SOUTHEAST OAHU REGIONAL SEDIMENT MANAGEMENT PROJECT

LANIKAI BEACH RESTORATION PILOT PROJECT STUDY DESIGN REPORT

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EXECUTIVE SUMMARY

The Honolulu District of the U.S. Army Corps of Engineers recently completed a Regional Sediment Management (RSM) plan on the southeast shores of Oahu. The project was co-sponsored by the State of Hawaii Department of Land and Natural Resources (DLNR) Office of Conservation and Coastal Lands (OCCL) and the study area ranges from Makapuu Point to Mokapu Peninsula. That study provided guidance on solving sediment problems in the region using a systems approach that considers the entire region from the mountains to the sea. The plan included conceptual demonstration projects at Kaelepulu Stream, Lanikai, Bellows AFS, Kaiona Beach, and Kaupo Beach.

The Southeast Oahu RSM culminated with the 2009 “Southeast Oahu Regional Sediment Management Plan” report. The Lanikai conceptual study within the RSM plan focused on beach restoration options for Lanikai Beach, including beach nourishment with and without structures. The RSM has been extended to include the design for a pilot beach restoration project on the southern Lanikai shoreline.

The pilot project is located at the Pokole Way beach access along a shoreline reach that has been hardened with seawalls and revetments. There is presently no dry beach along this reach due to chronic erosion. The erosion history of this shoreline reach has been analyzed by the University of Hawaii Coastal Geology Group, who produced erosion maps for southeast Oahu as part of the SEO/RSM report and found erosion rates of 9 feet/year near Wailea Point. As a result of the erosion, an extensive system of seawalls and revetments has been constructed to stabilize the shoreline. This report presents the design of a pilot beach project along the southern Lanikai shoreline.

Oceanographic design parameters were determined and design wave characteristics were produced based on these findings. A beach cell stabilized by two tuned T-head groins was designed at the Pokole Way beach access. The design includes a beach with a minimum 30-foot wide beach crest, where the beach crest is defined as the area between the vegetation line and the top of the beach face. The groin design utilizes geotextile tubes and the project requires a total of 22,100 cubic yards of sand.

Construction cost for the project is estimated to be $4,750,000. The cost estimate assumes that there is a suitable sand source available. The cost of $150 per cubic yard of sand utilized in this estimate was developed based on cost estimates for offshore sand recovery at other locations in Hawaii. A more detailed site-specific estimate from a dredging contractor could result in a different cost.

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LIST OF ABBREVIATIONS

ACES	Automated Coastal Engineering System
AFS	Air Force Station
CDIP	Coastal Data Information Program
CEDAS	Coastal Engineering Design and Analysis System
CGG	Coastal Geology Group
CHL	Coastal & Hydraulic Laboratory
cot	Cotangent
CRM	Concrete Rubble Masonry
Dir	Direction
DLNR	Department of Land and Natural Resources
E	East
ENE	East-Northeast
ESE	East-Southeast
ft	Feet
G	Gap Width
GIS	Graphical Information System
H	Horizontal
H	Wave Height
$H_{1/3}$	Significant Wave Height
$H_{1/10}$	Extreme Wave Height
H_s	Significant Wave Height
HDPE	High-density Polyethylene
K_D	Armor Stone Stability Coefficient

lb, lbs	Pounds
LiDAR	Light Detecting and Ranging
m	Meters
MHHW	Mean Higher High Water
MHW	Mean High Water
MLLW	Mean Lower Low Water
MLW	Mean Low Water
MSL	Mean Sea Level
N	North
NE	Northeast
NNE	North-Northeast
NNW	North-Northwest
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Service
OCCL	Office of Conservation and Coastal Lands
RSM	Regional Sediment Management
S	South
SE	Southeast
sec	Seconds
SEO	Southeast Oahu
SHOALS	Scanning Hydrographic Operational Airborne LiDAR Survey
S_r	Specific gravity
SSE	South-Southeast
SSW	South-Southwest
SW	Southwest
SWL	Still Water Level
T	Wave Period
T_s	Significant Wave Period
T_p	Peak Wave Period
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
V	Vertical
W	Weight
W	West
WIS	Wave Information Studies
WNW	West-Northwest
w_r	Unit Weight of Stone
WSW	West-Southwest

1. INTRODUCTION

1.1 Project Description

In 1999, the U.S. Army Corps of Engineers (USACE) began the Regional Sediment Management (RSM) Demonstration Project, which has local application on the southeast shores of Oahu (SEO). The SEO/RSM covered Makapuu Point to Mokapu Peninsula, with the stated tasks of:

1. Documenting long-term trends in nearshore wave climate for the windward side of Oahu
2. Modeling nearshore circulation
3. Developing a regional sediment budget
4. Developing a GIS along the southeast Oahu coast
5. Identifying suitable sand sources
6. Mapping shoreline change for the region

The SEO/RSM culminated with the 2006 “Regional Sediment Management Plan” report. The RSM report presented beach restoration options for Lanikai Beach, including beach nourishment with and without structures. The RSM has been extended to use the findings of the RSM plan and focus those findings to design a pilot beach restoration project on the southern Lanikai shoreline.

1.2 Study Area Location and Description

Lanikai is located on the windward side of Oahu between Kailua and Waimanalo (Figure 1-1). The 1.5 mile stretch of Lanikai coastline extends from Alala Point in the north to Wailea Point in the south (Figure 1-2). While the ends of the Lanikai shoreline have experienced severe erosion and presently have little or no sandy beach, the 0.5-mile long beach toward the center of Lanikai is considered one of Hawaii’s top beaches, benefitting from the protective nearshore reef and scenic views of sunrise over the Mokulua Islands. The protected nearshore waters and gentle breezes favor ocean sports like swimming and windsurfing.

The beach has experienced cycles of erosion and accretion and much of the private property fronting the shoreline has been hardened with an extensive system of seawalls and revetments. Homeowners and beach users have expressed interest in restoring Lanikai Beach to provide recreational value and shore protection to the community. Sea Engineering, Inc., as a consultant to the U.S. Army Corps of Engineers, Honolulu District, has conducted an evaluation of beach restoration alternatives.

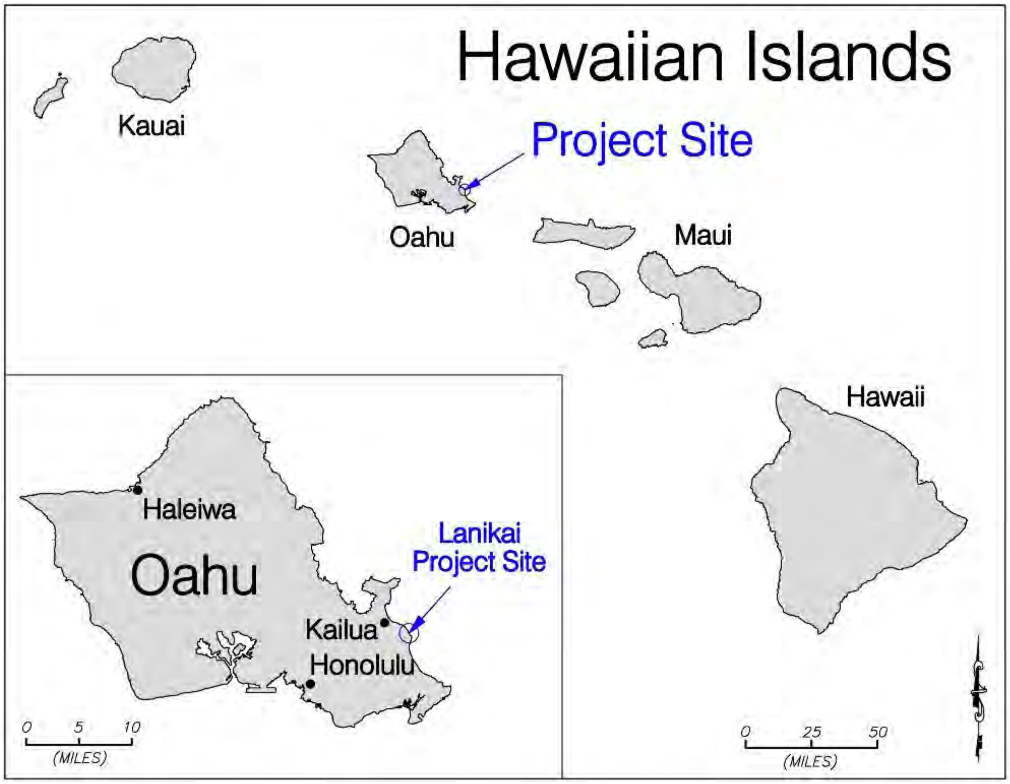


Figure 1-1 Project location map

Figure 1-2 Lanikai study location map

2. SITE CONDITIONS

2.1 Regional Setting

Lanikai is located on the windward shore of Oahu, between Waimanalo and Kailua. The project location was shown previously in Figure 1-2. The shoreline protrudes seaward in comparison to the adjacent Waimanalo and Kailua bays. Lanikai is bordered by Alala Point on the north and Wailea Point on the south. The Mokulua Islands are located approximately 4,000 to 4,500 feet offshore. The shallow fringing reef extends 2,500 to 3,500 feet offshore; Lipp (1995) reported that seaward portions of the reef are only a few inches below mean lower low water (MLLW), which is the annual average of the daily lower low tide level.

The waters landward of the reef edge are typically five to ten feet deep. The seafloor is composed of fossilized limestone or coral rubble with reef outcrops and channels. A thin layer of sand exists over much of this area, while areas with thicker sand deposits are also found.

Detailed nearshore bathymetry information is available from the U.S. Army Corps of Engineers' Scanning Hydrographic Operational Airborne LiDAR Survey (SHOALS) dataset. Data exists from the shoreline to a depth of approximately 120 feet. This bathymetry data was used as input to the numerical wave modeling discussed later in Section 6. A bathymetric map of the project site based primarily on LiDAR data is shown in Figure 2-1 overlaid on a geo-referenced satellite photograph. The depth contours are shown as yellow and orange lines and are in feet relative to MLLW.

The Lanikai shoreline is shaped by the prevailing tradewind waves. These waves experience refraction and diffraction past the Mokulua islands and over the shallow fringing reef, resulting in a very complex nearshore wave pattern. Bulges in the sandy shoreline are centered 1,900 feet and 3,800 feet south of Alala Point. These bulges are produced by convergent wave patterns caused by refraction and diffraction past the Mokulua Islands and the reef. A third bulge, centered opposite Lanipo Drive, has been armored. These complex wave patterns are shown in Figure 2-2, which illustrates the wave crest patterns computed by the numerical model BOUSS2D for deepwater wave conditions *Dir* = ENE, *H* = 6 feet, *T* = 8 seconds (details on the modeling techniques are covered in Section 6.2.3).

Oblique shoreline photographs and a complete description of the Lanikai shoreline were presented in the conceptual design report (Sea Engineering, 2008).

