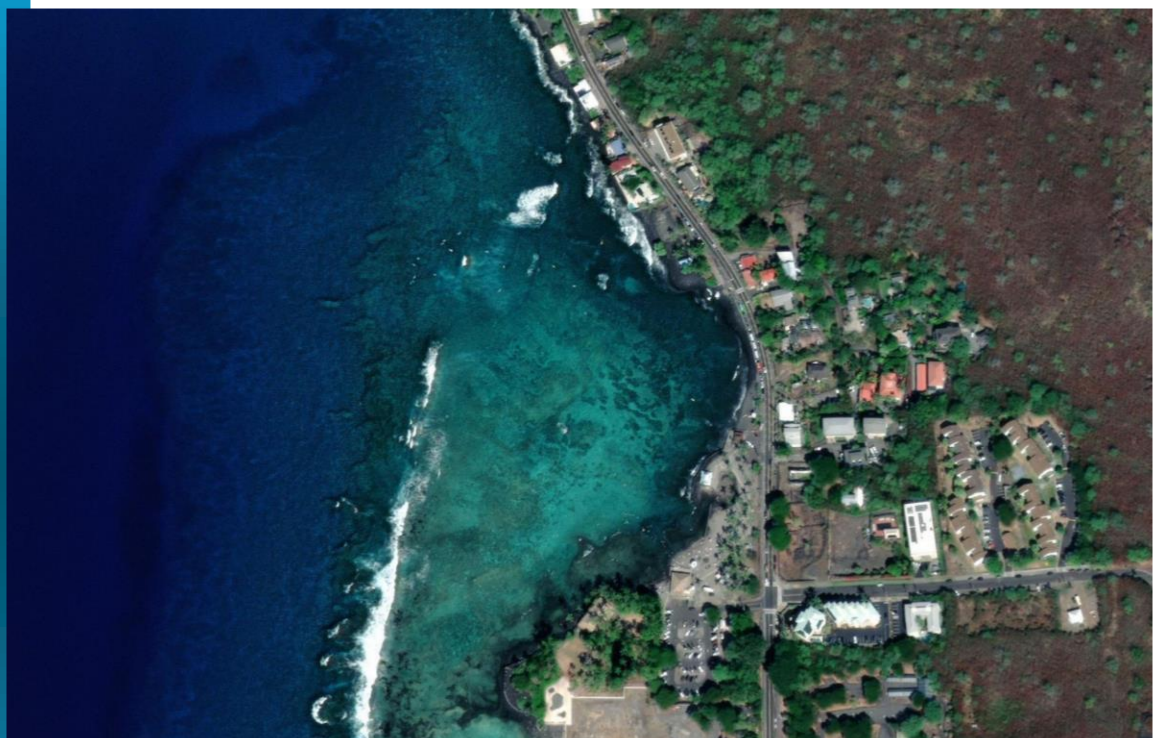
Final Report
Coastal Assessment for Kahaluu Beach Park Planning

Kahaluu-Keauhou, Hawaii Island, Hawaii

August 2021



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1. INTRODUCTION

1.1 Project Location and General Description

Kahaluu Beach Park is located on the west, or Kona, coast of the Island of Hawaii (Big Island), as shown in Figure 1-1. The park is approximately 4.5 miles south of the town of Kailua-Kona and is situated in the midst of a major resort area.

The park is owned and operated by the County of Hawaii and is heavily utilized by both residents and tourists. Sandy beach areas are relatively limited on the Big Island, and thus Kahaluu Beach Park is a valuable recreational resource. The Park is about 5.4 acres in size, with about 700 feet of shoreline and typically less than 0.5 acres of sandy beach area. Approximately one-third of the park shoreline is protected by grouted rock and concrete-ruble-masonry (CRM) walls, and there are backshore walls protecting park facilities and landscaping. Park facilities include pavilions, restrooms, showers, picnic areas, a basketball court, and parking area. Backshore elevations data, from the USACE SHOALS LiDAR dataset collected in 2013, shows backshore elevations in the park vary between +5 to +10 feet MSL. The parking lot area is lower than the surrounding park area with elevations between +4.3 to +4.9 feet MSL. The existing park facilities and backshore elevations are shown on Figure 1-2. The nearshore water in the vicinity of the park is very popular for swimming, snorkeling, and surfing.

The bay shoreline is fronted by a shallow basalt rock shelf, typically 2 to 4 feet deep at mean sea level (MSL) on the southern half and 4 to 6 feet deep on the northern half. The seaward side of the southern portion is elevated slightly with respect to the inner portion of the shelf and is lined with large boulders which protrude above the water surface (the “Menehune breakwater”). The origin of these boulders is unknown. The boulders and shallow water on the south side dissipate incident wave energy before it reaches shore, while the relatively deeper water on the north side of the bay permits more wave energy to enter the bay on this side.

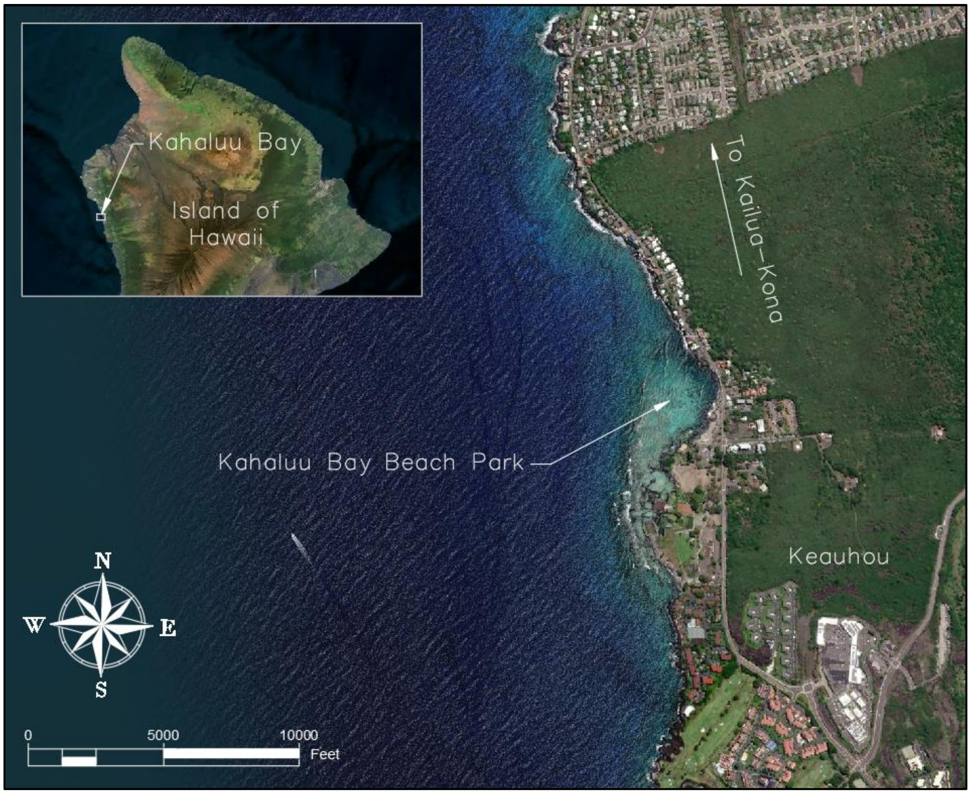

Figure 1-1. Kahaluu Bay Beach Park regional overview

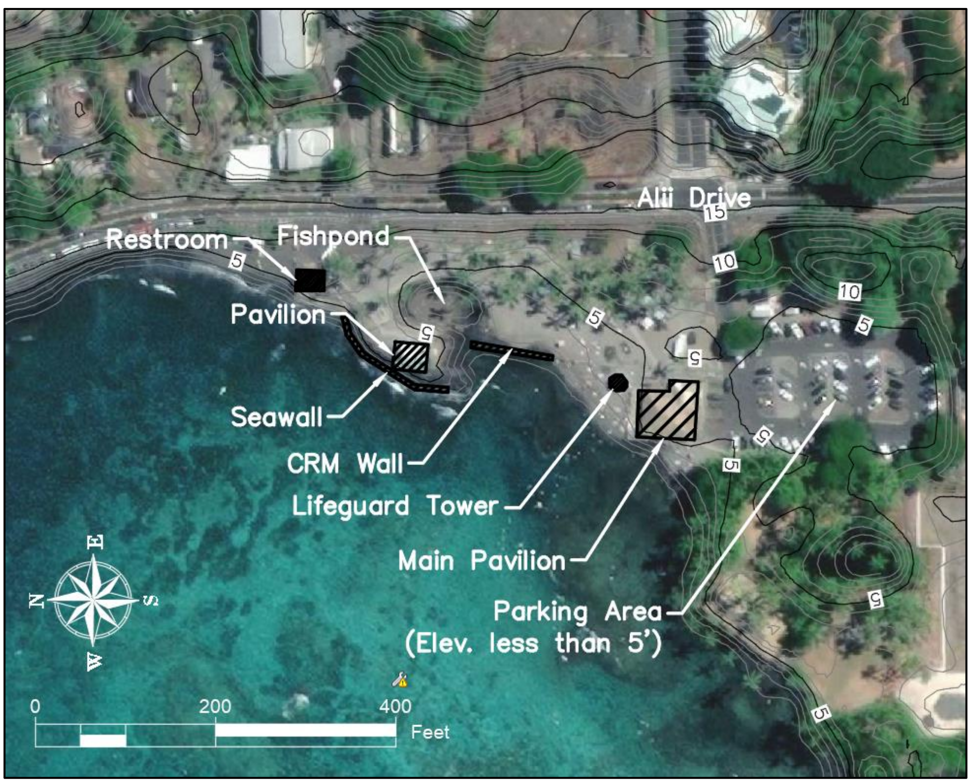

Figure 1-2. Existing park facilities (elevation contours in feet relative to MSL)

1.2 Project Purpose and Scope of Work

Planning for maintaining and improving Kahaluu Beach Park (KBP) has been on-going for more than 25 years. In 1992 Sea Engineering, Inc. (SEI) prepared a Coastal Engineering Evaluation to support preparation of a park master plan. In 2008 the University of Washington, Department of Landscape Architecture, developed the "Kahaluu Beach Park: Conceptual Master Plan". The park is directly tied to the coast and the sea, and the design parameters, considerations, and constraints imposed by the sea are integral to improvement planning. Recent identification of climate change and sea level rise as major factors in coastal planning for the future further emphasize the need for coastal engineering input to help guide the planning process. In 2013 the "Kahaluu Beach Park: Sequencing for Studies and Planning was prepared. This study for a Coastal Assessment and Hazards Analysis will support *Step 2. Collect Baseline Data, a. Conduct Coastal Studies*, of the planning sequence.

The project includes the following general work tasks:

- Available coastal and oceanographic information search and review
- Site investigations
- Evaluation of coastal planning and design parameters
- Improvement concept development
- Agency and Public coordination
- Preparation of a project report

This report presents the results of the coastal assessment, and oceanographic parameters and considerations pertinent to the design of park improvements and resiliency for expected future climate and sea level change. Also presented for consideration are preliminary concept plans for park changes in response to sea level change.

2. OCEANOGRAPHIC DESIGN PARAMETERS

2.1 Bathymetry

Bathymetry data for Kahaluu Bay was obtained from the USACE SHOALS LiDAR dataset which was collected in 2013 by aerial reconnaissance. Figure 2-1 shows the bathymetric contours offshore from the beach park. Kahaluu Bay is an embayment with a shallow reef flat (approx. 2-6 feet depth) which extends seaward by about 1,000 feet from the shoreline. A shallow reef edge separates the reef flat and the deeper offshore waters. From the reef edge the bottom gradually slopes down to a depth of about 50 feet. A steep drop-off exists between the 50 to 75-foot depth contour and then the depths gradually deepen seaward of the 75-foot depth. To the north of the bay, there exists a small submarine canyon which consists of more gradual slopes from the shoreline to the deeper offshore waters.

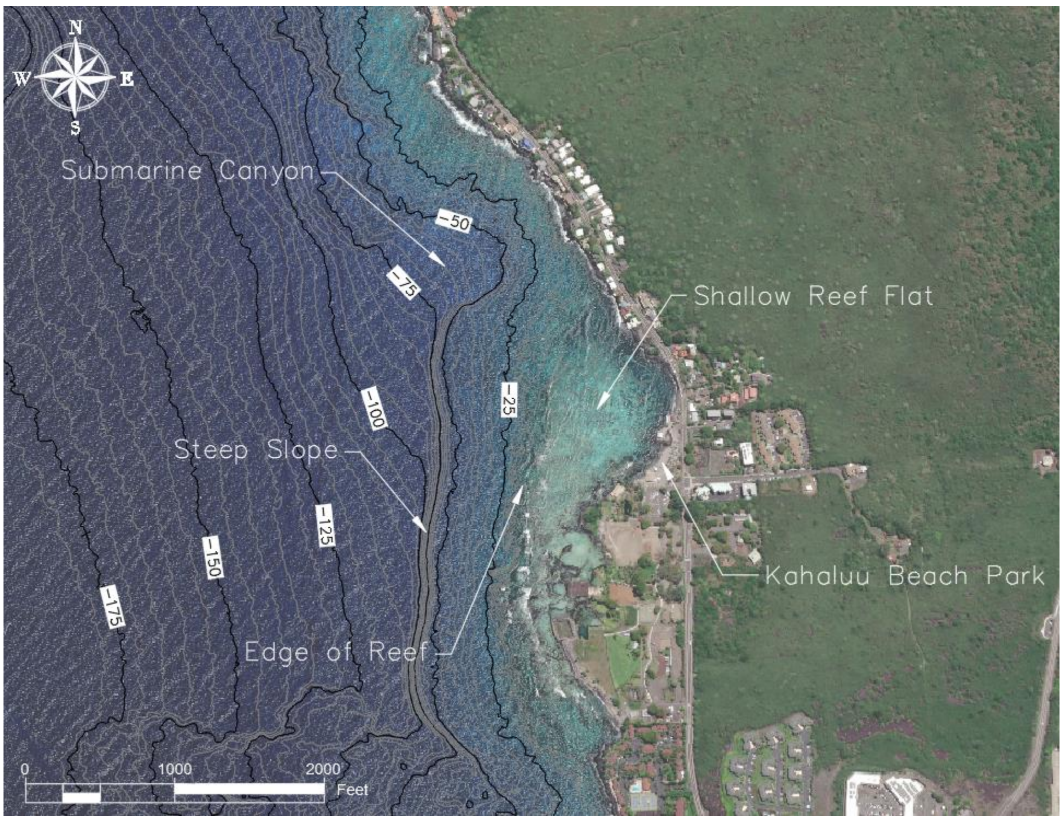

Figure 2-1. Kahaluu Bay bathymetry (elevations in feet relative to MSL)

2.2 Winds

The climate in Hawaii is characterized by two distinct seasons, primarily defined by the annual variation in persistence of the northeast tradewinds. During the summer months (May through September) the tradewinds predominate, blowing out of the northeast approximately 80 percent of the time with speeds generally from 10 to 25 mph. the winter season (November through March) is characterized by weakening of the tradewind persistence and the occurrence of southerly or westerly winds as a result of localized low pressure and frontal systems.

Kahaluu Bay is sheltered, as is most of the west coast of the island of Hawaii, from the prevailing tradewinds by large mountains of Mauna Kea and Mauna Loa. Light and variable winds prevail to about 40 miles offshore, with onshore-offshore land/sea breezes predominating due to the diurnal heating and cooling of the island.

The project area is directly exposed to Kona winds blowing from the south-southwest to west, generated by low pressure systems or cold fronts moving toward Hawaii. Periods of Kona winds are typically of short duration (1 to 3 days), and damaging Kona winds are not common. A severe Kona storm in January 1980, however, had sustained wind speeds of 30 knots or greater for a period of several days in the vicinity of Kahaluu Beach Park.

2.3 Water Levels

2.3.1 *Astronomical Tides*

Hawaii tides are semi-diurnal with pronounced diurnal inequalities (i.e., two high and low tides each 24-hour period with different elevations). Variation of the tidal range results from the relative position of the moon and the sun. During full moon and new moon phases, the moon and sun act together to produce larger "spring" tides; when the moon is in its first or last quarter, smaller "neap" tides occur. The cycle of spring to neap tides and back is half the 27-day period of the moon's revolution around the earth and is known as the fortnightly cycle. The combination of diurnal, semi-diurnal and fortnightly cycles dominates variations in sea level throughout the Hawaiian Islands.

Tidal predictions and historical extreme water levels are provided by the National Ocean and Atmospheric Administration (NOAA) NOS Center for Operational Oceanographic Products and Services. A tide station is located at Kawaihae (station ID: 1617433) approximately 32 miles north-northeast of Kahaluu Beach Park. Water level data based on the 1983 to 2001 tidal epoch is shown in Table 2-1.

Table 2-1 Water level data for Kawaihae, Station 1617433 (NOAA)

Datum	Elevation (feet, MLLW)	Elevation (feet, MSL)
Mean Higher High Water (MHHW)	+2.12	+1.20
Mean High Water (MHW)	+1.62	+0.70
Mean Sea Level (MSL)	+0.92	0.00
Mean Low Water (MLW)	+0.18	-0.74
Mean Lower Low Water (MLLW)	0.00	-0.92

Elevations in this study are referred to the MSL tidal datum.

2.3.2 *Sea Level Rise*

The present rate of global mean sea-level change (SLC) is +3.4 ± 0.4 mm/year (Sweet, 2017), where a positive number represents a rising sea level. SLC appears to be accelerating compared to the mean of the 20th Century. Factors contributing to the measured rise in sea level include decreasing global ice volume and warming of the ocean. Sea level, however, is highly variable. The historical sea level trend for Hilo Bay, Station 1617760, is shown in Figure 2-2 (NOAA, 2017). The mean historical rate of sea level change (RSLC) is +3.12 ± 0.29 mm/yr based on monthly data for the period 1927 to 2020. The tide gauge data also shows monthly mean sea level anomalies up to 0.5 feet (15 cm) above the linear trend in Hilo Bay that are likely mesoscale eddies.

The National Oceanic and Atmospheric Administration (NOAA) recently (NOAA, 2017) revised their sea level change projections through 2100 taking into account up-to-date scientific research and measurements. NOAA is projecting that global sea level rise as shown by their "Extreme" scenario could be as high as about 8 feet by 2100. NOAA's recent report also identifies specific regions that are susceptible to a higher than average rise in sea level. Hawaii has thus far experienced a rate of sea level rise that is less than the global average; however, this is expected to change. Hawaii is in the "far field" of the effects of melting land ice. This means that those effects have been significantly less in Hawaii compared to areas closer to the ice melt. Over the next few decades, this effect is predicted to spread to Hawaii, which will then experience sea level rise greater than the global average.

Figure 2-3 presents mean sea level rise scenarios for Hawaii based on the revised NOAA projections, taking into account the far-field effects. While the projections are based on the most current scientific models and measurements, discretion is necessary for selecting the appropriate scenario. Selecting the appropriate sea level change projection is a function of many parameters, including topography, coastal setting, criticality of infrastructure, the potential for resilience, budget, and function.

Sea level rise has the potential to impact beaches and shorelines in Hawaii. Rising sea level will result in greater wave energy at the shoreline. Impacts may include beach narrowing and beach loss, loss of land due to erosion, and infrastructure damage due to inundation and flooding. The impacts from anomalous sea level events (e.g., mesoscale eddies, storm surge) are also likely to increase. A 2015 study (Anderson et al., 2015) found that, due to increasing sea level rise, average shoreline recession (erosion) in Hawaii is expected to be nearly twice the historical extrapolation by 2050, and nearly 2.5 times the historical extrapolation by 2100 (Anderson et al., 2015).

