A Central San Francisco Bay Coastal Flood Hazard Study

San Mateo County, California

Coastal Analysis Report

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An Integrated Production Team

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1. Introduction

The Federal Emergency Management Agency (FEMA) in Region IX is conducting a coastal engineering study of the San Francisco Bay, which includes detailed modeling and analyses of coastal hazards, as part of the California Coastal Analysis and Mapping Project (CCAMP). The results from this study will be used to re-map the coastal flood hazards in for San Francisco Bay 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 referenced as Pacific G&S.

Michael Baker Jr., Inc. (Baker) conducted coastal flood hazard analyses for the San Mateo County coastline north of the San Mateo – Hayward Bridge as a part of the Central San Francisco Bay flood hazard study (Figure 1). The portion of San Mateo County south of the San Mateo – Hayward Bridge is being studied by AECOM as a part of the Bay Area Coastal—South flood hazard study.

The coastal hazard analysis builds upon the San Francisco Bay regional-scale wave and hydrodynamic modeling conducted by DHI Water & Environment (DHI). The regional modeling of San Francisco Bay was conducted in two phases. The first phase, focused on the North and Central Bay, was completed in 2011 (DHI, 2011). The second phase, focused on the South Bay, was completed in 2012 (DHI, 2012). Results from the North/Central Bay study were used in the coastal flood hazard analysis from the northern border of San Mateo County to the northern end of the San Francisco International Airport (SFO); the South Bay results are used from south of the airport to the San Mateo – Hayward Bridge.

A copy of this report along with all of the supporting data is stored on the FEMA Mapping Information Platform (MIP). The study ID number is 11-09-1227S. An annotated submission file structure directory is provided in Appendix D. For more information on accessing the MIP contact the FEMA mapping assistance help line at 1-877-FEMA-MAP.

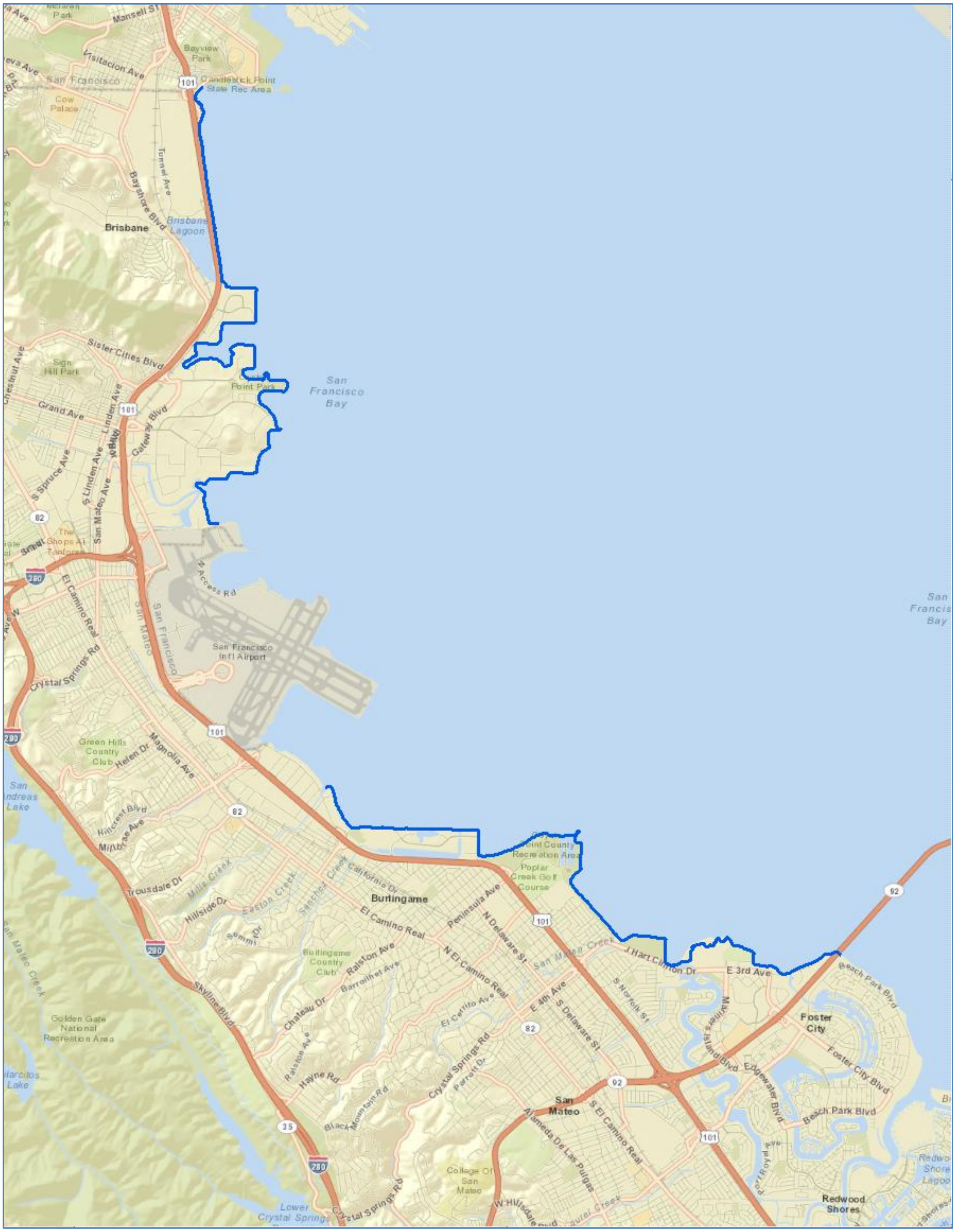

Figure 1. Central San Francisco Bay San Mateo County study area—studied shoreline shown in blue

2. Methodology Overview

Coastal flooding hazards were evaluated with one-dimensional (1D) transect-based models. Wave setup, runup, overtopping, and overland wave propagation were analyzed for 31 transects along the northern San Mateo County coastline (see Figure 2). Shore-perpendicular transects were placed with consideration of variations in topography, shoreline type, development density, land use, and incident wave conditions, with spacing between transects ranging from 500 feet to 1 mile (see Section 4. Transect Layout). All water levels and wave parameters used in the analyses come directly, or were derived from, the regional hydrodynamic and wave modeling effort for the North/Central San Francisco Bay (DHI, 2011).

The water levels from DHI included the effects of tide, storm surge, and riverine discharge. Wave setup was not included as a component of the water levels. DHI's hydrodynamic and wave modeling effort 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 1D transects were utilized to transform the waves through the surf zone.

The San Francisco Bay coast of San Mateo County is highly developed. In many areas the shoreline is hardened by engineered shore protection structures, primarily revetments, to limit or prevent erosion and to dampen wave energy. In other areas, the shoreline has been stabilized with the non-engineered placement of rubble or rip-rap. An important consideration for a flood hazard study is whether these types of shore protection structures have the ability withstand the forces associated with the base flood. Given the relatively sheltered wave environment of San Francisco Bay, it was assumed that all shore protection structures would remain predominately intact and therefore they were included in the flood hazard analysis using their full crest elevations and by recognizing the reduction in wave runup due to rubble and revetment surface roughness.

Erosion analyses were not conducted for San Mateo County since there are no dunes along the San Francisco Bay coastline.

Wave runup was calculated for transects with coastal armoring or steeply sloping ground profiles in the vicinity of the flooded shoreline. Wave runup was calculated for 28 of the 31 San Mateo County coastal flood hazard study transects, including four transects for which WHAFIS modeling was also conducted. Those four transects cross an initial shoreline on which runup is calculated and then extend across small harbors or low-lying areas where WHAFIS is used to evaluate wave regeneration and overland wave propagation. For this study, runup was calculated using one of two methods, depending on shoreline characteristics. As recommended in the Pacific G&S, 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) method (van der Meer 2002) was used for shorelines with shore protection structures and steeply sloping ($m \geq 0.125$) natural shorelines. The total runup elevation is also referred to as the total water level, or TWL. Annual TWL maxima were selected from the regional hydrodynamic and wave 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. A more detailed description of the wave runup analyses is presented in Section 5. Wave Runup.

Overland wave propagation modeling, using FEMA's 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 flood level alone. 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. Eight of the 31 transects were analyzed for overland wave propagation hazards. Of the eight transects, only transect 28 was evaluated for wave runup hazards inland along the transect. A more detailed description of the WHAFIS analysis is presented in Section 7. Overland Wave Propagation.

There are accredited and non-accredited levees along the San Mateo County coastline. Wave runup on the face of the levees was calculated for all levees. Overland wave propagation was modeled for the transects located along shorelines with non-accredited levees. The levees were left intact within the WHAFIS modeling, but the stillwater was extended behind the crest to its intersection with the prevailing ground and wave regeneration in the lee of the levee was evaluated. A more detailed discussion on the levees is presented in Section 8. Levees.

Final results from WHAFIS and wave runup analyses were merged to form a wave envelope profile of the most hazardous flood conditions expected along each transect for 1-percent-annual-chance flood conditions. This wave envelope profile defines the Base Flood Elevations (BFEs) along the transect.

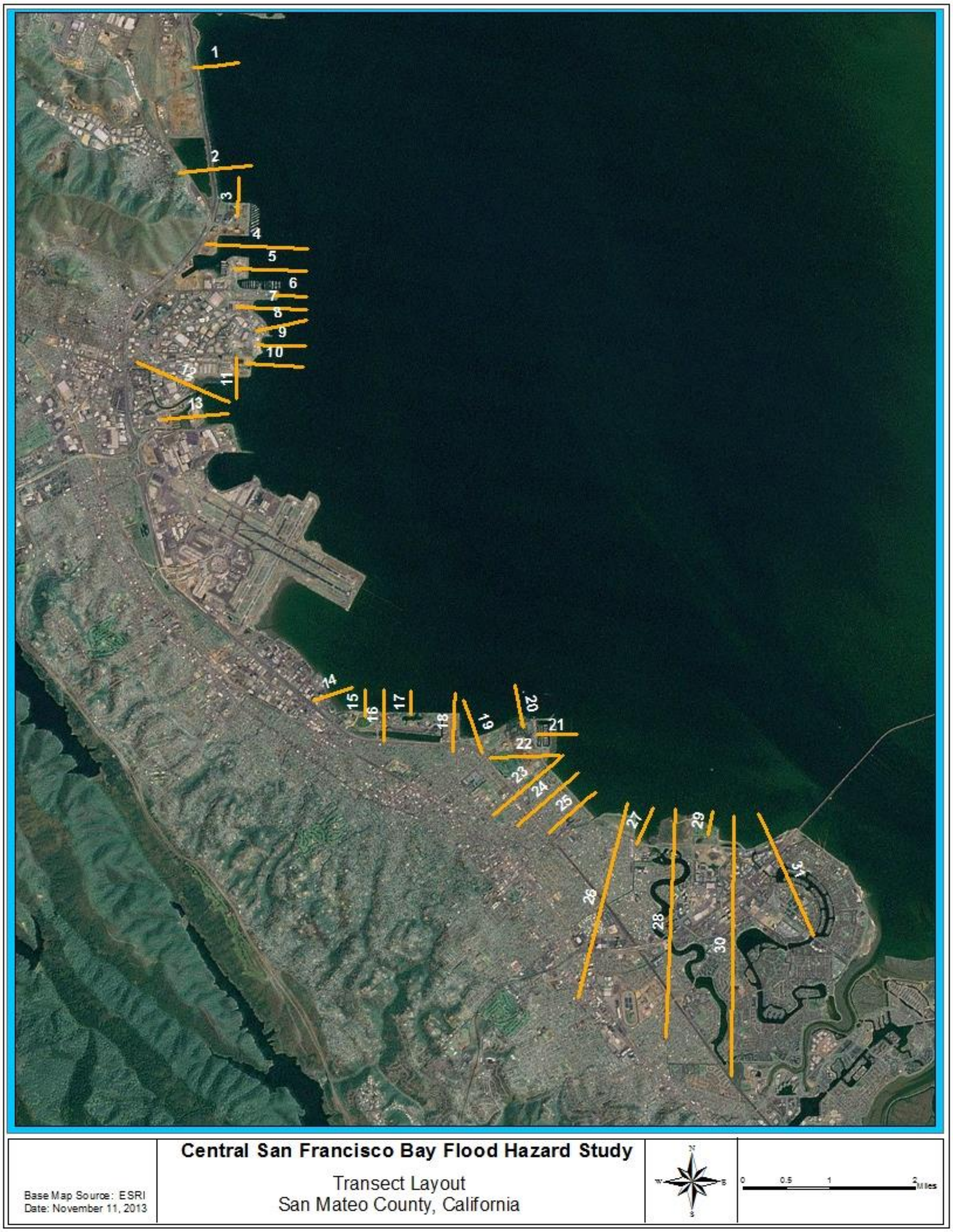

Figure 2. Northern San Mateo County Transect Layout

3. Data Sources

3.1. Topography and Bathymetry

Ground elevations were based on National Oceanic and Atmospheric Administration (NOAA) Northern San Francisco Bay Area LiDAR, collected February-April 2010. The same bathymetric data that was used in the regional hydrodynamic and wave study (DHI, 2011) was used for the San Mateo overland wave hazard analyses: 2003 U.S. Army Corps of Engineers (USACE) dredging surveys, the 2005 U.S. Geological Survey South Bay bathymetric survey, and bathymetric survey data from the NOAA/National Ocean Service Geophysical Data System. In areas where two datasets overlapped, the USACE data was given priority. See DHI, 2011 for more details on bathymetric data merging and prioritization. Figure 3 shows the bathymetric data coverage offshore of northern San Mateo County.

Levee crest elevations for transects 27 and 28 were obtained from the as-built Bayfront Levee Containment plan, entitled “City of San Mateo Bayfront Levee Profile B Alignment,” signed by Mr. Charles D. Anderson, P.E., and dated January 25, 2012.

Data from surveys performed by Wilsey Ham Civil Engineers between June 2008 and March 2011 were provided by the City of Foster City for crest elevations of the levee pedway surrounding the city. These data were used to supplement the LiDAR data to more accurately reflect the existing conditions of the levee pedway.

