evaluated for the maximum likelihood (ML) and probability weighted moments (PWM) solutions. In most cases, the ML method best fit the annual maxima data. For transects where this was not the case, the PWM solution was chosen instead. For transects with erosion or structure failure, the 1-percent-annual-chance TWLs from the worst case 1-percent TWL configuration are shown with the corresponding intact profile (intact), failed profile (failed), Most Likely Winter Profile (MLWP), or final eroded profile (final_eroded) case listed in the 'Transect' column. The mean runup slope was calculated from the runup slopes associated with the 50 annual maximum TWLs. These slopes represent the actual profile slopes where runup is present and are not capped. The roughness reduction factor for each transect was defined based on shore type. The TAW vertical wall reduction factor was used in cases where the runup slope exceeded 45 degrees and the TAW runup method was used in the annual TWL maxima output. The vertical wall reduction factor reported in Table 9 is the average of the vertical wall reduction factors used in the 50 annual maxima. The reduction factors for berms and porosity were set to 1 for all transects, and the angle of wave attack reduction factor changed with each time step, based on the refracted wave direction at the toe. Table 9 also indicates whether the structure or bluff is expected to be overtopped by the 1-percent wave runup event, that is, whether the 1-percent TWL exceeds the crest of the structure or bluff. Transects where an erosion analysis was conducted are noted as are transects where a PFD was identified. Results from the erosion analysis are provided in Appendix E. The location of the primary frontal dune heel may control the mapping of the VE zone if it is located farther landward than the VE zone limits calculated from wave runup and overtopping.

Open coast 1-percent-annual-chance TWLs ranged from 14.8 feet to 42.2 feet. Changes in TWL between transects were a result of transect exposure and orientation, shore type, profile steepness, and the use of reduction factors to account for structure roughness and steepness greater than 45 degrees.

Table 9: 1-Percent-Annual-Chance TWLs and Mean Runup Slopes for the 61 Open Coast Runup Transects

Transect	Stillwater Elevation Gauge No.	SHELF Input	Shore Type	Mean Runup Slope	γ_v	Erosion	PFD	1% Annual Chance TWL (ft, NAVD)	Over-topping
1	9419750	HU968	Sandy Beach / Dune / Bluff	0.02	-	-	-	17.6	-
2	9419750	HU942	Sandy Beach / Dune / Bluff	0.01	-	-	-	18.2	-
3	9419750	HU919	Bluff	0.09	-	-	-	19.1	-
4	9419750	HU895	Dune	0.02	-	-	YES	17.9	-
5_intact	9419750	HU888	Sandy Beach with Roadway	0.02	-	YES	YES	18.5	-
6_intact	9419750	HU874	Dune	0.02	-	YES	YES	18.0	-
7	9419750	HU865	Bluff	0.01	-	-	-	17.8	-
8	9419750	HU838	Dune	0.04	-	-	YES	18.3	-
9	9419750	HU821	Bluff	0.03	-	-	-	18.4	-
10	9419750	HU817	Bluff	0.05	-	-	-	18.9	-
11	9418767	HU785	Bluff	0.12	-	-	-	24.3	-
12	9418767	HU756	Bluff	0.06	-	-	-	22.0	-
13	9418767	HU747	Bluff	0.04	0.93	-	-	18.1	-
14	9418767	HU728	Bluff	1.07	0.97	-	-	20.9	-
15	9418767	HU728	Bluff	1.80	0.87	-	-	24.5	-
16	9418767	HU708	Bluff	5.65	0.73	-	-	25.0	-
17	9418767	HU694	Dune	0.01	-	-	YES	17.2	-
18_final_eroded	9418767	HU679	Dune / Bluff	0.01	-	YES	YES	17.8	-
19	9418767	HU670	Sandy Beach / Bluff	0.01	-	-	-	17.6	-
20	9418767	HU660	Sandy Beach	0.01	-	-	-	17.7	YES
21_intact	9418767	HU640	Dune	0.02	-	YES	YES	19.2	-
22	9418767	HU601	Dune	0.02	-	-	YES	18.4	-
23_intact	9418767	HU580	Dune	0.49	-	-	YES	29.3	-
24_intact	9418767	HU557	Dune	0.27	-	-	YES	28.8	YES
25	9418767	HU520	Dune	0.02	-	-	YES	18.5	-
26	9418767	HU507	Dune	0.02	-	-	YES	18.7	-
27	9418767	HU480	Dune	0.01	-	-	YES	18.7	-