The State of Hawaii recently published the *Sea Level Rise Vulnerability and Adaptation Report for Hawaii*, which discusses the anticipated impacts of projected future sea level rise on coastal hazards, and the potential physical, economic, social, environmental, and cultural impacts of sea level rise in Hawaii (Hawaii Climate Change Mitigation and Adaptation Commission, 2017). The University of Hawaii conducted numerical modeling to estimate the potential impacts that a 3.2-foot rise in sea level would have on coastal hazards including passive flooding, annual high wave flooding, and coastal erosion. For Hawaii Island only passive flooding was analyzed.

A sea level rise of 2.54 feet was chosen for design purposes at the project property. This corresponds to the *Intermediate* rate over a 50-year design life which is suitable for planning and design purposes for a project of this scale. While critical infrastructure such as roads, power plants, and hospitals may require the highest level of protection, it is reasonable to design coastal protection and stabilization structures for a lesser level, in this case a 50-year lifespan. Coastal structures require ongoing monitoring and regular maintenance due to their exposure to the degrading effect of marine processes. The basis of design parameters, and consequent design life are based on typical functional use of similar coastal structures. Designing for conditions, such as significantly higher sea levels, that are predicted for time periods that well exceed the design life of the structure will produce more robust installations but will well exceed their functional performance requirements during their serviceable lifespans. Designing for a lesser sea level rise is still consistent with the Mayor's directive, as the sea level rise that the coastal stabilization structures to be evaluated in a follow-up report are expected to experience during their design lifetime would likely be less than the 3.2 feet presented in the directive.

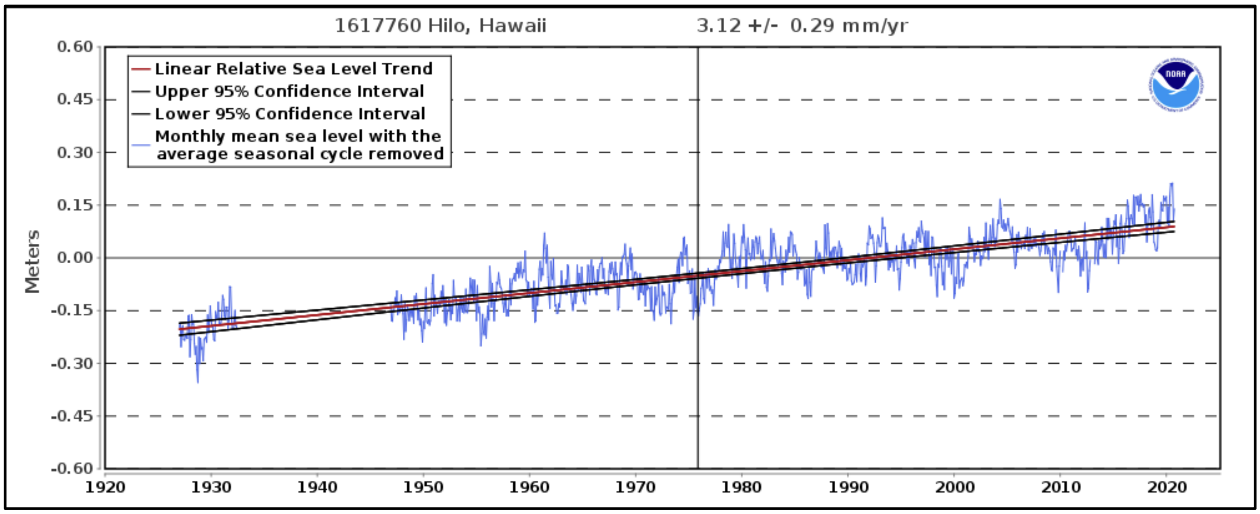

Figure 2-2 Mean sea level trend, Hilo Bay, Station 1617760, 1927 to 2020 (NOAA, 2017)

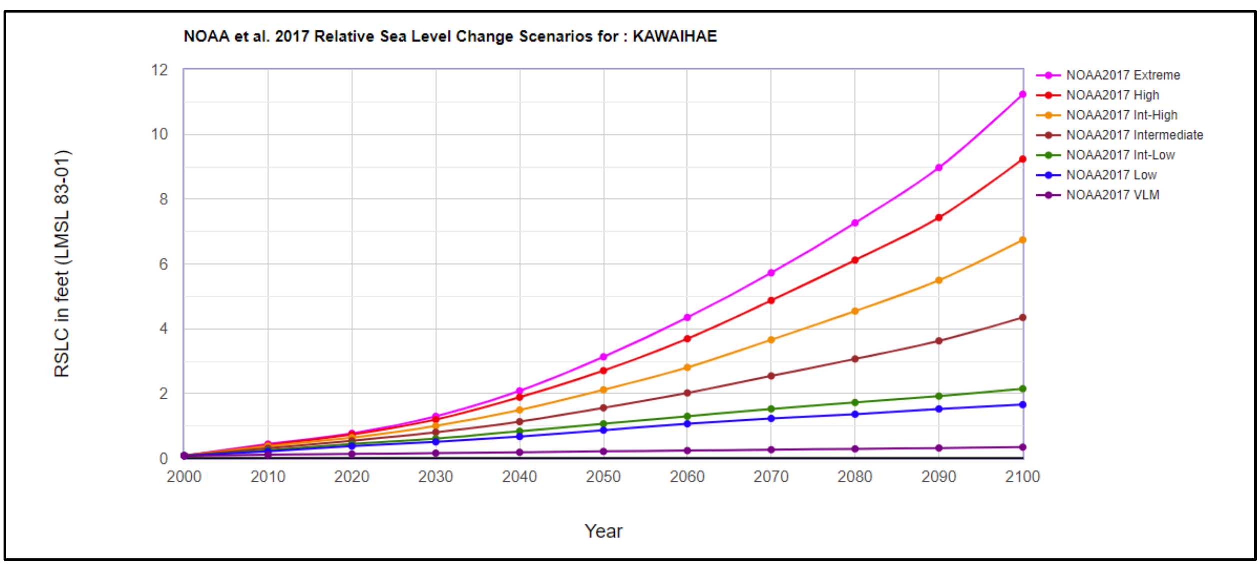

Figure 2-3 Hawaii sea level rise projections (adapted from NOAA, 2017)

Table 2-2 Hawaii Local Mean Sea Level rise scenarios for Kawaihae Tide Station (feet)

Scenario	2020	2030	2040	2050	2060	2070	2080	2090	2100
Extreme	0.77	1.30	2.09	3.14	4.35	5.73	7.27	8.98	11.24
High	0.74	1.20	1.89	2.71	3.69	4.87	6.12	7.43	9.24
Intermediate-High	0.64	1.00	1.49	2.12	2.81	3.66	4.55	5.50	6.74
Intermediate	0.54	0.81	1.13	1.56	2.02	2.54	3.07	3.63	4.35
Intermediate-Low	0.44	0.61	0.84	1.07	1.30	1.53	1.72	1.92	2.15
Low	0.38	0.51	0.67	0.87	1.07	1.23	1.36	1.53	1.66

2.3.3 *Sea Level Anomaly*

The ocean surface does not have a consistent elevation. In this report, sea level anomalies (SLA) are defined as the difference between the measured and predicted tides recorded at the Hilo Bay and Kawaihae NOAA tide stations. SLA exist as a result of such processes as El Nino, global warming, geostrophic currents due to the rotation of the earth, and mesoscale eddies that propagate across the ocean. Hawaii is subject to periodic extreme tide levels due to large oceanic eddies and other oceanographic phenomena that have recently been recognized and that sometimes propagate through the islands. Mesoscale eddies produce tide levels that can be up to 0.5 feet higher than normal for periods up to several weeks (Firing and Merrifield, 2004). An additional temporary sea level rise on the order of 0.5 feet has also been associated with phenomena related to the El Niño /Southern Oscillation (ENSO).

While “King Tides” received negative notoriety as being the cause of the inundation, they contributed only a small fraction of the water level rise. King Tides refer to the higher tide levels that are a result of the alignment of the earth, sun, and moon during the winter and summer months. During these times, high tide can reach an elevation of as much as +3.0 ft MLLW. In 2017, Hawaii experienced longer than expected elevated water levels, leading to significant coastal inundation, especially at low-elevation urban areas. The start of 2020 also marked an extended period of large SLA. Figure 2-4 shows the extreme water levels from January 11 and 12, 2020 measured at Hilo Bay. During this time period, sea level anomaly of 1.0 to 1.3 feet added to the winter King Tides resulting in the highest recorded water level at Hilo Bay to date of +3.1 ft MSL.

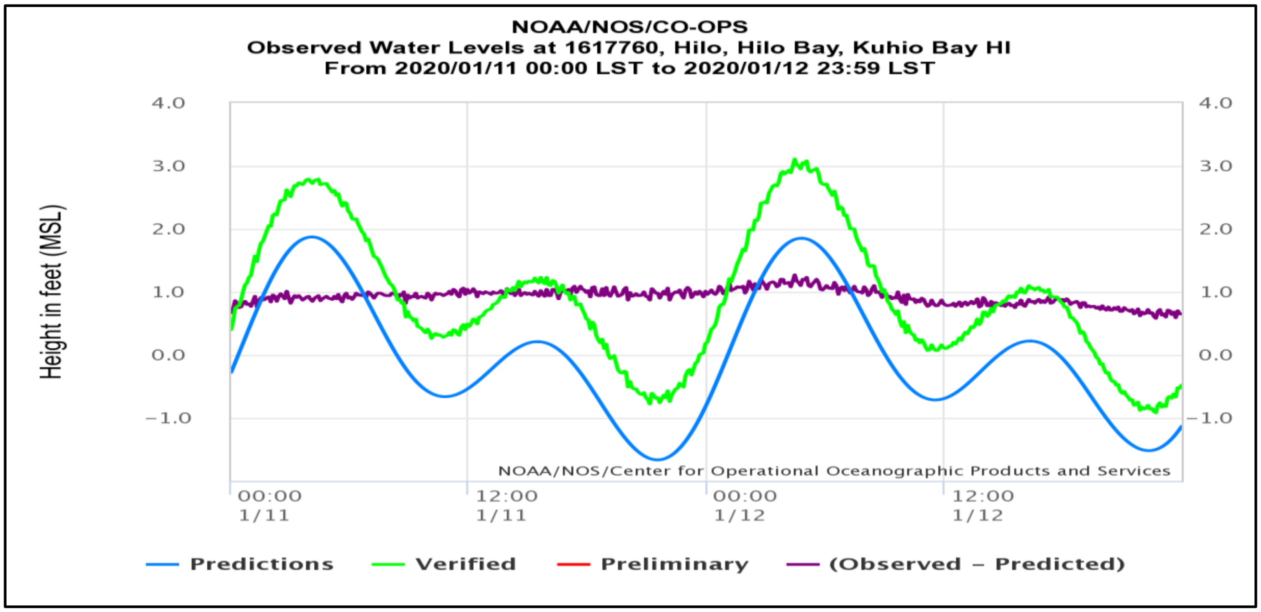

Figure 2-4 Predicted and measured tides at Hilo Bay (January 11-12, 2020)

2.4 Waves

2.4.1 *General Wave Climate*

The wave climate in Hawaii is dominated by long period swell generated by distant storm systems, by relatively low amplitude, short period waves generated by more local winds, and the occasional bursts of energy associated with intense local storms. Typically, Hawaii receives five general surface gravity wave types: 1) northeast tradewind waves, 2) North Pacific swell, 3) South Pacific swell, 4) southeast tradewind waves, and 5) Kona wind waves. The dominant swell regimes for Hawaii are shown in Figure 2-5.

Tradewind waves occur throughout the year and are the most persistent April through September when they usually dominate the local wave climate. These winds result from the strong and steady tradewinds blowing from the northeast quadrant over long fetches of open ocean. Tradewind deepwater waves are typically between 3 and 8 feet high with periods of 5 to 10 seconds, depending upon the strength of the tradewinds and how far the fetch extends east of the Hawaiian Islands. The direction of approach, like the tradewinds themselves, varies between north-northeast and east-southeast and is centered on the east-northeast direction. The project site is completely sheltered from prevailing tradewind waves.

During the winter months in the northern hemisphere, strong storms are frequent in the North Pacific in the mid-latitudes and near the Aleutian Islands. These storms generate large North Pacific swells that range in direction from west-northwest to northeast and arrive at the northern Hawaiian shores with little attenuation of wave energy. These are the waves that have made surfing beaches on the north shores of Oahu and Maui famous. Deepwater wave heights often reach 15 feet and in extreme cases can reach 30 feet. Periods vary between 12 and 20 seconds, depending on the location of the storm. The project site is well sheltered from most North Pacific swell, however, swell events approaching from the west to west-northwest may directly impact the project site.

South Pacific swell is generated by storms in the southern hemisphere and is most prevalent during the summer months of April through September. Traveling distances of up to 5,000 miles, these waves arrive with relatively low deepwater wave heights of 1 to 4 feet and periods of 14 to 20 seconds. Depending on the positions and tracks of the southern hemisphere storms, southern swells approach between the southeasterly and southwesterly directions. The project site is exposed to South Pacific swell; however, these waves would approach at an oblique angle to the coastline.

Kona storm waves directly approach the project site; however, these waves are fairly infrequent, occurring only about 10 percent of the time during a typical year. Kona waves typically range in period from 6 to 10 seconds with heights of 5 to 10 feet and approach from south to west. Deepwater wave heights during the severe Kona storm of January 1980 were about 17 feet. The project site is directly exposed to Kona storm waves which would typically approach the coastline head on.

Severe tropical storms and hurricanes have the potential to generate extremely large waves. Recent hurricanes impacting the Hawaiian Islands include Hurricane Iwa in 1982 and Hurricane Iniki in 1992. Iniki directly hit the island of Kauai and resulted in large waves along the southern and western shores of the Hawaiian Islands. Damage from these hurricanes was extensive. However, hurricane tracks typically travel from east to west across the Pacific and through Hawaii (Figure 2-6) and are unlikely to reverse direction towards the project site and generate large waves at the shoreline.

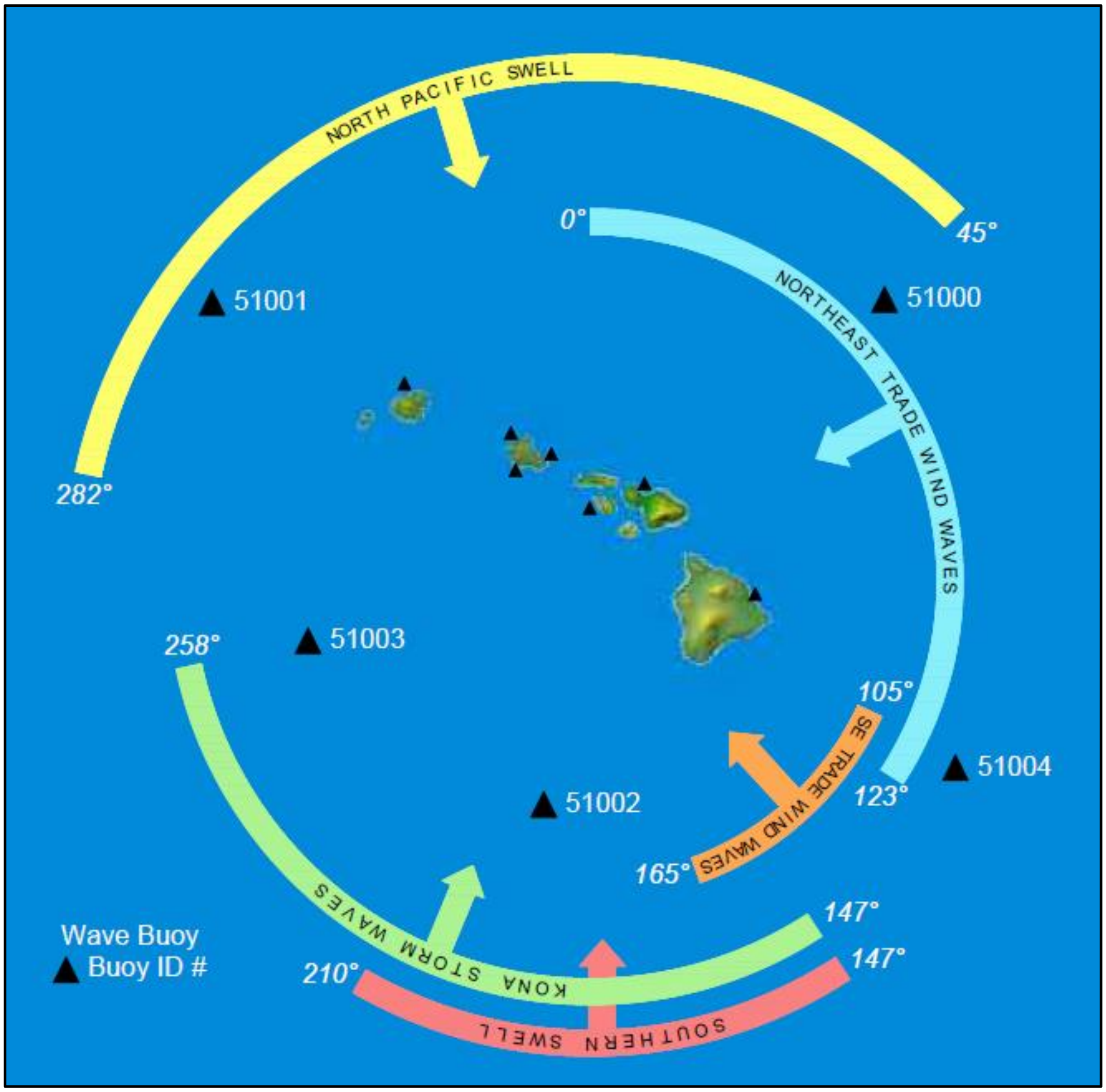

Figure 2-5. Hawaii dominant swell regimes

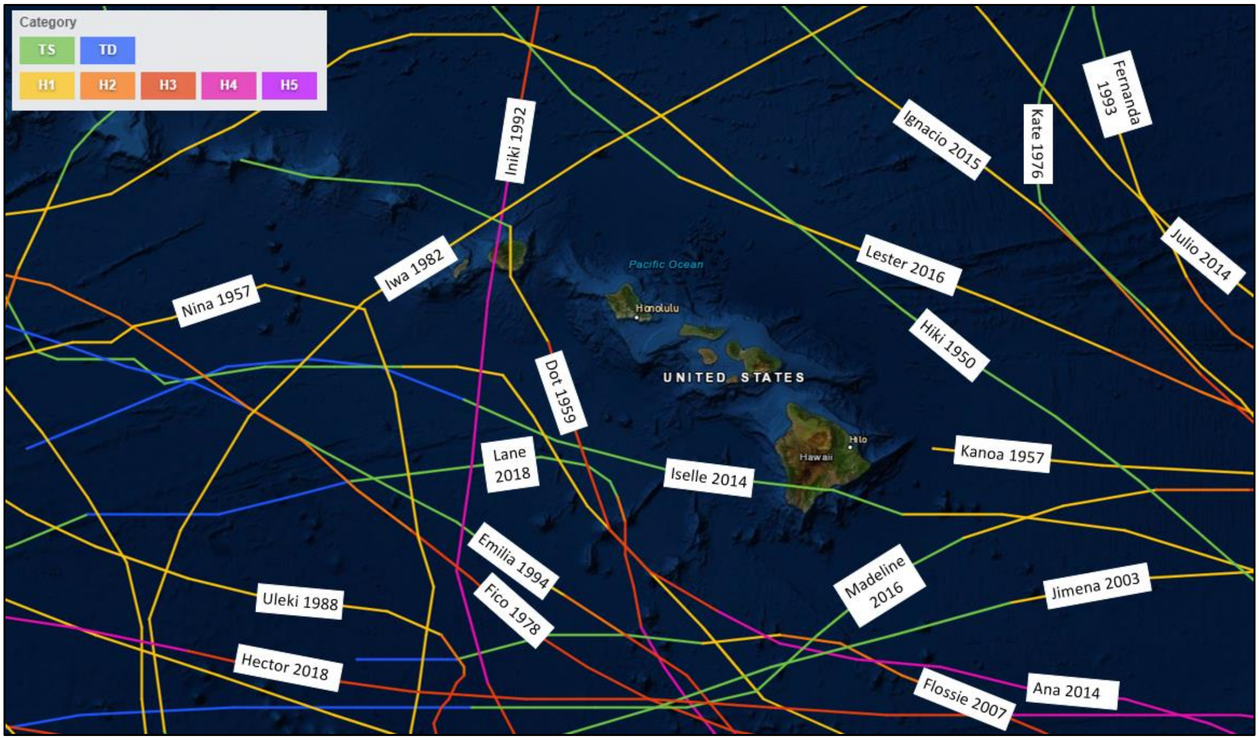

Figure 2-6. Hawaii historical hurricane tracks (1949 to 2018)

2.4.2 *Kona Coast Wave Climate*

Wave data available from the National Oceanographic and Atmospheric Administration (NOAA) was compiled and analyzed to identify the primary components of the wave climate affecting the project site. These data provide a 38-year wave record and were statistically analyzed to determine the frequency of occurrence of different wave heights, periods, and directions near the coast. The primary wave conditions occurring at the project site include winter North Pacific swell and summer South Pacific swell.