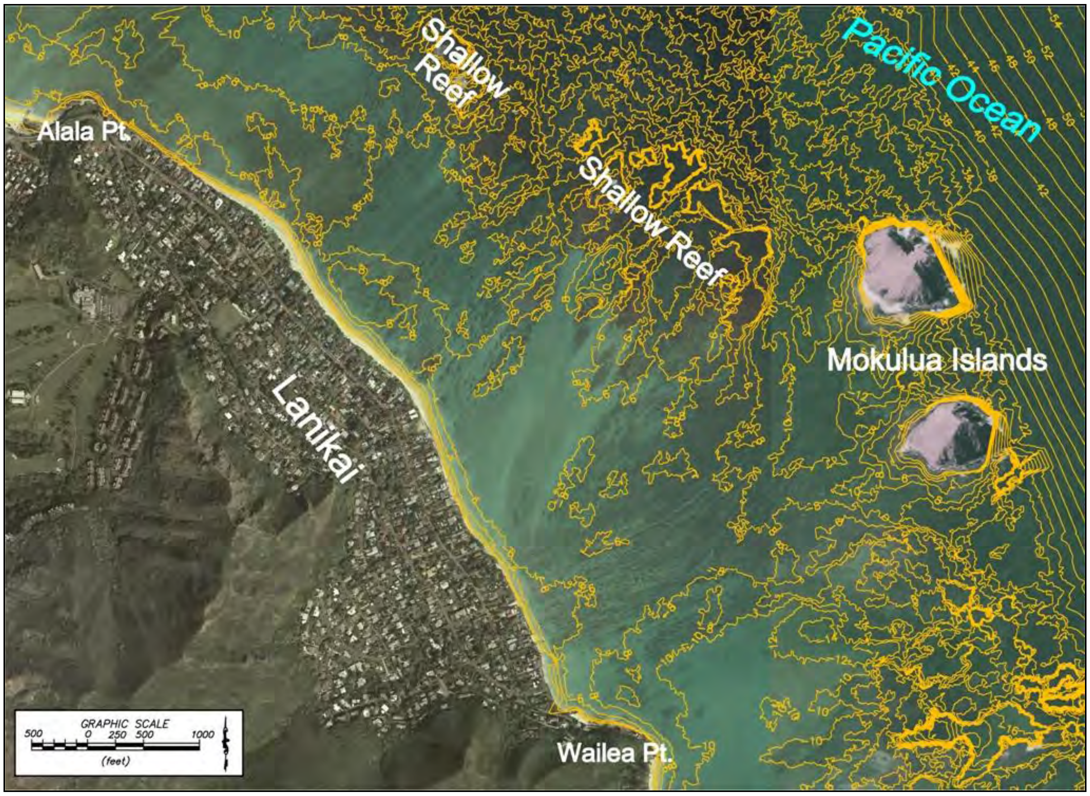


Figure 2-1 Bathymetric contours at Lanikai (contours are in feet relative to MLLW at two-foot intervals)

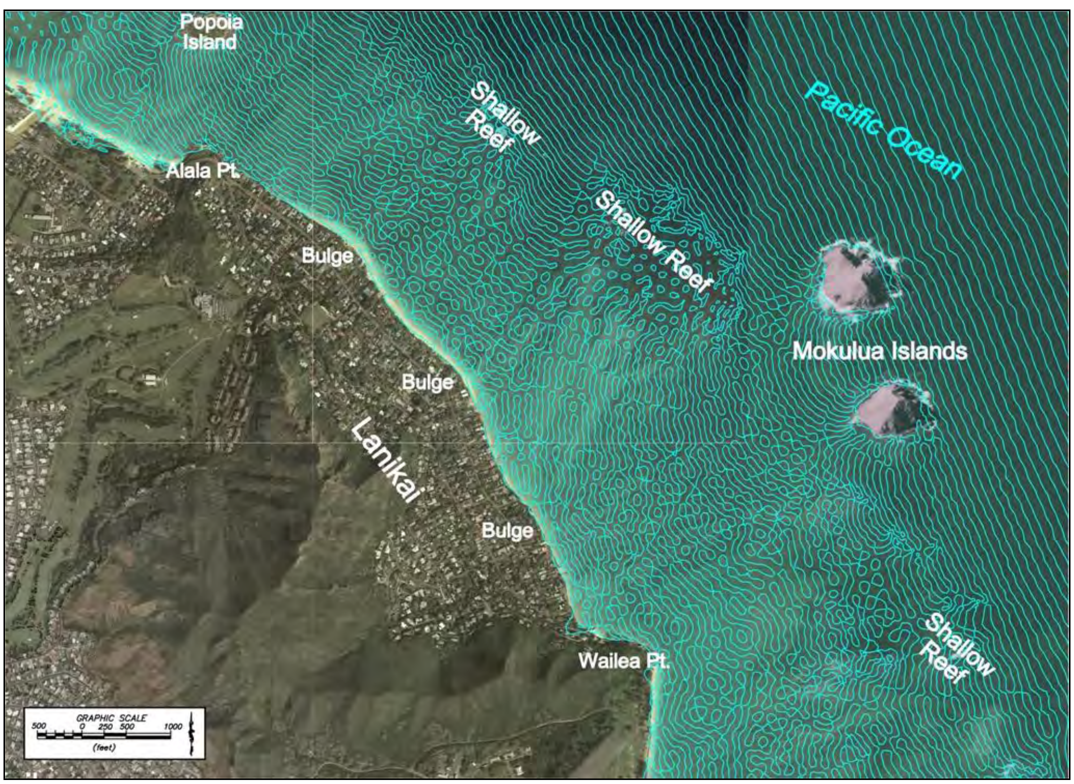


Figure 2-2 BOUSS2D wave crest orientation, *Dir* = ENE, *H* = 6 feet, *T* = 8 seconds

2.2 Project Site Description

The pilot beach demonstration project site was selected by the Lanikai Homeowners Association to be located at the beach access near the intersection of Mokulua Drive and Pokole Way. The project site is shown in Figure 2-3, along with property lines downloaded from the State of Hawaii GIS Program website overlaid on a geo-referenced satellite photo. The Pokole Way beach access, shown in Figure 2-4, is approximately 10 feet wide near Mokulua Drive; the access becomes effectively narrower toward the ocean, due to the presence of vegetation such as Naupaka.

The shoreline of the two properties to the north of the beach access is armored with vertical seawalls for a distance of about 145 feet (Figure 2-5). The seawalls transition into a non-engineered rock revetment that extends 300 feet further north (Figure 2-6). The shoreline to the south of the beach access contains a mix of vertical seawalls and non-engineered rock revetments (Figure 2-7).

Figure 2-3 Project site location


Figure 2-4 View of Pokole Way beach access from Mokulua Drive


Figure 2-5 Verticle seawalls on north side of beach access


Figure 2-6 Rock revetment to the north of the seawalls

Figure 2-7 View of shoreline south of beach access

Shoreline change along Lanikai Beach has been evaluated by the University of Hawaii's Coastal Geology Group (CGG). The CGG used historical aerial photographs dating from 1911 to 2005 to compare the shoreline change. The photographs have been ortho-rectified and geo-referenced and the low water marks on the photographs were digitized to provide a record of the long-term changes to that representative coastal feature. The Lanikai shoreline change map pertaining to the project site is shown in Figure 2-8.

The analysis for this section of Lanikai considers beach loss prior to 1989. The shoreline change map shows annual erosion rates in the project area (transects 90-97) of about 5.5 to 9.5 feet per year leading up to 1989. Since then, shore protection structures have prevented further retreat of the shoreline.

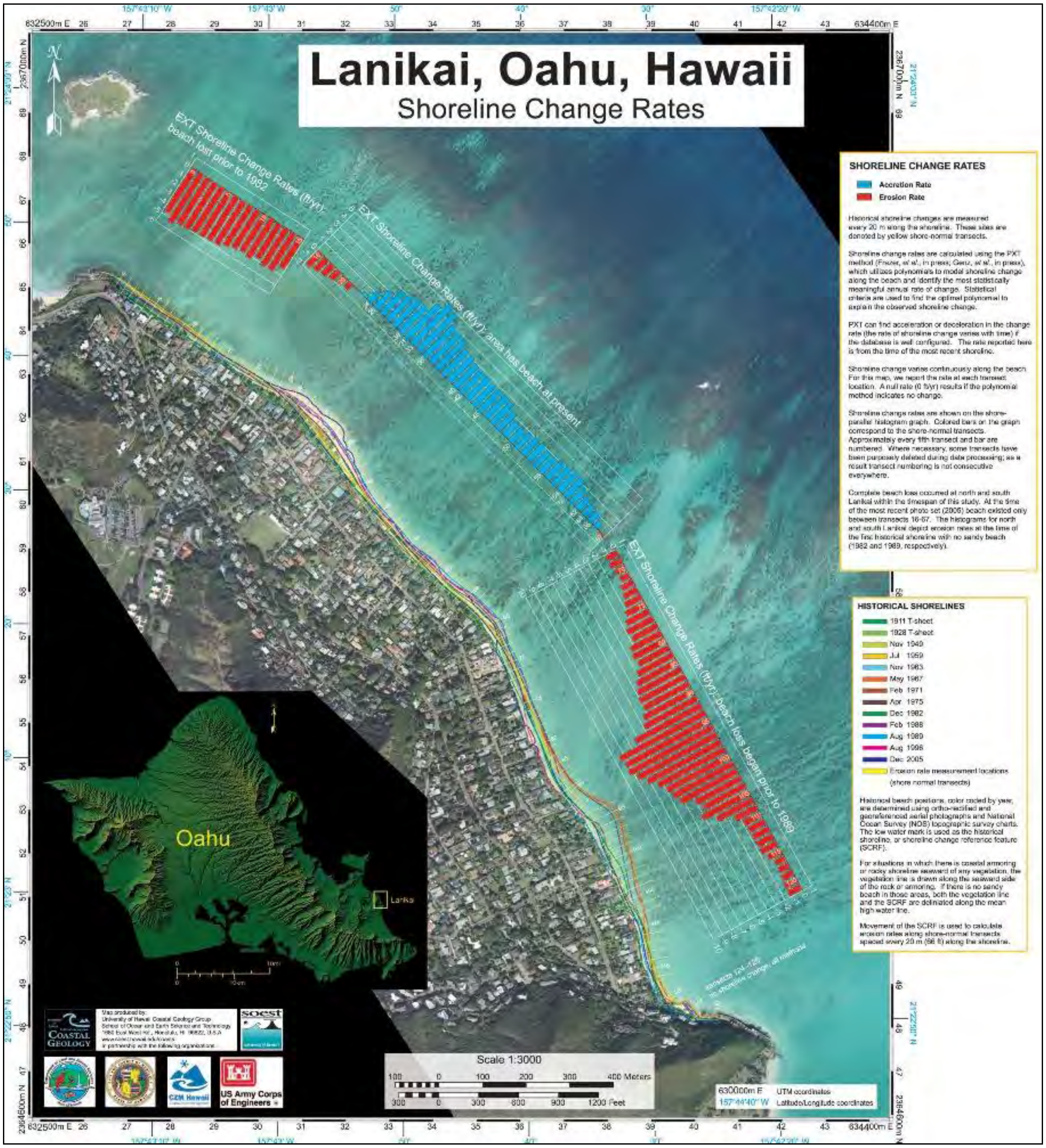


Figure 2-8 Shoreline Chage Map (Univ. of Hawaii Coastal Geology Group)

3. OCEANOGRAPHIC DESIGN PARAMETERS

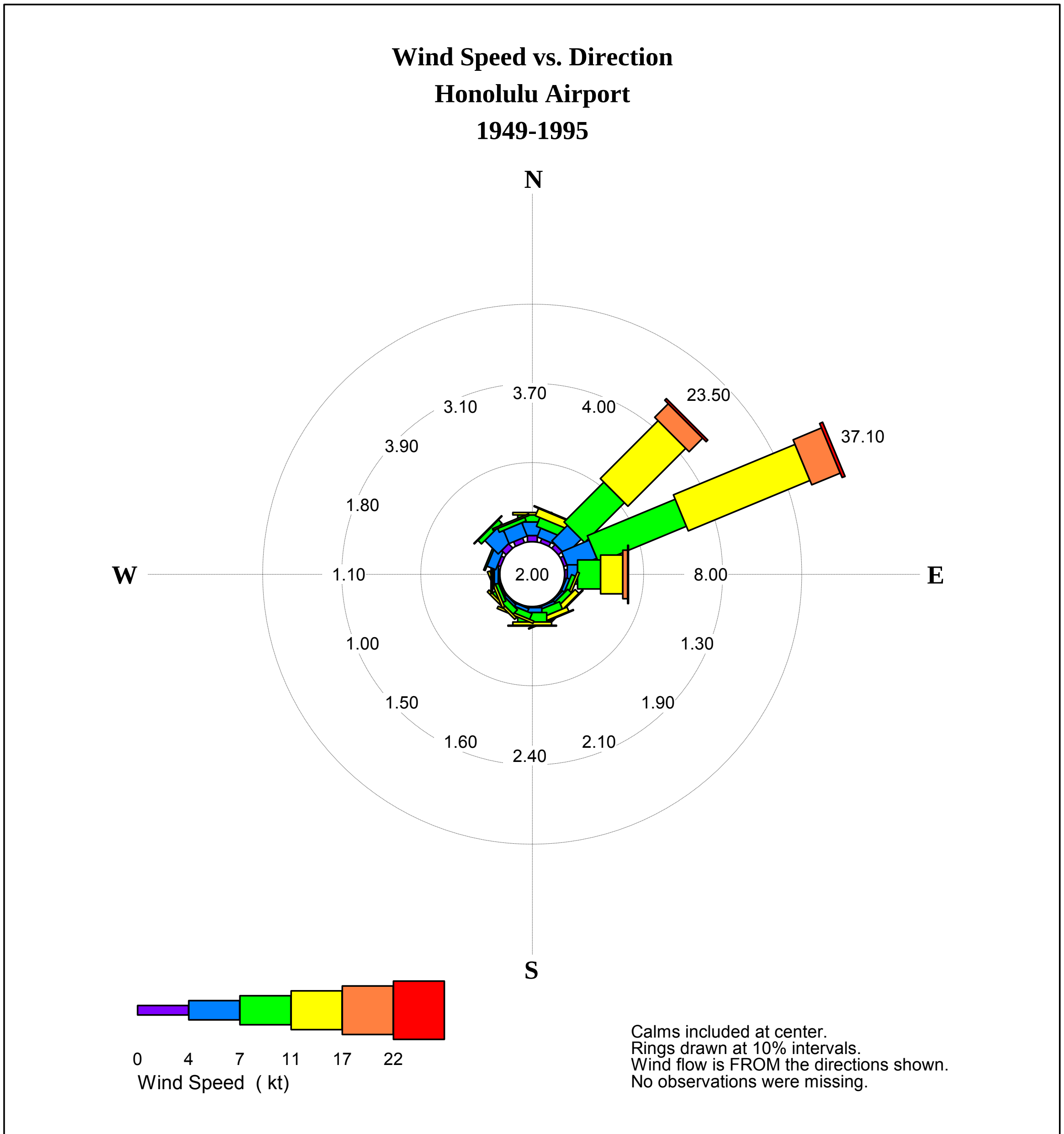
3.1 Winds

The prevailing wind throughout the year in the Hawaiian Islands is the northeasterly trade wind. Its average frequency varies from more than 90% during the summer season to only 50% in January, with an overall annual frequency of about 70%. Westerly, or Kona, winds occur primarily during the winter months, generated by low pressure or cold fronts that typically move from west to east past the islands.