Transect	Stillwater Elevation Gauge No.	SHELF Input	Shore Type	Mean Runup Slope	γ_v	Erosion	PFD	1% Annual Chance TWL (ft, NAVD)	Over-topping
28	9418767	HU457	Sandy Beach	0.05	-	-	-	18.7	YES
29	9418767	HU439	Dune	0.02	-	-	YES	18.1	-
30	9418767	HU428	Dune	0.02	-	-	YES	18.4	-
31	9418767	HU414	Bluff	2.98	0.79	-	-	24.7	-
32	9418767	HU400	Bluff	0.02	-	-	-	19.1	-
33	9418767	HU369	Bluff	0.03	-	-	-	19.6	-
34	9418767	HU359	Bluff	0.01	-	-	-	19.9	-
35	9418767	HU351	Bluff	0.01	-	-	-	20.3	-
36	9418767	HU340	Bluff	0.45	0.96	-	-	25.7	-
37	9418767	HU330	Bluff	0.01	-	-	-	19.7	-
38_failed	9418767	HU319	Revetment	0.50	-	-	-	32.7	YES
39	9418767	HU289	Dune/Bluff	0.02	-	-	YES	20.0	-
40	9418767	HU272	Bluff	0.02	-	-	-	22.1	-
41	9418767	HU254	Bluff	0.04	-	-	-	14.8	-
42	9418767	HU246	Dune	0.03	-	-	YES	19.9	-
43	9418767	HU230	Bluff	0.02	-	-	-	21.9	-
44	9416841	HU220	Bluff	0.03	-	-	-	23.2	-
45	9416841	HU198	Bluff	0.10	-	-	-	24.4	-
46	9416841	HU160	Bluff	0.04	-	-	-	22.1	-
47	9416841	HU136	Bluff	0.03	-	-	-	20.0	-
48	9416841	HU118	Bluff	0.05	-	-	-	22.0	-
49	9416841	HU104	Bluff	0.14	-	-	-	26.5	-
50	9416841	HU095	Bluff	2.69	0.78	-	-	30.1*	-
51	9416841	HU083	Bluff	0.03	-	-	-	19.9	-
52	9416841	HU155	Bluff	0.06	-	-	-	19.0	-
53	9416841	HU047	Bluff	0.09	-	-	-	18.9	-
54	9416841	HU043	Bluff	0.04	-	-	-	19.0	-
55	9416841	HU041	Bluff	1.99	0.86	-	-	37.2	-
56	9416841	HU036	Bluff	0.41	-	-	-	37.4*	-
57	9416841	HU031	Bluff	1.12	0.97	-	-	42.2	YES
58	9416841	HU027	Bluff	0.57	0.98	-	-	30.7*	-
59	9416841	HU026	Bluff	0.61	-	-	-	32.6	-
60	9416841	HU014	Bluff	0.03	-	-	-	19.0	-

Transect	Stillwater Elevation Gauge No.	SHELF Input	Shore Type	Mean Runup Slope	γ_v	Erosion	PFD	1% Annual Chance TWL (ft, NAVD)	Over-topping
61	9416841	HU007	Bluff	0.24	0.99	-	-	26.4	-

*1% Annual Chance TWL based on GEV/Probability Weighted Moments (PWM) since PWM provided better statistical fit than Maximum Likelihood (ML) method.

The 1-percent TWL exceeded the backshore structure, bluff or dune crest at 5 of the 44 open coast transects, resulting in wave overtopping. Figure 18 shows an example of runup and overtopping results.

Table 10 presents the results of the calculated splashdown distances (y_{Gouter}) and the landward extent of the flow where $hV^2 = 200\ ft^3/sec^2$, approximating the limits of the V zone, and where $h=0$, approximating the limit of the A zone. As the table shows, most splashdown and bore propagation distances ended near to the barrier crest, producing narrow flood zones. In fact, for some transects, the flow hV^2 was initially less than $200\ ft^3/sec^2$ at the barrier crest or upon splashdown (e.g., 24).