Wave hindcasting is a tool used to calculate past wave events based on weather models and historical data (Hubertz, 1992). With the proper inputs, wave hindcast models can calculate historical wave climates anywhere in the world. Hindcast model outputs are often recorded for a single location, known as a "virtual buoy".

WaveWatch III (WWIII) is a numerical wave model used to forecast and hindcast waves. Hindcast data for a 38-year period (1979-2017) are available around the Hawaiian Islands from NOAA/NCEP. For this study, hindcast data were obtained from virtual buoy Station HNL12, located approximately 37 miles west-northwest from the project site (Figure 2-7).

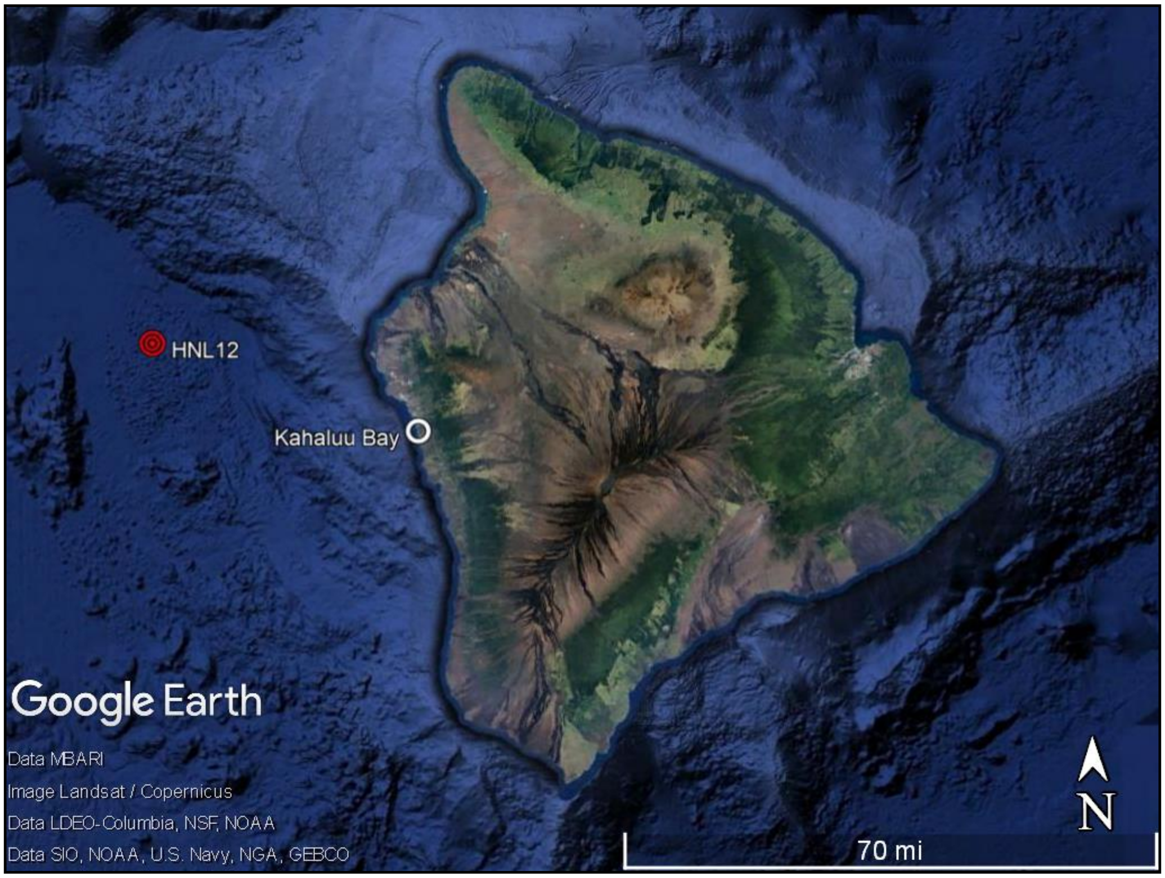

Figure 2-7. Project site and virtual buoy location

It is rare for the sea state to consist of a singular wave condition. Wave events are described by wave height, peak period, and peak direction. The wave parameters from the hindcast model are calculated from a modeled wave spectrum. The spectrum shows the distribution of wave energy relative to wave frequency (wave frequency is the inverse of wave period) and wave direction. This methodology allows multiple wave conditions to be accounted for at the same time for a more accurate description of the sea state. Figure 2-8 is a wave height rose diagram that shows the percent occurrence of wave height and direction for waves as measured at Station HNL12. Figure 2-9 is a wave period rose diagram that shows the percent occurrence of wave period and direction for waves as measured at Station HNL12. Tradewinds waves approaching from the northeast to southeast directions were filtered out of the dataset because these waves do not reach the Kahaluu Bay shoreline. Prevailing waves are the most common or average wave conditions occurring at a specific site. Prevailing waves during the winter months have a significant wave height at HNL12 of about 2 feet with a peak period of 13 seconds and typically approach from between the west-northwest to northwest direction. Prevailing waves during the summer months have a significant wave height at HNL12 of about 1 foot with a peak period of 14 seconds and typically approach from between the south to southwest direction.

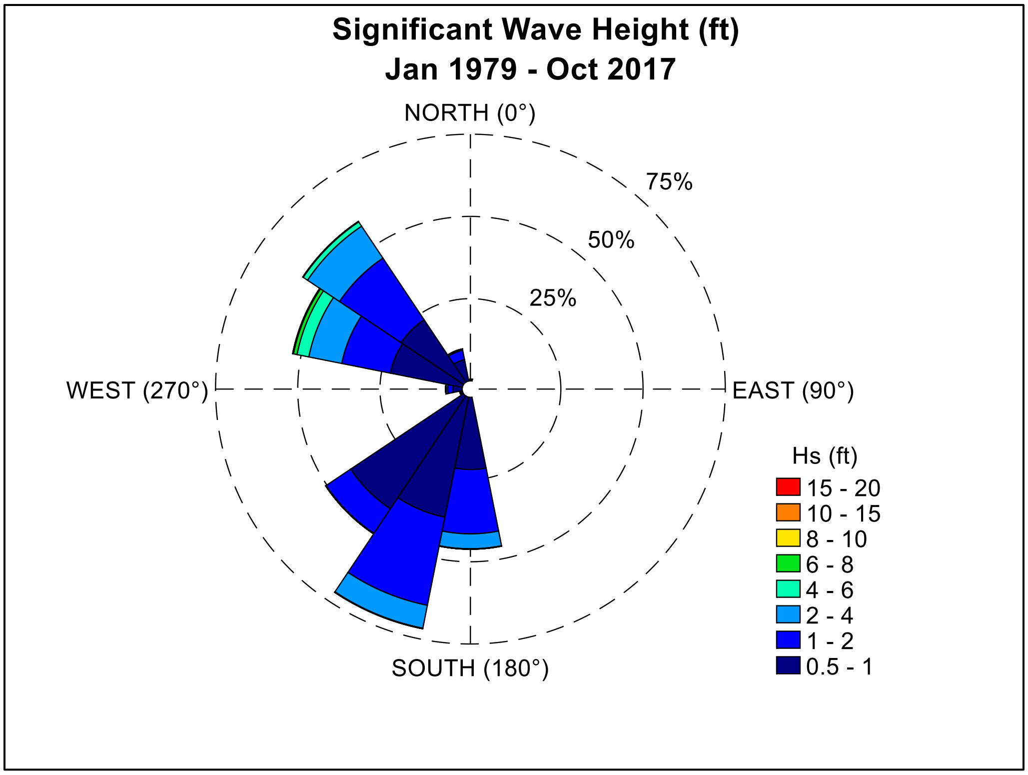

Figure 2-8. Station HNL12 virtual buoy wave height rose from Jan 1979 - October 2017

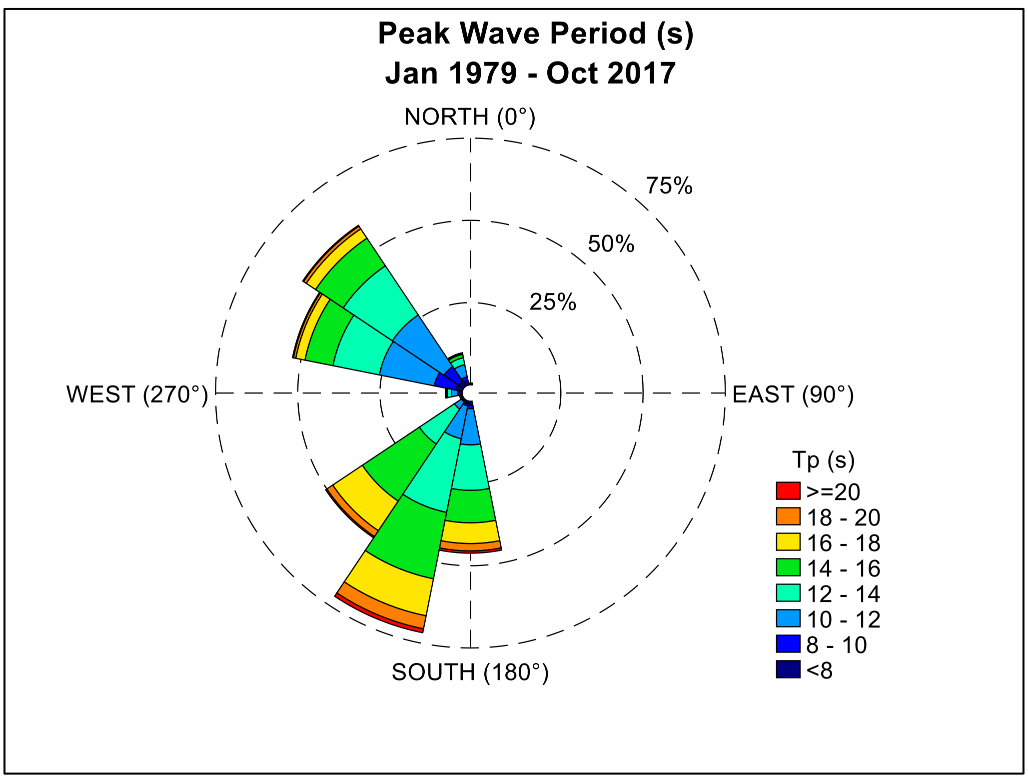

Figure 2-9. Station HNL12 virtual buoy wave period rose from Jan 1979 - October 2017

2.4.3 *Extreme Deepwater Waves*

Historical wave buoy data allows the prediction of extreme wave events. These are infrequent, large, powerful, low probability wave events that are typically used for design purposes. For example, a 50-year return period wave event is an extreme event with a 1/50 (i.e., 2%) chance of occurring in any given year.

Wave conditions throughout the Hawaiian Islands are measured by moored wave buoys. Each buoy measures and records its motion due to passing waves. This data is used to compute spectral wave energy and direction at half-hour increments and derives important wave parameters such as significant wave height, peak direction, and peak period from those measurements. Wave buoys throughout the islands are operated by the Pacific Islands Ocean Observation System (PacIOOS).

For this study, multiple wave buoys were used to determine extreme wave conditions specific to the project site based on the buoy location and wave exposure. The primary wave conditions occurring at the project site include winter North Pacific swell and summer South Pacific swell. For north pacific swell wave conditions, buoy station 202 located 7.2 miles northwest of Hanalei Bay, Kauai, was used. This buoy was chosen because of its complete exposure to North Pacific swell particularly swell approaching from the west-northwest direction which would directly impact the project site. For South Pacific swell wave conditions, buoy stations 146 and 239 (replaced 146) located 2.5 miles southwest of Kaumalapau Harbor, Lanai. These buoys were chosen because of their exposure to South Pacific swell and Kona seas. The location of these buoys relative to the project site are shown in Figure 2-10.

Buoy wave height data was used to generate a Weibull extreme value distribution for return period wave heights. The Weibull Distribution is a tool for relating the size of wave to the frequency of occurrence at a given location. The analysis requires a long-term data set with well-documented wave events. These events are then sorted by size, and frequency of occurrence can be assessed by how often these events occur in the record. The relationship is logarithmic, and a linear fit can be established with a best fit linear regression of the data. Though not all wave events will be co-located on the line, its general trend represents the nature of the size and frequency relationship of wave events at a specific location. An extreme wave return period analysis using the Weibull Distribution was performed for waves associated with North Pacific swell, South Pacific swell, and Kona storm waves.

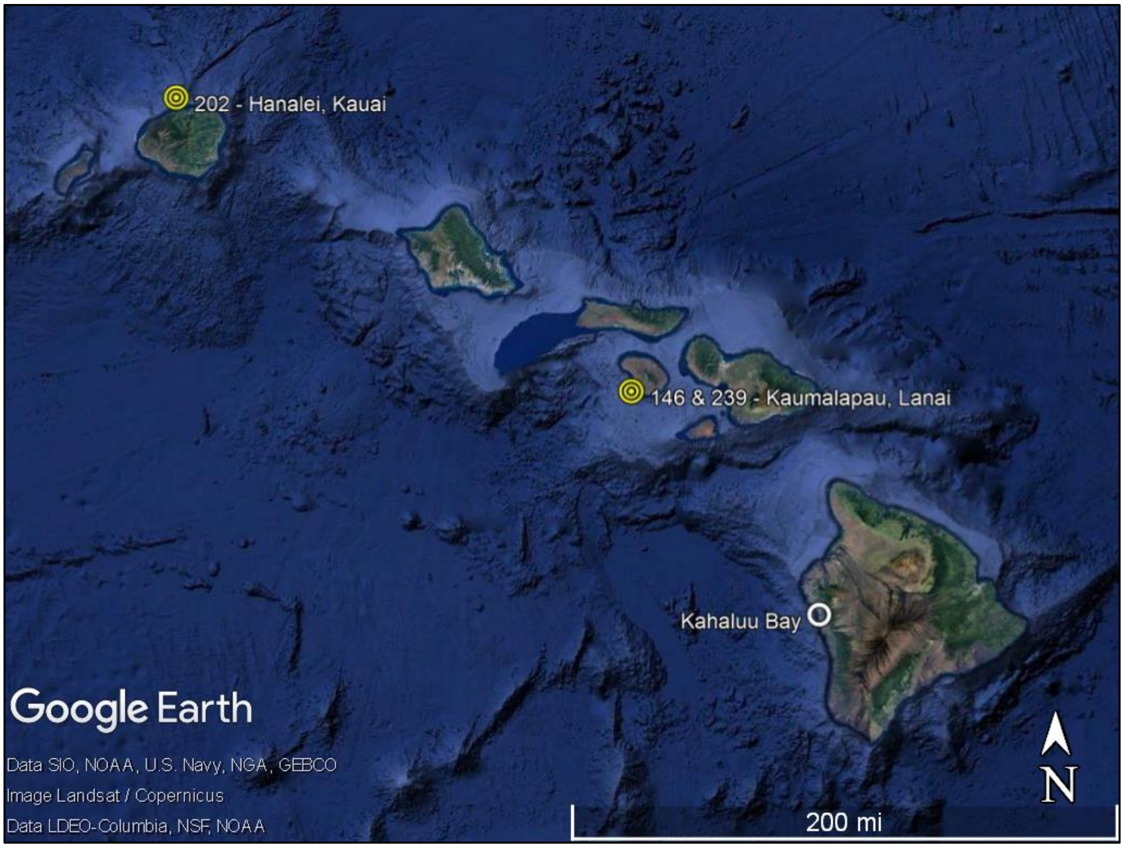

Figure 2-10. Location of wave buoys relative to Kahaluu Bay Beach Park

2.4.3.1 North Pacific Swell

For extreme deepwater waves associated with North Pacific swell, wave buoy data was compiled from buoy station 202 located offshore from Hanalei Bay, Kauai. Extreme wave heights were investigated by filtering the buoy data by direction and period for waves approaching from the west-northwest direction, with periods greater than 10 seconds. The distribution of wave height versus return period is shown on Figure 2-11 and tabulated in Table 2-3. The ten largest wave events approaching from the west-northwest direction during the period of record are summarized in Table 2-4.

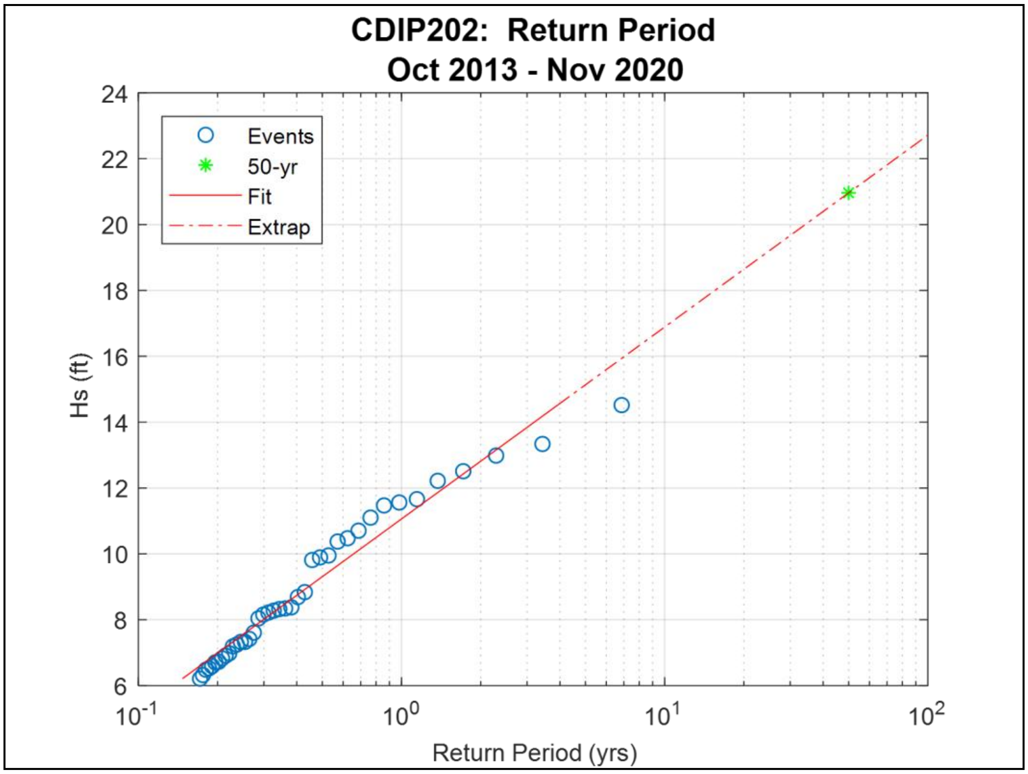

Figure 2-11. Significant wave height vs. return period, Station 202 (Hanalei buoy), October 2013 to November 2020

Table 2-3. Significant wave height vs. return period for WNW swell events recorded at Station 202 (Hanalei buoy)

Return Period	Hs (ft)
1	11.1
2	12.8
5	15.1
10	16.9
25	19.2
50	21.0

Table 2-4. Top 10 WNW swell events recorded at Station 202 (Hanalei buoy)

Date	Hs (ft)	Tp (sec)	Dp (deg. TN)
2016-01-26	14.5	14	297
2017-01-14	13.3	17	297
2014-01-18	13.0	14	294
2016-03-02	12.5	15	293
2017-01-19	12.2	15	297
2020-01-26	11.7	17	299
2019-01-15	11.6	15	299
2018-02-04	11.5	15	290
2017-04-01	11.1	15	297
2013-12-20	10.7	18	294

2.4.3.2 South Swell

For extreme deepwater waves associated with South Pacific swell, wave buoy data was compiled from buoy stations 146 and 239 located 2.5 miles southwest of Kaumalapau Harbor, Lanai. Extreme wave heights were investigated by filtering the buoy data by direction and period for waves approaching from the south to southwest directions, with periods of 12 seconds or greater. Wave height versus return period is shown on Figure 2-12 and Table 2-5. The ten largest wave events associated with south swell during the period of record are shown in Table 2-6.