Tradewinds are produced by the outflow of air from the Pacific Anticyclone high pressure system, also known as the Pacific High. The center of this system is located well north and east of the Hawaiian chain and moves to the north and south seasonally. In the summer months, the center moves to the north, causing the tradewinds to be at their strongest from May through September. In the winter, the center moves to the south, resulting in decreasing tradewind frequency from October through April. During these months, the tradewinds continue to blow; however, their average monthly frequency decreases to 50%.

During the winter months, wind patterns of a more transient nature increase in prevalence. Winds from extratropical storms can be very strong from almost any direction, depending on the strength and position of the storm. The low pressure systems associated with these storms typically track west to east across the North Pacific north of the Hawaiian Islands. At Honolulu Airport, wind speeds resulting from these storms have on several occasions exceeded 60 mph. Kona winds are generally from a southerly to southwesterly direction and occur when low pressure systems have a close approach to the islands. These storms are often accompanied by heavy rains.

Figure 3-1 shows a wind rose diagram applicable to the site based on wind data recorded at Honolulu International Airport between 1949 and 1995. The wind rose shows that the winds there come from the east through northeast nearly 70% of the time.



PERCENT OCCURRENCE: Wind Speed (kt)
LOWER BOUND OF CATEGORY

DIR	0	4	7	11	17	22
N	0.80	1.70	0.90	0.30	0.00	0.00
NNE	0.50	1.20	1.20	1.00	0.10	0.00
NE	0.60	2.80	7.10	10.30	2.40	0.30
ENE	0.40	3.80	12.00	16.60	3.90	0.40
E	0.30	1.30	2.90	2.80	0.60	0.10
ESE	0.10	0.40	0.50	0.30	0.00	0.00
SE	0.10	0.30	0.70	0.70	0.10	0.00
SSE	0.10	0.30	1.00	0.60	0.10	0.00

TOTAL OBS = 134736 MISSING OBS = 0

PERCENT OCCURRENCE: Wind Speed (kt)
LOWER BOUND OF CATEGORY

DIR	0	4	7	11	17	22
S	0.10	0.60	1.20	0.40	0.10	0.00
SSW	0.10	0.40	0.80	0.30	0.00	0.00
SW	0.10	0.30	0.80	0.30	0.00	0.00
WSW	0.10	0.20	0.40	0.30	0.00	0.00
W	0.20	0.50	0.20	0.20	0.00	0.00
WNW	0.40	1.20	0.20	0.00	0.00	0.00
NW	0.80	2.40	0.60	0.10	0.00	0.00
NNW	0.60	1.80	0.50	0.20	0.00	0.00

CALM OBS = 2695 PERCENT CALM = 2.00

Figure 3-1 Wind rose diagram for Honolulu International Airport, 1949-1995

3.2 Waves

3.2.1 Prevailing Waves

The wave climate in Hawaii is typically characterized by four general wave types. These include northeast tradewind waves, southern swell, North Pacific swell, and Kona wind waves. Tropical storms and hurricanes also generate waves that can approach the islands from virtually any direction. Unlike winds, any and all of these wave conditions may occur at the same time.

Tradewind waves occur throughout the year and are most persistent April through September when they usually dominate the local wave climate. They 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 to 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 directly exposed to tradewind wave energy.

Southern 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 sheltered from southern swell by the island itself, as well as by the southern Hawaiian islands.

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 shore of Oahu 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 not directly exposed to north swell; however, this wave energy does refract and diffract around the island and impact the site.

Waves that approach from the southeasterly to southwesterly direction associated with Kona winds and Kona lows are known as Kona storm waves. Kona storms occur when the winter low pressure systems that travel across the North Pacific Ocean dip south and approach the islands. Strong southerly and southwesterly winds generated by these storms result in large waves on exposed shorelines and often heavy rains. These events are infrequent; however, they can result in very large waves with deepwater heights up to 15 feet (Noda, 1991). Periods typically range from 6 to 10 seconds. The project site is not directly exposed to Kona storm waves.

3.2.2 Prevailing Deepwater Wave Climate

The Lanikai project site faces northeast and is primarily affected by tradewind waves; however, during the winter, north Pacific swell can refract and also affect the site. Selection of the appropriate prevailing wave conditions was necessary to evaluate the beach planforms for the conceptual beach restoration design. Measured directional wave data is available for Buoy 098 of the Coastal Data Information Program (CDIP), which is located 2.8 miles northeast of Lanikai. Semi-hourly readings of significant wave height, period, and direction are available for August 2000 to present and the proximity of this buoy to the project site make the data directly applicable. Joint frequency of wave height and period are produced for 22.5° direction bands and are shown in Table 3-1 and corresponding wave height and period roses are shown in Figure 3-2 and Figure 3-3. The wave roses show the occurrence of north swell and northeast tradewind waves. The predominant wave direction is east-northeast, occurring 32.5% of the time from that direction. The most frequently occurring wave condition is 6 to 8 feet high with a period of 8 to 10 seconds from the east-northeast.

Wave information is also available in the form of hindcast data sets provided by the U.S. Army Corps of Engineers' Wave Information Studies (WIS). WIS results are generated by numerical simulation of past wind and wave conditions. WIS information produces records of wave conditions based on historical wind and wave conditions at numerous stations around the Hawaiian Islands. These hourly records of wave conditions are available for the years 1981 through 2004.

The Lanikai project site is exposed to waves from approximately the north to south-southeast. WIS Station 99, located 45 miles north-northeast of Lanikai, was chosen as being representative for comparison with the CDIP data. This station was the closest to the project site and is fully exposed to the range of waves at Lanikai. The data, however, contains a significant amount of energy from the northwest direction that does not impact the project site. Table 3-2 shows the frequency of occurrence of wave height and period for the WIS data. Additionally, the wave height and wave period distributions for the full WIS 99 data set are presented as roses in Figure 3-4 and Figure 3-5. As with the CDIP buoy, the wave roses for WIS 99 show the north swell and tradewind waves. Since the WIS station is located further from Oahu, it is exposed to waves from a greater direction range. The waves between north-northwest and south-southeast represent 63.5% of all waves at that station. Considering only this range of wave directions, the east-northeast direction band would contain 30.2% of the energy, which is comparable to the 32.5% shown by the CDIP buoy.

Table 3-1 CDIP Buoy 098 deepwater waves, 2000-2007. Percent frequency of occurrence: significant wave height H_s (feet) vs. significant period T_s (seconds)