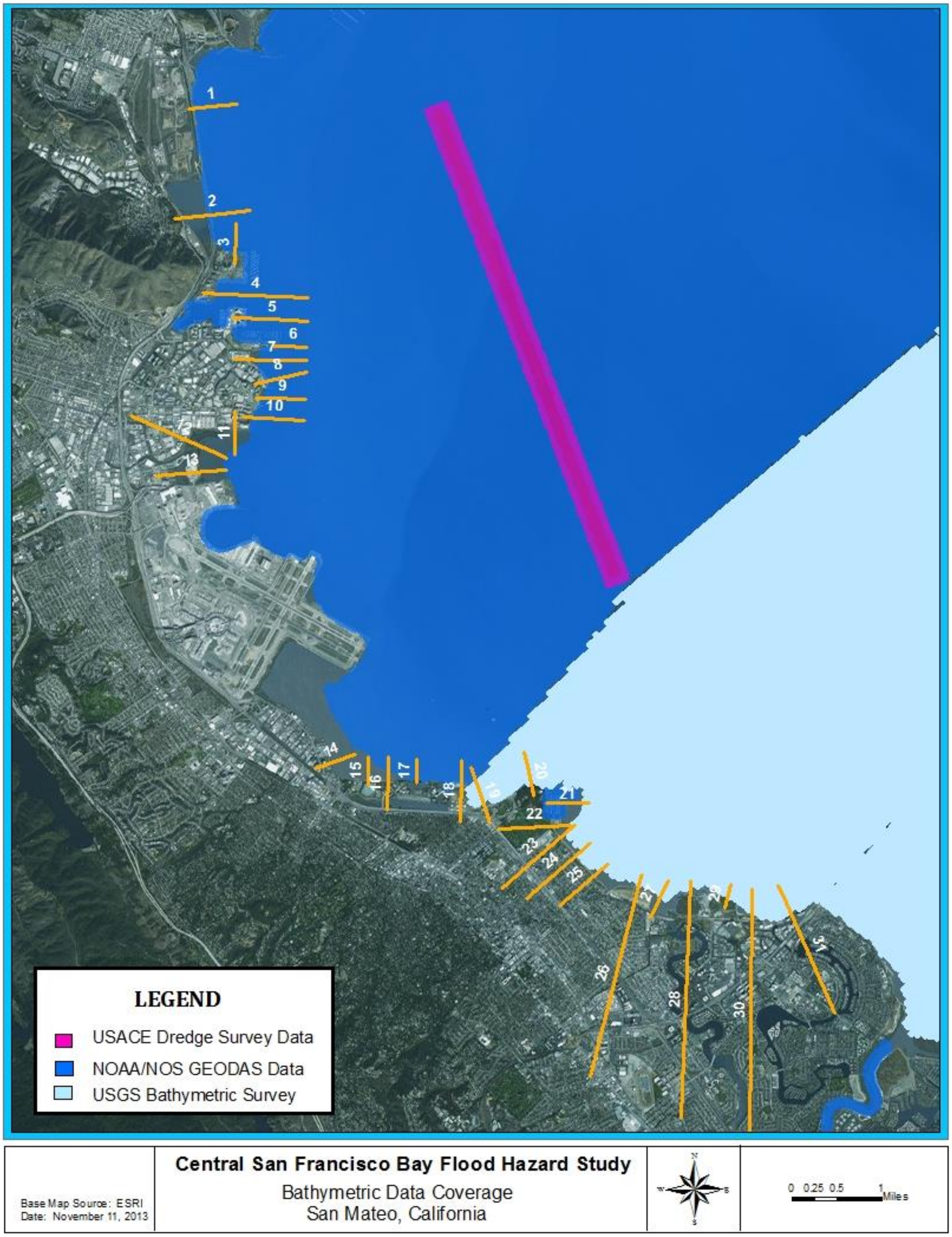

Figure 3. Bathymetric data coverage offshore of San Mateo County

3.2. Water-Level and Wave Starting Conditions

Water-level and wave conditions were obtained directly or derived from the regional hydrodynamic and wave modeling results. The regional modeling of San Francisco Bay was conducted in two phases. The first phase, focused on the North and Central Bay, was completed in 2011 (DHI, 2011). The second phase, focused on the South Bay, was completed in 2012 (DHI, 2012). Results from the North/Central Bay study were used in the coastal flood hazard analysis from the northern border of San Mateo County to the northern end of San Francisco International Airport (transects 1-13); the South Bay results were used from south of the airport to the San Mateo – Hayward Bridge (transects 14-31).

The methodologies and model setup of the two regional modeling studies were very similar. Two notable differences between the two studies are the simulation period and the wave models. The North/Central Bay study simulated a 31-year period from 1973 to 2003 and modeled both Pacific Ocean swell and locally generated wind-waves (seas). The South Bay study simulated a 54-year period from 1956 to 2009 and only modeled the locally generated wind-waves. The South Bay study did not model swell waves because swell from the Pacific Ocean do not penetrate that far south into the bay.

DHI's results were provided at pass points located approximately at the zero NAVD88 contour (the zero pass point). In areas where this contour is either too close to the shoreline or it does not exist at all (e.g. in the vicinity of harbor quays), a second line of pass points was defined 300 meters away from the normal shoreline (the 300m pass point); 300m pass points were not specified for all shoreline reaches, particularly within narrow embayments, as shown in Figure 4. The nearest zero and 300m pass points, where available, were assigned to each transect to provide inputs. Water-level data were taken from the zero pass point for each transect. Wave height data for each transect were also taken from the zero pass point, where possible. Due to the relatively coarse resolution of the regional wave model grids, the bed-level elevations near the land/water grid boundaries are not well resolved in some areas. Results at pass points with artificially high bed levels were affected by wetting/drying issues and other issues in the wave transformation calculations related to inaccurate depths, such as depth-limited breaking. Based on these observations, the wave information for transects with zero pass points with bed levels higher than 2 feet (NAVD 88) was taken from the 300m pass point. An elevation of 2 feet NAVD is between mean low water and mean sea level (MSL) in the bay and was considered a reasonable elevation to ensure that the point is wet and has sufficient depth at higher water levels to capture wave height events of interest. Table 1 contains the pass point assignments for the northern San Mateo County transects and indicates whether the wave data for both the locally generated wind waves (seas) and the Pacific Ocean swell were taken from the zero pass point or the 300m pass point.

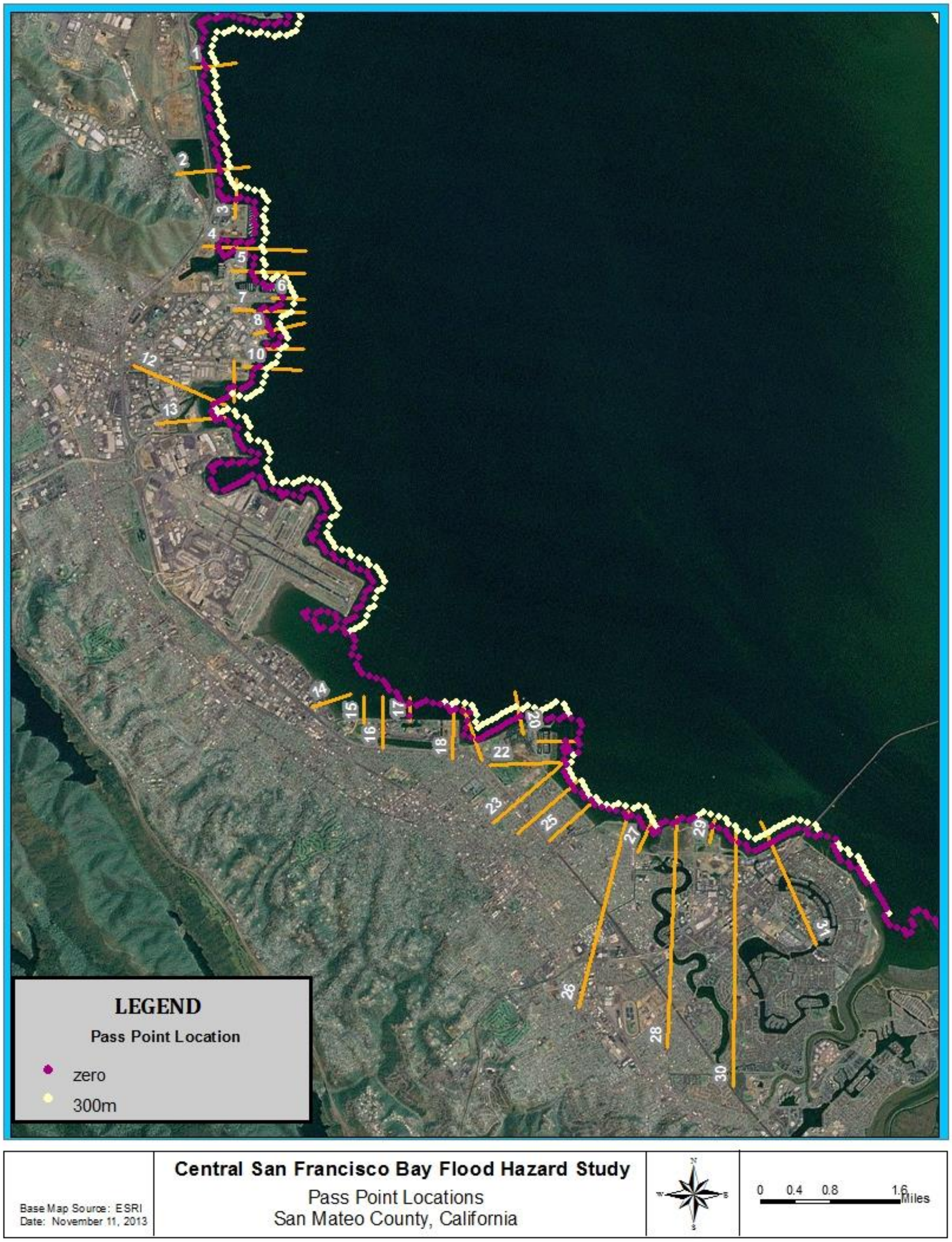

Figure 4. Pass point locations for DHI's surge and wave modeling results

Table 1. Pass Point Assignments

Transect Number	DHI Pass Point ID Number		Seas Pass Point Location	Swell Pass Point Location
	Zero	300m		
1	1088	401	0	0
2	1039	351	0	0
3	1025	342	300	300
4	1010	316	300	300
5	987	308	0	0
6	972	293	0	0
7	970	289	0	0
8	959	283	300	300
9	950	276	0	0
10	946	270	0	0
11	942	267	0	0
12	934	257	0	0
13	929	251	0	0
14	746	0	0	-
15	746	0	0	-
16	735	0	0	-
17	727	0	0	-
18	710	118	0	-
19	698	104	300	-
20	696	112	300	-
21	666	0	0	-
22	655	92	0	-
23	653	92	0	-
24	645	88	0	-
25	637	84	0	-
26	624	79	300	-
27	618	74	300	-
28	611	0	0	-
29	598	69	300	-
30	588	63	0	-
31	576	54	300	-

3.3. Wind Speed

Wind speed is an input requirement for the WHAFIS model and wave overtopping calculations. For WHAFIS modeling, this study developed wind speed values for each transect from the same wind data used in the regional hydrodynamic and wave study (DHI, 2011) to drive the storm surge and wind wave (seas) models. For overtopping calculations, a default wind speed of 30 mph was used. The Pacific G&S (FEMA, 2005) recommends 30 mph as a minimum value for splash overtopping calculations. Use of this default value was considered reasonable given the study area’s relatively sheltered exposure.

The DHI study thoroughly investigated the available wind data and chose three National Climatic Data Center wind observation stations to provide wind coverage for the bay area: San Francisco

International Airport (SFO), Oakland Metro International Airport (OAK), and Travis Field Air Force Base. These wind stations were chosen for the following reasons (DHI, 2011):

- They appear to be most representative of over-water conditions (i.e., consistently strongest);
- They cover the entire length of the hindcast period, from January 1973 to December 2003;
- They have relatively few gaps in the wind record; and
- They provided the forcing for the storm surge and wave modeling that gave the best results in terms of calibration.

The regional modeling study divided the bay into three areas and assigned a wind station to each area. The same subarea assignments were utilized for this study. Northern San Mateo County is within the subareas for both OAK and SFO; however, all WHAFIS transects fall within the SFO subarea (Figure 5). Section 7.2 provides a more detailed discussion of how wind speed values were developed for the WHAFIS modeling.

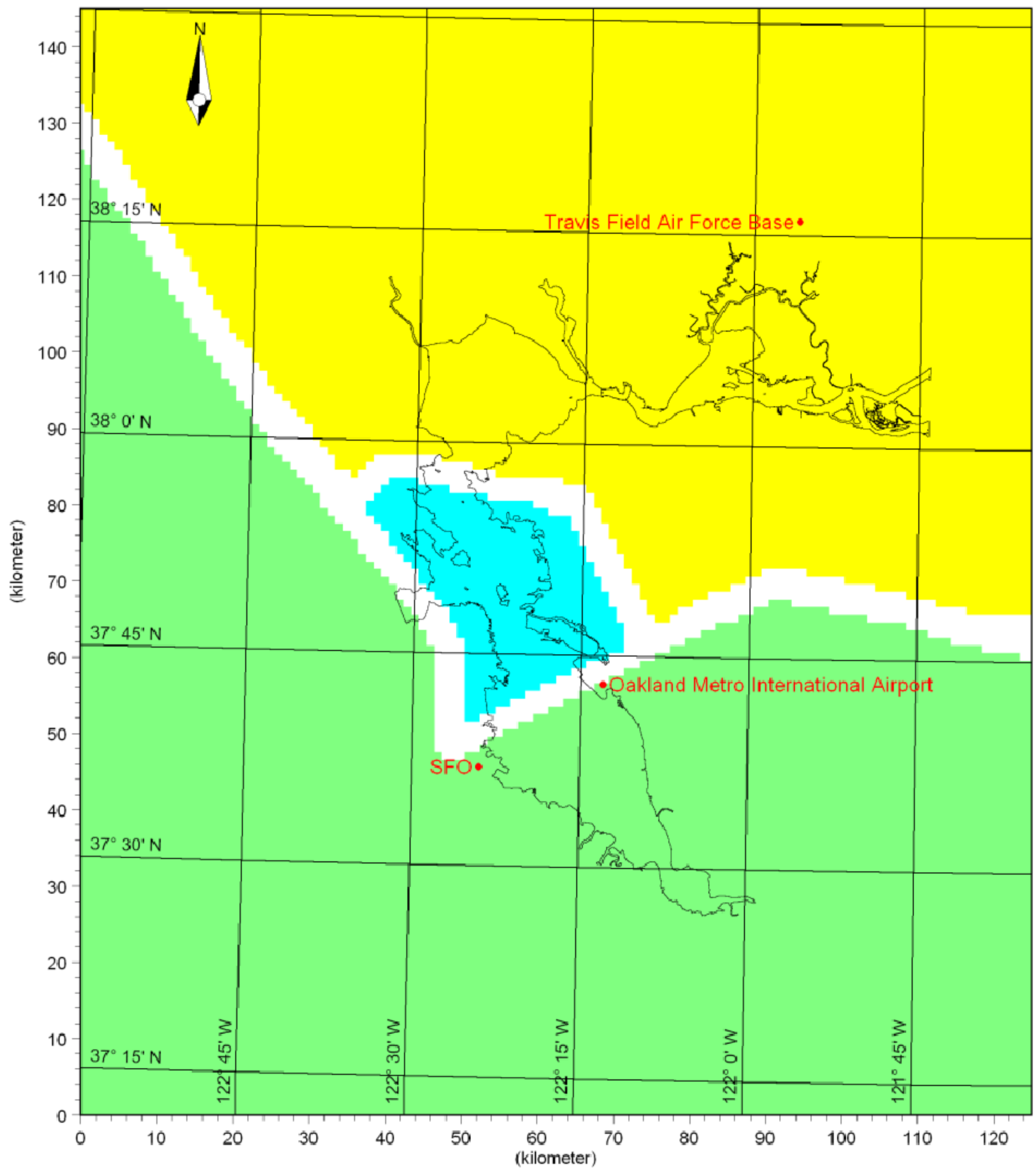

Figure 5. Areas used to assign measured wind data. Yellow area: Travis Field AFB, green area: SFO Airport and cyan area: OAK Airport. White areas indicate transition zones for wind parameters (DHI, 2011)

3.4. Land Use Information

A land use data layer was constructed through the visual inspection of detailed aerial imagery and through field visits conducted in September 2010. The San Mateo County coastal floodplain is relatively narrow and is heavily developed in low-lying areas susceptible to flooding. As a result, areas vulnerable to flooding could be accessed with relative ease in order to confirm land use determinations based on aerial photography.