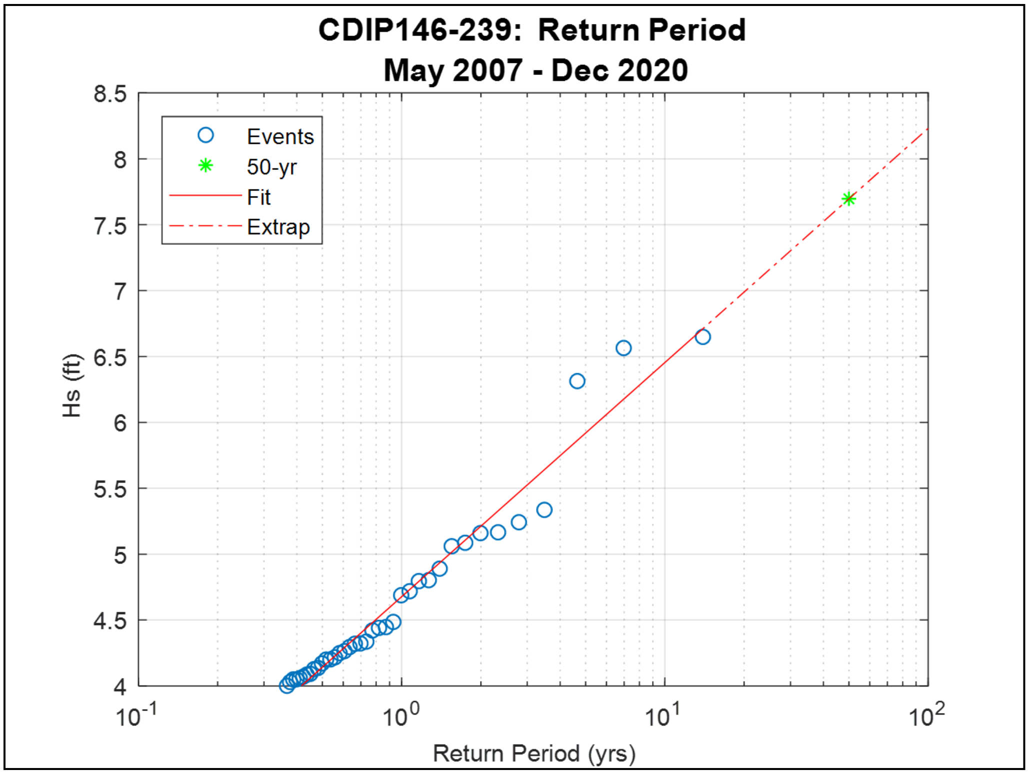

Figure 2-12. Significant wave height vs. return period, station 146/239 (Lanai buoy), filtered for south swell, May 2007 to December 2020

Table 2-5. Significant wave height vs. return period, station 146/239 (Lanai buoy), filtered for south swell, May 2007 to December 2020

Return Period	Hs (ft)
1	4.7
2	5.2
5	5.9
10	6.5
25	7.2
50	7.7

Table 2-6. Top 10 south swell events recorded at CDIP 165/238 (Barbers Point buoy)

Date	Hs (ft)	Tp (sec)	Dp (deg. TN)
2019-07-14	6.6	18	186
2019-05-13	6.6	17	186
2013-05-17	6.3	17	185
2019-08-20	5.3	18	195
2018-10-27	5.2	15	175
2008-09-07	5.2	15	196
2014-05-17	5.2	17	195
2018-08-08	5.1	15	174
2012-07-05	5.1	15	200
2016-05-08	4.9	17	199

2.4.3.3 Kona Storm Waves

For extreme deepwater waves associated with Kona storm waves (seas), wave buoy data was compiled from buoy stations 146 and 239 located 2.5 miles southwest of Kaumalapau Harbor, Lanai. Extreme wave heights were investigated by filtering the buoy data by direction and period for waves approaching from the west to south directions, with periods of 10 seconds or less. Wave height versus return period is shown on Figure 2-13 and Table 2-7. The ten largest wave events associated with Kona seas during the period of record are shown in Table 2-8.

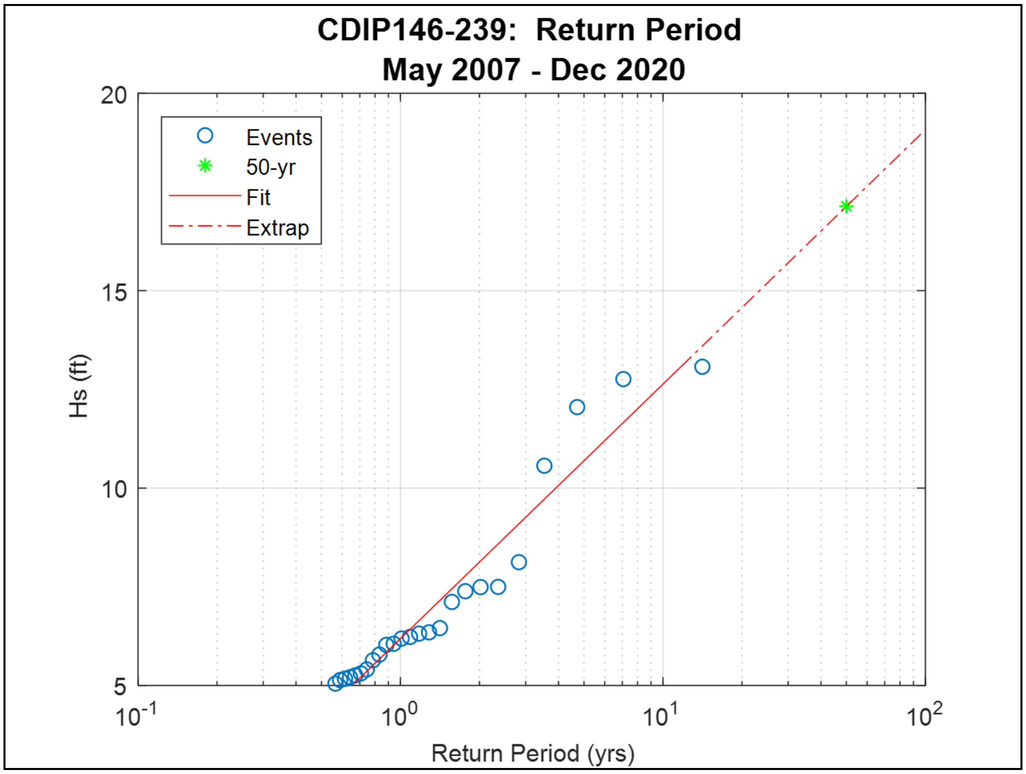

Figure 2-13. Significant wave height vs. return period, stations 146/239 (Lanai buoys), filtered for Kona storm waves, May 2007 to December 2020

Table 2-7. Significant wave height vs. return period, stations 146/239 (Lanai buoys), filtered for Kona storm waves, May 2007 to December 2020

Return Period	Hs (ft)
1	6.2
2	8.1
5	10.7
10	12.6
25	15.2
50	17.1

Table 2-8. Top 10 Kona storm waves events recorded at stations 146/239 (Lanai buoys)

Date	Hs (ft)	Tp (sec)	Dp (deg. TN)
2007-12-05	13.1	8	224
2019-02-11	12.8	8	273
2015-01-03	12.1	8	257
2015-02-14	10.6	8	247
2014-01-23	8.1	8	282
2020-03-17	7.5	5	283
2009-01-17	7.5	8	273
2017-02-12	7.4	7	238
2017-02-07	7.1	8	287
2011-01-13	6.5	7	203

2.5 Wave Transformation to Shore

As deepwater waves propagate toward shore, they begin to encounter and be transformed by the ocean bottom. In shallow water, the wave speed becomes related to the water depth. As waves slow down with decreasing depth, the process of wave shoaling steepens the wave and increases the wave height. Wave breaking occurs when the wave profile shape becomes too steep to be maintained. This typically occurs when the ratio of wave height to water depth is about 0.78 and is a mechanism for dissipating the wave energy. Wave energy is also dissipated due to bottom friction. The phenomenon of wave refraction is caused by differential wave speed along a wave crest as the wave passes over varying bottom contours and can cause wave crests to converge or diverge and may locally increase or decrease wave heights. Not strictly a shallow water phenomenon, wave diffraction is the lateral transmission of wave energy along the wave crest and would cause the spreading of waves in a shadow zone, such as occurs behind a breakwater or other barrier. Numerical models are useful tools to assess the potential impacts of future sea level rise by simulating specific wave events under various water levels. Two numerical wave models, SWAN and XBeach-NH were utilized for this study to simulate the wave transformation from deep water to the project site for current and future sea level rise scenarios for various wave conditions.

SWAN Model

Simulating Waves Nearshore (SWAN) is a third-generation wave model developed by Delft University of Technology that computes random, short-crested wind-generated waves in coastal regions and inland waters (*Booij, et al, 1999*). The SWAN model can be applied as a steady state or non-steady state model and is fully spectral (over the total range of wave frequencies). Wave propagation is based on linear wave theory, including the effect of wave generated currents. SWAN provides many output quantities, including two-dimensional spectra, significant wave height and mean wave period, and average wave direction and directional spreading. For this project, the SWAN model was used to transform waves from deep water to intermediate water depths just offshore from the project site. SWAN model results were used to provide wave parameter input for a nearshore numerical wave and flow model, XBeach-NH.

XBeach-NH Model

As waves move into shallow water, bathymetry has a greater influence on wave behavior. Waves interact with the bottom, dissipating more energy through depth-induced breaking and bottom friction. Results of the SWAN model for the prevailing wave, annual wave, and 50-year wave conditions were modeled from just offshore of project site into the nearshore region using the XBeach non-hydrostatic (XBeach-NH) numerical model. XBeach is an open-source numerical wave model originally developed to simulate hydrodynamic and morphological processes along sandy shorelines. The XBeach-NH module (*Stelling and Zijlema, 2003*) computes the depth-averaged flow due to waves and currents using the non-linear shallow water equations and includes a non-hydrostatic pressure term. The governing equations are valid from intermediate to shallow water and can simulate most of the phenomena of interest in the nearshore zone and in harbor basins, including shoaling and refraction over variable bathymetry, reflection and diffraction near structures, energy dissipation due to wave breaking and bottom friction, breaking-induced longshore/cross-shore ("rip") currents, and harbor oscillations. XBeach-NH is a phase resolving model, meaning that wave crests and troughs are modeled and propagated in time and space. The result is an accurate representation of wave heights and wave patterns across the domain.

2.5.1 *Offshore Wave Transformation*

For this study, the SWAN wave model was used to simulate the deepwater waves from North Pacific swell, South Pacific swell, and Kona storms. For each of the three wave regimes, the 50-year and 1-year (annual high wave) recuring waves were simulated (see section 2.4.3) which represent storm conditions and a once-per-year seasonal wave, respectively. An unstructured mesh grid was developed for the SWAN model and covers the main Hawaiian Islands to account for island shadowing for waves approaching from the west-northwest direction. Unstructured grids allow for coarse resolution in offshore regions where waves are not influenced by the seafloor and fine resolution nearshore where wave transformation is more affected by the shallower water depths. The resolution of the SWAN domain varies from 3.1 miles (5,000 m) offshore in deepwater to 984 ft (300 m) near islands to 164 ft (50 m) nearshore surrounding Kahaluu Beach Park (see Figure 2-14).

Figure 2-14. SWAN unstructured mesh for the Hawaiian Islands and Kahaluu Bay (red outline shows finer mesh around Kahaluu Beach Park)

Table 2-9 lists the deepwater wave parameters (see Section 2.4.3) used as input for the SWAN model for each scenario wave condition. These parameters are applied along all boundaries of the unstructured SWAN grid. For the 50-year return period wave, the Kona seas case generated the largest wave heights just offshore from Kahaluu Bay. For the 1-year annually recuring wave, the WNW swell case generated the largest wave heights just offshore from Kahaluu Bay. The SWAN output for the 50-year Kona seas and annual WNW swell are shown in Figure 2-15 and Figure 2-16, respectively.

Table 2-9. Modeled wave cases (inputs to SWAN model)

Wave Case	Deepwater Wave Parameters		
	H_s (feet)	T_p (seconds)	Dir. (deg. TN)
50yr WNW Swell	21.0	16	295
50yr SSW Swell	7.7	16	195
50yr Kona Seas	17.0	9	260
Annual WNW Swell	11.1	16	295
Annual SSW Swell	4.7	16	195
Annual Kona Seas	6.2	8	260

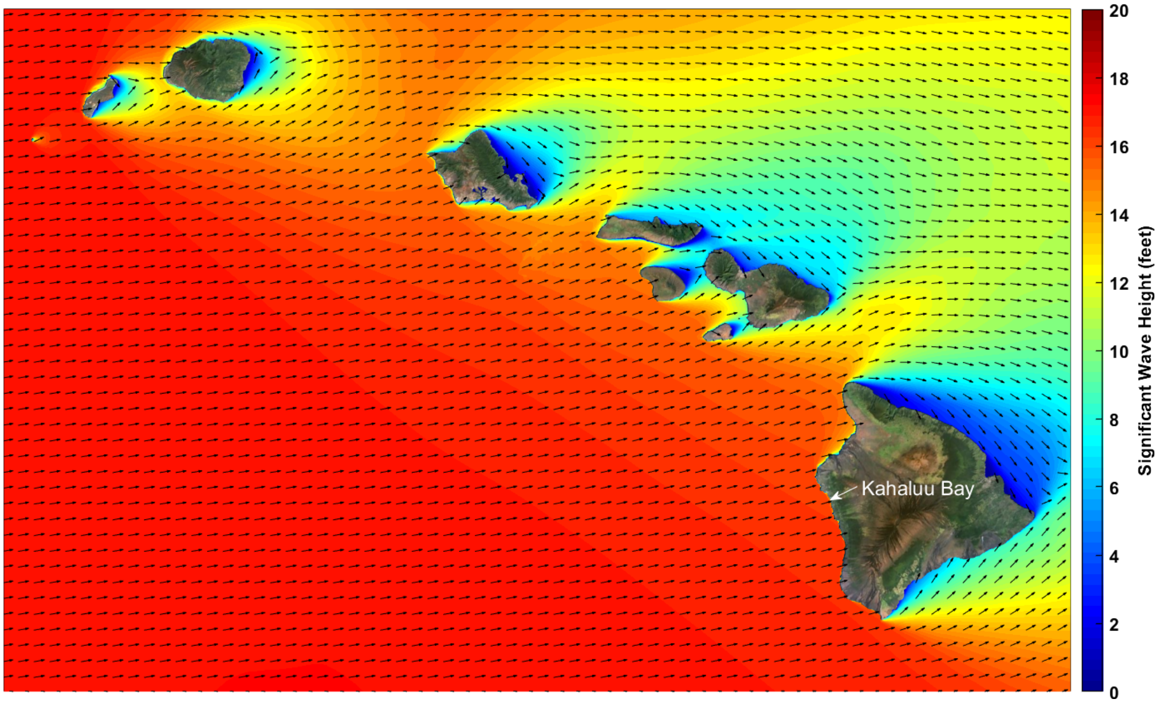

Figure 2-15. Significant wave height from SWAN model for the 50yr Kona storm event

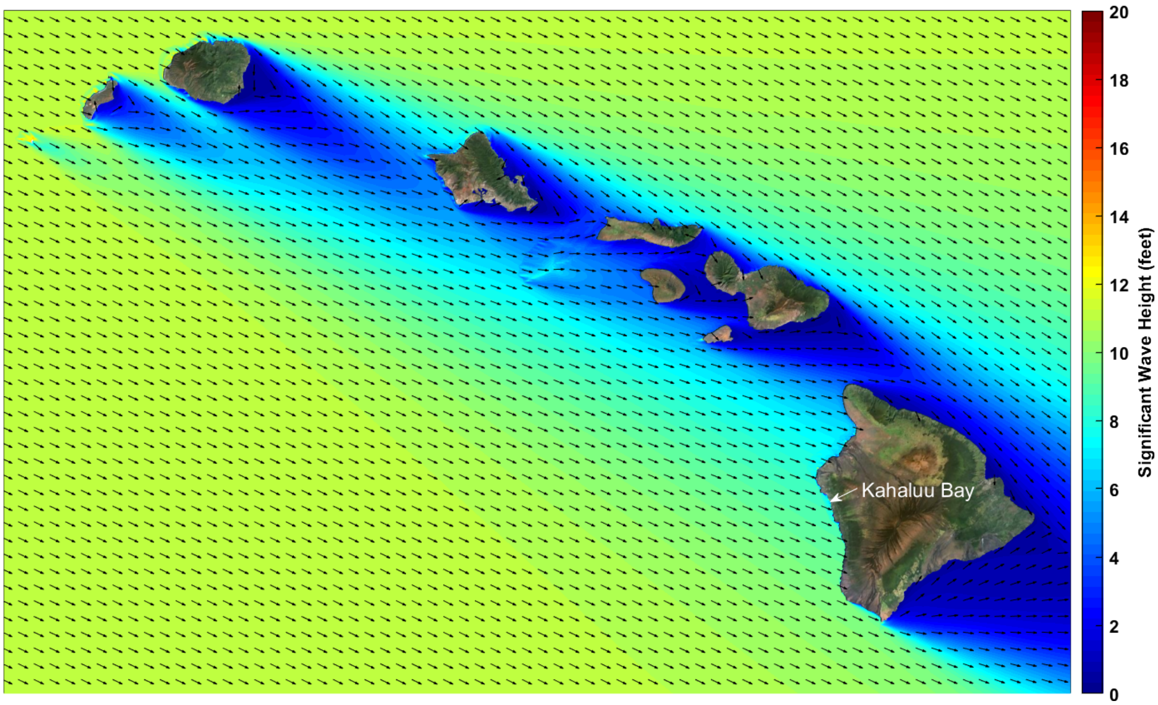

Figure 2-16. Significant wave height from SWAN model for the annual WNW swell

2.5.2 *Nearshore Wave Transformation*

For this study, the numerical model XBeach-NH is used to assess nearshore wave conditions at Kahaluu Beach Park. A telescoping grid was developed with variable grid resolution. The grid resolution within and surrounding Kahaluu Beach Park is about 5 feet while the grid resolution offshore is about 10 feet. Results from the SWAN model were used as input to the XBeach-NH model at its offshore boundary. The orientation of the model grid was adjusted to match the incoming wave direction for each wave case. Figure 2-17 shows the model domain extents oriented for Kona seas and WNW swell. Model bathymetry was adapted from the 2013 USACE SHOALS Lidar dataset (see Figure 2-18).