CDIP Mokapu Buoy

		Period (s)													
Dir (°TN)	Hs\Ts	0-2	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	22-24	24-26	Total%
NNW	0-2	-	-	-	-	-	-	-	-	-	-	-	-	-	**0.00**
326.25 -	2-4	-	-	0.00	-	0.05	0.29	0.17	0.07	0.00	0.00	-	-	-	**0.59**
348.75	4-6	-	-	-	-	0.08	0.81	0.93	0.46	0.01	0.00	-	-	-	**2.30**
	6-8	-	-	-	0.00	0.02	0.35	0.71	0.44	0.05	0.01	-	-	-	**1.58**
	8-10	-	-	-	0.00	0.00	0.06	0.14	0.12	0.02	0.00	-	-	-	**0.35**
Hs (ft)	10-12	-	-	-	-	0.00	0.02	0.02	0.02	0.01	0.00	-	-	-	**0.07**
	12-14	-	-	-	-	-	0.00	0.00	0.00	-	-	-	-	-	**0.00**
	Total%	**0.00**	**0.00**	**0.00**	**0.00**	**0.16**	**1.54**	**1.97**	**1.12**	**0.09**	**0.01**	**0.00**	**0.00**	**0.00**	**4.89**
N	0-2	-	-	-	-	0.00	0.00	0.00	-	-	-	0.00	-	-	**0.01**
-11.25 -	2-4	-	-	-	0.03	0.52	0.86	0.41	0.20	0.04	0.02	0.01	0.01	-	**2.10**
+11.25	4-6	-	-	0.00	0.03	1.05	2.80	2.08	1.15	0.26	0.11	0.06	0.01	-	**7.55**
	6-8	-	-	0.01	0.06	0.33	1.73	1.90	1.42	0.30	0.19	0.05	0.01	0.00	**6.00**
	8-10	-	-	-	0.02	0.12	0.54	0.52	0.52	0.13	0.07	0.03	0.01	-	**1.95**
Hs (ft)	10-12	-	-	-	0.00	0.05	0.13	0.14	0.11	0.06	0.04	0.01	-	-	**0.53**
	12-14	-	-	-	-	0.00	0.04	0.05	0.06	0.02	0.01	0.00	-	-	**0.18**
	14-16	-	-	-	-	0.00	0.02	0.02	0.01	0.01	0.00	-	-	-	**0.06**
	16-18	-	-	-	-	-	0.00	0.00	0.00	0.00	0.00	-	-	-	**0.01**
	Total%	**0.00**	**0.00**	**0.01**	**0.14**	**2.06**	**6.12**	**5.12**	**3.46**	**0.83**	**0.44**	**0.16**	**0.04**	**0.00**	**18.39**
NNE	0-2	-	-	-	-	0.00	0.00	-	-	-	-	0.00	-	-	**0.01**
11.25 -	2-4	-	-	0.00	0.07	0.47	0.26	0.05	0.01	0.00	0.00	0.00	-	-	**0.86**
33.75	4-6	-	-	0.04	0.18	0.98	0.94	0.34	0.09	0.01	0.00	0.00	0.00	-	**2.58**
	6-8	-	-	0.01	0.21	0.56	0.66	0.26	0.07	0.01	-	0.00	-	-	**1.79**
	8-10	-	-	-	0.04	0.23	0.44	0.15	0.05	-	-	-	0.00	-	**0.92**
Hs (ft)	10-12	-	-	-	0.01	0.11	0.13	0.05	0.02	0.00	-	-	-	-	**0.32**
	12-14	-	-	-	-	0.02	0.07	0.03	0.01	-	0.00	-	-	-	**0.14**
	14-16	-	-	-	-	0.00	0.02	0.01	0.00	0.00	0.00	-	-	-	**0.04**
	16-18	-	-	-	-	-	0.00	0.00	0.00	-	0.00	-	-	-	**0.01**
	Total%	**0.00**	**0.00**	**0.05**	**0.51**	**2.39**	**2.52**	**0.89**	**0.25**	**0.02**	**0.01**	**0.01**	**0.00**	**0.00**	**6.66**
NE	0-2	-	-	-	-	0.00	-	-	-	-	-	-	-	-	**0.00**
33.75 -	2-4	-	-	0.03	0.26	0.65	0.17	0.01	0.00	-	-	-	-	-	**1.13**
56.25	4-6	-	-	0.18	1.48	2.46	0.62	0.06	0.02	0.00	-	-	-	-	**4.82**
	6-8	-	-	0.04	1.03	1.84	0.61	0.09	0.03	0.00	0.00	-	-	-	**3.64**
	8-10	-	-	0.00	0.27	0.85	0.38	0.06	0.02	-	-	-	-	-	**1.57**
Hs (ft)	10-12	-	-	-	0.01	0.33	0.14	0.01	0.00	-	-	-	-	-	**0.50**
	12-14	-	-	-	0.00	0.07	0.06	0.02	0.02	-	-	-	-	-	**0.17**
	14-16	-	-	-	-	0.00	0.01	0.00	0.02	0.00	-	-	-	-	**0.04**
	16-18	-	-	-	-	-	0.00	-	0.01	0.01	0.00	-	-	-	**0.02**
	18-20	-	-	-	-	-	-	-	0.00	0.01	0.01	-	-	-	**0.02**
	Total%	**0.00**	**0.00**	**0.25**	**3.06**	**6.19**	**1.99**	**0.26**	**0.12**	**0.03**	**0.01**	**0.00**	**0.00**	**0.00**	**11.91**
ENE	0-2	-	-	-	0.00	0.00	-	-	-	-	-	-	-	-	**0.00**
56.25	2-4	-	-	0.12	0.72	0.73	0.08	0.01	0.00	-	-	-	-	-	**1.67**
78.75	4-6	-	-	1.80	5.41	4.70	0.34	0.02	0.01	0.01	-	-	-	-	**12.29**
	6-8	-	-	0.41	5.59	6.30	0.48	0.02	0.00	0.00	-	-	-	-	**12.80**
	8-10	-	-	0.01	1.07	2.78	0.47	0.01	-	-	-	-	-	-	**4.34**
Hs (ft)	10-12	-	-	-	0.06	0.67	0.34	0.02	0.00	-	-	-	-	-	**1.10**
	12-14	-	-	-	0.00	0.06	0.10	0.02	-	-	-	-	-	-	**0.19**
	14-16	-	-	-	-	0.01	0.04	0.02	-	-	-	-	-	-	**0.06**
	16-18	-	-	-	-	0.00	0.02	0.00	-	-	-	-	-	-	**0.03**
	Total%	**0.00**	**0.00**	**2.34**	**12.85**	**15.26**	**1.90**	**0.13**	**0.01**	**0.01**	**0.00**	**0.00**	**0.00**	**0.00**	**32.50**
E	0-2	-	-	-	0.00	-	-	-	-	-	-	-	-	-	**0.00**
78.75 -	2-4	-	0.00	0.35	0.66	0.76	0.06	0.09	0.04	0.00	-	-	-	-	**1.96**
101.25	4-6	-	-	2.14	3.07	4.14	0.40	0.14	0.08	0.01	-	-	-	-	**9.98**
	6-8	-	-	0.41	3.49	4.62	0.46	0.03	0.07	0.00	-	-	-	-	**9.08**
	8-10	-	-	0.00	0.53	1.40	0.36	0.01	-	-	-	-	-	-	**2.29**
Hs (ft)	10-12	-	-	-	0.03	0.20	0.18	0.00	-	-	-	-	-	-	**0.41**
	12-14	-	-	-	0.00	0.00	0.02	0.01	-	-	-	-	-	-	**0.04**
	14-16	-	-	-	-	-	0.01	0.01	-	-	-	-	-	-	**0.01**
	Total%	**0.00**	**0.00**	**2.90**	**7.78**	**11.12**	**1.49**	**0.29**	**0.19**	**0.02**	**0.00**	**0.00**	**0.00**	**0.00**	**23.78**
ESE	0-2	-	-	0.00	-	-	0.00	-	-	-	-	-	-	-	**0.00**
101.25 -	2-4	-	0.00	0.05	0.08	0.07	0.02	0.08	0.06	-	-	-	-	-	**0.37**
123.75	4-6	-	-	0.07	0.12	0.16	0.02	0.01	0.02	-	0.00	-	-	-	**0.39**
Hs (ft)	6-8	-	-	0.00	0.03	0.06	0.01	0.00	-	0.00	0.00	-	-	-	**0.11**
	8-10	-	-	-	0.00	0.02	0.00	-	-	-	0.00	-	-	-	**0.02**
	Total%	**0.00**	**0.00**	**0.12**	**0.24**	**0.30**	**0.05**	**0.10**	**0.08**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	0.89
SE	0-2	-	-	-	-	-	0.00	0.00	-	-	-	-	-	-	**0.00**
123.75 -	2-4	-	-	0.00	0.02	0.01	0.02	0.12	0.26	0.02	0.01	0.00	-	-	**0.46**
146.25	4-6	-	-	0.01	0.05	0.02	0.00	0.02	0.13	0.05	0.02	0.00	-	-	**0.30**
Hs (ft)	6-8	-	-	-	0.01	0.01	-	-	0.01	0.03	0.01	-	-	-	**0.07**
	8-10	-	-	-	-	0.00	0.00	-	-	-	-	-	-	-	**0.00**
	Total%	**0.00**	**0.00**	**0.02**	**0.07**	**0.04**	**0.03**	**0.14**	**0.40**	**0.11**	**0.04**	**0.01**	**0.00**	**0.00**	**0.84**
SSE	0-2	-	-	-	-	-	-	-	-	-	-	-	-	-	**0.00**
146.25 -	2-4	-	-	-	-	-	0.00	0.02	0.06	0.01	-	-	-	-	**0.09**
168.75	4-6	-	-	0.01	0.00	-	-	0.00	0.04	0.00	-	-	-	-	**0.05**
Hs (ft)	6-8	-	-	-	0.00	0.00	-	-	-	0.00	-	-	-	-	**0.00**
	Total%	**0.00**	**0.00**	**0.01**	**0.01**	**0.00**	**0.00**	**0.03**	**0.10**	**0.01**	**0.00**	**0.00**	**0.00**	**0.00**	**0.15**
														Total	**100.00**

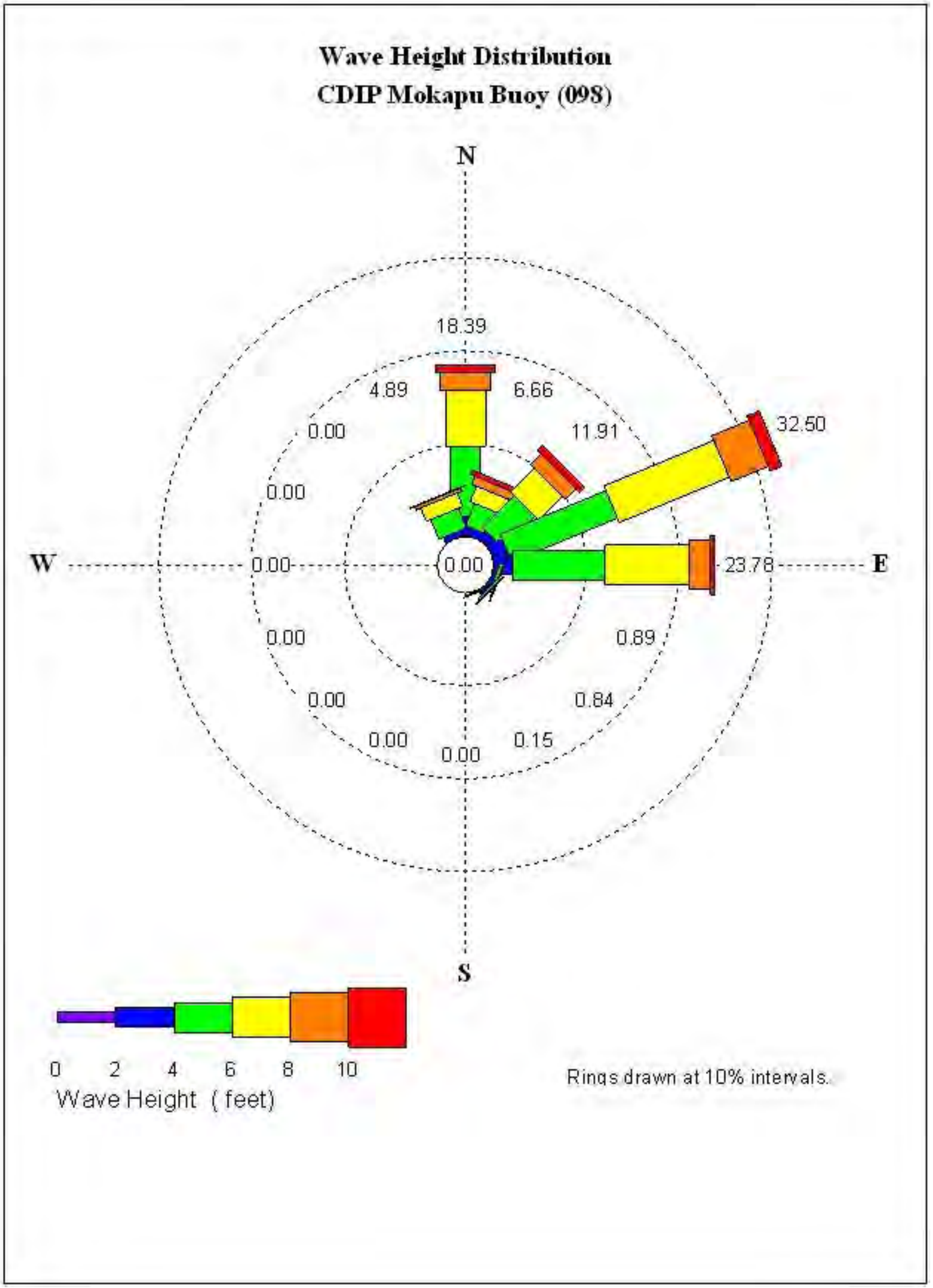


Figure 3-2 Wave height rose for CDIP Buoy 098, Mokapu

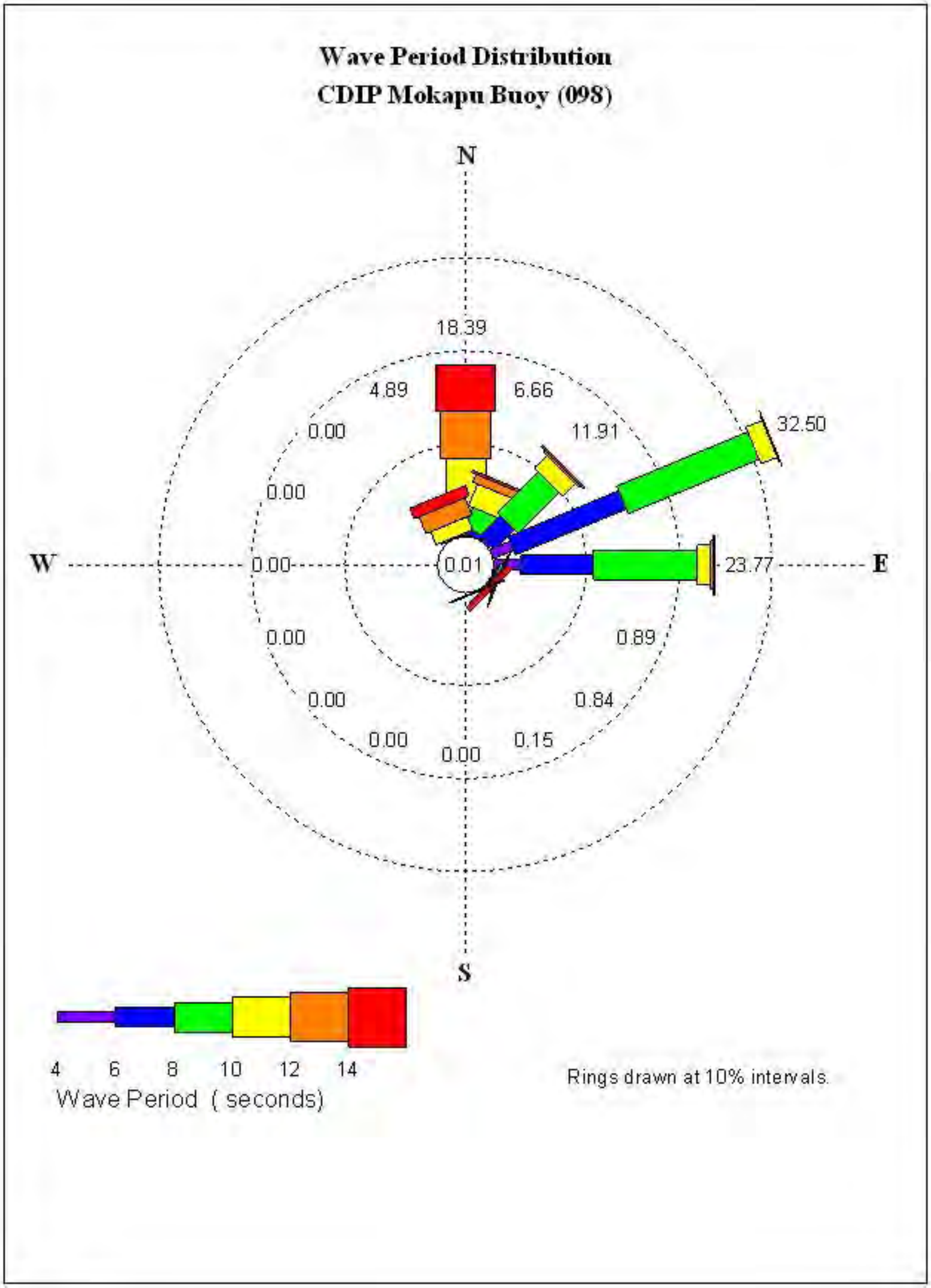


Figure 3-3 Wave period rose for CDIP Buoy 098, Mokapu

Table 3-2 WIS Station 99, hindcast deepwater waves, 1981-2004. Percent frequency of occurrence: significant wave height H_s (feet) vs. peak period T_p (seconds)