ESRI's World Imagery data layer was used as the 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).

4. Transect Layout

Coastal flooding hazards were evaluated with one-dimensional (1D) transect-based models. Wave setup, runup, overtopping, and overland wave propagation were analyzed for 31 transects along the northern San Mateo County coastline (see Figure 2. Northern San Mateo County Transect Layout). Transects were placed at locations that are representative of a reach of shoreline, with consideration of the study area features that are important to the wave hazard processes that impact the area. In areas where wave runup might be significant, transects were oriented approximately perpendicular to local topographic and bathymetric contours and were placed with consideration of variations in shoreline and nearshore slopes, shoreline structures, structure or bluff crest elevations, shoreline orientation, and incident wave conditions.

In areas where overland wave propagation is anticipated to be a flood hazard, transects were oriented along the dominant direction of wave propagation, and with consideration of variations in topography and land cover (i.e., buildings, vegetation, and other factors) that can influence wave transformation.

Shoreline orientation was an important consideration for all of San Mateo County in the siting of transects due to the highly irregular geometry of the bay. The shoreline orientation plays a significant role in the wave exposure with coves and embayments that are somewhat sheltered from the greater bay being protected from higher energy waves that impact the less protected reaches of coast.

In northern San Mateo County, spacing between transects ranged from 500 feet to 1 mile. Transects were more closely spaced in areas of higher development density and areas with more heterogeneous shoreline characteristics.

5. Wave Runup

Wave runup is the culmination of the wave breaking process, whereby the broken wave surges up the beach, bluff, or structure face along the shoreline. 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 elevation reached by the total runup above the SWEL. 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} .

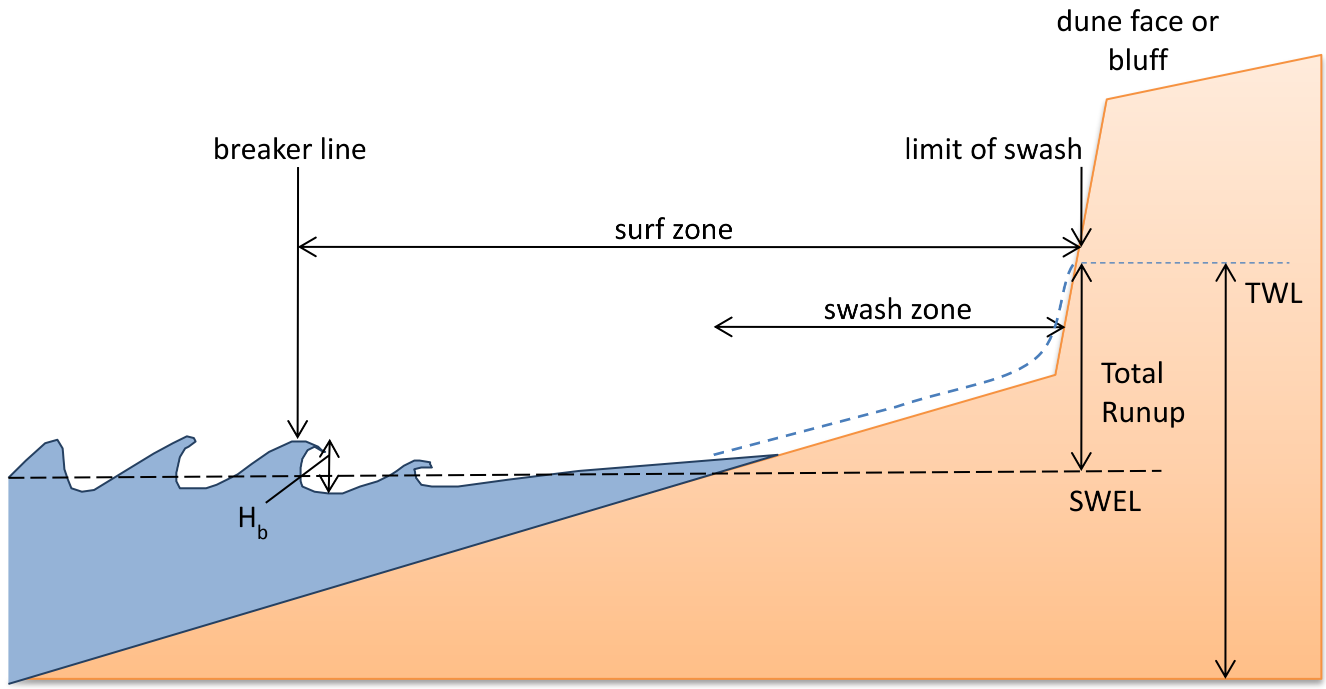

Figure 6. Conceptual model showing the components of wave runup associated with incident waves (modified from Pacific G&S, 2005)

Static wave setup and dynamic wave setup were calculated for all shore types using DIM, as described in the Pacific G&S. Incident wave runup was calculated with one of two methods. DIM was used for areas of beach and gently sloping natural shoreline (m<0.125). The TAW method (van der Meer, 2002) was used to calculate runup on shorelines with shore protection structures or steeply sloping ($m \geq$0.125) natural shorelines.

For transects 1-13, which are in the North/Central Bay regional hydrodynamic and wave model study area, wave runup was computed at each hourly time step in the 31-year time series for wind-generated waves (seas), swells, and a combination of the two. For transects 14-31, which are in the South Bay regional hydrodynamic and wave model study area, wave runup was computed at each hourly time step in the 54-year time series for wind-generated waves (seas) only.

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 G&S. This runup elevation is a short-term statistic associated with a group of waves or a particular storm. It is a standard definition of runup, commonly denoted as $R_{2\%}$. This 2 percent is different from the 1-percent-annual-chance condition that is associated with long-term extreme value statistics. The 1-percent condition has a 1-percent annual probability of occurrence, which corresponds approximately to the 100-year condition, while the runup statistic corresponds to a 2-percent exceedance occurrence in several hours of waves. Unless otherwise indicated, the runup referred to in this study is the 2-percent runup.

Wave setup and runup were combined with coincident water level values taken from the DHI surge model output to develop the TWL values. The following sections describe the calculation of TWLs using each of the three wave runup methods. A statistical extreme value analysis (EVA) was performed on the TWL to determine the 1-percent-annual-chance TWL (See Section5.5. Extreme Value Analysis).

5.1. Input Parameters

The basic input information required for calculating the total runup includes water level and starting wave conditions at the pass point; transect geometries, such as slopes; crest and toe locations, where applicable; and the transect's orientation to the shoreline. Table 2 summarizes the input values used in the wave setup and runup analyses.

Table 2. Input Parameters for Wave Setup and Runup Analyses

Method	Input Requirements
DIM, Setup and Runup	Equivalent deepwater significant wave height, H_0 Peak spectral period, T_p Nearshore slope, *m*, i.e. *(*$\tan\alpha$*)* Spectral peakedness parameter, *Gamma*
TAW, Runup	Spectral significant wave height at the structure toe, H_{m0} Spectral wave period, $T_{m-1.0}$ Structure slope, $\tan\alpha$ Incident wave direction Shoreline roughness Presence of berm

Water Levels and Wave Conditions: Water levels and wave conditions were taken directly from the regional hydrodynamic and wave models for each hourly time step simulation. Water levels, or SWELs, the spectral significant wave height, H_{m0}, the peak period, T_p, and the direction of wave propagation are provided explicitly in the regional wave modeling output. Waves were initially filtered by the direction of wave propagation, so that only waves propagating onshore were considered in these analyses. Onshore propagation was defined as propagating within 90 degrees of either side of the transect orientations, which are approximately shore perpendicular.

Wave Transformations: Each different runup method requires a different wave value as an input to the calculations. The equivalent deepwater significant wave height, H_0, is the required input parameter for DIM setup and runup calculations. The TAW method requires the spectral significant wave height, H_{m0}, at the structure toe. Wave shoaling and refraction transformations were conducted on the regional wave model output to obtain these required wave parameters.

H_0 was obtained by first refracting and shoaling the wave height at the pass point onshore to its breaking point. The wave was then de-shoaled offshore to deepwater conditions. $\overline{H_0'}$ was calculated from H_0 using the conversion factor 0.626 (FEMA, 2007).

H_{m0} at the toe of the structure, used for TAW runup calculations, was obtained by shoaling and refracting the wave from the pass point onshore to the toe of the structure or steep shoreline, using the water depth at the toe for each time step. The water depth at a given pass point required for shoaling calculations was obtained from the regional wave model output. If the shoaled wave height exceeded 0.78 times the water depth at the structure toe, the waves were assumed to be depth-limited at the toe.

Once waves had been transformed, a secondary directional filter was applied to further refine the selection of waves propagating onshore. The transformed waves had to be within +/-45 degrees of the transect orientation to be included in the runup analyses.

Combining Waves: For transects in the North/Central Bay regional model study area (transects 1-13), the wave heights and periods were combined when coincident seas and swells were found to be propagating landward and within 90 degrees of one another, as recommended in *Guidance for Flood Hazard Analyses in Sheltered Waters* (FEMA, 2008). The method described in FEMA, 2008 was modified to account for the difference in wave direction between the seas, the swell, and their direction of propagation relative to the shoreline. The combined wave height H_{ss}, wave period T_{ss}, and combined wave angle θ_{ss} were estimated as:

$$\theta_{ss} = tan^{-1}\left[\frac{H_1^2\, sin\theta_1 + H_2^2\, sin\theta_2}{H_1^2\, cos\theta_1 + H_2^2\, cos\,\theta_2}\right] \quad \text{(Equation 1)},$$

$$\alpha_1 = \theta_1 - \theta_{ss} \quad \text{(Equation 2a)},$$

$$\alpha_2 = \theta_2 - \theta_{ss} \quad \text{(Equation 2b)},$$

$$H_{ss} = (H_1^2 cos\alpha_1 + H_2^2 cos\alpha_2)^{1/2} \quad \text{(Equation 3), and}$$

$$T_{ss} \approx \frac{T_1 H_1^2 cos\alpha_1 + T_2 H_2^2 cos\alpha_2}{H_1^2 cos\alpha_1 + H_2^2 cos\alpha_2} \quad \text{(Equation 4)},$$

where (H_1, T_1) and (H_2, T_2) are associated with the seas and swells, respectively. θ_1 and θ_2 are the angles between the refracted wave direction of the seas and swells and the orientation of the transect. These angles were calculated at breaking for DIM and at the toe for TAW. α_1 and α_2 are the angles between the refracted wave direction and the combined wave direction for seas and swell. The values of H and T were calculated for the condition needed for input calculations.

The combined deepwater equivalent wave heights were calculated from the individual deepwater equivalent seas and swells. The combined spectral significant wave height at the toe of the structure was calculated from the individual spectral significant wave heights at the toe for seas and swell. For use with DIM, the combined peak spectral wave period, $T_{p_{ss}}$, was calculated from the T_p provided in DHI's results for seas and swells. The combined wave period for use in TAW, $T_{m-1.0}$, was calculated from $T_{p_{ss}}$ using the relationship $T_{m-1.0} = T_{p_{ss}}/1.1$.

Profile Geometry: Bathymetric and LiDAR data were used to generate transect profiles. From these plots, the nearshore and structure slopes were selected for use in the DIM and TAW equations, respectively. Structure crest and toe locations were selected graphically, based on a visual inspection.

5.2. Wave Setup

Both the static and dynamic components of wave setup were calculated using DIM. As presented in the Pacific G&S, 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 beach slope. The equations for static and dynamic wave setup 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:

$$\bar{\eta} = 4.0 F_H F_T F_{Gamma} F_{Slope} = 4.0 \left(\frac{H_0}{26.2}\right)^{0.8} \left(\frac{T_p}{20.0}\right)^{0.4} (1.0) \left(\frac{m}{0.01}\right)^{0.2} \quad \text{(Equation 5)}$$

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

$$\eta_{rms} = 2.7 G_H G_T G_{Gamma} G_{Slope} = 2.7 \left(\frac{H_0}{26.2}\right)^{0.8} \left(\frac{T_p}{20.0}\right)^{0.4} (Gamma)^{0.16} \left(\frac{m}{0.01}\right)^{0.2} \quad \text{(Equation 6)}$$

The wave parameters required as input for DIM are the deepwater significant wave height and the spectral peak wave period. The equivalent deepwater wave is the deepwater wave that includes the reduction in height due to refraction. To obtain the deepwater wave height, the nearshore wave height from the regional wave modeling must be transformed. The wave height given at the pass point was shoaled and refracted onshore to its breaking point and then de-shoaled to determine the equivalent deepwater significant wave height, H_0, used in Equations 5 and 6. The spectral peak wave period, T_p, was obtained directly from the regional wave model output.