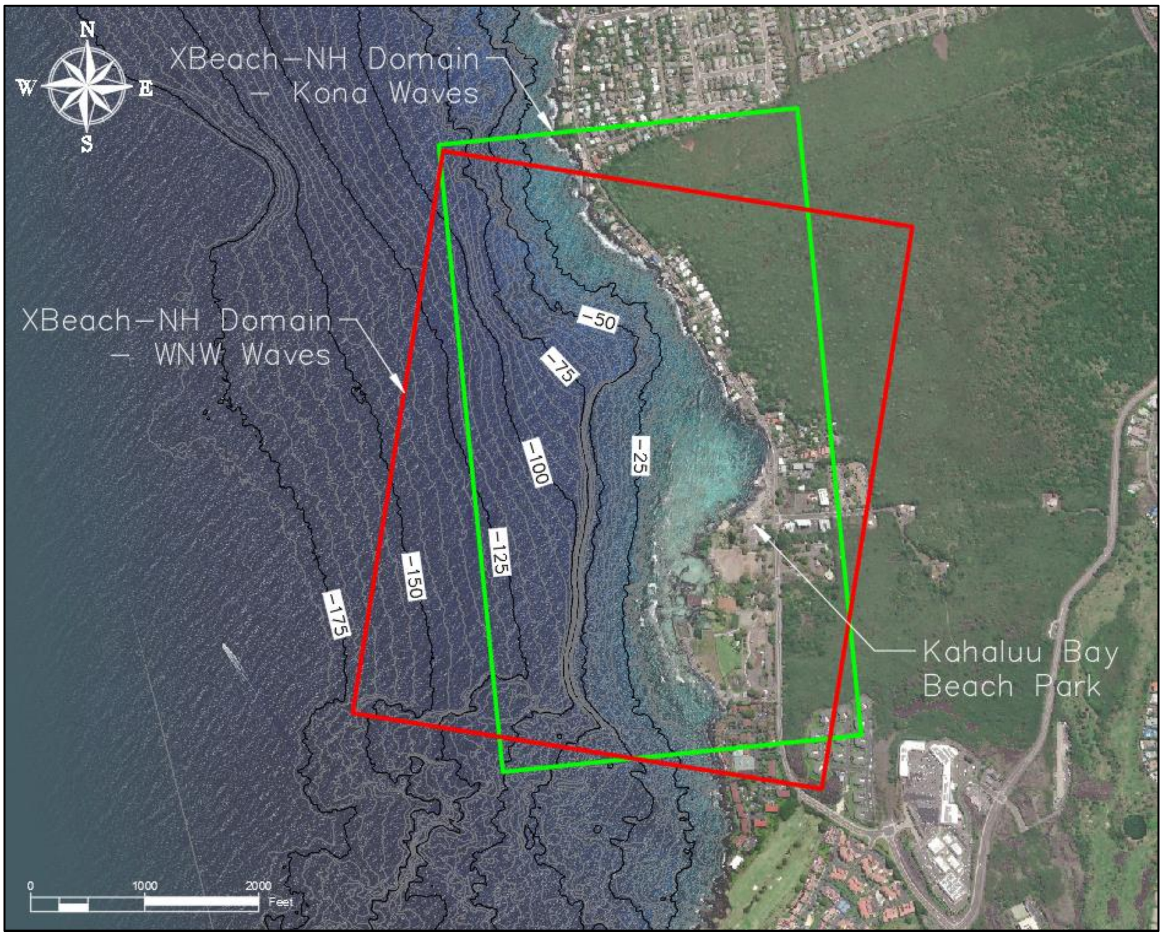

Figure 2-17. XBeach-NH model domain extents (green - oriented for Kona waves, red - oriented for WNW waves, elevations in feet relative to MSL)

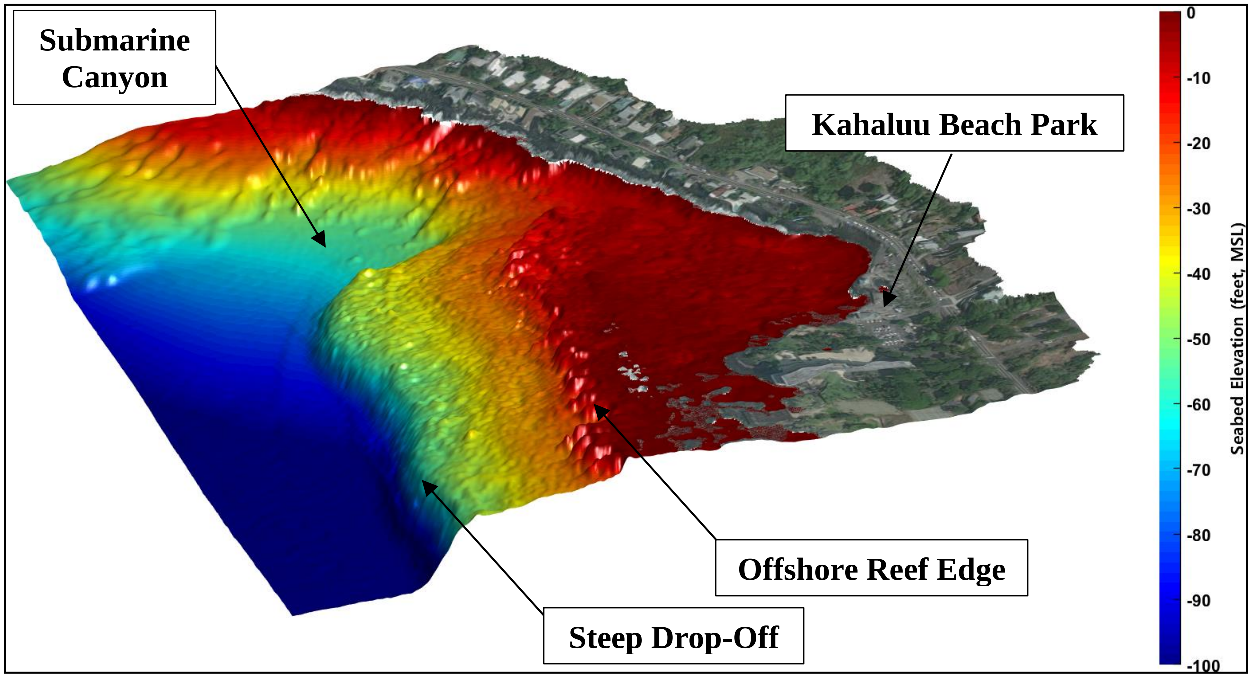

Figure 2-18. XBeach-NH model bathymetry

2.5.2.1 50-year Kona Storm

For the 50-year Kona storm simulation, water level input to the model includes a combination of high tide (+1.2 feet), a +0.5-foot SLA, and the intermediate 2070 SLR scenario (+2.54 feet) for a total design water level of +4.24 feet MSL. Model output of the wave heights nearshore will be used for structural evaluation of shoreline structures in a follow-up study. Figure 2-19 and Figure 2-20 show the output nearshore significant wave height and water surface elevation, respectively, for the 50-year Kona Storm event. Breaking wave heights offshore reach up to about 16 feet while wave heights near the shoreline are approximately 3-4 feet.

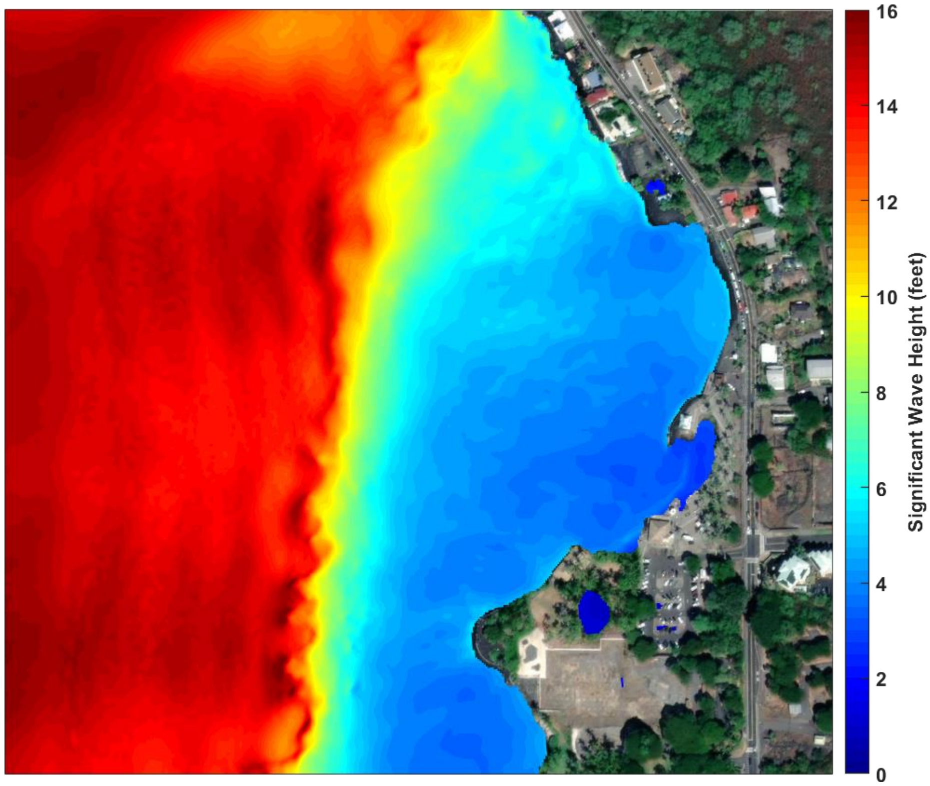

Figure 2-19. XBeach-NH modeled significant wave height for the 50-year Kona Storm event

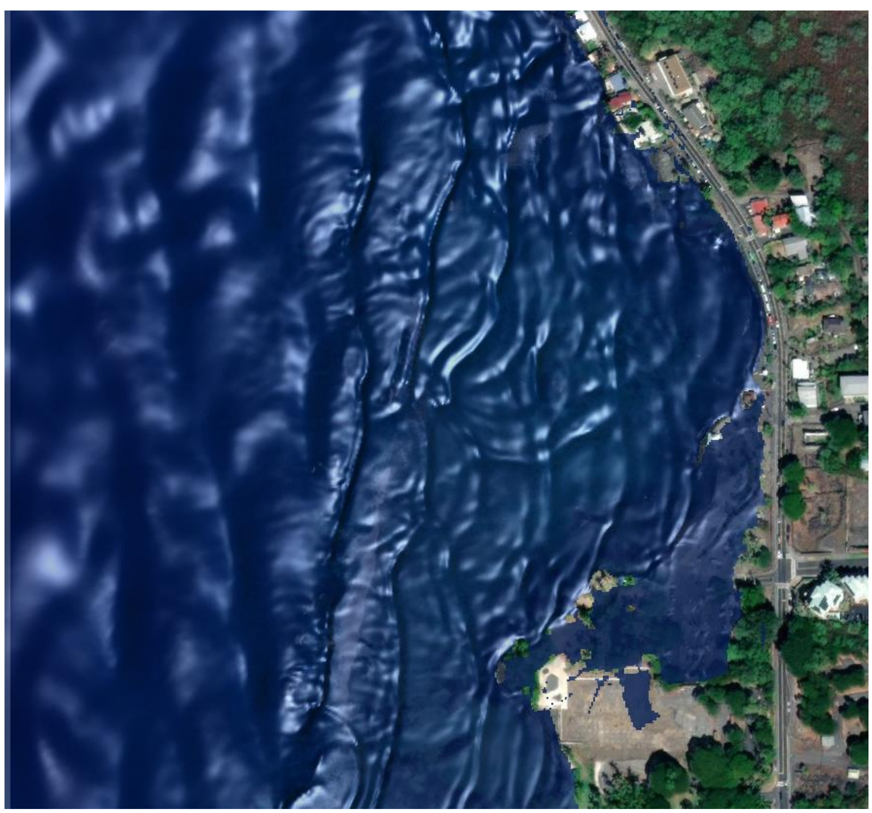

Figure 2-20. XBeach-NH modeled water surface elevation for the 50-year Kona Storm event

2.5.2.2 Annual WNW Swell

For the annual WNW swell case the initial model was simulated for current sea level including high tide and a 0.25-foot SLA. Model output for the annual wave heights nearshore will be used for evaluation of wave inundation under various SLR scenarios (see section 2.6). Figure 2-21 and Figure 2-22 show the output nearshore significant wave height and water surface elevation, respectively, for the annual WNW swell event. Breaking wave heights offshore are about 10-12 feet while nearshore wave heights are about 2-3 feet.

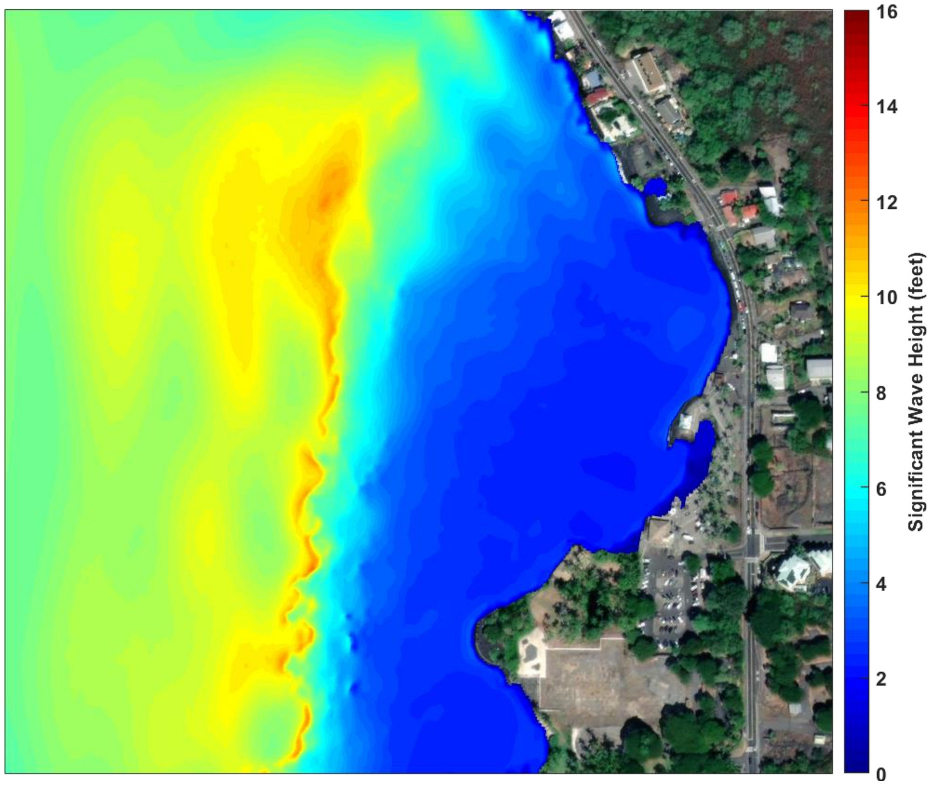

Figure 2-21. XBeach-NH modeled significant wave height for the annual WNW swell event

Figure 2-22. XBeach-NH modeled water surface elevation for the annual WNW swell event

2.5.2.3 Prevailing WNW Swell

For the prevailing WNW swell case the initial model was simulated for current sea level including high tide and a 0.25-foot SLA. Model output for wave heights and circulation nearshore will be used to assess existing sediment transport and coastal processes. Figure 2-23 and Figure 2-24 show the output nearshore significant wave height and water surface elevation, respectively, for a prevailing WNW swell. Breaking wave heights offshore are about 2-3 feet while nearshore wave heights are about 0.5-1 feet.

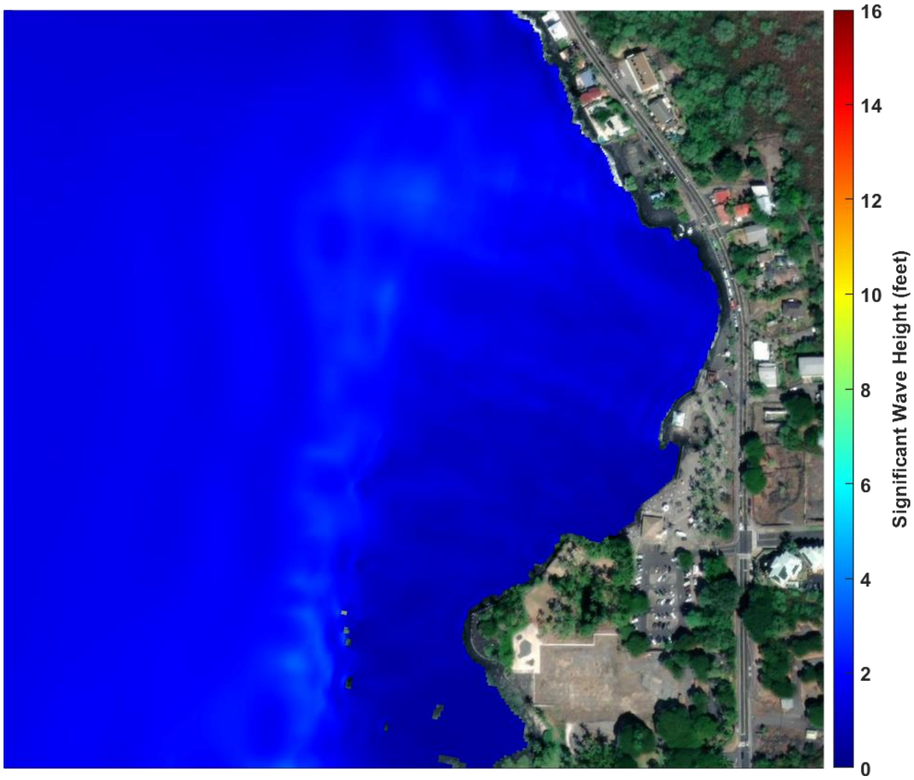

Figure 2-23. XBeach-NH modeled significant wave height for a prevailing WNW swell

Figure 2-24. XBeach-NH modeled water surface elevation for a prevailing WNW swell

2.6 Wave Inundation

A key part of this study is the assessment of wave impacts at the project site and how potential future SLR may increase wave runup and inundation into the park. For this study, the XBeach-NH model is used to assess flooding potential at Kahaluu Beach Park under future projections of SLR.

2.6.1 *Model Validation*

Numerical models require some level of validation/calibration before they can be used as part of the design and planning process. Wave inundation models are often difficult to validate because field data for runup and inundation is typically limited and difficult to collect. For this study, video footage provided by The Kohala Center taken during a strong swell event on December 21, 2013 is used to qualitatively compare model results with field observations. Figure 2-25 shows a still from the video which shows wave runup and inundation in the backshore area at Kahaluu Beach Park adjacent to the pond. The XBeach-NH model was used to simulate the waves during this event around the same time of day the video was taken. Wave parameters for this particular event were compiled from buoy station 202 and the deepwater waves were propagated to the XBeach-NH boundary using SWAN. Figure 2-26 shows a snapshot of modeled water level at a single time within the simulation period. The model appears to reasonably reproduce the observed wave runup and inundation extent for this particular wave event. Based on this model verification, it is considered reasonable to assume the model will accurately represent the wave inundation hazard for different wave conditions and future SLR.