Table 10 includes all transects for which the barrier crest elevation was exceeded by the TWL during any of the 50 TWL annual maxima events or the 1-percent-annual-chance TWL. The number of TWL annual maxima events for which this criterion was met is listed as "Number of Wave Overtopping Events." The overtopping limits and overtopping rate were calculated for each overtopped transect using the methods described in Section 4.6.2.

In addition to TWLs determined for the base (1-percent-annual-chance) flood, an extreme value analysis of the annual maxima was conducted to determine the 50-percent-, 20-percent-, 10-percent-, 4-percent-, 2-percent-, and 0.2-percent-annual-chance flood elevations. The TWLs for these frequencies are presented in Appendix F.

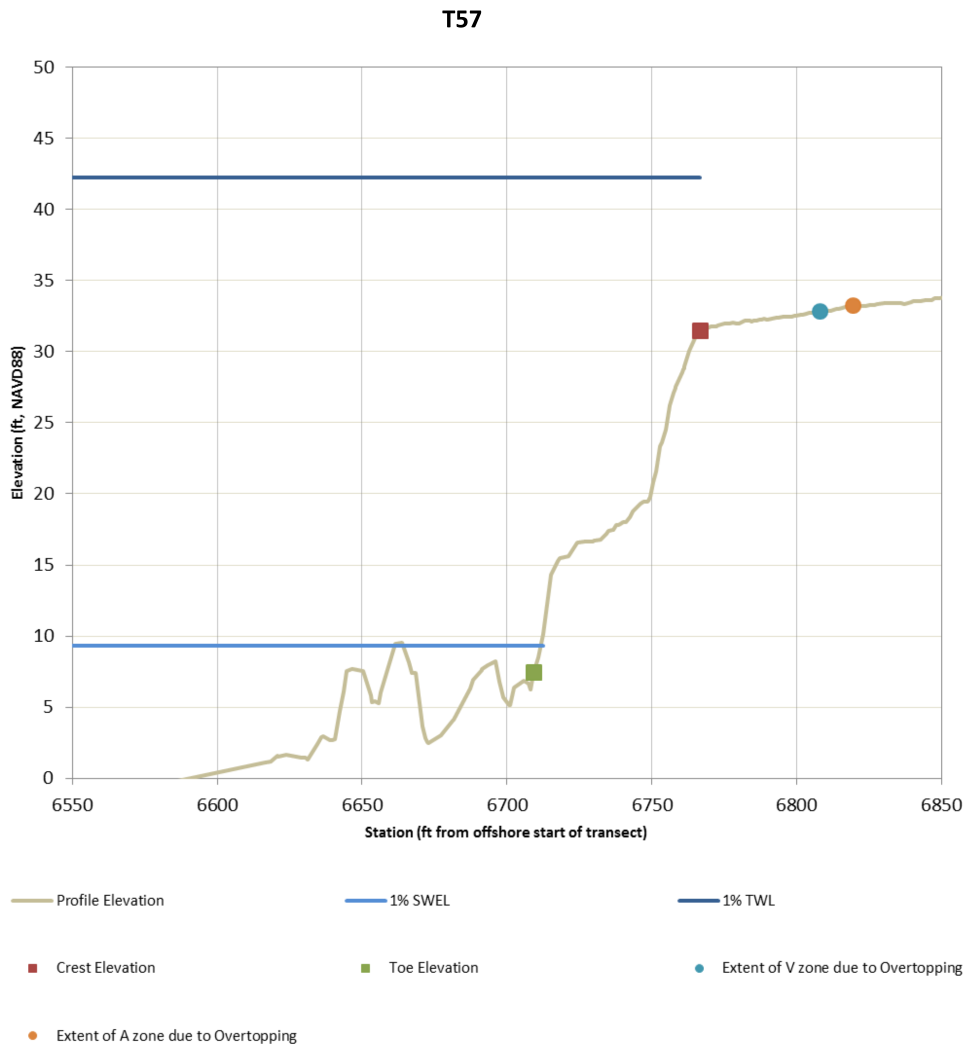

Figure 18: Example of Runup and Overtopping Results Showing the 1-percent Runup Elevation and Overtopping Limits for Transect 57

Table 10: Overtopping Rates and Splashdown and Hazard Zone Limits for the 1-Percent-Annual-Chance TWLs at Overtopped Transects