For transects with structures or steep bluffs, the nearshore slope, m, was approximated as the slope between the structure or bluff toe and a point offshore at an elevation equal to the toe elevation minus 1.5 times the spectral significant wave height, H_{m0} , as prescribed in van der Meer, 2002. For gently sloping, natural profiles, the nearshore slope is the average slope between the runup limit and twice the break point of the deepwater significant wave height (van der Meer, 2002; FEMA, 2005).

The spectral peakedness parameter for JONSWAP spectra, Gamma, is used to calculate the dynamic component of setup (Equation 2). The raw wave energy spectra were not available from the regional wave modeling effort, so Gamma could not be calculated explicitly for this study. Instead, a Gamma of 3.3 was used for the dynamic setup calculations: 3.3 is the average of the spectra entering into the development of the JONSWAP spectrum. Sensitivity testing with a range of Gamma values showed that Gamma had little effect on the statistical TWLs.

5.3. DIM Runup Calculations

Runup on gently sloping, natural shorelines (m<0.125) was calculated with the DIM method. The 2-percent runup calculation is based on the standard deviations of the oscillating wave setup (dynamic 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, σ_1, calculated from Equation 6. The standard deviation of the incident wave oscillations (wave runup), σ_2, on natural beaches is given in the Pacific G&S (FEMA, 2005) as:

$$\sigma_2 = 0.3\xi_0 H_0 \qquad \text{Equation 7)}$$

The total oscillating component to the 2-percent total wave runup, $\hat{\eta}_T$, is determined as the combination of the two standard deviations of the fluctuating components:

$$\hat{\eta}_T = 2.0\sqrt{{\sigma_1}^2 + {\sigma_2}^2} \qquad \text{(Equation 8)}$$

Combining the results from Equations 5 and 8 yields the 2-percent total wave runup, and when combined with the surge component, results in the TWL:

$$TWL = \bar{\eta} + \hat{\eta}_T + Surge$$

5.4. TAW Runup Calculations

Runup on barriers, including steep dune features, bluffs, and coastal armoring structures such as revetments, was calculated according to 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. To replicate the wave tank measurements, the wave setup component of the TWL in this work is calculated seaward of the toe of the structure, and wave setup landward of the toe of the structure is not included. See Appendix A for a more detailed discussion of the calculation of wave setup in the presence of barriers. Wave setup seaward of the toe of the structure was computed with DIM, using the nearshore slope. Wave setup was not included for cases where waves had not broken prior to reaching the toe of the structure.

The reference water level at the toe of the barrier for runup calculations is the 2-percent Dynamic Water Level (DWL2%). The dynamic water level (DWL) is the sum of the measured stillwater, 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, 2013a). 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, 2013b).

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:

$$(H_b)_{rms} = \left(\frac{\kappa}{g}\right)^{1/5} \left(\frac{H_0'^2 C_0}{2}\right)^{2/5} /1.4 \qquad \text{(Equation 9)}$$

where κ, the breaker criterion, is 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:

$$\overline{\eta_0} = 0.189(H_b)_{rms} \qquad \text{(Equation 10)}$$

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

$$\bar{\eta}(h) = 0.189(H_b)_{rms} - 0.186h \qquad \text{(Equation 11)}$$

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:

$$\gamma_\eta = {}^{\bar{\eta}(h)}/_{\overline{\eta_0}} \qquad \text{(Equation 12)}$$

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, $\bar{\eta}'$:

$$\bar{\eta}' = \gamma_\eta \bar{\eta} \qquad \text{(Equation 13)}$$

DWL2% is therefore:

$$DWL2\% = \bar{\eta}' + 2\eta_{rms} + Surge \qquad \text{(Equation 14)}$$

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 was calculated iteratively across the zone between the stillwater elevation (SWEL) minus $1.5H_{m0}$ and the runup limit. Since the runup limit was initially unknown, the SWEL $+1.5H_{m0}$ was chosen as a first estimate (Figure 7). H_{m0} is the spectral significant wave height at the toe of the structure and was determined by shoaling and refracting the wave height provided at the pass point to the structure toe. If the DWL2% depth at the structure toe was 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 the common industry breaker index of 0.78 ($H_{m0} = 0.78 \times h_{toe}$).

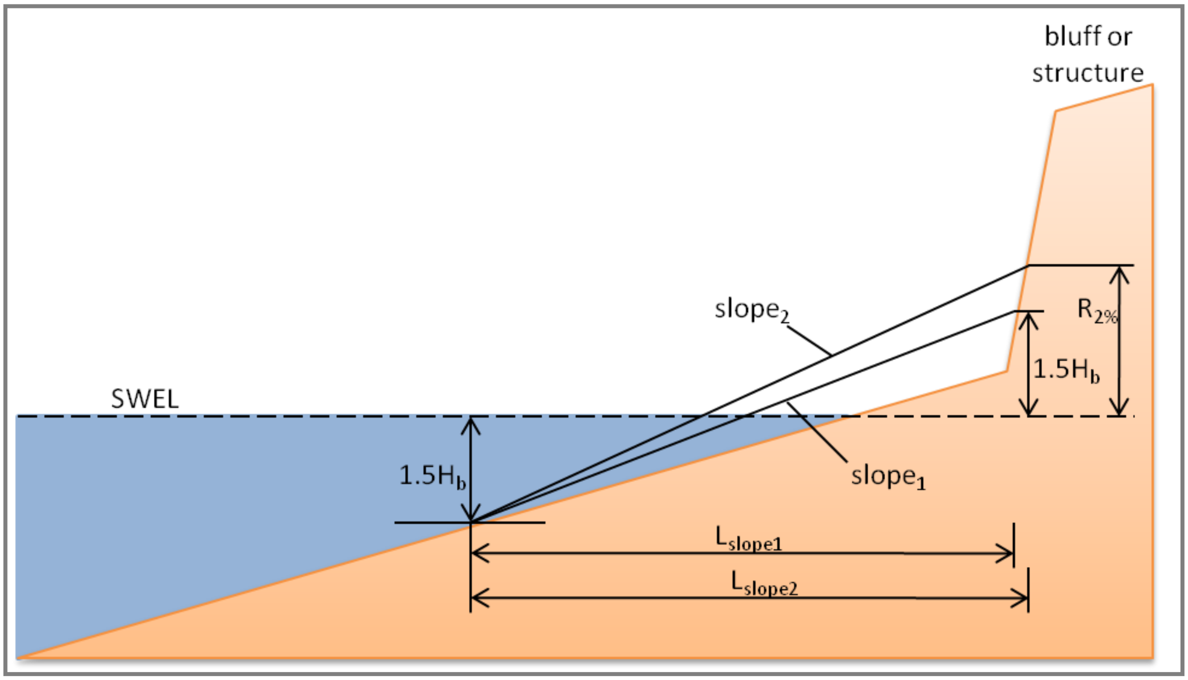

Figure 7. Determination of an average slope based on an iterative approach—first estimate is initially based on SWEL ± $1.5H_{m0}$ (or $1.5H_b$ where appropriate), and second estimate is the runup limit calculated with the first slope estimate, etc. (modified from van der Meer, 2002)

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

$$R_{2\%} = H_{m0} \begin{Bmatrix} 1.77\gamma_r\gamma_b\gamma_\beta\xi_{0m}, & 0.5 \le \gamma_b\xi_{0m} < 1.8 \\ \gamma_r\gamma_b\gamma_\beta\left(4.3 - \frac{1.6}{\sqrt{\xi_{0m}}}\right), & 1.8 \le \gamma_b\xi_{0m} \end{Bmatrix} \quad \text{(Equation 15)}$$

Where:

- $R_{2\%}$ = 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 (not applicable in San Mateo County—no berms observed in transect profile geometries)

- γ_β = influence factor for oblique wave attack
- ξ_{0m} = Iribarren number ($tan\alpha/(\frac{H_{m0}}{L_{m-1.0}})^{0.5}$)
- $tan\alpha$ = barrier slope
- $L_{m-1.0}$ = spectral wave length ($gT_{m-1.0}^2/2\pi$)
- $T_{m-1.0}$ = spectral wave period

Influence factors for roughness and oblique wave attack were selected according to Table D.4.5-3 in the Pacific G&S. The roughness reduction factors used to account for the surface roughness of structures are summarized in Table 3.

Table 3. Roughness Reduction Factor Values

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

The influence factor for oblique wave attack was calculated at each time step, relating the direction of wave propagation to the transect orientation. Waves were assumed to be short crested. 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 spectral wave period, $T_{m-1.0}$, and its associated wavelength, $L_{m-1.0}$, were calculated from the spectral peak wave period, T_p, which was provided as output from the DHI wave models. $T_{m-1.0}$ is calculated as:

$$T_{m-1.0} = T_p/1.1 \qquad \text{(Equation 16)}$$

This relationship is based on single-peaked spectra and is described in van der Meer, 2002. The resultant value for runup on a barrier is added to DWL2% to yield the TWL.

5.5. Extreme Value Analysis

The GEV distribution was selected to determine the statistical 1-percent-annual-chance TWL from the hindcast data. A fundamental requirement of extreme value theory with annual maxima is the independence of storm events. Use of the January-to-December calendar year for annual maxima selection allowed for the possibility that a single storm event extending from late December into January would produce the annual maxima for two consecutive years. Therefore, a July-to-June year was chosen to guarantee the independent selection of the predominantly winter storm events in San Francisco Bay. TWL annual maxima were selected for each of the 12-month periods between July 1 and June 30. An additional TWL annual maximum was selected for a synthetic "year" created by combining the two remaining half-years, January to June 1973 and July to December 2003 for the North/Central Bay study area and January to June 1956 and July to

December 2009 for the South Bay study area. The annual TWL results are summarized in Appendix B.

The GEV distribution was selected to determine the statistical 1-percent-annual-chance TWL. The advantages of using the GEV distribution over alternative approaches (method of independent storms, peak-over-threshold, etc.) are that few decisions are required in the calculation of the distribution parameters, and the extremes maintain their independence and distribution. The 31 or 54 annual maxima per transect exceeded the suggested minimum of 20 years of data for reliable results (Palutikof et al., 1999). The cumulative distribution function (CDF) of the GEV family of distributions is given as:

$$F(z) = \exp\left\{-\left[1+\xi\left(\frac{z-\mu}{\sigma}\right)\right]^{-1/\xi}\right\} \quad \text{(Equation 17)}$$

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 (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 $\xi,$ and the fit of the resulting CDF to the annual maxima were evaluated for the maximum likelihood solutions.

5.6. Runup Results

The maximum annual TWL events for each transect are provided in Appendix B. The 1-percent-annual-chance TWLs for each transect, resulting from the TWL extreme value analysis, are summarized in Table 4. Table 4 also indicates whether any shore protection structures are present at the shoreline where runup is calculated. The mean runup slope was calculated from the annual maxima runup slopes. The roughness reduction factor, γ_r, is provided for each transect. 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 5 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.

Table 4. 1-Percent-Annual-Chance TWLs, Mean Runup Slopes, and TAW Reduction Factors Used for the 28 Runup Transects

Transect	Structure Description	Mean Runup Slope	Roughness Reduction Factor γ_r	1% TWL (ft, NAVD88)	Overtopped
1	Revetment	0.49	0.6	13.68	YES
2	Revetment	0.28	0.6	12.63	YES
3	Revetment	0.39	0.6	10.25	—
4	Revetment	0.39	0.6	13.71	YES
5	Revetment	0.50	0.6	13.87	YES
6	Revetment	0.07	0.6	10.68	—
7	Revetment	0.35	0.6	13.31	YES
8	Revetment	0.09	0.6	11.10	YES
9	NA	0.23	1.0	12.25	—
10	Revetment	0.34	0.6	13.80	
11	Revetment	0.31	0.6	11.49	—
12	NA	0.07	1.0	10.42	—
13	NA	0.20	1.0	12.97	YES
14	NA	0.10	1.0	10.56	YES
15	Revetment	0.24	0.6	10.67	—
16	Revetment	0.54	0.6	12.02	YES
17	Revetment	0.45	0.6	12.26	YES
18	Revetment	0.63	0.6	12.48	YES
19	Revetment	0.34	0.6	11.81	YES
20	NA	0.55	1.0	15.89	—
23	Levee/ Revetment	0.39	0.6	12.68	—
24	Levee/ Revetment	0.32	0.6	12.46	—
25	Levee/ Revetment	0.26	0.6	12.49	—
26	Revetment	0.36	0.6	12.38	—
27	Levee/ Revetment	0.47	0.6	12.13	—
29	Levee/ Revetment	0.49	0.6	12.58	YES
30	Levee/ Revetment	0.58	0.6	13.00	YES
31	Levee/ Revetment	0.46	0.6	12.96	YES

6. Overtopping

Overtopping occurs when the wave runup exceeds profile crest elevation, which can result in flooding landward of the crest. 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, $y_{G\ outer}$

6.1. 1-Percent-Annual-Chance Overtopping Conditions

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 the overtopping flow. The Pacific G&S recommends the Cox and Machemehl (1986) method (C-M method) to determine these values for splash and bore overtopping.

The required input parameters for the C-M method are the TWL, the wave period, and the DWL. 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. As a statistical value, however, the 1-percent TWL is not associated with a specific wave period or DWL. Therefore, an appropriate wave period and DWL must be chosen for use with the 1-percent TWL to estimate the 1-percent overtopping hazard.

The function of the DWL in the C-M method calculation is in determining whether the overtopping is splash or bore in nature. This classification is important because bore overtopping results in greater depths and extents than splash overtopping. Using the maximum DWL from the TWL annual maxima maximizes the overtopping depths and extents resulting from the 1-percent TWL. This pairing is not considered overly conservative, however, since it is reasonable to expect that the 1-percent TWL could result from the maximum DWL and a moderate wave condition. The remaining parameter needed to calculate wave overtopping hazards is the wave period. The mean wave period from the TWL annual maxima provides a moderate wave condition to pair with the maximum DWL and 1-percent TWL. Thus, the 1-percent overtopping hazard can be estimated with the C-M method using the 1-percent TWL and the maximum DWL and the mean peak wave period from the TWL annual maxima.

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 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 8 illustrates splash overtopping, with bore propagation landward of splashdown. The variables involved in the determination of the limits of overtopping and the hazard zones landward of the barrier crest are also shown.