Figure 2-25. Video footage from December 21, 2013 showing inundation in the backshore area (looking south)

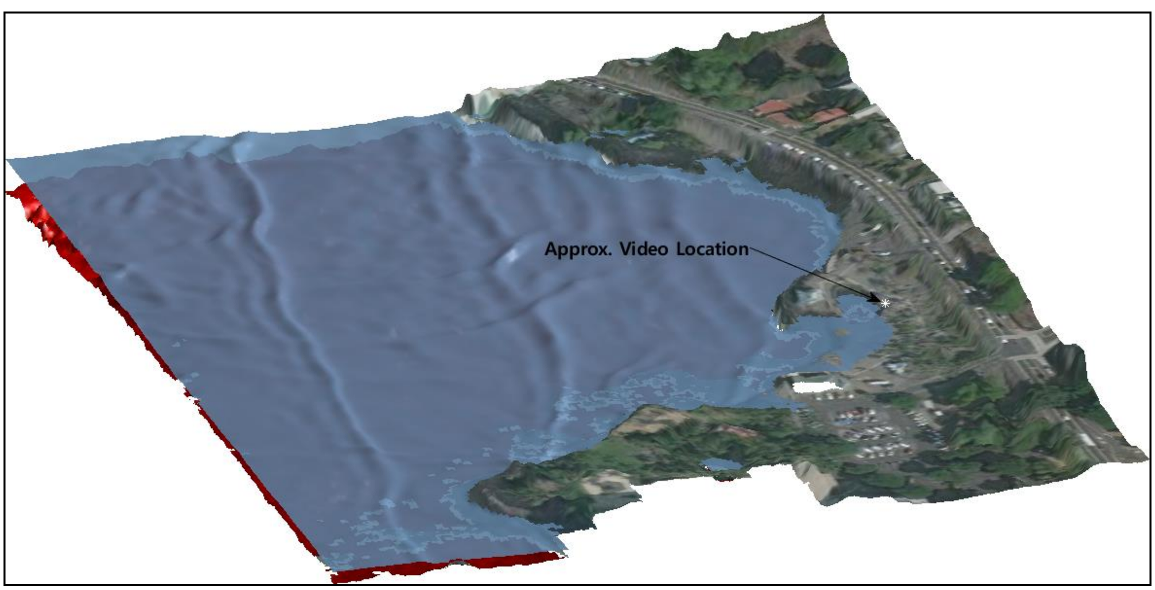

Figure 2-26. Snapshot of water level output from XBeach-NH model (December 21, 2013 wave simulation)

2.6.2 *Model Application for Prevailing (Annual) Event*

The validated XBeach-NH wave model was applied to simulate wave inundation at Kahaluu Beach Park for various sea level rise (SLR) projections under an annually recurring (1-year) WNW swell wave event. Output from the SWAN model just offshore from the project site was used as boundary conditions to the XBeach-NH model. The XBeach-NH model was simulated for current sea level and 2 additional SLR levels using the NOAA Intermediate projections for the years 2040 and 2060. SLR levels for these years were vertically adjusted to account for the measured sea level rise at the Hilo Bay tide station which shows sea level rise is about 0.25 feet above the existing MSL datum. Table 2 10 summarizes the input water levels into each of the 3 model cases. Model results of flooding extent are summarized in Table 2-11 and shown in Figure 2- 24 through Figure 2- 26 for each SLR scenario for the annual WNW swell event. Detailed flooding parameters are shown in Figure 2-30 through Figure 2-33 and include maximum flood depth, flood velocity, flood level elevation, and spatial frequency of flooding.

Table 2-10. Input water levels for the 3 simulation scenarios

Input Water Levels (feet, MSL)			
Year:	2020	2040	2060
MHHW	1.20	1.20	1.20
SLA	0.25	0.25	0.25
SLR	0.54	1.13	2.02
Total:	1.99	2.58	3.47

Table 2-11. Modeled annual wave inundation summary throughout park

Year	SLR	Modeled Inundation Summary
2020	+0.54 ft	Inundation extends up to 90 feet inland from current shoreline, inundation of the north end of the parking lot area.
2040	+1.13 ft	Inundation extends up to 110 feet inland from current shoreline, partial inundation of the pavilion, inundation of the northern half of the parking lot area.
2060	+2.02 ft	Inundation of virtually the entire park.

Figure 2-27. Modeled extent of wave inundation extent at current sea level under annual WNW swell wave conditions

Figure 2-28. Modeled extent of wave inundation with +1.13 ft of sea level rise (equivalent to the year 2040 for Intermediate projection) under annual WNW swell wave conditions

Figure 2-29. Modeled extent of wave inundation with +2.02 ft of sea level rise (equivalent to the year 2060 for Intermediate projection) under annual WNW swell wave conditions

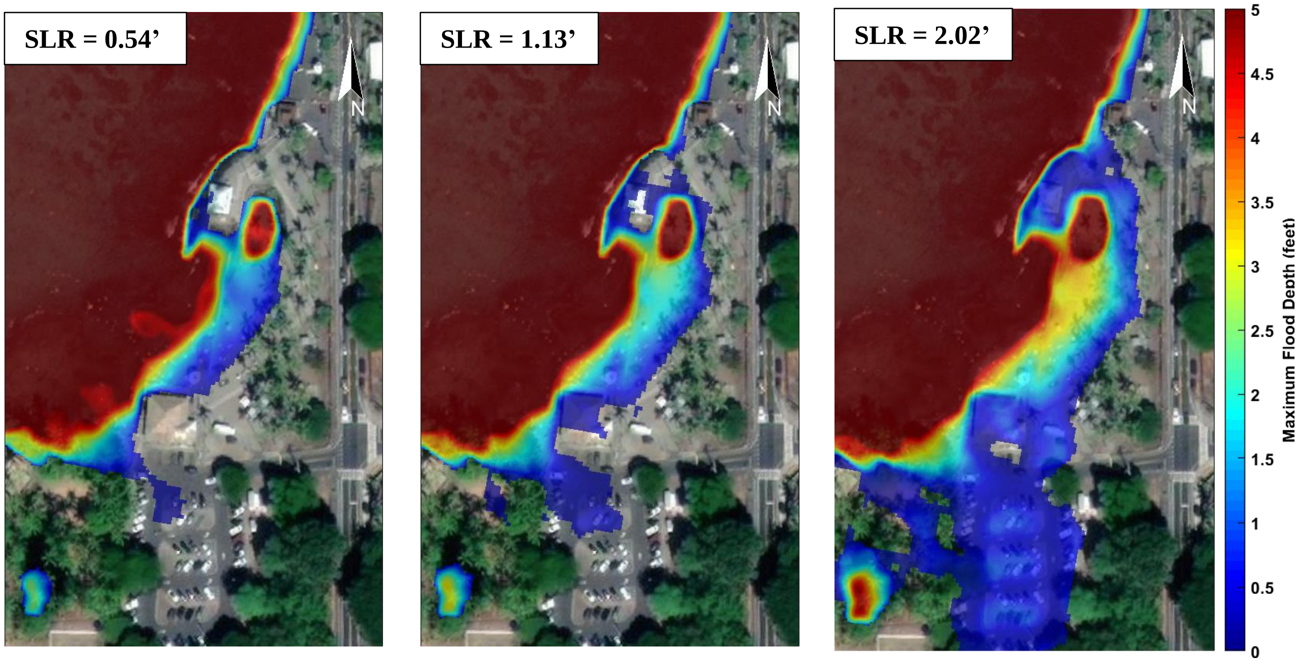

Figure 2-30. Modeled maximum inundation flood depth for the annual WNW swell

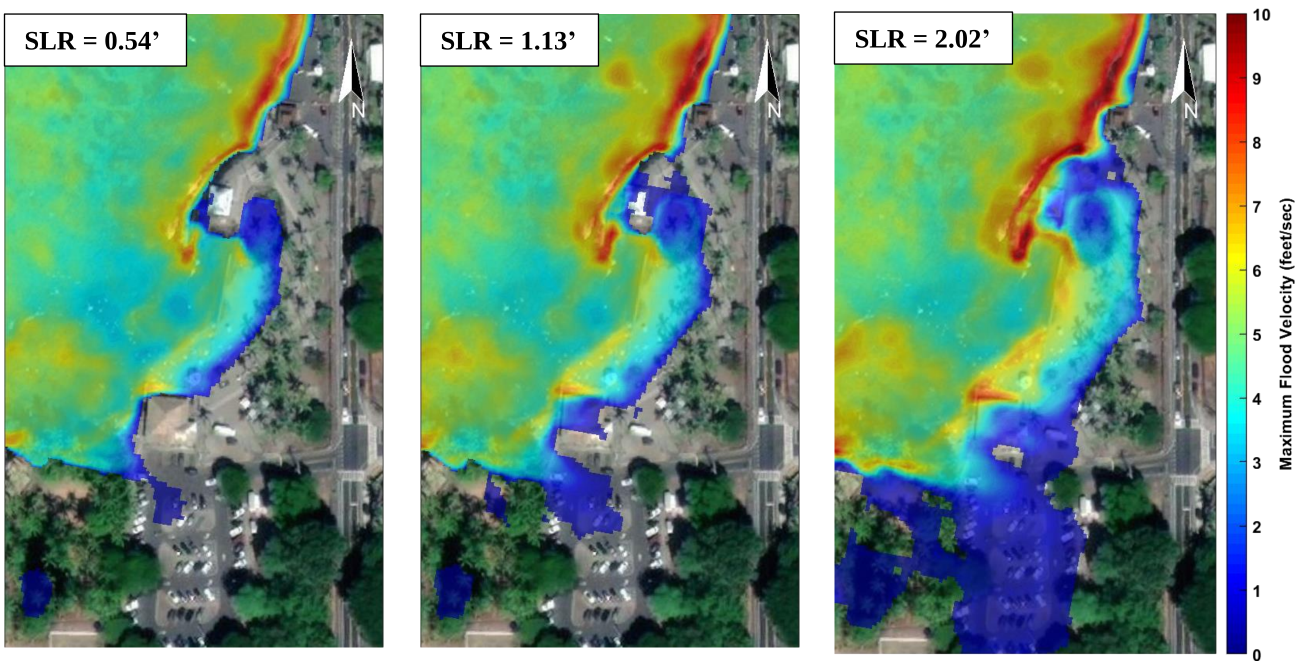

Figure 2-31. Modeled maximum inundation flood velocity for the annual WNW swell

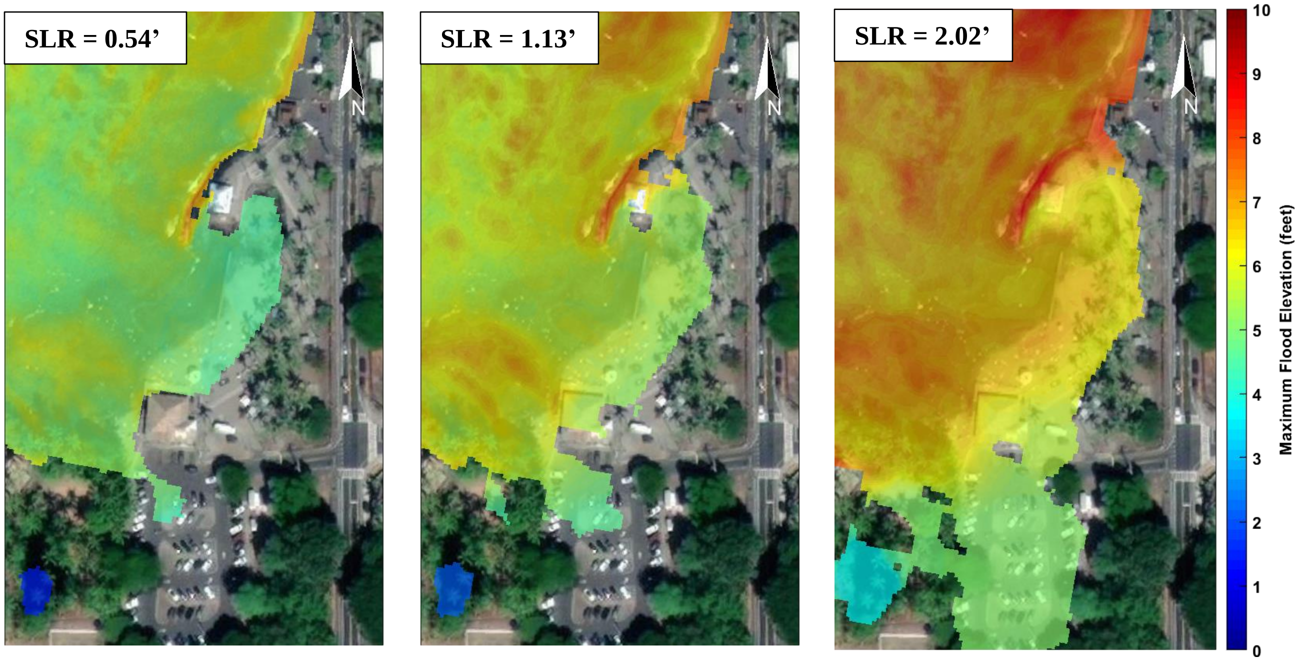

Figure 2-32. Modeled maximum inundation flood elevation (feet, MSL) for the annual WNW swell

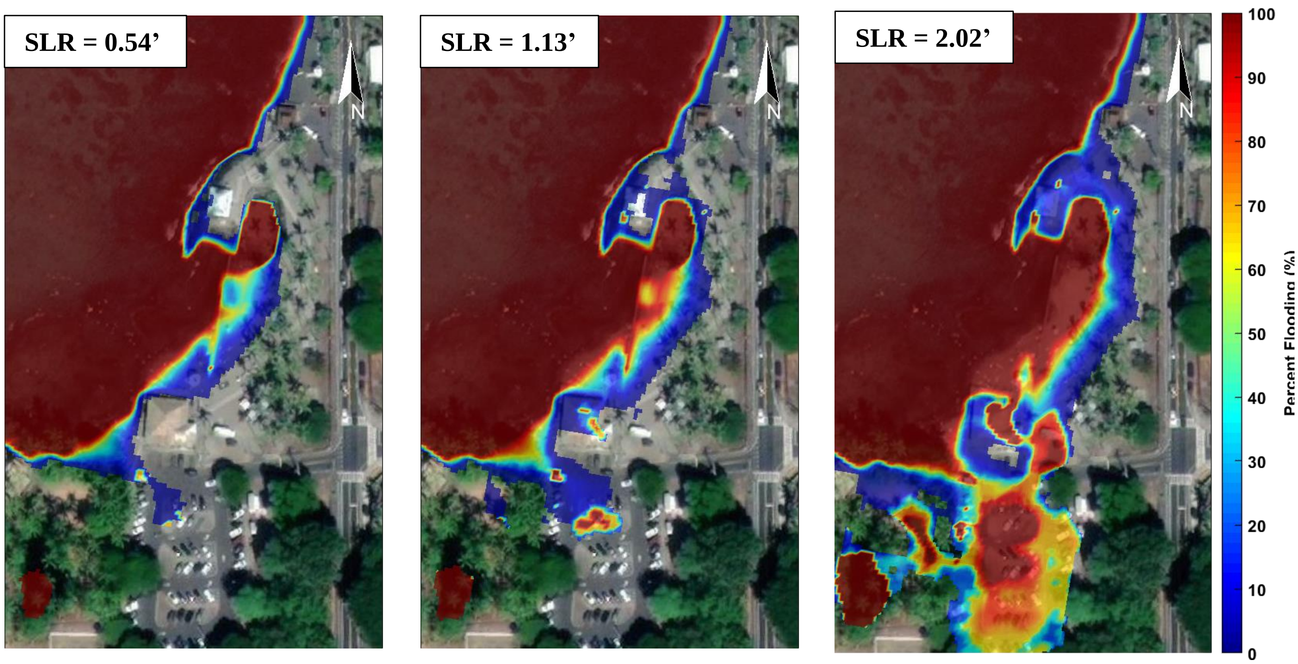

Figure 2-33. Modeled frequency of wave inundation during simulation period for the annual WNW swell

In summary, with SLR of about 2 feet, currently estimated to occur about the year 2060, virtually the entire park will be flooded up to the +6 foot elevation by an annually occurring wave event, resulting in water depths of 1 to 2 feet throughout the park. The water velocity from wave uprush will be about 2 to 5 feet/second, resulting in a force on structure foundations and walls. Recommendation: Elevate or relocate park features above the +8 foot elevation.

2.6.3 *Model Application for Extreme (50-year) Event*

The XBeach-NH wave model was also applied to simulate wave inundation at Kahaluu Beach Park for various sea level rise (SLR) projections under the extreme (50-year) Kona wave event. Output from the SWAN model just offshore from the project site was used as boundary conditions to the XBeach-NH model. The XBeach-NH model was simulated for the same water level projection listed above in Table 2-10. Model results of flooding extent are summarized in Table 2-12 and shown in Figure 2-34 through Figure 2-36 for each SLR scenario for the 50-year Kona wave event. Detailed flooding parameters are shown in Figure 2-37 through Figure 2-40 and include maximum flood depth, flood velocity, flood level elevation, and spatial frequency of flooding.

Table 2-12. Modeled extreme wave inundation summary throughout park

Year	SLR	Modeled Inundation Summary
2020	+0.54 ft	Inundation of virtually the entire park with maximum flood depths up to 3.0 feet.
2040	+1.13 ft	Inundation of virtually the entire park with maximum flood depths up to 3.5 feet.
2060	+2.02 ft	Inundation of virtually the entire park with maximum flood depths up to 4.0 feet.