WIS 099

		Period (s)												
Dir (°TN)	**Hs\Ts**	**<4**	**4-6**	**6-8**	**8-10**	**10-12**	**12-14**	**14-16**	**16-18**	**18-20**	**20-22**	**22-24**	**>24**	**Total%**
NNW	<4	-	-	-	0.0	0.0	0.0	-	0.0	0.0	-	-	-	**0.1**
326.25 -	4-6	-	-	-	0.9	0.6	0.1	0.0	0.0	0.0	-	-	-	**1.7**
348.75	6-8	-	-	-	0.5	2.5	0.8	0.2	0.1	0.0	0.0	-	-	**4.1**
	8-10	-	-	-	0.1	1.6	1.4	0.3	0.1	0.0	0.0	-	0.0	**3.6**
Hs (ft)	10-12	-	-	0.0	0.0	0.6	1.1	0.4	0.1	0.0	0.0	0.0	0.0	**2.2**
	12-14	-	-	-	0.0	0.2	0.6	0.3	0.1	0.0	0.0	0.0	0.0	**1.3**
	14-16	-	-	-	-	0.0	0.3	0.3	0.1	0.0	0.0	0.0	0.0	**0.7**
	16-18	-	-	-	-	0.0	0.1	0.2	0.1	0.0	-	-	-	**0.3**
	>18	-	-	-	-	0.0	0.0	0.0	0.1	0.0	-	-	-	**0.2**
	Total%	**0.0**	**0.0**	**0.0**	**1.5**	**5.5**	**4.5**	**1.8**	**0.7**	**0.2**	**0.0**	**0.0**	**0.0**	**14.1**
N	<4	-	-	0.0	0.1	0.0	0.0	-	-	-	-	-	-	**0.1**
-11.25 -	4-6	-	-	0.1	1.0	0.4	0.1	0.0	-	-	-	-	-	**1.5**
+11.25	6-8	-	-	0.0	0.6	1.0	0.2	0.0	0.0	-	-	-	-	**1.9**
	8-10	-	0.0	0.0	0.2	0.8	0.3	0.1	0.0	-	-	-	-	**1.4**
Hs (ft)	10-12	-	-	0.0	0.1	0.5	0.3	0.1	0.0	-	-	-	-	**1.0**
	12-14	-	-	0.0	0.0	0.2	0.3	0.1	0.0	-	-	-	-	**0.6**
	14-16	-	-	-	0.0	0.1	0.1	0.1	0.0	-	-	-	-	**0.3**
	>16	-	-	-	0.0	0.0	0.0	0.0	0.0	-	-	-	-	**0.1**
	Total%	**0.0**	**0.0**	**0.1**	**2.0**	**3.0**	**1.3**	**0.4**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**7.0**
NNE	<4	-	-	0.1	0.1	0.0	-	-	-	-	-	-	-	**0.1**
11.25 -	4-6	-	-	0.2	0.9	0.2	0.0	0.0	0.0	-	-	-	-	**1.2**
33.75	6-8	-	-	0.1	0.8	0.4	0.1	0.0	0.0	-	-	-	-	**1.5**
	8-10	-	-	0.0	0.3	0.4	0.1	0.0	0.0	-	-	-	-	**0.9**
Hs (ft)	10-12	-	-	0.0	0.1	0.3	0.1	0.0	-	-	-	-	-	**0.5**
	12-14	-	-	0.0	0.1	0.2	0.1	0.0	-	-	-	-	-	**0.3**
	>14	-	-	0.0	0.0	0.1	0.0	0.0	-	-	-	-	-	**0.1**
	Total%	**0.0**	**0.0**	**0.4**	**2.3**	**1.5**	**0.4**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**4.7**
NE	<4	0.0	0.0	0.1	0.1	0.0	-	-	-	-	-	-	-	**0.2**
33.75 -	4-6	-	0.0	0.9	1.1	0.1	0.0	0.0	-	-	-	-	-	**2.2**
56.25	6-8	-	0.0	1.1	1.5	0.3	0.0	0.0	-	-	-	-	-	**2.9**
	8-10	-	-	0.3	0.7	0.3	0.0	0.0	-	-	-	-	-	**1.3**
Hs (ft)	10-12	-	-	0.1	0.3	0.2	0.0	0.0	-	-	-	-	-	**0.6**
	12-14	-	-	0.0	0.1	0.1	0.0	0.0	-	-	-	-	-	**0.3**
	14-16	-	-	0.0	0.1	0.1	0.0	0.0	-	-	-	-	-	**0.2**
	>16	-	-	-	0.0	0.1	-	-	-	-	-	-	-	**0.1**
	Total%	**0.0**	**0.0**	**2.5**	**3.9**	**1.1**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**7.7**
ENE	<4	0.0	0.0	0.2	0.1	0.0	0.0	-	-	-	-	-	-	**0.3**
56.25	4-6	-	0.2	2.9	1.2	0.1	0.0	0.0	-	-	-	-	-	**4.5**
78.75	6-8	-	0.1	5.0	2.8	0.2	0.0	0.0	-	-	-	-	-	**8.1**
	8-10	-	-	1.3	2.3	0.2	0.0	-	-	-	-	-	-	**3.8**
Hs (ft)	10-12	-	-	0.2	0.9	0.2	0.0	-	-	-	-	-	-	**1.2**
	12-14	-	-	0.0	0.4	0.2	0.0	-	-	-	-	-	-	**0.6**
	14-16	-	-	0.0	0.1	0.2	0.0	0.0	-	-	-	-	-	**0.4**
	>16	-	-	-	0.0	0.1	0.0	-	-	-	-	-	-	**0.2**
	Total%	**0.0**	**0.3**	**9.7**	**7.8**	**1.3**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**19.1**
E	<4	-	-	0.1	0.1	0.0	0.0	0.0	-	-	-	-	-	**0.3**
78.75 -	4-6	-	0.0	1.1	1.3	0.3	0.2	0.0	0.0	-	-	-	-	**2.9**
101.25	6-8	-	0.0	1.4	2.0	0.3	0.3	0.1	0.0	-	-	-	-	**4.0**
	8-10	-	-	0.2	1.0	0.3	0.0	0.0	-	-	-	-	-	**1.5**
Hs (ft)	10-12	-	-	0.0	0.4	0.2	0.0	-	-	-	-	-	-	**0.7**
	12-14	-	-	-	0.1	0.2	0.0	-	-	-	-	-	-	**0.2**
	>14	-	-	-	0.0	0.1	0.0	0.0	-	-	-	-	-	**0.1**
	Total%	**0.0**	**0.0**	**2.8**	**4.9**	**1.3**	**0.6**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**9.8**
ESE	<4	-	-	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	**0.0**
101.25 -	4-6	-	-	0.0	0.2	0.1	0.1	0.0	0.0	-	-	-	-	**0.3**
123.75	6-8	-	-	-	0.0	0.1	0.1	0.1	0.0	-	-	-	-	**0.3**
	8-10	-	-	-	0.0	0.0	0.1	0.0	-	-	-	-	-	**0.1**
Hs (ft)	>10	-	-	-	0.0	0.0	0.0	0.0	-	-	-	-	-	**0.0**
	Total%	**0.0**	**0.0**	**0.0**	**0.2**	**0.2**	**0.3**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.8**
SE	0-2	-	-	-	-	-	-	-	-	-	-	-	-	**0.0**
123.75 -	2-4	-	-	-	0.0	0.0	0.0	0.0	-	-	-	-	-	**0.0**
146.25	4-6	-	-	0.0	0.0	0.0	0.1	0.0	-	-	-	-	-	**0.1**
Hs (ft)	>6	-	-	-	-	0.0	0.0	0.0	-	-	-	-	-	**0.1**
	Total%	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.2**
SSE	0-2	-	-	-	-	-	-	-	-	-	-	-	-	**0.0**
146.25 -	2-4	-	-	-	0.0	0.0	0.0	0.0	-	-	-	-	-	**0.0**
168.75	4-6	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	**0.1**
Hs (ft)	>6	-	-	-	0.0	0.0	0.0	0.0	0.0	-	-	-	-	**0.1**
	Total%	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.2**
S	0-2	-	-	-	-	-	-	-	-	-	-	-	-	0.0
168.75 -	2-4	-	-	-	0.0	0.0	0.0	0.0	0.0	-	-	-	-	0.1
191.25	4-6	-	-	-	0.0	0.1	0.3	0.2	0.1	0.0	0.0	-	-	0.7
Hs (ft)	>6	-	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.0	0.0	-	-	0.3
	Total%	**0.0**	**0.0**	**0.0**	**0.0**	**0.1**	**0.4**	**0.3**	**0.2**	**0.0**	**0.0**	**0.0**	**0.0**	**1.1**
SSW	0-2	-	-	-	-	-	-	-	-	-	-	-	-	0.0
191.25 -	2-4	-	-	-	-	0.0	0.0	0.0	0.0	-	-	-	-	0.1
213.75	4-6	-	-	-	0.0	0.0	0.3	0.2	0.1	0.0	-	-	-	0.7

Table 3-2 (continued)