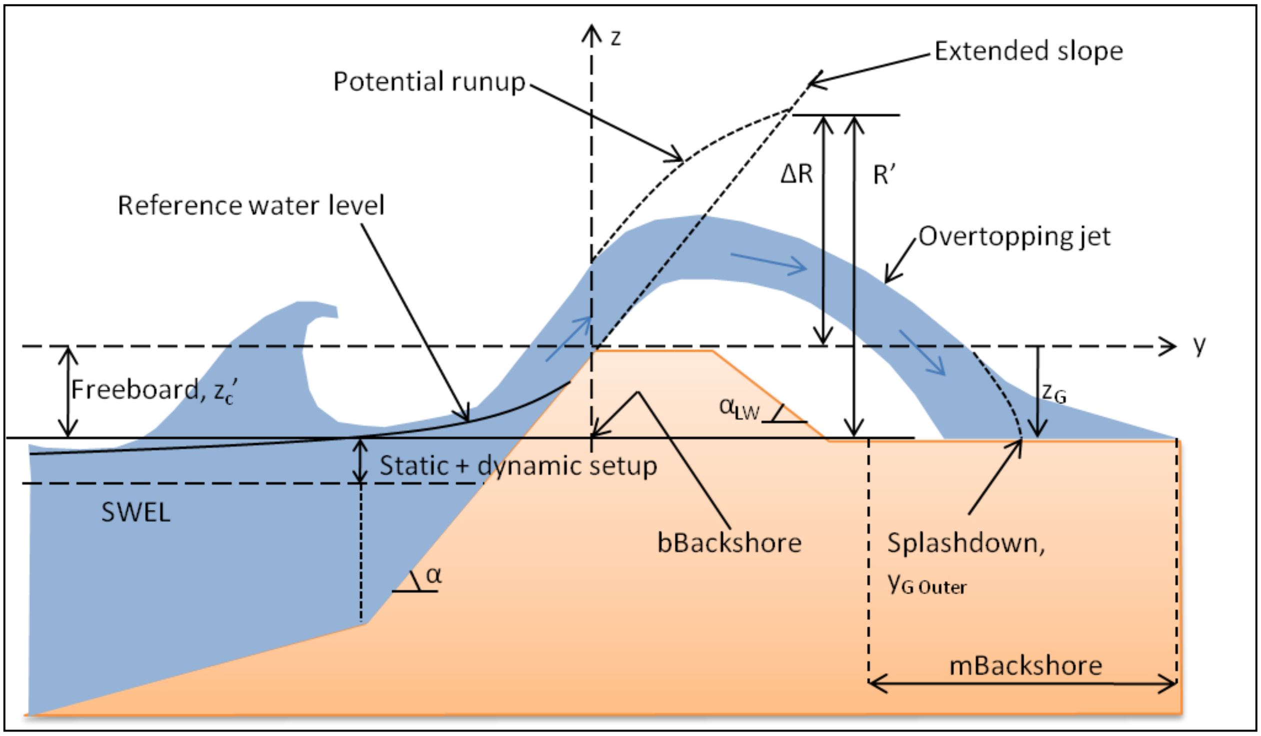

Figure 8. Illustration of splash overtopping and its associated variables (modified from Pacific G&S, 2005)

The landward limit of a 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 G&S. 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 G&S to allow for automation of the approximation of the outer limit of the splash region, $y_{G\ outer}$:

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

and

$$z_G = bBackshore + mBackshore * y_{G\ outer}$$

where α is the seaward slope of the structure in degrees, *bBackshore* is the intercept for the backshore slope adjacent to the barrier crest, and *mBackshore* is the backshore slope (DOGAMI, 2010). In cases of splash overtopping, the onshore wind speed used to calculate an enhanced onshore water velocity was taken to be 30 mph, following the suggested minimum in the Pacific G&S, Section D.4.5.2.5.1. This value was deemed appropriate due to the relatively sheltered location of the study area. Use of this value is also supported by the wind speed analysis conducted for WHAFIS modeling, since it falls within the range of values derived for input in those analyses (See Section 6.2).

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:

$$h(y) = \left[\sqrt{h_0} - \frac{5(y-y_0)}{A_m\sqrt{gT^2}}\right]^2 \qquad \text{(Equation 18)}$$

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

6.3. Overtopping Results

Figure 9 shows an example of runup and overtopping results. Table 5 presents the results of the calculated splashdown distances (y_{Gouter}), the landward extent of the flow where $hV^2=200\ ft^3/s^2$, approximating the limit of the V zone, and where $h=0$, approximating the limit of the A zone. As the table shows, all splashdown and bore propagation distances ended very near to the barrier crest, producing narrow flood zones. In fact, for some transects, the flow hV^2 was initially less than $200\ ft^3/s^2$ at the barrier crest or upon splashdown (e.g., transects 2 and 17). The narrow overtopping distances and low severity of bore flow is likely due to the small waves in the study area. Table 5 includes all transects for which the barrier crest elevation was exceeded by the TWL during any of the 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.”

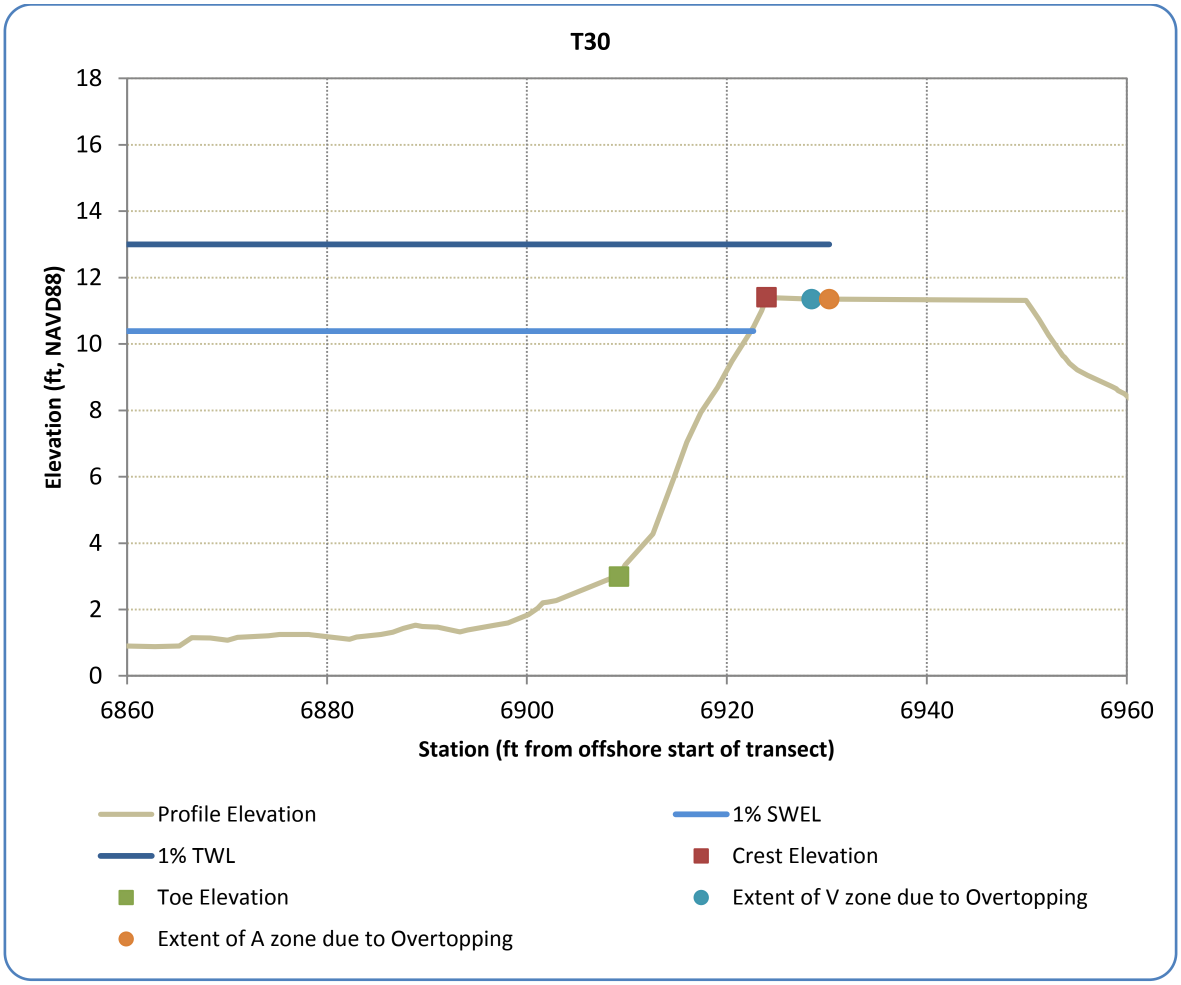

Figure 9. Example of runup and overtopping results showing 1% runup elevation (1% TWL) and overtopping limits for Transect 30

Table 5. Splashdown and Hazard Zone Limits for the 1-Percent-Annual-Chance TWLs at 17 Overtopped Transects

Transect	Number of Wave Overtopping Events	1% Overtop-ping Event DWL2% (ft, NAVD88)	Crest Elevation (ft, NAVD88)	1% TWL (ft, NAVD88)	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)	Backshore Slope Coefficient Am	zG (ft)
1	3	9.24	12.60	13.68	2.26	0.00	2.26	2.39	4.65	0.95	0.05
2	3	9.36	11.50	12.63	0.00	0.00	0.00	0.00	0.00	0.95	1.50
4	5	9.25	11.90	13.71	0.43	0.00	0.43	2.39	2.81	0.74	0.06
5	26	9.24	9.74	13.87	0.00	0.51	0.51	4.60	4.60	0.95	-
7	27	9.26	9.90	13.31	0.00	0.00	0.00	4.50	4.50	1.04	-
8	0	9.37	11.00	11.10	0.07	0.00	0.07	0.60	0.60	0.99	0.00
13	21	9.28	9.90	12.97	0.00	0.00	0.00	4.13	4.13	1.10	-
14*	10	9.38	9.30	10.56	-	-	-	-	-	-	-
16	19	8.84	10.20	12.02	0.00	0.00	0.00	2.56	2.56	0.90	-
17	0	9.21	12.22	12.26	0.00	0.00	0.00	0.33	0.33	0.83	-0.07
18	23	8.84	10.50	12.48	0.00	0.00	0.00	3.06	3.06	1.03	-
19	6	9.25	10.20	11.81	0.00	0.00	0.00	2.38	2.38	1.04	-
29	7	8.91	11.29	12.58	3.25	0.00	3.25	2.45	5.69	1.00	0.00
30	8	9.29	11.40	13.00	4.44	0.00	4.44	2.75	7.19	1.01	-0.02
31	5	9.44	11.57	12.96	3.17	0.00	3.17	2.56	5.73	1.01	-0.02

*Crest is inundated by DWL2%

7. Overland Wave Propagation

Overland wave propagation was evaluated using WHAFIS, Version 4.0, model (FEMA, 1988; Divoky, 2007). WHAFIS uses representative transects to compute wave heights, wave periods, and wave crest elevations as the wave propagates inland from the shoreline. WHAFIS analyses were performed for eight of the 31 San Mateo County transects. Figure 10 shows the transects for which overland wave propagation flood hazards were analyzed.

7.1. WHAFIS Modeling

WHAFIS was developed to predict wave heights associated with overland propagation during flooding events. The model was based on the methodology outlined in the 1977 National Academy of Sciences report, *Methodology for Calculating Wave Action Effects Associated with Storm Surges*, and is fully documented in FEMA, 1988. Updates to the model are documented in Divoky, 2007. The model is based on formulations that include a wave energy conservation equation and a conservation of waves equation that expresses the variation in spectral peak wave period over the length of the transect. The formulations include the effects of barriers to wave transmission (buildings and rigid vegetation) and the regeneration of waves over water and flooded land areas. This includes the effects of dissipation due to marsh plants. Updates in version 4.0 (Divoky, 2007) include the user's ability to specify varying wind conditions at the initiation of the model. These conditions cannot be varied spatially along the transect.

The WHAFIS analysis yields three pieces of information that are used to map flood hazards from overland wave propagation: the inland extent of inundation, the wave height variation along the transect, and the wave crest elevation along the transect. The extent of inundation determines the floodplain boundary and is controlled by the SWEL and topography. The wave height determines the flood zone designation—Zone VE for areas with wave heights 3 feet or greater and Zone AE for wave heights less than 3 feet. The wave crest elevation determines the BFE.

Figure 10. Transects in San Mateo County analyzed with WHAFIS for overland wave propagation

The WHAFIS model is typically applied using a single event with the 1-percent-annual-chance water level and the 1-percent-annual-chance wave conditions, which are assumed to be coincident.

The assumption of coincidence of peak waves and water levels is appropriate for open coasts on the Atlantic Ocean and the Gulf of Mexico, where flood events are associated with fast-moving hurricanes, and extreme water levels and waves arrive together, born of the same forcing (i.e., the hurricane). This assumption is not necessarily valid for the sheltered shorelines of San Francisco Bay, where the forcing mechanisms for elevated water levels and large wave events are distinctly different. Large waves within most of the bay are caused by extended wind events along specific fetch orientations. Water levels in the bay are sensitive to these same winds blowing along specific fetches, but water levels are also influenced by tidal forcing. Investigations of the coincidence of water level and wave events for San Francisco Bay within the regional hydrodynamic and wave study data confirmed that elevated water levels are often decoupled from the local wave and wind events. Therefore, pairing the 1-percent water level with the 1-percent wave height is not appropriate and may result in the overestimation of the 1-percent flood hazard from overland wave propagation. Analysis showed that locally generated seas pose a greater overland wave propagation hazard than Pacific Ocean swells for all WHAFIS transects in San Mateo County. As such, WHAFIS modeling was performed using starting water-level and wave conditions developed from the locally generated seas information. The WHAFIS model was only run for transects with starting wave heights greater than 0.5 feet since waves smaller than this threshold can be considered an insignificant component of the flood hazard.

7.2. Input Parameters

The basic input information required by WHAFIS includes SWELs, wave and wind conditions, ground elevations, and land use classifications with the corresponding vegetation or building parameters.

Water Levels and Wave Conditions: To account for decoupled water levels and wave conditions in the bay, a dual-event-based approach was selected to analyze overland wave propagation. Since water levels control floodplain extent, and wave heights control flood zone designations, the following two water level and wave height combinations account for the variability in wave and water level combinations that present the most hazardous conditions for overland wave propagation:

- *Scenario 1: The 1-percent-annual-chance SWEL paired with an appropriate wave condition.*
- *Scenario 2: The 1-percent-annual-chance wave height and related period paired with an appropriate SWEL.*

Scenario 1 maximizes floodplain width as a result of 1-percent-annual-chance stillwater flooding, while Scenario 2 maximizes the wave height, and therefore the VE zone width, for the 1-percent-annual-chance flood. The wave condition paired with the 1-percent-annual-chance SWEL and the SWEL paired with the 1-percent-annual-chance wave height were derived from the 1-percent-annual-chance wave crest elevation using the relationship that 70 percent of the wave height is above the SWEL (NAS, 1977).