Figure 2-34. Modeled extent of wave inundation extent at current sea level for the extreme 50-year Kona wave event

Figure 2-35. Modeled extent of wave inundation with +1.13 ft of sea level rise (equivalent to the year 2040 for Intermediate projection) for the extreme 50-year Kona wave event

Figure 2-36. Modeled extent of wave inundation with +2.02 ft of sea level rise (equivalent to the year 2060 for Intermediate projection) for the extreme 50-year Kona wave event

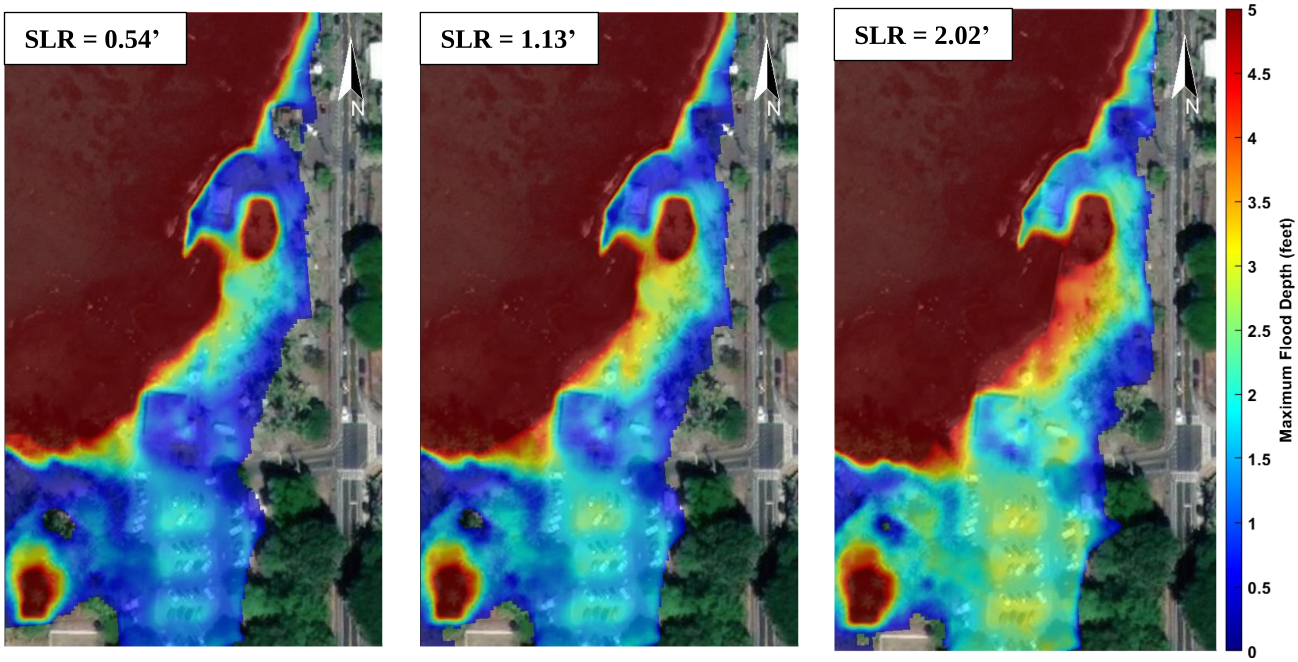

Figure 2-37. Modeled maximum inundation flood depth for the extreme 50-year Kona wave event

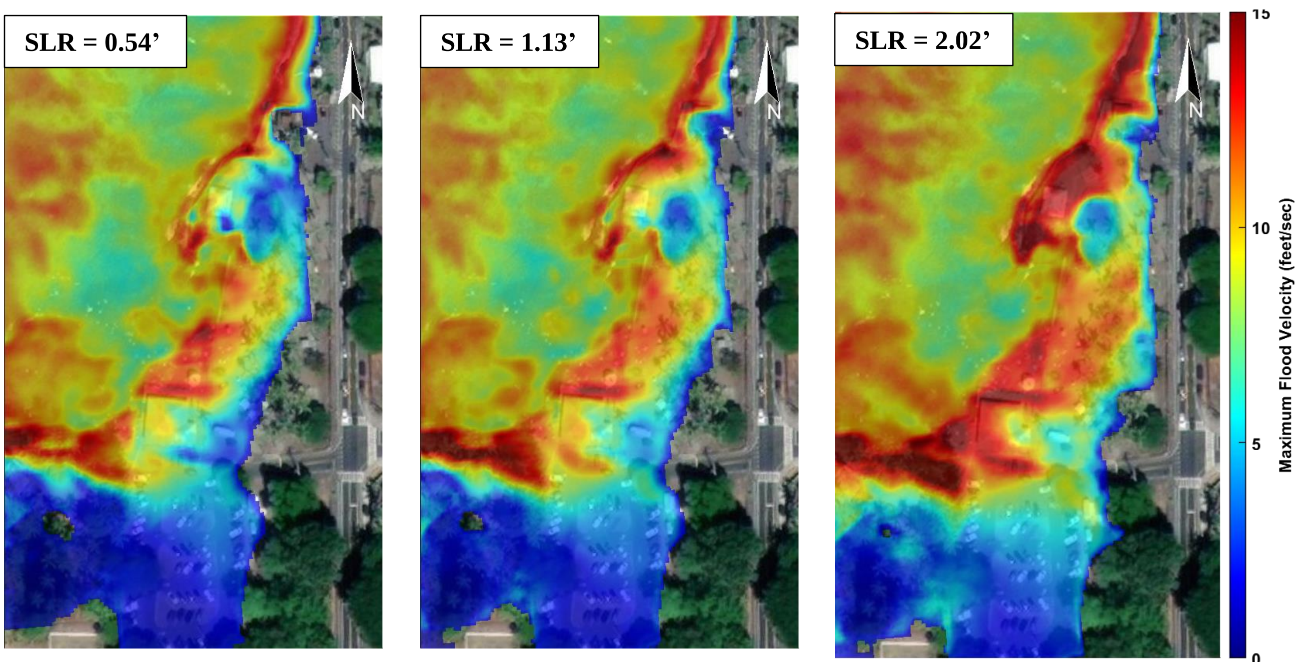

Figure 2-38. Modeled maximum inundation flood velocity for the extreme 50-year Kona wave event

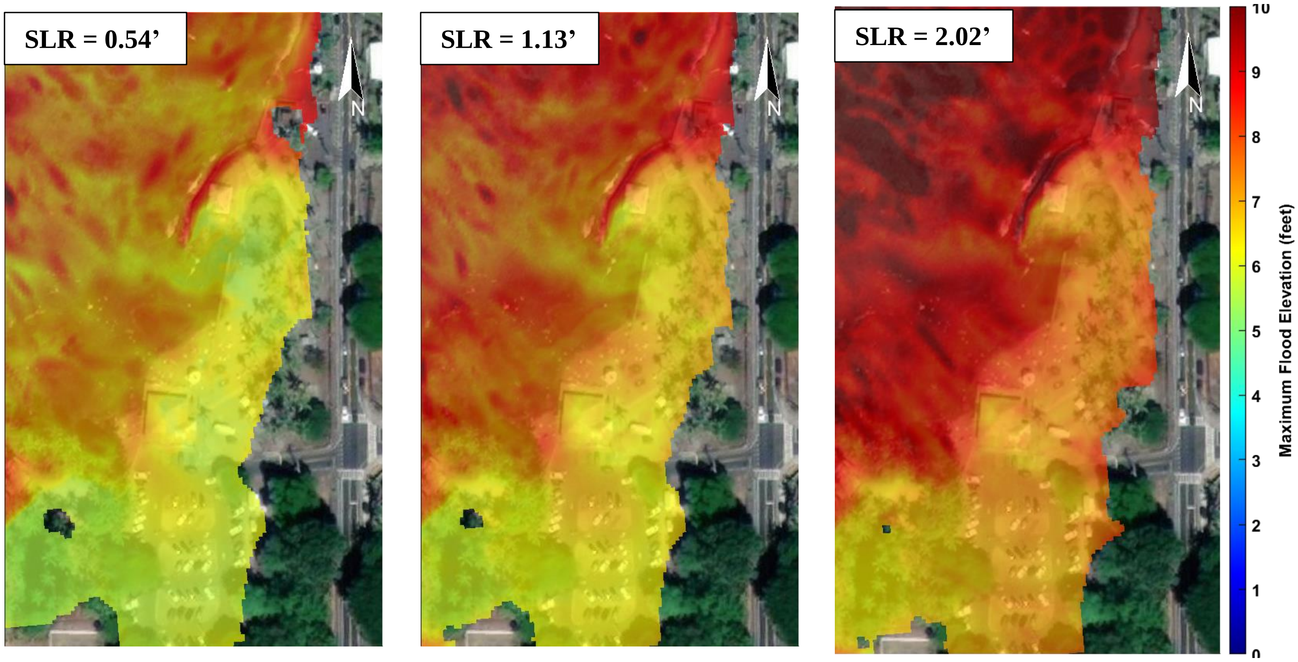

Figure 2-39. Modeled maximum inundation flood elevation (feet, MSL) for the extreme 50-year Kona wave event

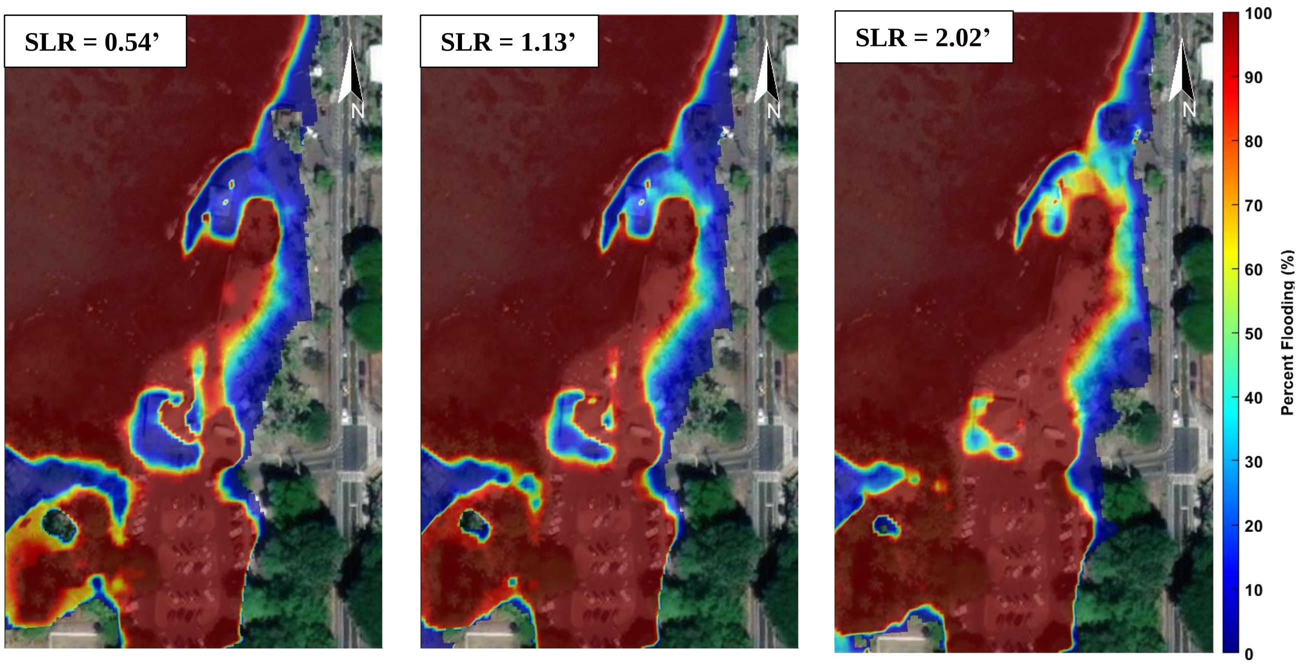

Figure 2-40. Modeled frequency of wave inundation during simulation period for the extreme 50-year Kona wave event

In summary, with SLR of about 2 feet, currently estimated to occur about the year 2060, virtually the entire park will be flooded up to about the +8 to +9 foot elevation by the extreme 50-year Kona wave event, resulting in water depths of 3 to 4 feet throughout the park. The water velocity from wave uprush will be about 10 to 15 feet/second, resulting in a force on structure foundations and walls. Recommendation: Elevate or relocate park features above the +8 to +9 foot elevation.

2.7 Coastal Processes

The shoreline in the project areas is generally a basalt lava flow, with cover of sand in the center of the bay which forms the beach area. The beach sand composition is roughly 60 percent basalt (lava) and 40 percent calcareous (coral and shell fragments), with medium to coarse grain size. Sand also covers most of the backshore area of the park, with thin sand troughs, coral colonies and basalt rubble scattered over the bay floor.

Figure 2-41 shows the XBeach-NH simulated nearshore wave-generated currents for a prevailing WNW swell event. The result shows that currents typically flow south to north through the bay and out of the bay to the north through the deeper waters there. Currents at the shoreline are generally weak under prevailing conditions. During more energetic wave events and/or higher water levels, larger waves reach the shoreline which may mobilize shoreline sand seaward into the bay. The sand may then be deposited on the reef flat or transported out of the system to the north by the north flowing currents.

Although there are anecdotal reports by long time residents that there was once an extensive sand beach along the bay shoreline, there is little evidence of that today. There is no natural source of sand or littoral transport of sand along this coastline to naturally nourish or replenish the beach. And the bay is directly exposed to storm waves and high water levels, and strong wave generated currents. Sand lost during storm periods does not return, and over time the beach has diminished in size. The limited remaining existing sand is primarily held in place by shoreline structures and walls. Additional sand placed along the park shoreline would not be expected to stay in place without stabilizing structures to prevent sand movement. And the potential movement of sand fill from the beach offshore onto the reef flat could adversely affect marine flora and fauna.

Figure 2-41. XBeach-NH modeled wave generated current vectors for the prevailing WNW swell

3. RECOMMENDATIONS

3.1 Existing Park Facilities Condition

A structural condition assessment of the existing park facilities has been accomplished, and the results are provided in Appendix A. The assessment shows the following signs of deterioration and possible failure.

- All the existing rock walls (main pavilion foundation, center shore parallel CRM wall, north seawall, and north restroom foundation wall) show significant signs of deterioration and possible near-term failure – foundation undermining, leaning, sink holes behind the walls, missing stone, failing concrete, missing wall sections. Portions of the walls are considered potentially hazardous to park users.
- The main pavilion structural elements (foundation, posts and beams, roof) show significant deterioration – corrosion, dry rot, termites. It also does not appear to meet current building code requirements for vertical and lateral load resistance. It is likely the cost of repair would equal the cost of replacement. The main pavilion's close proximity to the waterline, its low floor elevation, and its damaged/deteriorated condition makes its replacement and relocation a high priority. Its location and elevation will result in regular flooding during periods of high tides and annual wave conditions by about the year 2040.
- The north pavilion is severely deteriorated and is currently closed to use. This should be removed.
- The north restroom, while still functional, is showing signs of deterioration – the foundation is severely undermined and collapsing, the concrete block walls are spalling, and wood roof members are showing signs of rot. By about the year 2040 the restroom vicinity will flood during annual high water and wave conditions.

3.2 Planning Considerations

The following general considerations and assumptions are used to guide development of preliminary concept plans for park improvements.

1. Maintain a functional park until the approximate year 2060, or longer if climate and sea level change permit. This would require
    - Relocating the main pavilion to higher ground, minimum +8-foot ground elevation, and provide for a floor elevation of +10 feet to avoid flooding during an extreme wave event.
    - Elevation of at least a portion of the park grounds to a minimum +8 feet to avoid annual flooding.
    - Elevating or relocating the parking area.
2. Provide for access to the water from the backshore park area.
3. Minimize incursion beyond the current waterline and into nearshore waters to minimize impacts to marine habitat.

3.3 Concept Improvement Plans

The following suggested improvement plans are preliminary concepts, based on the results of the oceanographic and coastal processes analysis. The concepts present alternative actions to adapt the park to the coastal water level and wave inundation expected to occur over the next 40 to 50 years. They are intended to initiate discussion with the community and county agencies on topics such as what is desired for the park's future, what facilities are desired, how much change in the park and its appearance is acceptable/desired, etc.

3.3.1 *Plan 1*

Plan 1 would provide minimal facilities and prepare the park to be functional under expected future coastal water level conditions until approximately 2060. The primary improvement elements would consist of the following, and are shown on Figure 3-1.

- Construct a new CRM wall at the existing 5-foot ground elevation contour, with a crest elevation of +10 feet, and fill mauka of the wall to the +8 foot elevation. Construct stairs and/or a ramp from the wall crest down to existing solid ground (approximate MSL elevation) to provide access to the water.
- Construct a new main pavilion/restroom landward of the new CRM wall. A 50-year storm wave event would flood the coastline up to the +8 to +9 foot elevation, thus the pavilion should be designed to accommodate an infrequent flooding event to this elevation to avoid damage to the structure.
- Construct a new rock rubblemound revetment foundation protection for the north restroom to prolong its useful life. This facility would eventually be removed when its maintenance/repair is no longer cost effective.
- Remove the existing main pavilion, north pavilion, and CRM wall along the shoreline. Over time the existing sandy shoreline can be expected to erode back to the new CRM wall.
- Initially the main pavilion foundation wall and slab should remain, however as this continues to deteriorate it will eventually have to be removed. Similarly, the north sea wall should remain, however it can be expected to continue to deteriorate until its eventual collapse.
- Elevate or relocate the parking lot, say by 2040.

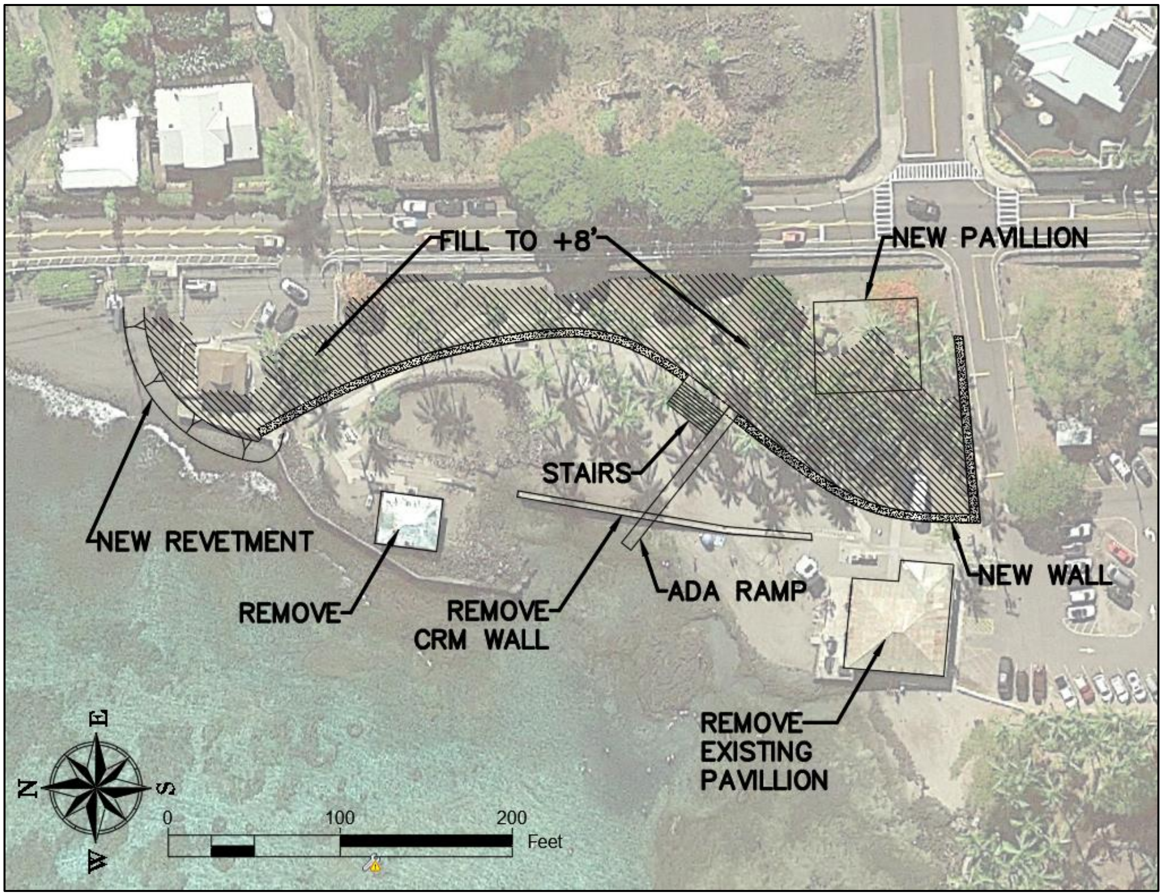

Figure 3-1. Plan 1 Park Improvements

3.3.2 *Plan 2*

Plan 2 would consist of the elements of Plan 1, plus replacing the north seawall with a new rock rubblemound revetment/groin, as shown on Figure 3-2. This would provide for a small stable sand beach.