Hs (ft)	10-12			0.2	0.9	0.2	0.0							**1.2**
	12-14	-	-	0.0	0.4	0.2	0.0	-	-	-	-	-	-	**0.6**
	14-16	-	-	0.0	0.1	0.2	0.0	0.0	-	-	-	-	-	**0.4**
	>16	-	-	-	0.0	0.1	0.0	-	-	-	-	-	-	**0.2**
	Total%	**0.0**	**0.3**	**9.7**	**7.8**	**1.3**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**19.1**
E	<4	-	-	0.1	0.1	0.0	0.0	0.0	-	-	-	-	-	**0.3**
78.75 -	4-6	-	0.0	1.1	1.3	0.3	0.2	0.0	0.0	-	-	-	-	**2.9**
101.25	6-8	-	0.0	1.4	2.0	0.3	0.3	0.1	0.0	-	-	-	-	**4.0**
	8-10	-	-	0.2	1.0	0.3	0.0	0.0	-	-	-	-	-	**1.5**
Hs (ft)	10-12	-	-	0.0	0.4	0.2	0.0	-	-	-	-	-	-	**0.7**
	12-14	-	-	-	0.1	0.2	0.0	-	-	-	-	-	-	**0.2**
	>14	-	-	-	0.0	0.1	0.0	0.0	-	-	-	-	-	**0.1**
	Total%	**0.0**	**0.0**	**2.8**	**4.9**	**1.3**	**0.6**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**9.8**
ESE	<4	-	-	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	**0.0**
101.25 -	4-6	-	-	0.0	0.2	0.1	0.1	0.0	0.0	-	-	-	-	**0.3**
123.75	6-8	-	-	-	0.0	0.1	0.1	0.1	0.0	-	-	-	-	**0.3**
	8-10	-	-	-	0.0	0.0	0.1	0.0	-	-	-	-	-	**0.1**
Hs (ft)	>10	-	-	-	0.0	0.0	0.0	0.0	-	-	-	-	-	**0.0**
	Total%	**0.0**	**0.0**	**0.0**	**0.2**	**0.2**	**0.3**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.8**
SE	0-2	-	-	-	-	-	-	-	-	-	-	-	-	**0.0**
123.75 -	2-4	-	-	-	0.0	0.0	0.0	0.0	-	-	-	-	-	**0.0**
146.25	4-6	-	-	0.0	0.0	0.0	0.1	0.0	-	-	-	-	-	**0.1**
Hs (ft)	>6	-	-	-	-	0.0	0.0	0.0	-	-	-	-	-	**0.1**
	Total%	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.2**
SSE	0-2	-	-	-	-	-	-	-	-	-	-	-	-	**0.0**
146.25 -	2-4	-	-	-	0.0	0.0	0.0	0.0	-	-	-	-	-	**0.0**
168.75	4-6	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	**0.1**
Hs (ft)	>6	-	-	-	0.0	0.0	0.0	0.0	0.0	-	-	-	-	**0.1**
	Total%	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.2**
S	0-2	-	-	-	-	-	-	-	-	-	-	-	-	0.0
168.75 -	2-4	-	-	-	0.0	0.0	0.0	0.0	0.0	-	-	-	-	0.1
191.25	4-6	-	-	-	0.0	0.1	0.3	0.2	0.1	0.0	0.0	-	-	0.7
Hs (ft)	>6	-	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.0	0.0	-	-	0.3
	Total%	**0.0**	**0.0**	**0.0**	**0.0**	**0.1**	**0.4**	**0.3**	**0.2**	**0.0**	**0.0**	**0.0**	**0.0**	**1.1**
SSW	0-2	-	-	-	-	-	-	-	-	-	-	-	-	0.0
191.25 -	2-4	-	-	-	-	0.0	0.0	0.0	0.0	-	-	-	-	0.1
213.75	4-6	-	-	-	0.0	0.0	0.3	0.2	0.1	0.0	-	-	-	0.7
Hs (ft)	>6	-	0.0	-	-	0.0	0.0	0.1	0.1	0.0	0.0	-	-	0.3
	Total%	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.4**	**0.4**	**0.3**	**0.0**	**0.0**	**0.0**	**0.0**	**1.1**
SW	<4	-	-	-	-	-	0.0	0.0	-	-	-	-	-	0.0
213.75 -	4-6	-	-	-	0.0	0.0	0.0	0.0	0.0	-	-	-	-	0.1
236.75	6-8	-	-	0.0	-	0.0	0.0	0.0	0.0	-	-	-	-	0.0
Hs (ft)	>8	-	-	0.0	-	-	-	0.0	0.0	-	-	-	-	0.0
	Total%	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.1**
WSW	<4	-	-	-	-	0.0	-	-	-	-	-	-	-	0.0
236.75 -	4-6	-	-	-	0.0	0.0	0.0	0.0	-	-	-	-	-	0.1
258.75	6-8	-	-	-	-	0.0	0.0	0.0	0.0	-	-	-	-	0.0
Hs (ft)	>8	-	-	0.0	0.0	0.0	-	0.0	0.0	-	-	-	-	0.0
	Total%	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.1**
W	<4	-	-	-	-	0.0	0.0	0.0	-	-	-	-	-	0.0
258.75 -	4-6	-	-	-	-	0.0	0.1	0.0	0.0	-	-	-	-	0.2
281.25	6-8	-	0.0	-	-	0.0	0.1	0.0	0.0	-	-	-	-	0.1
Hs (ft)	>8	-	-	0.0	0.0	-	0.0	0.0	0.0	-	-	-	-	0.0
	Total%	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.1**	**0.1**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.3**
WNW	<4	-	-	-	0.0	0.0	0.0	0.0	0.0	-	-	-	-	0.1
281.25 -	4-6	-	-	-	0.0	0.2	0.2	0.1	0.0	0.0	-	-	-	0.5
303.75	6-8	-	0.0	0.0	0.0	0.2	0.4	0.1	0.0	0.0	0.0	0.0	0.0	0.7
	8-10	-	-	0.0	0.0	0.0	0.6	0.2	0.0	0.0	0.0	-	-	0.8
Hs (ft)	10-12	-	-	-	0.0	0.0	0.2	0.3	0.1	0.0	0.0	-	-	0.6
	12-14	-	-	-	-	0.0	0.1	0.2	0.1	0.0	0.0	-	-	0.4
	14-16	-	-	-	-	0.0	0.0	0.1	0.1	0.0	-	-	-	0.2
	>16	-	-	-	-	-	0.0	0.0	0.0	0.0	-	-	-	0.1
	Total%	**0.0**	**0.0**	**0.0**	**0.1**	**0.5**	**1.6**	**0.9**	**0.3**	**0.1**	**0.0**	**0.0**	**0.0**	**3.4**
NW	<4	-	-	-	0.0	0.0	0.0	0.0	0.0	-	-	-	-	0.1
303.75 -	4-6	-	-	-	0.4	1.3	0.4	0.1	0.0	0.0	0.0	-	-	2.3
326.25	6-8	-	-	-	0.2	2.7	2.2	0.5	0.2	0.0	0.0	0.0	0.0	5.8
	8-10	-	-	0.0	0.0	1.6	4.4	1.1	0.4	0.1	0.0	0.0	0.0	7.7
Hs (ft)	10-12	-	-	-	0.0	0.4	3.4	1.6	0.5	0.2	0.0	0.0	0.0	6.2
	12-14	-	-	-	-	0.1	1.6	1.5	0.4	0.1	0.0	0.0	0.0	3.8
	14-16	-	-	-	0.0	0.0	0.5	1.0	0.4	0.1	0.0	0.0	-	2.1
	16-18	-	-	-	-	0.0	0.2	0.5	0.4	0.1	0.0	0.0	-	1.1
	18-20	-	-	-	-	0.0	0.1	0.2	0.2	0.1	0.0	-	0.0	0.5
	>20	-	-	-	-	-	0.0	0.1	0.1	0.0	0.0	0.0	-	0.3
	Total%	**0.0**	**0.0**	**0.0**	**0.6**	**6.2**	**12.7**	**6.7**	**2.7**	**0.7**	**0.1**	**0.0**	**0.0**	**29.9**
													Total	**100.0%**

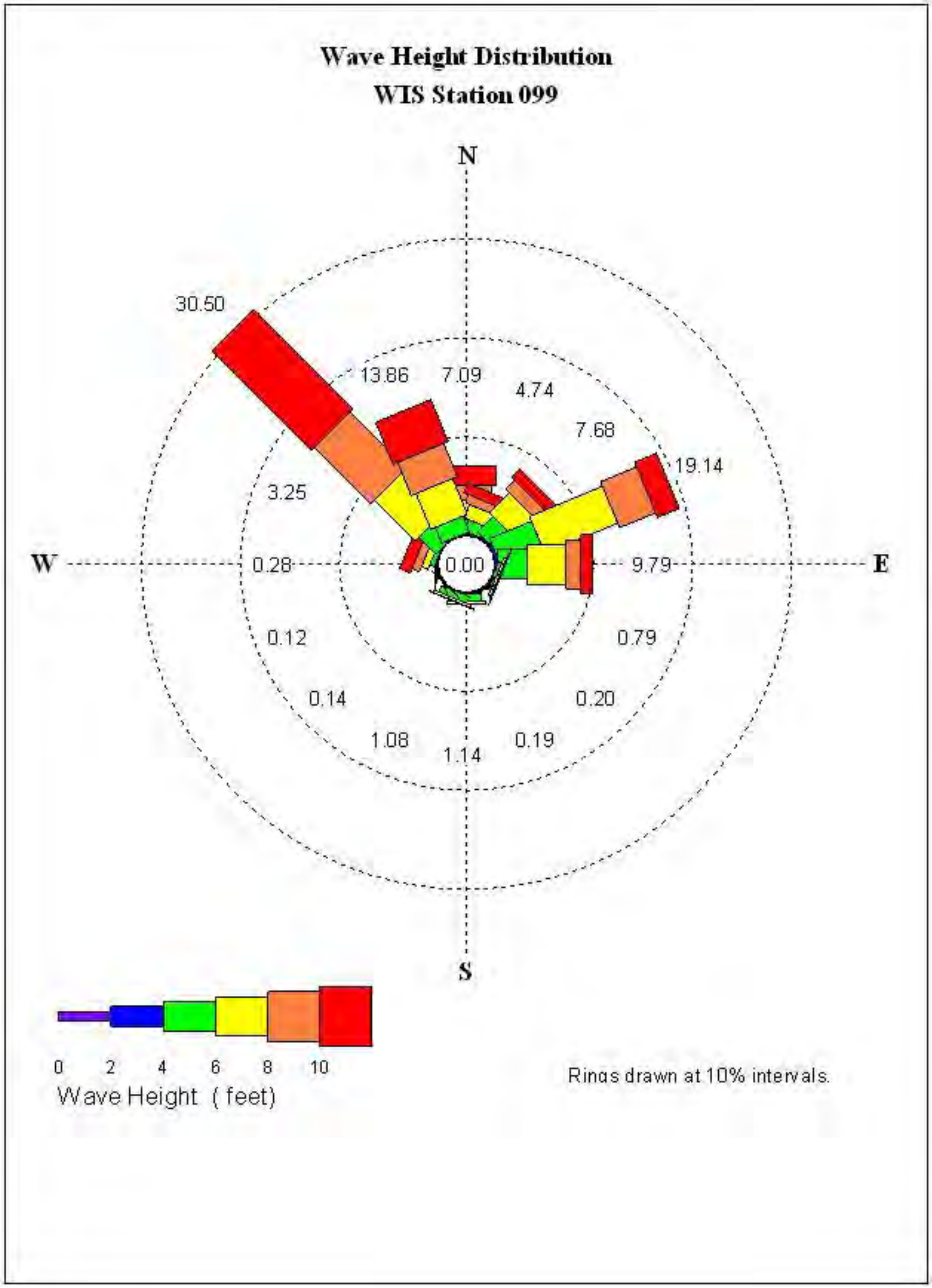


Figure 3-4 Wave height rose for WIS Station 99

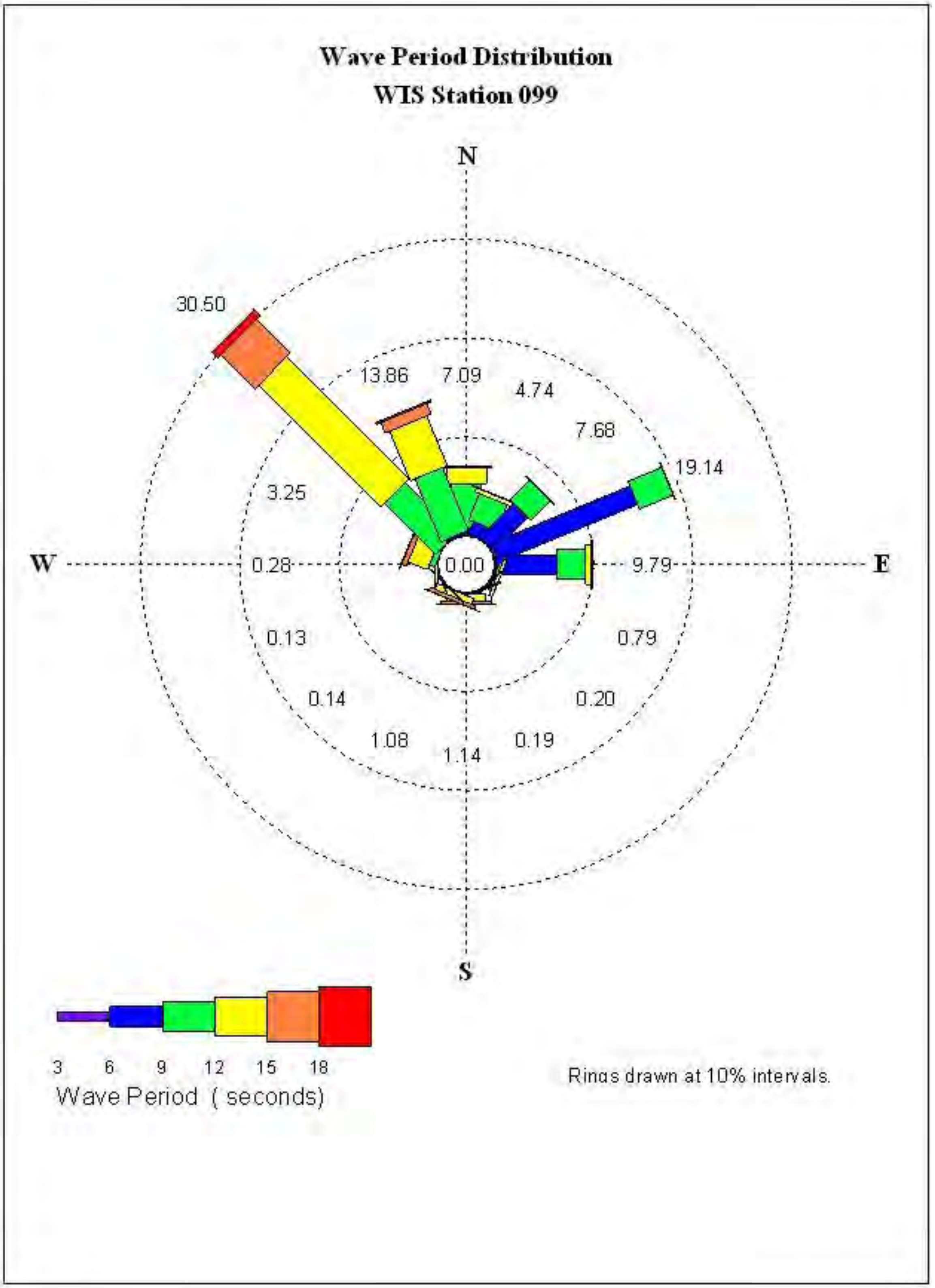


Figure 3-5 Wave period rose for WIS Station 99

3.2.3 Extreme Deepwater Wave Height

The Hawaiian Islands are annually exposed to severe storms and storm waves generated by passing low pressure systems, tropical storms including hurricanes, and large swell waves generated by distant north or south Pacific storms. Storms and high wave events considered here include:

- One-year return period wave
- Fifty-year return period wave
- Close approach hurricane generated waves

The WIS hindcast wave data set presented previously can be further analyzed using a Gumbel distribution of extreme events to obtain design wave heights and return periods. While north swell energy refracts and impacts the project site, a significant portion of this energy is lost during wave transformation, resulting in lower energy waves at the project site versus the offshore WIS station. The data set was therefore filtered for waves whose approach direction was between northeast and southeast; these are the wave directions considered to have the most effect on the project site. The highest annual waves from the filtered data were obtained and these 24 waves ranged from 13.5 feet to 23.5 feet; the wave periods corresponding to these waves ranged from 8.6 seconds to 12.8 seconds. The design wave heights and return periods based on the Gumbel analysis are shown in Table 3-3. For comparison, Vitousek and Fletcher (2008) determined the maximum annually recurring wave height in the direction range of 60° to 90° TN to be 17.6 feet, which is consistent with the annual wave presented in Table 3-3.

Table 3-3 Wave heights vs. return periods

Return Period (years)	Wave Height (feet)
1	17.8
5	21.0
10	22.4
25	24.2
50	25.5
75	26.3
100	26.9

Within the 24 years of data, the five largest annual waves have periods of 10.9 to 12.3 seconds; thus, the wave period of the 50-year wave is taken to be the average of these five wave periods, or 11.8 seconds. The average direction of these waves is approximately east-northeast, which is also the prevailing wave direction for the complete data set.

The design wave conditions selected for further analysis are summarized in Table 3-4.

Table 3-4 Selected design wave conditions

Type of Wave	Deepwater Wave Height (feet)	Breaking Wave Height (feet)	Wave Period (sec.)
Prevailing Wave	6.0	8.3	8.0
1-Year Wave	17.8	22.7	11.8
50-Year Wave	25.5	29.5	11.8

3.3 Nearshore Water Levels

3.3.1 Wave Transformation in Shallow Water

As deepwater waves approach the shoreline, they begin to transform due to the effects of shoaling, bottom friction, refraction, and diffraction. As waves shoal, heights increase and the wave crests steepen, to the point that the waves become unstable, leading to breaking and dissipation of wave energy. Wave energy can also be attenuated due to bottom friction. The approach direction can change as the wave front refracts, or becomes oriented parallel to the existing bathymetric contours. Lateral spreading of energy, known as diffraction, can occur behind a natural or man-made barrier.

The breaking wave values given in Table 3-4 for the selected design wave conditions reflect the shoaling and refraction characteristics of these waves at the project site as determined though ACES using site bathymetry.

To better analyze the waves shoreward of the shallow fringing reef and the Mokulua islands, the wave model BOUSS2D was employed. BOUSS2D, a component of the Surface-water Modeling System suite of modeling products, is a shallow-water nonlinear wave model that also includes the processes of wave shoaling, refraction, diffraction, and breaking. BOUSS2D is a time domain model and has been shown to be particularly useful in modeling wave/structure interaction. The model has the capability of outputting an animation of water surface elevations, which is very effective in showing the wave propagation to shore.

BOUSS2D requires input of the nearshore bathymetry. Bathymetry is represented in the model through the creation of a grid and the specification of depth at each grid point. Selection of model grid spacing and domain width requires a tradeoff between desired resolution, spatial coverage, and computer computational capabilities. The 15,520 ft by 14,760 ft domain was oriented parallel to the selected wave approach and was centered to capture the primary wave dynamics affecting the project site. The width and position of the domain are believed to be sufficient to limit lateral boundary effects. Model grid spacing of 33 ft was sufficient to resolve the structure of the reef and show the wave propagation to shore.

3.3.2 Tide

Hawaii tides are semi-diurnal with pronounced diurnal inequalities (i.e., two high and low tides each 24-hour period with different elevations). Tidal predictions and historical extreme water levels are given by the Center for Operational Oceanographic Products and Services, NOS, NOAA, website. The nearest tide station is at Mokuoloe (Coconut) Island in Kaneohe Bay; correction factors can be applied to estimate the tides at Waimanalo Bay, which is adjacent to Lanikai Beach. The water level data corrected for Waimanalo, based on the 1983-2001 tidal epoch, is shown in Table 3-5.

Table 3-5 Water level data for Waimanalo

Mean Higher High Water	1.80 feet
Mean High Water	1.35 feet
Mean Tide Level	0.80 feet
Mean Low Water	0.25 feet
Mean Lower Low Water	0.00 feet

Hawaii is also subject to periodic extreme tide levels due to large scale oceanic eddies that propagate through the islands. These eddies produce tide levels up to 0.5 to 1 foot higher than normal for periods of up to several weeks.

3.3.3 Still Water Levels and Nearshore Wave Heights

During high wave conditions, the nearshore water level may be elevated above the tide level by the action of breaking waves. This water level rise, termed wave setup, could be as much as 1 to 2 feet during severe storm wave conditions. During hurricane conditions, an additional water level rise due to wind stress and reduced atmospheric pressure can occur. Collectively termed “storm surge,” this can potentially add another 1 to 2 feet to the stillwater level. For example, during the 1992 passage of Hurricane Iniki over Port Allen Harbor on the island of Kauai, a National Weather Service tide gauge recorded a water level rise of 4.9 feet above the predicted tide elevation.

During storm or large wave conditions, there may be multiple zones of wave breaking. Wave heights are said to be *depth-limited* because once the water depth becomes shallow enough the wave breaks, losing size and energy. The wave, however, may reform before it reaches the shoreline and break again when the depth-limited ratio is again attained. The still water level rise during storm events is an important design consideration because it allows larger wave heights to reach the shoreline than during lower water levels.

Estimation of still water level rise for the 50-year wave event may be accomplished by traditional methodology which uses bathymetry and wave heights as inputs. Still water level rise at the shoreline is a combination of astronomical tide, storm surge, and wave setup. The astronomical tide level chosen for design conditions is MHHW due to its frequency of occurrence. MHHW was presented earlier as 1.80 feet in adjacent Waimanalo Bay (Table 3-5).

Wave setup is a function of the breaking wave height, period, and bottom topography. The mass transport of water due to breaking waves produces wave setup—the increase in water depth shoreward of the breaker zone. The available analytical methods for calculating wave setup have been simplified and assume long, straight, parallel bathymetric contours, continuous breaking waves, and breaker zones relatively near shore; these methods are presented in the Shore Protection Manual (1984) and Coastal Engineering Manual (2006). Experience has shown that these methods tend to over-predict wave setup, because the natural environment has discontinuous breaking zones, irregular bathymetry, channels, and gaps in the reef that allow for a relief of wave setup.

Calculation of the design wave inshore of the reef, however, requires a more sophisticated analysis, due to the presence of the shallow fringing reef and the Mokulua islands. Results from three methods were compared to determine the appropriate design nearshore wave height. The first method, which is the most conservative, assumes a depth-limited condition controlled by the water depth approximately 300 feet from shore. This produces a breaking wave height of 6.6 feet, which is believed to be too large, as waves of this size would initially break on the shallower fringing reef, losing a substantial amount of energy.

The second method considers the largest waves that could propagate over the reef and between the islands. Residents have reported that the most damaging waves approach from between the Mokulua islands, where the controlling water depth is about 10 feet, which would produce a maximum wave height of about 7 feet between the islands. The wave diffracts toward shore, with a diffraction coefficient of about 0.5 determined from nomographs presented in the Shore Protection Manual, resulting in an inshore maximum wave height of 3.5 feet. Residents reported that the largest waves they have seen nearshore are approximately two to three feet high.

The third method uses BOUSS2D modeling of the 50-year event to determine the design wave heights. Time series' of water surface elevation were recorded at several locations within the model domain, including two sites approximately 300 feet offshore of the project site. The modeling showed a low-frequency (infra-gravity) oscillation that was believed to be an artifact of the model and therefore was removed from the record, leaving only the gravity wave signal. The significant wave height (H_s or $H_{1/3}$) of these waves was then calculated to be 2.6 feet and the more extreme $H_{1/10}$ was calculated to be 3.3 feet. Similar computations were made for the prevailing wave and the annual wave and are presented in Section 3.3.4.

The models were run at water levels to account for tides, wave setup, and eddies. Based on the modeling results and confirmed by observations, a design wave height of 3.3 feet is selected.

3.3.4 Design Still Water Level

The project site is exposed to waves from north-northwest through south-southeast as presented in Section 3.2.2. While all of these waves would lose some energy through refraction, a wave approaching with a deepwater direction from the east-northeast would experience the least refraction. For design purposes, the design wave is considered to approach from the east-northeast, which was found in Section 3.2.3 to be the average direction of the five largest annual waves from the WIS data set.

A summary of design parameters for the four wave conditions discussed in Section 3.2 is presented in Table 3-6.

Table 3-6 Design wave conditions

	Prevailing wave	1-year wave	50-year wave
Deepwater Wave Height H_o (ft) Breaking Wave Height H_b (ft)	6 8.3	17.8 22.7	25.5 29.5
Still Water Level Rise Astronomical tide (ft) Large-scale eddy (ft) Wave setup (ft)	 1.8 0.5 0.4	 1.8 0.5 0.9	 1.8 0.5 1.3
Total SWL rise (ft)	2.7	3.2	3.6
Nominal Water Depth (ft)	6.0	6.0	6.0
Design Water Depth (ft)	8.5	9.2	9.6
Design Wave Height *H* (design, ft)	1.3	2.2	3.3

3.3.5 Seafloor Investigations

Sea Engineering performed investigations of the seafloor conditions on February 23, 2009. The investigations included measurements of water depth and jet probe penetration into the bottom. The jet probe system consisted of a 2-inch pump, fire hose, and a 20-foot long, 1.25-inch diameter probe. The probe was calibrated in 0.1-foot intervals. Measurements were performed from a 25-foot Boston Whaler.

Probes and water depths were measured at 22 locations along five transects. Approximate groin orientation had previously been determined and the probe locations, shown in Figure 3-6, were selected accordingly. At each location, the field crew logged time, position, water depth, sediment thickness, and a description of the bottom. The crew also probed several times at each location to confirm the findings. The details of the probing are presented in Table 3-7. In the table, “hard refusal” refers to the probe penetration stopping upon encountering hard material such as reef rock; “soft refusal” refers to a probe penetration that ends without noticeably encountering solid material. The measured water depths were found to be consistent with the closest LiDAR data points.

Sand thickness was generally greatest near shore, where a low-elevation sand beach is found. The sand thickness generally decreased in the offshore direction, with no measureable sand thickness at any of the “a” points. The elevations of penetration are important for the structure foundation and these values are presented in Figure 3-7. As highlighted by that figure, the range of refusal varied from -4.7 to -6.8 feet MLLW and 14 of the 22 elevations are within 0.25 feet of -6 feet MLLW.