Transect	Number of Wave Overtopping Events	Mean Tp (s)	DWL2% (ft, NAVD88)	1% TWL (ft, NAVD88)	R'/zc'	Overtopping Type	Maximum Splashdown, yGouter (ft)	Bore Propagation Distance from yGouter to hV2=200 (ft)	V Zone Limit from Crest (ft)	Bore Propagation Distance from yGouter to h=0 (ft)	A Zone Limit from Crest (ft)	Crest Elevation (ft, NAVD88)	Am	zG (ft)	Overtopping Rate, q (cfs/ft)
20*	7	15.15	18.35	17.74	0.25	Inundated	-	-	-	-	-	15.70	-	-	-
24_intact	2	15.28	18.66	28.82	1.48	Splash	0.00	0.00	0.00	16.52	16.52	25.55	1.15	1.45	0.78
28*	1	15.37	18.58	18.66	-0.07	Inundated	-	-	-	-	-	17.39	-	-	-
38_failed	3	14.82	19.52	32.66	1.23	Splash	6.23	0.00	6.32	17.47	23.79	30.18	1.05	-0.12	0.28
57	9	13.99	19.58	42.19	1.91	Splash	21.52	20.11	41.63	30.61	52.13	31.43	0.96	0.05	<0.01

*The dynamic water level exceeded the crest elevation at this transect such that the crest was inundated.

5.2 Sheltered Waters Analysis

For each of the 20 sheltered waters transects, the maximum TWL calculations for analyzed fetches are provided in Appendix G. Also included are the results of the swell propagation and runup analyses at Transects 76 and 77. Because the swell analysis was run in a response-based method, the output in Appendix G for Transects 76 and 77 contains 50 annual maxima and the statistical 1-percent TWL value for each transect. The 1-percent-annual-chance TWLs for each transect, resulting from the analyses are summarized in this section. For transects analyzed using the wave runup methods described in this report, the highest 1-percent-annual-chance TWLs are shown (

Table 11). For transects analyzed using WHAFIS, the controlling wave crest elevations at Station 0 are shown (Table 12). Table 13 includes transects for which the barrier crest elevation was exceeded by the calculated 1-percent TWL. As with the open coast analysis, the overtopping limits and overtopping rate were calculated for each overtopped transect using the methods described in Section 4.6.2. The wave height will determine flood zone designations for mapping purposes at WHAFIS transects: Zone VE for areas with wave heights 3 feet or greater, and Zone AE for wave heights less than 3 feet. The 1-percent TWL and controlling wave crest elevations will determine the BFEs.

Table 11: Sheltered Waters Runup Results

Transect Number	Shore Type	Runup Slope	Reduction Factors		1% TWL (ft NAVD)
			γ_r	γ_v	
63	Mudflats	0.26	1.0	-	13.3
65	Beach / Earthen Berm	0.36	1.0	-	14.1
66	Revetment	0.51	0.6	-	14.0
67	Mudflat	0.05	1.0	-	10.9
68	Earthen Berm	0.53	1.0	-	17.6
69	Marsh	0.03	1.0	-	10.7
70	Bluff	0.51	1.0	-	14.8
71	Revetment	0.44	0.6	-	11.1
72	Vertical Wall	-	1.0	-	12.1
73	Revetment	0.40	0.6	-	12.2
74	Beach	0.33	1.0	-	13.3
75	Revetment	0.44	0.6	-	11.9
76_swell	Revetment	0.48	0.6	-	18.1
77_swell	Dune	0.04	1.0	-	13.0
79	Beach / Rip Rap	0.38	0.6	-	12.8
81	Mudflat / Bluff	0.37	1.0	-	14.9

Table 12: Sheltered Waters WHAFIS Results

Transect Number	Shore Type	1% SWEL (ft NAVD)	Static Wave Setup (ft)	1% Wave Crest Elevation (ft NAVD)
62	Beach	10.2	0.11	11.6
64	Marsh / Earthen Berm	10.2	0.18	13.2
78	Beach	10.2	0.25	12.4
80	Mudflat / Earthen Berm	10.2	0.17	13.0

Table 13: Overtopping Rates and Splashdown and Hazard Zone Limits for the 1-Percent-Annual-Chance TWLs at Overtopped Transects in Sheltered Waters