For Scenario 1 (Figure 11), the 1-percent-annual-chance SWEL is subtracted from the 1-percent-annual-chance wave crest elevation, and the remainder is assigned as 0.7*H_{m0}, yielding the wave height to be paired with the 1-percent-annual-chance SWEL. For Scenario 2, 70 percent of the 1-percent-annual-chance wave height is subtracted from the 1-percent-annual-chance wave crest elevation, and the remainder is assigned as the water depth, yielding the SWEL to be paired with the 1-percent-annual-chance wave height (Figure 12). Constraining the combined elevation of the water level/wave height pair for each scenario with the 1-percent wave crest elevation ensures that neither scenario overestimates the 1-percent flood condition. The results from the two scenarios are merged to form a single wave crest elevation profile of the most hazardous conditions along the length of the transect.

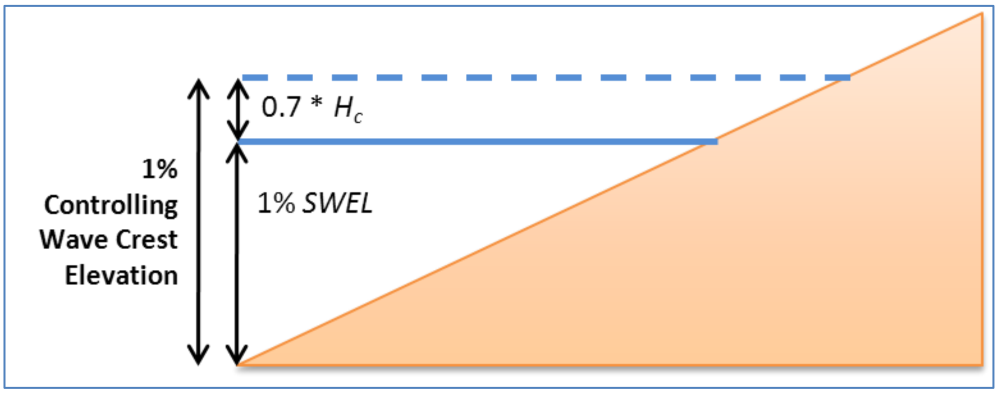

Figure 11. Scenario 1 -1-percent-annual-chance SWEL paired with an appropriate wave condition

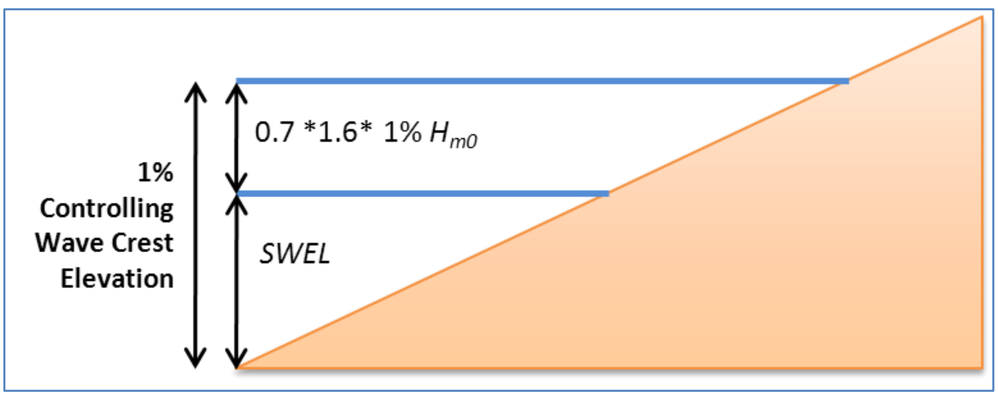

Figure 12. Scenario 2 – 1-percent-annual-chance wave height paired with an appropriate SWEL

For each WHAFIS transect, the 1-percent-annual-chance SWEL, wave height, and wave crest elevation were calculated from the regional hydrodynamic and wave modeling results. Waves were refracted from the DHI pass point onshore to the first station of the WHAFIS transect. Only waves propagating onshore were considered in these analyses. Onshore propagation was defined as the refracted wave at the WHAFIS starting point propagating within +/- 45 degrees of the transect orientation. Scenarios 1 and 2 both use statistically derived wave height values that do not have associated wave periods. A reasonable peak wave period, T_P, to be associated with a given wave height, can be obtained using the spectral relationship noted in Goda (2000), $T_P = 1.1 * (2.21\sqrt{H_{m0}})$.

An EVA was performed to estimate the 1-percent-annual-chance water levels, refracted wave heights, and refracted wave crest elevations for each transect. FEMA's Pacific G&S recommends that 1-percent-annual-chance statistics be calculated on annual maxima from at least 30 years of data using the GEV distribution with the Maximum Likelihood (ML) method. All statistical estimates were made using the WAFO toolbox for Matlab (WAFO-group, 2000).

Wind: Non-default wind speeds were used in this WHAFIS analysis. These wind speeds differ for the two scenarios for some transects. For Scenario 1, the wind speed is the maximum of the wind speeds occurring at the time of the surge annual maxima. For Scenario 2, the wind speed is the maximum of the wind speeds occurring at the time of the wave height annual maxima. Wind speeds were modified to account for directionality, since only the onshore component of the wind contributes to wave generation along a WHAFIS transect. The wind velocity in the direction of the transect, U_t, was calculated as

$$U_t = U_0 cos\theta \qquad \text{(Equation 19)}$$

where $U_0$is the reported wind speed from the assigned wind station (see Section 0), and θ is the angle between the transect and the reported wind direction.

A single wind speed was assigned to each transect for all *IF* and *VH* cards for each scenario. *OF* cards were not used in this study. It is appropriate not to distinguish between *IF* and *OF* cards, because the transects are short and the wind data originated from onshore locations (i.e., airports). A summary of the WHAFIS wind speeds is included in Table 6. Winds associated with wave events (Scenario 2) range from 30 to 35 mph. Winds associated with surge events range from 17 to 22 mph.

Wave Setup: The water levels used in the WHAFIS analysis do not include a wave setup component. Wave setup is not included because the waves from the regional model are sufficiently small in the bay such that wave breaking rarely occurs offshore of the modeled transect due to depth limitations. The two most common shore types for WHAFIS transects are transects with broad marshes or transects with a dike at the shoreline. In the case of the marshes, the waves are attenuated by the grass rather than broken. In the case of the dikes, the waves break on the structure and setup/runup will be evaluated separately from the WHAFIS analysis. For these reasons it was decided that it is most appropriate for water levels in the WHAFIS analysis not to include a wave setup component.

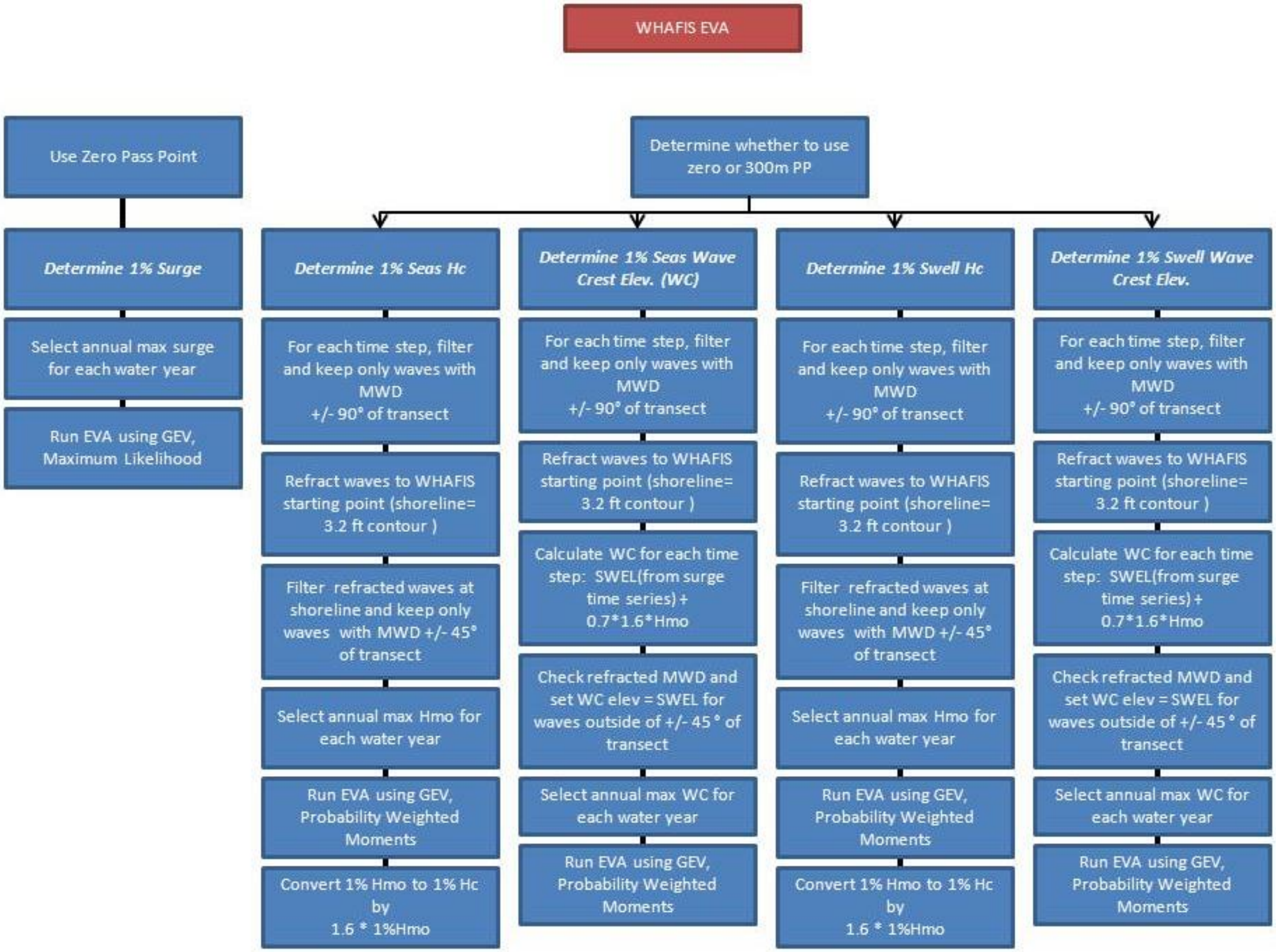

Figure 13. WHAFIS Starting water level and wave condition extreme value analysis flow chart

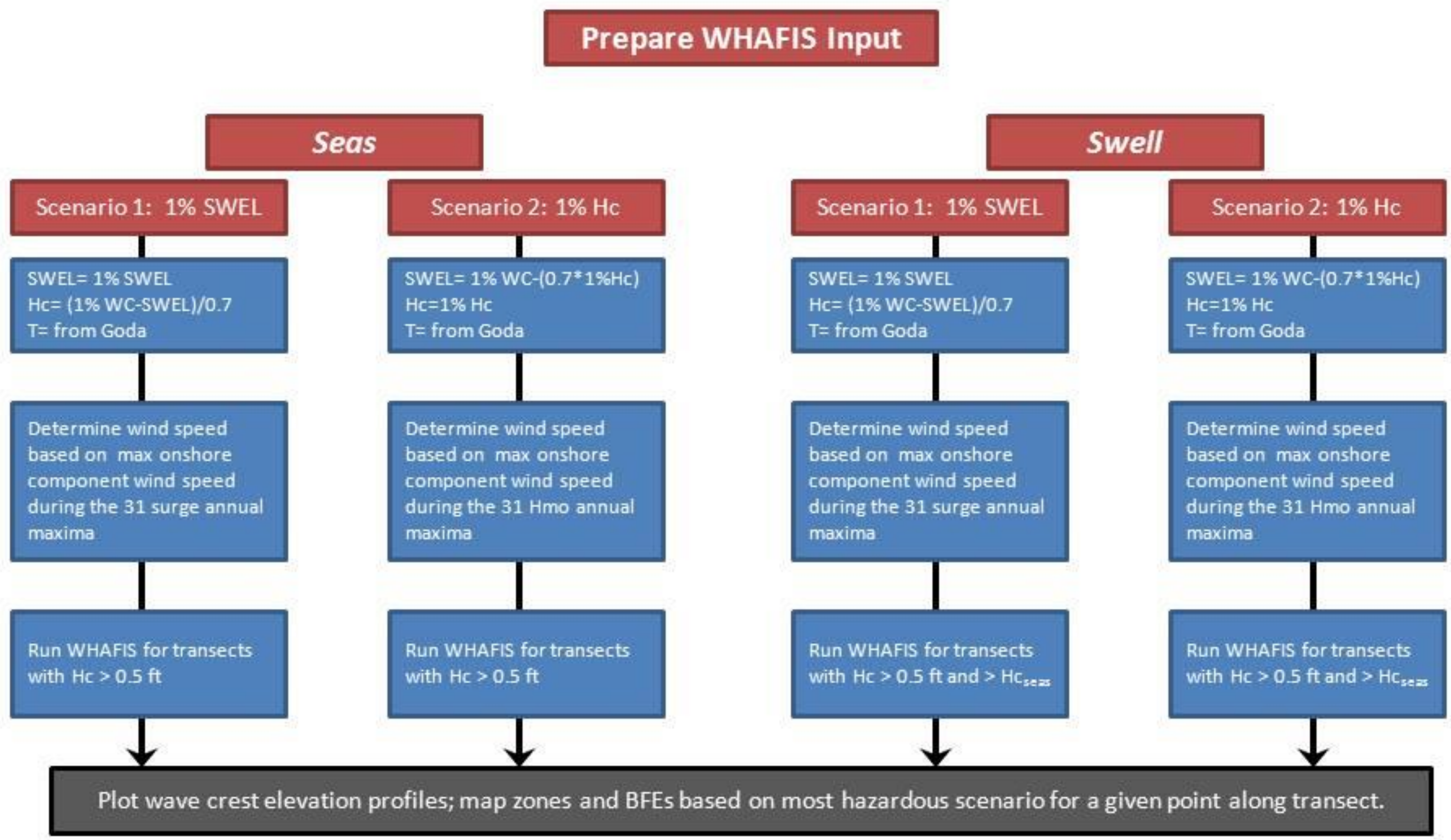

Figure 14. WHAFIS Starting Wave Condition development flow chart

Table 6. Stillwater Elevations, Starting Wave Conditions, and Wind Speeds Used for the 6 WHAFIS Transects

Transect Number	1% Wave Crest Elevation (ft NAVD)	Scenario 1				Scenario 2			
		1% SWEL (ft NAVD)	H_c (ft)	T (s)	Wind Speed (mph)	SWEL (ft NAVD)	1% H_c (ft)	T (s)	Wind Speed (mph)
5	11.6	10.4	1.8	2.6	20	8.9*	-	-	-
19	11.3	10.2	1.5	2.4	17	8.0*	-	-	-
21	11.5	10.3	1.7	2.5	20	9.1	3.4	3.5	31
22	11.2	10.3	1.3	2.2	19	8.9	3.3	3.5	30
23	11.3	10.3	1.5	2.3	22	8.9	3.5	3.6	35
24	11.1	10.3	1.1	2.1	22	8.8	3.3	3.5	35
25	11.3	10.3	1.4	2.3	22	8.8	3.6	3.7	35
28	11.1	10.4	1.1	2.0	20	8.5	3.7	3.7	34
*WHAFIS not run for Scenario 2 because transect not inundated by SWEL									

Shoreline: The MSL contour, which falls at 3.2 feet NAVD, was chosen to be the transect baseline for WHAFIS transect analyses. The transect baseline designates the location of the first WHAFIS station, Station 0. Typically, the 0-foot contour is chosen to be the transect baseline; however, in the San Francisco Bay the 0-foot NAVD contour is approximately equivalent to the Mean Lower Low Water datum. The 0-foot contour is often located hundreds of feet seaward of the wet/dry shoreline visible on aerial imagery. Since it is required that the transect baseline be shown on the Digital Flood Insurance Rate Map (FIRM), the great discrepancy between the location of the wet/dry shoreline and the 0-foot contour could cause confusion for end-users and result in a map that is not aesthetically pleasing. Thus, although FEMA's guidelines recommend the use of the 0-foot contour for the transect baseline, the MSL contour at 3.2 feet NAVD was identified as a more appropriate baseline for these analyses.