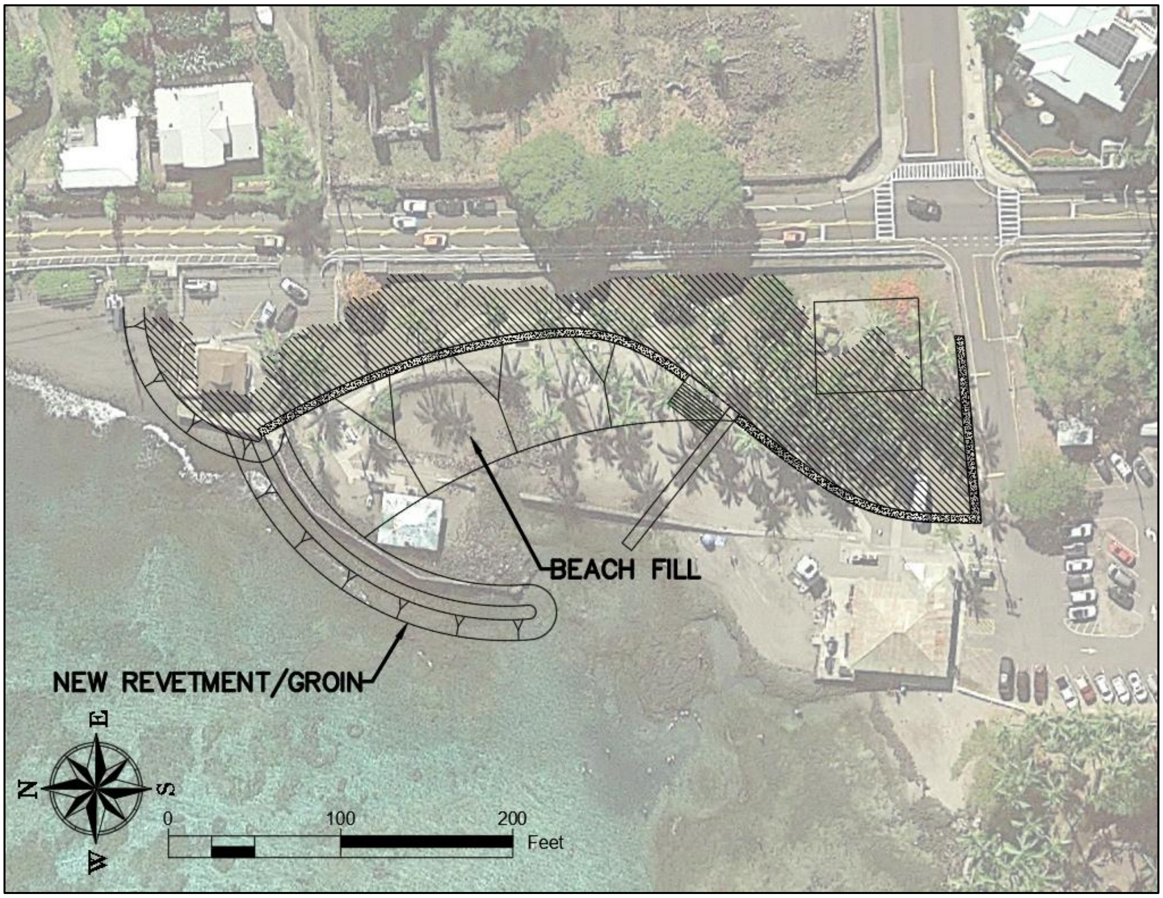

Figure 3-2. Plan 2 Park Improvements

3.3.3 *Plan 3*

Plan 3 would consist of the elements of Plans 1 and 2, plus construction of a new rock rubblemound groin at the location of the existing main pavilion and beach sand fill to create a stable concave beach between the two groins as shown on Figure 3-3.

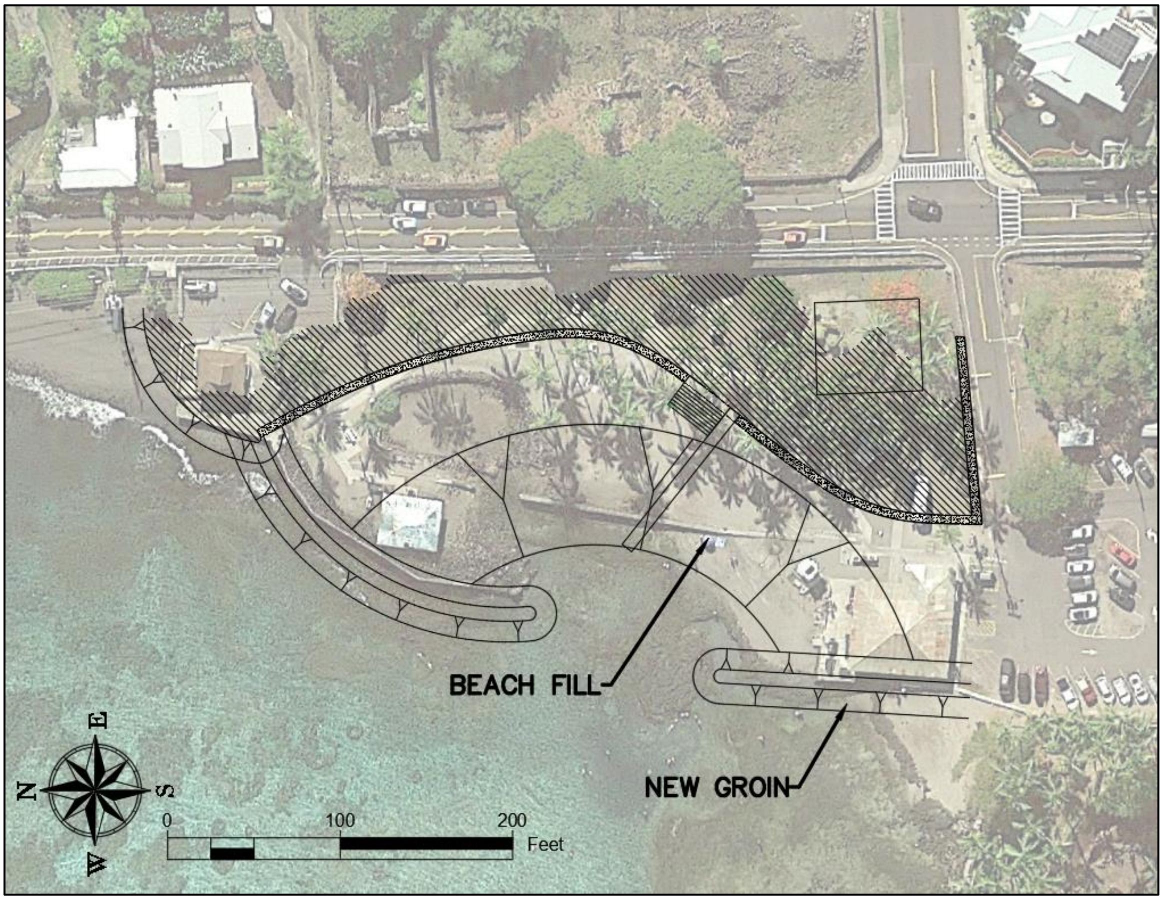

Figure 3-3. Plan 3 Park Improvements

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5. APPENDIX A: STRUCTURAL CONDITION ASSESSMENT

Kahalu'u Beach Park Structural Condition Assessment
Kahaluu-Keauhou, Island of Hawaii, Hawaii

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Structural Engineers

Glenn Miyasato, P.E.

Prepared for:
Sea Engineering, Inc.

July 28, 2021

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DESCRIPTION AND SCOPE OF WORK

Kahalu'u Beach Park is a County of Hawaii park located in Kahaluu-Keauhou on the west coast of the Island of Hawaii. The primary structures at the park consist of a main pavilion at the south end, restroom and pavilion at the north end, and CRM seawalls along the shoreline fronting the main pavilion, north pavilion, north restroom, and beach area between the main and north pavilions. Due to concerns about the current structural condition of the structures, it was requested we perform a visual assessment to identify deterioration, damage or structural deficiencies and provide preliminary recommendations for repairs. For this assessment, our scope of work consisted of the following items:

- Perform a half-day site visit on July 26, 2021 to observe existing conditions that could be viewed from ground level at interior public areas and the exterior grounds around the structures and to discuss various repairs and related concerns with Sea Engineering. Hidden conditions such as those covered with roof, ceiling, wall and floor coverings or ground were not reviewed during our visit.
- Qualitatively assess the current structural condition of the aforementioned structures based on visible deterioration, damage or other distress observed during our site visit.
- Provide preliminary structural repair recommendations.
- Prepare a short report summarizing the results of our assessment and associated repair recommendations.

OBSERVATIONS

The following conditions were observed during our review:

<u>Main Pavilion (Photo 1)</u>

- The one-story pavilion is about 50 ft by 40 ft in plan with the restroom extending out about 12 ft by 22 ft at the southeast corner. The double-pitched hip roof is constructed of corrugated steel roofing panels spanning to 2x3 flat purlins at 2 ft on centers and wood rafters and/or trusses at 4 ft on centers that span to perimeter 6x10 beams and 3½ in. diameter interior posts and 4½ in. diameter corner posts at 10 ft on centers. The interior framing is covered with a ceiling. The restroom and interior kitchen are enclosed by wood stud, CMU and CRM walls. No hurricane clips were visible. The building foundation appears to be a thickened edge concrete slab. The pavilion was possibly built in the 1960s or early 1970s.
- Significant corrosion resulting in holes through the metal was observed in the roof panels (Photo 2). Some panels have been replaced.
- Significant termite deterioration and dry rot was observed at the 2x6 rafter tails (Photo 3) and perimeter beams (Photo 4).
- Significant corrosion was observed in perimeter posts (Photo 5). The bases of several posts have been repaired (Photo 6).

Photo 1. Northeast Elevation

Photo 2. Corroded and Replaced Roof Panels

Photo 3. Deteriorated Eave Rafter

Photo 4. Deteriorated Beam

Photo 5. Corroded Post

Photo 6. Repaired Post Base

North Pavilion (Photo 7)

- The north pavilion appears to be of similar construction as the main pavilion. The one-story structure is about 24 ft by 18 ft in plan with 5 ft roof eaves. The double-pitched hip roof is constructed of corrugated steel roofing panels spanning to 2x3 flat purlins at 2 ft on centers and wood rafters and/or trusses at 4 ft on centers that span to perimeter beams and 3½ in. diameter posts at 5 to 8 ft on centers. No hurricane clips and shear walls are visible. The building foundation appears to be a thickened edge concrete slab on sand and cobbles.
- Significant corrosion resulting in holes through the roof panels was observed (Photo 8).
- Severe dry rot deterioration was observed at the rafter tails and perimeter purlins (Photo 9).
- Significant corrosion was observed in perimeter posts (Photo 10).
- Undermining of up to 1 to 2 ft deep was observed along west and south slab edges. The edges have been shored up with cobble (Photo 11).
- The pavilion has been fenced off (Photo 12).

Photo 7. Northeast Elevation

Photo 8. Corroded Roof Panels

Photo 9. Deteriorated Eave Rafters

Photo 10. Corroded Posts

Photo 11. Slab Edge Undermining

Photo 12. Fencing Around Pavilion

North Restroom (Photo 13)

- The one-story building is about 22 ft by 22 ft in plan and is constructed of asphalt shingle covered wood planks spanning to 4x8 wood rafters at 6 ft on centers supported on 6 in. CMU walls bearing on a thickened edge concrete slab-on-grade. No hurricane clips are visible.
- Dryrot and termite deterioration was observed at roof framing (Photo 14).
- Roof framing connections (Photo 15) and roofing trim (Photo 16) are corroded.
- Spalling was visible along the base of all perimeter CMU walls (Photo 17). Some of the cracked and delaminated areas appear to be previously patched spalls that have spalled again.
- Deteriorated CMU faces were observed at the base of the east perimeter wall (Photo 18).
- Sinkholes and undermining of up to 10 ft in from the retaining wall west (Photo 19) and north (Photo 20) of the restroom was observed. The undermining may extend to the building.

Photo 13. Northeast Elevation

Photo 14. Termite Deteriorated Beam

Photo 15. Corroded Connection

Photo 16. Corroded Roof Trim

Photo 17. CMU Spalling

Photo 18. Deteriorated and Spalling CMU

Photo 19. West Side Sinkhole

Photo 20. North Side Sinkhole

Main Pavilion Seawall (Photo 21)

- CRM retaining walls up to a 3 ft maximum height support the retained soil and concrete lanai slabs at the west and north sides of the structure. The approximately 50 ft long west facing retaining wall appears to be bearing largely on rock while the approximately 40 ft long north facing wall is bearing on sand and cobbles.
- Undermining of up to one foot deep by six inches high was observed at isolated locations of the north facing retaining wall (Photo 22).

Photo 21. Northwest Elevation

Photo 22. Undermining at Wall Toe

North Pavilion Seawall (Photo 23)

- The CRM seawall is approximately 180 ft long by about 7 to 8 ft high and extends from the south end of the north restroom wall, curving around the north pavilion to end about 60 ft south of the pavilion. The wall is approximately 18 in. at the top mortar cap with an approximate 1:12 front batter and 3:12 rear batter.
- An approximate 30 ft long by 4 ft high top section of wall at the south end has collapsed (Photo 24). The next 50 ft long middle section appears stable as it may be bearing on rock (Photo 25). The remaining 100 ft northern section has experienced some undermining of up to 1 to 3 ft in from the toe (Photo 26 and 27) with an isolated 4 ft long by 4 ft high partial collapse at the base (Photo 28), as this section of wall appears to bear largely on cobbles and sand. Cobble has been placed behind the wall apparently due to washout of sand substrate.

Photo 23. North Elevation

Photo 24. Partial Collapse of Top Section

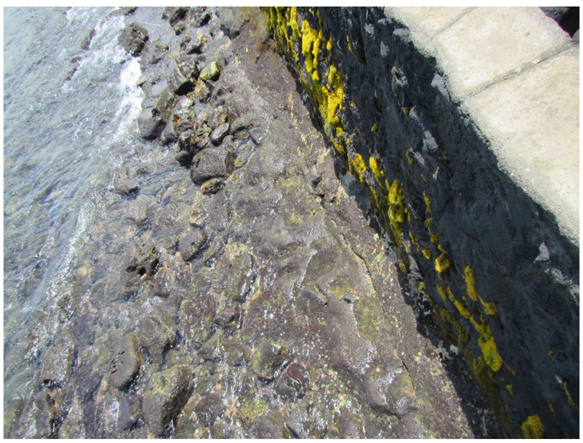

Photo 25. Non-undermined Section

Photo 26. Undermined Section

Photo 27. Undermined Section

Photo 28. Partial Collapse at Base

North Restroom Seawall (Photo 29)

- The CRM seawall consists of an approximately 50 ft long by about 4 to 5 ft tall west facing section and approximately 30 ft long by 7 ft high north facing section. The walls generally appear to be bearing on sand and cobbles.
- The west facing section has rotated out to create a reverse batter of between 2:12 at the middle section to more than 4:12 at the ends (Photo 30). The north facing wall is undermined with the undermining extending 1 to 2 ft in from the face (Photo 31). Sinkholes are visible behind the walls and the northwest corner and south end are partially collapsed (Photo 32). Portions of concrete pavement behind the walls are likely undermined.

Photo 29. North Elevation

Photo 30. Outward Rotation of West Wall

Photo 31. Undermining at North Wall

Photo 32. Partial Collapse at Both Ends

Middle Seawall between North and Main Pavilions (Photo 33)

- The CRM seawall is approximately 250 ft long by 3 to 4 ft tall and 2½ ft wide at the top with no significant front or rear batter. The wall appears to bear largely on sand and cobble.
- An approximate 40 ft portion of wall fronting the pond has collapsed (Photo 34).

- An approximate 100 ft long portion to the south of the collapse portion is significantly undermined (Photos 35 and 36).

Photo 33. West Elevation

Photo 34. Collapsed Section

Photo 35. Undermining at Front Toe

Photo 36. Undermining at Rear

STRUCTURAL ASSESSMENT AND PRELIMINARY REPAIR RECOMMENDATIONS

Due to wind and seismic upgrades to building codes over the years, it is unlikely that a structure meets current code requirements for lateral load resistance, unless the structure was recently designed. An assessment and preliminary repair recommendations for each structure is provided below. Due to their similarity in construction and condition, the north and main pavilion are grouped together for assessment purposes.

<u>Main and North Pavilions</u>

From the observed small and widely spaced roof framing, lack of hurricane clips and apparent lack of lateral load resisting elements, the building structural system does not appear to meet code requirements with regard to vertical and lateral load resistance. These inadequacies are further exacerbated by the significant termite deterioration and dryrot decay observed in the roof framing from intermittent water exposure and corrosion of roof framing and post supports. For the north pavilion, undermining of the west and south slab edges compromises the foundation bearing support for the perimeter posts at those sides. Preliminary repair recommendations are as follows:

- Replace the corroded roof panels and sister and retrofit deteriorated and substandard roof framing. As several posts have already been repaired, it is recommended that all posts be replaced. As this work would affect a large portion of the building structure above the slab, replacement of the entire building structure should be considered to provide a new structure that can be designed and constructed to meet current structural code requirements. In addition, the large extent of repair work may trigger upgrade of the entire structure to meet current structural codes, in which case a new structure may also be more cost effective.
- The foundation slabs could be incorporated into the new structures if they are determined to be adequate to support the loads from that structure. In the case of the north pavilion, the undermined slab edges should be resupported and protected against future undermining. This possibly could be accomplished by installing a concrete cutoff wall below the undermined slab edges that extends down to less erodible substrate.

<u>North End Restroom</u>

As the roof framing appear to be engineered, the structure may meet requirements for vertical load resistance. However, due to the apparent lack of hurricane clips, it may not meet requirements for lateral load resistance for the code in effect at the time of its construction. The spalling at the base of perimeter walls appears to be caused by expansive corrosion of the embedded reinforcing steel. This corrosion may be due to infiltration of chlorides into the concrete from exposure to salt laden air from the marine environment. Corrosion of roof framing connections and trim may also be accelerated from direct exposure to salt laden air in combination with deterioration of surface protective coatings. Deterioration of exterior CMU block faces at the base of the east wall may be due to poor quality block. Preliminary repair recommendations are as follows:

- Repair spalling at the base of perimeter walls to protect the reinforcing from further corrosion that can adversely affect the wall capacity over time. Repairs should consist of removing and squaring the unsound CMU and grout, removing additional concrete around the exposed reinforcing, cleaning the reinforcing of corrosion, coating the reinforcing with an anti-corrosion agent and filling the void with an appropriate polymer modified repair mortar.
- Deteriorated CMU block should also be repaired by removing the soft surfaces and patching the voids in lifts with a polymer modified repair mortar.
- If sufficient section remains, clean steel framing connections of corrosion and recoat with an appropriate anti-corrosion coating to protect against future corrosion.
- Severely termite and dryrot deteriorated roof framing should be spliced or replaced to restore the original integrity of the framing.
- Sinkholes behind and voids extending under and through the adjacent north and west seawalls should be investigated further to confirm whether they extend under the building foundation. Any identified voids should be filled with appropriately consolidated subgrade material to restore bearing capacity under the building foundations and surrounding pavement.

<u>Main Pavilion Seawall</u>

As the wall profiles are not visible, the adequacy of the walls with respect to resisting code specified loads is unknown. Undermining of the north facing seawall appears to be due to erosion of the underlying sand and cobble substrate due to wave action. For repairs, it is recommended that the wall base be extended down to an erosion resistant substrate such as rock by removing erodible material under the wall in phases and filling the void with concrete.

North Pavilion Seawall (Photo 33)

The collapsed upper portion of wall at the south end appears to be due to inadequate wall mass to resist lateral forces from wave action. Undermining of the majority of the wall appears due to erosion of the underlying sand and cobble substrate due to wave action. For repairs it is recommended that the wall base be extended down to erosion resistance substrate such as rock by removing erodible material under the wall in phases and filling the void with concrete. It is also recommended that the mass of the wall be increased by widening the wall thickness through the addition of mortared stone behind the wall. Due to the extent of repairs, replacement of this wall may also be considered. Repair or replacement of this wall is required to maintain the integrity of the north pavilion foundation.

North Restroom Seawall

The undermining, significant outward rotation and sinkholes at the wall appear to be caused by erosion and loss of bearing of the underlying sand and cobble substrate below the wall due to wave action. Due to the amount of outward rotation, it is recommended the west facing wall be demolished and replaced with a new wall bearing on erosion resistant substrate. The north facing wall should be extended down to erosion resistant substrate by removing erodible material under the wall in phases and filling the void with concrete. Repair or replacement of this wall is required to protect the integrity of the north restroom foundation.

Middle Seawall between North and Main Pavilions

The collapsed portion of wall appears to be due lateral forces from wave action and possible undermining of the wall. The undermining over the majority of the remaining wall appears to be caused by erosion of the underlying sand and cobble substrate below the wall due to wave action. Due to the extent of the collapse and undermining, it is recommended the wall be demolished and replaced with a new wall bearing on erosion resistant substrate.

The opinions formulated during this assessment are based on observations made at the time of the investigation. No guarantee or warranty as to future life, performance, or need for repair of any reviewed conditions is expressed or implied. This assessment does not address any other portions of the structure other than those areas specifically mentioned, nor does it include an assessment of architectural, geotechnical, mechanical, electrical, civil, hazardous materials or other nonstructural conditions. Compliance with any specifications and legal or code requirements, except as expressly noted, is specifically excluded from this assessment.

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