Transect	Mean Tp (s)	DWL2% (ft, NAVD88)	1% TWL (ft, NAVD88)	R'/z_c'	Overtopping Type	Maximum Splashdown, y_{Gouter} (ft)	Bore Propagation Distance from y_{Gouter} to hV^2=200 (ft)	V Zone Limit from Crest (ft)	Bore Propagation Distance from y_{Gouter} to h=0 (ft)	A Zone Limit from Crest (ft)	Crest Elevation (ft, NAVD88)	Am	z_G (ft)	Overtopping Rate, q (cfs/ft)
65	2.39	10.16	14.13	1.16	Splash	3.08	0.00	3.08	1.86	4.94	13.60	1.20	-0.31	<0.01
67	2.90	10.35	10.91	41.36	Bore	0.00	0.00	0.00	2.01	2.01	10.37	1.35	-	0.06
70	2.23	10.16	14.78	3.77	Bore	0.00	0.00	0.00	3.02	3.02	11.38	1.05	-	0.31
72	1.90	10.16	12.07	1.33	Splash	4.33	0.00	4.33	1.20	5.54	11.59	1.01	-0.32	0.01
73	2.26	10.16	12.23	1.59	Splash	2.12	0.00	2.12	1.34	3.46	11.46	1.00	0.04	0.03
74	2.33	10.16	13.26	2.04	Bore	0.00	0.00	0.00	2.09	2.09	11.68	1.02	-	<0.01
75	1.91	10.16	11.87	1.26	Splash	2.25	0.00	2.25	0.99	3.24	11.52	1.08	-0.11	0.01
76	12.85	11.09	18.05	1.06	Splash	3.63	0.00	3.63	10.60	14.23	17.65	1.26	-0.48	0.01
79	2.54	10.16	12.83	2.18	Bore	0.00	0.00	0.00	2.33	2.33	11.38	1.09	-	<0.01

6 Conclusion

Coastal hazard analyses were conducted for the Humboldt County Pacific coast and the sheltered waters of Humboldt-Arcata Bay. Coastal flooding hazards were evaluated using one-dimensional transect-based methods. Wave setup, runup, erosion, overtopping, and overland wave propagation were considered as potential flooding hazards along 81 transects. Runup was the dominant flood hazard on the open Pacific coast transects. Both runup and overland wave propagation affected the sheltered embayments.

6.1 Open Pacific Coast

Wave runup for the open Pacific coast was modeled using a response-based approach using DIM, TAW, or SPM methods, depending on the shoreline characteristics. Transects were adjusted as necessary for erosion, and structures were considered where necessary. Runup was added to the DWL to produce TWLs for each time step in the 50-year hindcast, and a statistical analysis was conducted on the annual maxima to determine 1-percent-annual-chance TWLs at each transect. The 1-percent-annual-chance runup ranged from 14.8 feet to 42.2 feet on the open coast. Wave overtopping was evaluated for five transects where the runup elevation exceeded the structure, bluff or dune crest.

6.2 Sheltered Waters

Flooding hazards in the sheltered embayments were analyzed for runup and overtopping and/or overland wave propagation. The 1-percent-annual-chance SWEL was coupled with an appropriate wave condition that was calculated based upon the wind speeds and fetch lengths specific to each studied site within the embayment. Runup was analyzed using an event-based approach to apply DIM or TAW as appropriate. Runup ranged from 10.7 to 18.1 feet in the sheltered embayments. Wave overtopping was evaluated for nine sheltered waters transects where the runup elevation exceeded the structure or bluff crest. WHAFIS was used to analyze four transects subject to overland wave propagation.

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Appendix A Transect Layout Maps

Appendix B Wind Roses

Appendix C Sheltered Waters ACES Inputs and Outputs

Appendix D Open Coast Total Water Level Tables

See Excel workbook file “Appendix D_Humboldt_Runup_Tables.xlsx.

Appendix E Erosion Results

See Excel workbook file “Appendix E_Erosion_Output.xlsx.

Appendix F Total Water Levels for All Return Periods

See Excel workbook file “Appendix F_TWL_Frequency_Table.xlsx.

Appendix G Sheltered Waters Total Water Level Tables and WHAFIS Results