Land Use: Land use information was derived from field visits and aerial imagery. Marsh vegetation parameters were obtained from a Northwest Hydraulic Consultants technical memorandum, titled *North San Francisco FEMA Mapping Vegetation Parameters for WHAFIS Modeling* (NHC, 2011). Marsh vegetation was classified based on field observations and according to characteristic marsh habitat ranges; the marsh vegetation WHAFIS parameters are listed in Table 7. Tidal datums reported in Table 7 are averages from NOAA San Francisco and Alameda tide gages. These elevation ranges are intended as a general rule of thumb for assignment of plant types to marsh land use classifications. A specific WHAFIS card with a given plant type may not have a ground elevation that falls within its listed range. However, the plant type should be consistent with the average elevation over the broader area presented by the land use polygon associated with the marsh card.

Rigid vegetation parameters were estimated from aerial imagery. Areas of tree or scrub/shrub vegetation types are minimal throughout the study area.

Narrow, raised linear features, such as marsh dikes, were represented with the *DU* card in the WHAFIS model when inundated by the modeled SWEL.

Table 7. Marsh Vegetation WHAFIS Parameters

Marsh Habitat	Range	Elevation Range (ft. NAVD)	WHAFIS Card	Vegetation Type	Effective Drag Coefficient (Default Value = 0.1)	Mean Unflexed Height of Stem (ft.)	Plant Density (plants per sq. ft.)	Base Stem Diam. (in.)	Mid-Stem Diam. (in.)	Top-Stem Diam. (in.)	Ratio of Total Frontal Area of Cylindrical Part of Leaves to Frontal Area of Main Stem
Salt Marsh Low	MSL-MHW	3.2-5.5	VH	Cord Grass (SALM)	0.1	3.5	6	0.5	0.5	0.50	1.59
Salt Marsh Mid	MHW-MHHW	5.5-6.4	VH	Pickleweed (PICK)	0.1	2	28	0.4	0.4	0.125	0.10

7.3. WHAFIS Results

Profiles showing resulting wave crest elevations, wave heights, and SWELs for Scenarios 1 and 2 are included in Appendix C (*Sets 1* and *2*, respectively). Note that ground topography above SWELs was excluded from these figures. Since the WHAFIS model is only concerned with inundated areas, ground elevations greater than the stillwater are carded as "Above Surge" and reflected in the model with an elevation equal to the stillwater.

Set 3 of the profiles in Appendix C compares the wave crest elevations and wave heights of Scenarios 1 and 2 for each transect. The comparison profiles demonstrate which scenario produces the highest wave crest elevation and wave height along a given reach of transect. For a given point along a transect, the higher of the two wave crest elevations will be used to map the BFE. In general, wave crest elevations were dominated by Scenario 1 conditions, due to the presence of higher SWELs. As shown in Table 6, Scenario 1 SWELs are larger than those for Scenario 2 for all transects, generally on the order of 1.0 to 2.0 feet.

Reaches with wave heights of 3 feet or greater will be mapped as VE zones. All of the transects had starting wave conditions with wave heights greater than 3 feet for Scenario 2.

7.4. Runup on an Inland Slope

The response-based approach for calculating wave runup (described in Section 5), which uses nearshore wave conditions from the regional wave and hydrodynamic modeling study, cannot be used to evaluate wave runup on emergent sloping ground that is inland of obstructions such as buildings, vegetation, or topographic features. The nearshore wave conditions may be significantly altered when the wave encounters the obstructions. For these inland secondary shorelines an event-based approach was adopted. To evaluate wave runup, the 1-percent-annual-chance flood scenarios were first modeled in WHAFIS to determine the wave height and period at the inland slope. The wave runup elevation was calculated with the appropriate wave runup method (DIM or TAW) for the slope and shore type of the emergent ground. Runup on an inland slope was necessary to be calculated for one of the eight WHAFIS transects. Table 8 summarizes the inland wave runup calculation for transect 28. The runup elevation is shown on the wave height profile in Set 3 of Appendix C and will be incorporated into the wave envelope profile that defines the BFEs along the transect.

Table 8. Summary of Wave Runup Calculations Inland Along WHAFIS Transects

Transect	More Hazardous Condition	Approximate Station of Wave Runup	H_c (ft)	T_p (s)	slope, m	SWEL at Toe (ft)	Runup Method	Runup (ft)	TWL (ft)
28	Scenario 1	1324	0.85	2.1	0.53	10.35	TAW	1.8	12.2

8. Levees

A levee is a man-made structure, usually an earthen embankment, designed and constructed in accordance with sound engineering practices to contain, control, or divert the flow of water so as to provide protection from temporary flooding. For purposes of the NFIP, FEMA will only recognize in its flood hazard and risk mapping effort those levee systems that meet, and continue to meet, minimum design, operation, and maintenance standards that are consistent with the level of protection sought through the comprehensive floodplain management criteria established by NFIP regulations, CFR 44 60.3. It is the responsibility of the community or other party seeking recognition of a levee system at the time of a flood risk study to provide the data outlined in 44 CFR 65.10. If the levee owner provides adequate information to certify that the levee provides protection from the base flood, FEMA considers the levee accredited, thus showing on the FIRM the protection provided by the levee for the 1-percent-annual-chance flood.

8.1. Levee Status and Flood Hazard Analysis Study Approach

Within northern San Mateo County, levee systems are currently identified along the San Francisco Bay in the communities of the City of San Mateo and the City of Foster City. The levee system fronting the community of North Shoreview in the City of San Mateo is not accredited on the effective FIRM. The Bayfront levee in the City of San Mateo, located between transect 26 and the City of San Mateo/Foster City boundary was accredited by FEMA in a letter addressed to the Honorable Brandt Grotte, Mayor of San Mateo, dated March 15, 2012. The Foster City levee, located at transects 29, 30, and 31, was accredited by FEMA in a letter addressed to the Ray Towne, Director of Public Works for the City of Foster City, dated July 23, 2007. The Foster City levee is shown as an accredited levee on the effective FIRM.

The effective FIRM for San Mateo County north of the San Mateo – Hayward Bridge shows the City of Foster City and the southern half of the City of San Mateo with flood zone designations of Zone X (Protected by Levee). The northern half of the City of San Mateo is mapped with a flood hazard designation of Zone AE with a BFE of 10 feet (NAVD).

For this study, wave runup ($R_{2\%}$) was evaluated at the shorelines of all levees for the purpose of establishing a BFE seaward of the levees. Maximum wave runup was also calculated for the purpose of evaluating the levees against FEMA's freeboard requirement (see following section).

The non-accredited levee in the City of San Mateo near North Shoreview was assumed not to provide protection from base flood inundation and overland wave propagation was modeled with WHAFIS for transects 22 through 25. The North Shoreview levee was included in the overland wave propagation analysis at its full crest elevation, but the stillwater elevation was extended inland past the crest to the intersection with the prevailing ground. Wave regeneration in the lee of the levee was calculated. This approach is meant to represent a partial failure or breach of the feature. In the event of a breach, floodwaters would inundate the low-lying areas behind the dikes, but the remaining sections of the dike would provide protection from wave hazards. Inundation and overland wave propagation was not evaluated for areas that are behind currently accredited levees and mapped as Zone X (Protected by Levee).

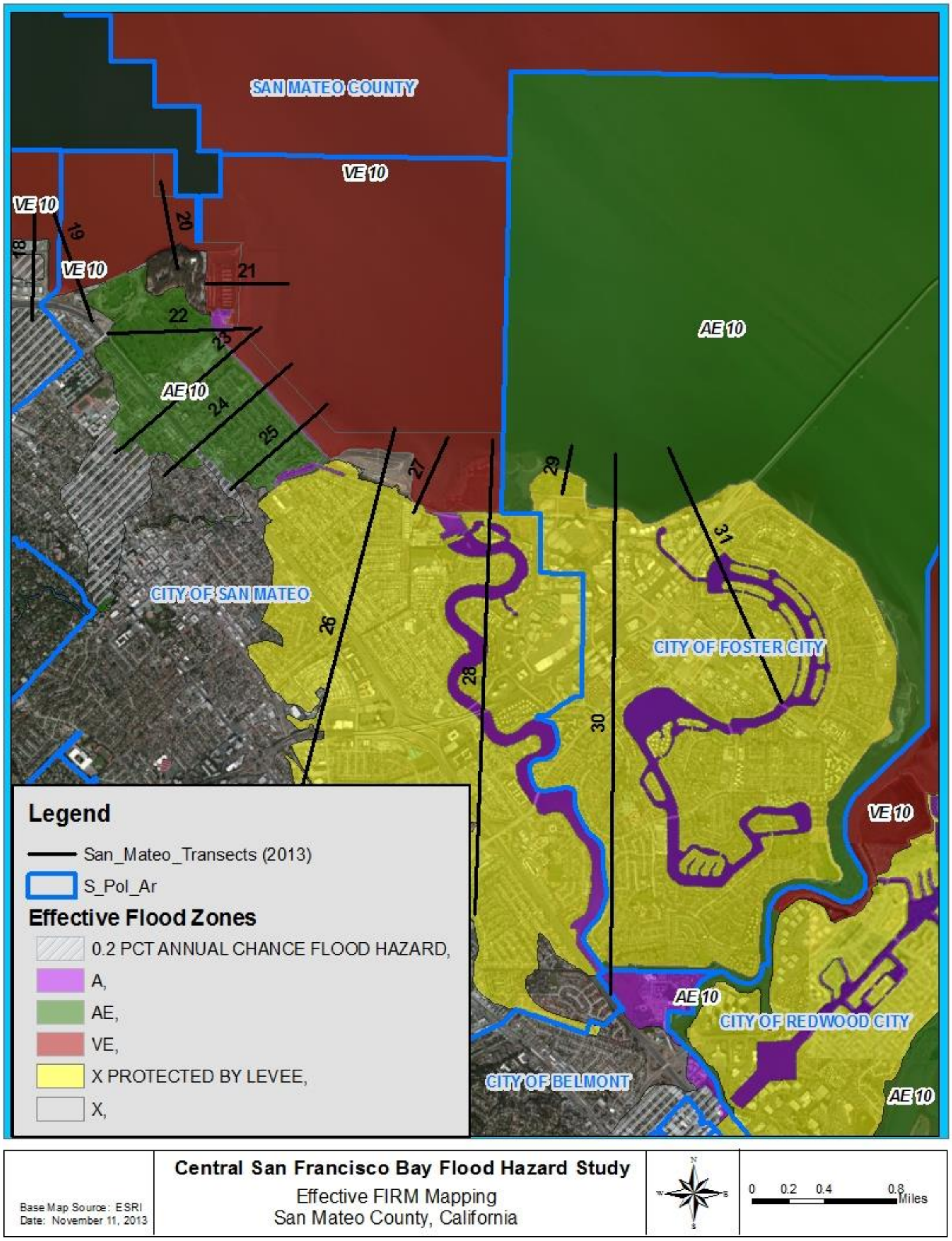

Figure 15. Effective mapping and new study transects in area of interest due to levees

8.2. Levee Freeboard Assessment

All levees were evaluated to determine if the levee heights meet levee freeboard requirements, per Title 44 of the Code of Federal Regulations (CFR), 65.10 (b)(1)(iii):

> For coastal levees, the freeboard must be established at 1 foot above the height of the 1-percent-annual-chance wave or the maximum wave runup (whichever is greater) associated with the 1-percent-annual-chance stillwater surge elevation at the site.

A corollary to the above, which is important in areas with relatively small wave action, further stipulates that:

> Under no circumstances will a freeboard of less than 2 feet above the 1-percent-annual-chance stillwater surge elevation be accepted. [44CFR 65.10 (b)(1)(iv)]

In addition, the maximum wave runup elevation was evaluated for all transects that intersect bayfront levees in the area shown in Figure 15. Maximum wave runup was calculated by converting the calculated $R_{2\%}$ for the annual maxima to R_{max} and then re-evaluating the statistical 1-percent-annual-chance TWLs. The following relationships from Walton (1992) were used to convert from $R_{2\%}$ to R_{max}.

$$R_{2\%} = 2.2 \times \bar{R} \quad \text{and} \quad R_{max} = 2.9 \times \bar{R}$$

Table 9 lists the approximate levee crest elevations in the vicinity of the transect. Levee crest elevations for transects 23-25 are based on the 2010 LiDAR. Levee crest elevations for transects 27 and 28 were obtained from the as-built Bayfront Levee Containment plan, entitled "City of San Mateo Bayfront Levee Profile B Alignment," signed by Mr. Charles D. Anderson, P.E., and dated January 25, 2012. Levee crest elevations for transects 30-32 were taken from surveys performed by Wilsey Ham Civil Engineers between June 2008 and March 2011. The surveys were provided by the City of Foster City for crest elevations of the levee pedway.

Table 9 also summarizes the maximum wave runup, controlling wave crest elevation, and 1-percent-annual-chance-stillwater surge elevations for the transects that intersect levees. Associated levee height requirements in order to meet FEMA's freeboard criteria are indicated and the levees are evaluated at each transect for whether the levee height is adequate to meet the freeboard criteria.

The North Shoreview and City of San Mateo levees meet the freeboard height requirements at the modeled transects for both the 2-feet above stillwater and the 1-foot above maximum wave runup elevation criteria. The Foster City levee does not meet the height requirements at the modeled transects for either the stillwater or the wave runup criterion.

Table 9. Levee height requirements to meet freeboard regulations

Transcect Number	Community	Max Ground (Levee Crest) Elevation (ft., NAVD88)	1% Annual Chance Stillwater Elevation (ft., NAVD88)	Ground vs. Stillwater Elevation Difference (ft.)	Approximate[2] 1% Annual Chance Controlling Wave Crest Elevation (ft., NAVD88)	Maximum Wave Runup Elevation (ft., NAVD88)	Max Ground (Levee Crest) vs. Wave Runup Elevation Difference (ft.)	Freeboard Requirement Satisfied?			
								2 ft. Above Stillwater Elevation		1 ft. Above Wave Runup	
								Levee Height Required (ft., NAVD88)	Height Met?	Levee Height Required (ft., NAVD88)	Height Met?
23	City of San Mateo--North Shoreview	14.8	10.3	4.5	11.3	13.4	1.4	12.3	YES	14.4	YES
24	City of San Mateo--North Shoreview	14.5	10.3	4.2	11.1	13.2	1.3	12.3	YES	14.2	YES
25	City of San Mateo--North Shoreview	14.8	10.3	4.5	11.3	13.0	1.8	12.3	YES	14.0	YES
26	City of San Mateo	*not applicable*[1]	10.3	*not applicable*[1]	11.1	*not applicable*[1]	*not applicable*[1]	*not applicable*[1]	*not applicable*[1]	*not applicable*[1]	*not applicable*[1]
27	City of San Mateo	14.2	10.4	3.8	11.5	13.1	1.1	12.4	YES	14.1	YES
28	City of San Mateo	14.7	10.4	4.3	11.0[3]	12.8	1.9	12.4	YES	13.8	YES
29	Foster City	11.1	10.4	0.7	11.2	14.0	-2.9	12.4	NO	15.0	NO
30	Foster City	11.4	10.4	1.0	11.4	14.5	-3.1	12.4	NO	15.5	NO
31	Foster City	11.6	10.4	1.2	11.8	14.4	-2.8	12.4	NO	15.4	NO

NOTE: This table should not be used to determine levee design criteria. It is intended to be used only for assessment of whether the levee system meets freeboard requirements.

[1] Transect crosses through Seal Point Park and does not actually intersect the levee feature.

[2] Reported wave crest elevations should be considered approximate. Wave crest elevations determined by extreme value analysis and taken from assigned regional wave study pass point location with the exception of transect 30.

[3] Wave crest elevation at levee toe from WHAFIS model results.

9. Results

Sections 5 through 7 summarize the runup, overtopping, and overland wave propagation results for the applicable transects. Those results are compiled to provide a summary of the dominant flood hazard at each transect. The 1-percent-annual-chance SWEL, wave crest elevation, runup elevation at the shoreline, and overtopping potential are provided for all San Mateo County transects in Table 10. For all transects, the fit of the resulting CDF to the annual maxima was evaluated for the ML and probability weighted moments solutions. In all cases, the ML method best fit the annual maxima data.

The SWEL, wave crest elevation, and runup elevation are statistical representations of the 1-percent-annual-chance value. For some of the transects with minimal wave action (e.g. 12), the 1-percent-annual-chance runup elevation is less than the 1-percent-annual-chance SWEL. It is important to note that these are statistical quantities and have no relevance in considering actual physical processes. For example, in the case outlined here, it would be physically impossible to represent a storm event with a runup elevation that was less than the SWEL; however, in a statistical sense, this could be the overall representation of the system. These results are a product of the statistical analysis used to calculate these values. Since the analysis contains only 31 years (or 54 years) of annual maxima, the 1-percent-annual-chance conditions have to be extrapolated from the extremal CDF. There is a certain amount of error intrinsic to fitting a CDF to data, and in conditions where there is minimal wave action and the TWL is similar (only slightly larger than the SWEL), this error can introduce unrealistic results, as are observed. For example, if there is a +/-2 percent error in the CDF fit for SWEL, a +/- 2 percent error in the CDF fit for TWL, and the TWL is typically only about 1 percent higher than the SWEL due to small wave action, it is easy to see how the extrapolated SWEL value could be higher than the TWL and still be within the error bounds. This is also why it is generally not seen in areas with higher wave energy.

Table 10. Summary of Results

Transect Number	Shoreline Structure	Runup Method	WHAFIS	1% SWEL (ft NAVD)	0.2% SWEL (ft NAVD)	1% Wave Crest Elevation (ft NAVD)	1% Runup Elevation (ft NAVD)	Overtopping (Y/N)
1	Revetment	TAW	—	10.26	11.61	11.84	13.68	Y
2	Revetment	TAW	—	10.29	11.69	11.52	12.63	Y
3	Revetment	TAW	—	10.28	11.69	10.40	10.25	N
4	Revetment	TAW	—	10.35	11.82	11.91	13.71	Y
5	Revetment	TAW	YES	10.35	11.82	11.61	13.87	Y
6	Revetment	DIM	—	10.36	11.83	12.02	10.68	N
7	Revetment	TAW	—	10.41	11.95	11.63	13.31	Y
8	Revetment	DIM	—	10.41	11.94	12.07	11.10	Y
9	NA	TAW	—	10.43	11.99	11.76	12.25	N
10	Revetment	TAW	—	10.45	12.03	11.93	13.80	N
11	Revetment	TAW	—	10.47	12.06	10.27	11.49	N
12	NA	DIM	—	10.46	12.04	11.62	10.42	N
13	NA	TAW	—	10.46	12.05	11.67	12.97	Y
14	NA	DIM	—	10.18	11.26	11.48	10.56	Y
15	Revetment	TAW	—	10.18	11.26	10.50	10.67	N
16	Revetment	TAW	—	10.20	11.31	10.71	12.02	Y
17	Revetment	TAW	—	10.20	11.33	10.87	12.26	Y
18	Revetment	TAW	—	10.22	11.37	11.00	12.48	Y
19	Revetment	TAW	YES	10.23	11.39	11.29	11.81	Y
20	NA	TAW	—	10.24	11.42	11.14	15.89	N
21	NA	-	YES	10.30	11.55	11.48	—	—
22	NA	-	YES	10.31	11.58	11.23	—	—
23	Levee/ Revetment	TAW	YES	10.31	11.58	11.34	12.68	N

Transect Number	Shoreline Structure	Runup Method	WHAFIS	1% SWEL (ft NAVD)	0.2% SWEL (ft NAVD)	1% Wave Crest Elevation (ft NAVD)	1% Runup Elevation (ft NAVD)	Overtopping (Y/N)
24	Levee/ Revetment	TAW	YES	10.32	11.60	11.12	12.46	N
25	Levee/ Revetment	TAW	YES	10.33	11.63	11.34	12.49	N
26	Revetment	TAW	—	10.34	11.66	11.09	12.38	N
27	Levee/ Revetment	TAW	—	10.36	11.70	11.47	12.13	N
28	Levee	-	YES	10.37	11.72	11.13	—	—
29	Levee/ Revetment	TAW	—	10.38	11.74	11.20	12.58	Y
30	Levee/ Revetment	TAW	—	10.39	11.77	11.40	13.00	Y
31	Levee/ Revetment	TAW	—	10.41	11.80	11.85	12.96	Y

10. Conclusion

Coastal flood hazard analyses were conducted for the northern San Mateo County coast of San Francisco Bay, north of the San Mateo – Hayward Bridge. Wave setup, runup, overtopping, and overland wave propagation were considered as potential flooding hazards along 31 transects. Wave runup was found to be to the dominant flood hazard for 25 of the transects. Stillwater inundation flooding and overland wave propagation was the dominant flood hazard for three transects. The remaining five transects were affected by both wave runup and overland wave propagation. The potential for wave overtopping was identified for 15 of the 31 transects for which wave runup was evaluated at the shoreline.

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Appendix A. Wave Setup Contribution to Total Water Levels at Structures

FEMA's *Guidelines for Coastal Flood Hazard Analysis and Mapping for the Pacific Coast of the United States* (FEMA, 2005) specifies that total runup includes three components: static wave setup, dynamic wave setup, and incident wave runup (Equation D.4.5-1). Section D.4.2 defines the total water level (TWL) as the sum of the stillwater level, the wave setup, and wave runup. Following Equation D.4.5-1, the incident wave runup (R_{inc}) for structures is added to the wave setup values (static and dynamic) statistically based on the application of DIM to find the total runup. DIM is also applied to estimate the setup water surface at the toe of the structure.

The recommended approach for calculating incident wave runup on structures is the TAW method and is presented in Equation D.4.5-19. The reference water level at the toe of the barrier for runup calculations is DWL2% and includes the stillwater level, the static wave setup, and the 2-percent dynamic wave setup. To avoid double-counting the wave setup influence, the dynamic wave setup at the toe of the structure is adjusted based on the value of the JONSWAP Gamma.

In Section D.4.5.1.6, sample computations of total runup are presented. Included in this section are two examples of runup on structures. For both, the combined dynamic setup and incident wave runup are added to the static setup to determine a total runup height above the SWEL.

Dean (2010) investigates whether wave setup should be calculated separately and added to the runup results obtained from the TAW runup calculation procedure. Because TAW is based on wave tank tests, which inherently include wave setup in the runup measurements landward of the toe of the structure, wave setup should not be added explicitly to the runup calculations in the region landward of the toe. However, wave and water levels at the toe of the structure must be known, in order to compute runup. This reference level should be the water level, which includes wave setup seaward of the toe of the structure.

Dean summarizes that wave setup landward of the toe of the structure clearly should not be included separately in the wave runup calculations by the TAW methodology. To do so would result in including setup twice. It is, therefore, recommended that the combined storm surge, astronomical tide, and any wave setup at the toe of the structure be the water level to which the wave runup determined by the TAW methodology is added.

Appendix B. Total Water Level Tables

See Excel workbook file "AppendixB_Runup_Tables.xlsx" that accompanies this report.

Appendix C. WHAFIS Results Profiles

See stand-alone document “SanMateo_Appendix_C.pdf” that accompanies this report.

Appendix D. Coastal Analysis Submission File Directory

Coastal Analysis Submission Directory			
J:\R09\CALIFORNIA_06\SAN_MATEO_06081\SAN_MATEO_081C\11-09-1227S\SubmissionUpload\Coastal\2161815			
SUB DIRECTORY			DESCRIPTION
J:\R09\CALIFORNIA_06\SAN_MATEO_06081\SAN_MATEO_081C\11-09-1227S\SubmissionUpload\Coastal\2161815\SanFranciscoBay\CentralBay_northernSanMateo\Correspondence			
	J:\R09\CALIFORNIA_06\SAN_MATEO_06081\SAN_MATEO_081C\11-09-1227S\SubmissionUpload\Coastal\2161815\SanFranciscoBay\CentralBay_northernSanMateo\Correspondence\Meetings		minutes from study-related meetings
	J:\R09\CALIFORNIA_06\SAN_MATEO_06081\SAN_MATEO_081C\11-09-1227S\SubmissionUpload\Coastal\2161815\SanFranciscoBay\CentralBay_northernSanMateo\Correspondence\QC		external QA/QC review forms
J:\R09\CALIFORNIA_06\SAN_MATEO_06081\SAN_MATEO_081C\11-09-1227S\SubmissionUpload\Coastal\2161815\SanFranciscoBay\CentralBay_northernSanMateo\General			study reporting and supporting documentation files
	J:\R09\CALIFORNIA_06\SAN_MATEO_06081\SAN_MATEO_081C\11-09-1227S\SubmissionUpload\Coastal\2161815\SanFranciscoBay\CentralBay_northernSanMateo\General\Central SF Bay Field Survey		various files describing field reconnaisance effort
	J:\R09\CALIFORNIA_06\SAN_MATEO_06081\SAN_MATEO_081C\11-09-1227S\SubmissionUpload\Coastal\2161815\SanFranciscoBay\CentralBay_northernSanMateo\General\Supporting Documentation		technical memos referenced in study documentation
J:\R09\CALIFORNIA_06\SAN_MATEO_06081\SAN_MATEO_081C\11-09-1227S\SubmissionUpload\Coastal\2161815\SanFranciscoBay\CentralBay_northernSanMateo\Nearshore_Wave_Models			data files for 1D wave hazard modeling
	J:\R09\CALIFORNIA_06\SAN_MATEO_06081\SAN_MATEO_081C\11-09-1227S\SubmissionUpload\Coastal\2161815\SanFranciscoBay\CentralBay_northernSanMateo\Nearshore_Wave_Models\Runup		results and summary tables for response-based runup calculations
		\LeveeSurveyData	topographic survey data and levee accredidation letters for Foster City and San Mateo Bayfront levees
		\Simulations	annual TWL maxima tables
		\Spatial Files	full-length transects used to define runup profiles
	J:\R09\CALIFORNIA_06\SAN_MATEO_06081\SAN_MATEO_081C\11-09-1227S\SubmissionUpload\Coastal\2161815\SanFranciscoBay\CentralBay_northernSanMateo\Nearshore_Wave_Models\WHAFIS		input/output and summary tables for WHAFIS modeling; including inland runup calculations
		\Simulations\EVA_for WHAFIS Input	extreme value analysis summary table and annual maxima for SWEL, wave height, and wave crest elevations
		\Simulations\Profiles	wave envelope profiles
		\Simulations\S1 Input-Output	WHAFIS input/output for Scenario 1
		\Simulations\S2 Input-Output	WHAFIS input/output for Scenario 2
		\Simulations\Wind	data files for WHAFIS wind speed analysis
		\Spatial Files	GIS databases with WHAFIS-related data
J:\R09\CALIFORNIA_06\SAN_MATEO_06081\SAN_MATEO_081C\11-09-1227S\SubmissionUpload\Coastal\2161815\SanFranciscoBay\CentralBay_northernSanMateo\Transects			GIS transect layer