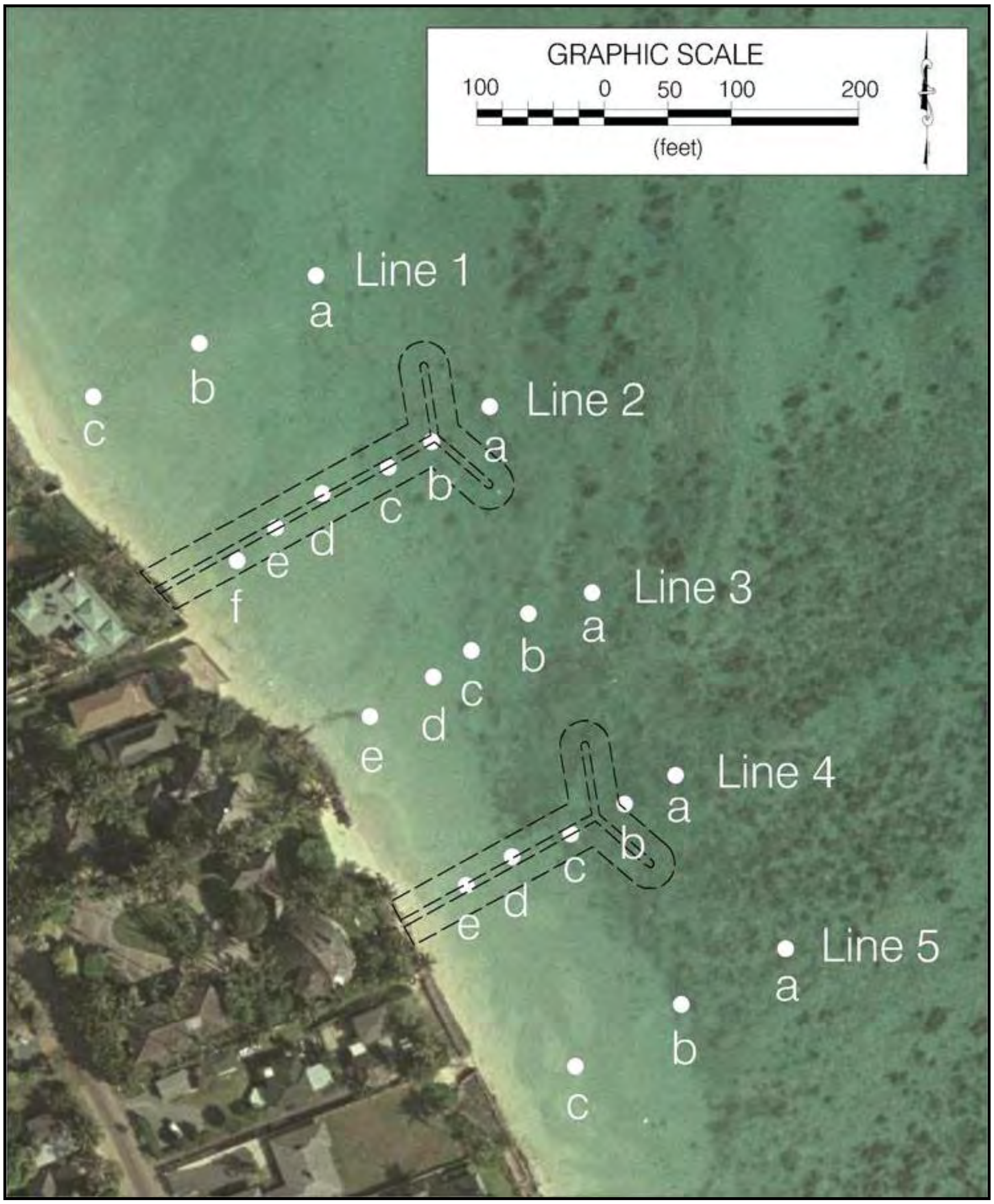


Figure 3-6 Seafloor investigation locations

Table 3-7 Jet probe details

Point	Water depth (feet MLLW)	Sand thickness (feet)	Description
1a	5.84	0	1 probe to 1.1 ft, 1 probe to 4.1, several to 0
1b	4.89	1.5	Hard refusal
1c	3.34	2.5	Hard refusal
2a	6.82	0	Hard refusal
2b	6.32	0	Hard refusal
2c	5.85	0	Hard refusal
2d	4.35	2	Coarse
2e	4.34	2	Coarse; hard refusal
2f	3.34	3.25	Fine to coarse; soft refusal
3a	5.48	0	Hard refusal
3b	6.18	0.5	Coarse, then hard refusal
3c	5.64	0	Hard refusal
3d	5.30	0.8	Hard refusal
3e	3.58	2	Hard refusal
4a	5.80	0	Hard refusal
4b	6.29	0	Hard refusal
4c	5.79	0	Hard refusal
4d	4.56	1.2	Hard refusal
4e	3.74	1	Hard refusal
5a	6.03	0	Hard refusal
5b	5.48	0	Hard refusal
5c	3.34	2.5	Fine to coarse, then soft refusal

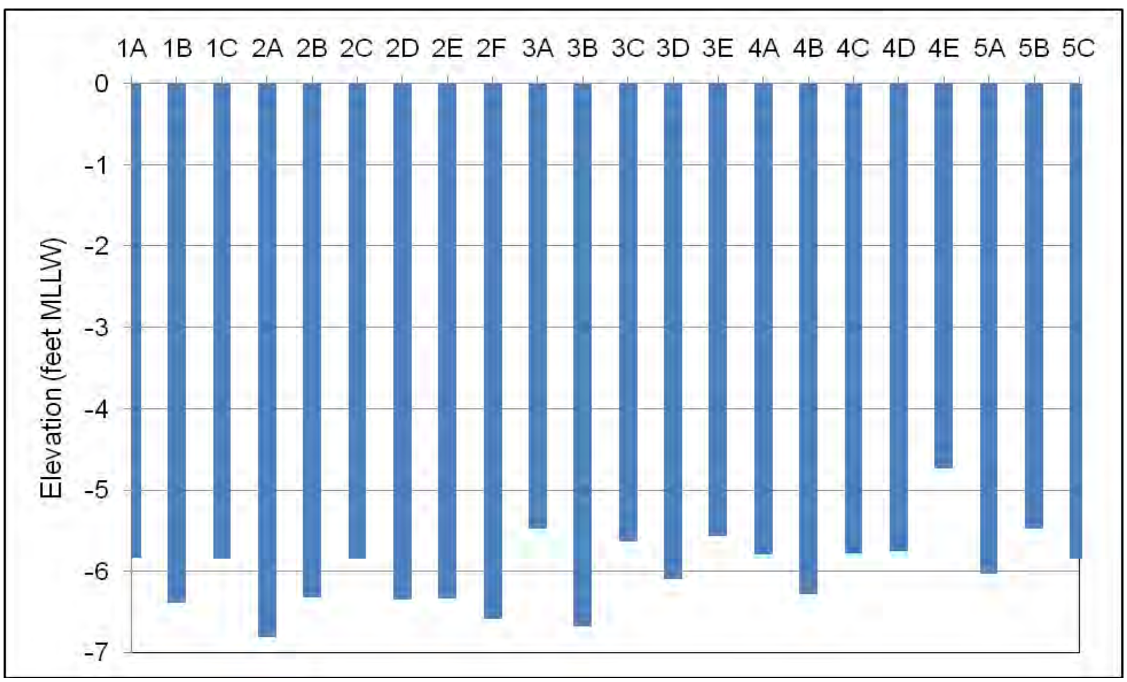


Figure 3-7 Elevation of probe refusal

4. STRUCTURE DESIGN ANALYSIS

The Lanikai Beach Restoration Pilot Project includes nourishing the beach with sand and constructing groins to provide stability for the beach. Objectives of this project include determining appropriate oceanographic design parameters and applying these design parameters to determine structure characteristics. Oceanographic design parameters were presented in Section 3. Key structure design parameters include height, slope, composition, stone size, and crest width.

Another objective of the project is for the groins to be temporary structures, constructed of material other than rock. These materials will be discussed in Section 5. In general, the design guidance for these alternative methods is limited and designs must be based on a small empirical database of existing installations. The guidance for designing rock rubblemound structures, however, is plentiful. To assist in the design of the groins, the dimensions and stone size of rock rubblemound groins will first be determined. This will assist us in selecting the proper design characteristics for the alternative structures.

In this section, the structures are designed as rock rubblemounds with side slopes of 1V:1.5H, which is the steepest slope recommended by the Coastal Engineering Manual (2006). Crest width is taken to be three stones.

4.1 Crest Elevation and Wave Runup

The elevation of the structures determines the amount of wave overtopping that will occur during prevailing wave conditions, which were presented in Section 3. While larger structures will reduce the overtopping, they present a larger footprint and are more costly. Additionally, from an aesthetics perspective, structures with lower crest elevations produce less visual impact.

The structures are designed to allow minimal overtopping during prevailing conditions. Runup elevation and overtopping rate were calculated using the Automated Coastal Engineering System (ACES) module in the Coastal Engineering Design and Analysis System (CEDAS) package, both of which were developed by the U.S. Army Corps of Engineers' Coastal & Hydraulic Laboratory (CHL).

Runup is a function of the wave height at the project site at the prevailing water level; the shallow reef limits the wave height that can impact the structures. Due to the complexity of the wave environment, the prevailing wave height at the structure location was determined using BOUSS2D, similar to the method described in Section 3.3.3. The significant wave height H_s for the prevailing wave was shown by the modeling to be 1.3 feet, for which ACES produces a runup elevation of +4.6 feet MLLW at high water level conditions.

Based on the calculated wave runup, a structure crest height of +4.5 feet is selected for design of the groins. This elevation allows minimal overtopping during prevailing conditions and is lower than the beach crest and vegetation line elevations for the adjacent beaches.

4.2 Stone Size

The stone size is calculated for comparative purposes and is based on extreme wave conditions discussed in Section 3.2. The modeling of the 50-year wave event showed H_s = 2.6 feet and $H_{1/10}$ = 3.3 feet. Selecting $H_{1/10}$ as the more conservative value, the required groin armor stone weight for stability under this wave height is given by the Hudson Formula (Coastal Engineering Manual, 2006):

$$W = \frac{w_r H^3}{K_D (S_r - 1)^3 \cot\theta}$$

where,

W = weight in pounds of an individual armor stone
w_r = unit weight of the stone, 160 lb/ft^3
H = wave height, 3.3 feet
K_D = armor stone stability coefficient, 2 for two layers randomly placed
S_r = specific gravity of the stone relative to seawater, use 2.5
cot θ = cotangent of the groin side slope, use 1.5

The resultant armor stone weight would be approximately 530 lbs with a corresponding nominal diameter of 1.5 feet. A range of +/- 25% of the median weight is typically utilized, which yields a stone weight range of 400 to 660 lbs.

The geotechnical conditions at the shoreline indicate a thin layer of sand above hard limestone. In this situation, use of an underlayer would not be necessary.

5. BEACH RESTORATION ALTERNATIVES/GROIN ALTERNATIVES

5.1 Introduction

Groins in Hawaii have generally been constructed of rock, with the intention of being permanent structures. The present pilot project is a demonstration of the effectiveness of tuned T-groins to produce a stable beach cell. As such, the client will have the option of removing the groins at the end of the demonstration or in the event that the groins do not perform as expected. The groins designed are therefore considered temporary and materials other than rock are considered necessary to produce groins capable of being removed without excessive impact on the environment. Existing temporary construction materials and methods have been evaluated for application to the Lanikai Beach Restoration Pilot Project. These are described in the following sections.

This report does not address the mechanism for removing the structure should the client desire to do such. The costs presented within each section are for the material and fill material (sand or rock).

5.2 Geotextile tubes

Description

Geotextile tubes are shore protection structures most commonly used as revetment-types of structures; however, they have also been used as groins, breakwaters, dikes, dune cores, and artificial reefs. As temporary containers, geotextile tubes are used for dewatering and storing dredge material. A photograph of a T-groin constructed from a geotextile tubes is shown in Figure 5-1.

MacTubes, manufactured by Maccaferri USA, are examples of geotextile tubes that have been used for shore protection. MacTubes are composed of high-tenacity polyester yarns, which are woven into a rip-stop stable network such that the yarns retain their relative position. The material is inert to biological degradation and resistant to naturally encountered chemicals, alkalis, and acids. Tubes exposed to sunlight can be coated with Polyurea, an elastomer coating frequently used in the marine environment. Standard tube lengths are 50, 100, 200, and 250 feet. Other manufacturers and suppliers of geotextile tubes include Bradley Industrial Geotextiles and Flint Industries.

A typical cross section of a groin constructed from a geotextile tube is shown in Figure 5-2. The structure consists of a main tube filled with sand. Anchor tubes with geotextile mats are typically attached to the tubes that form the base of the structure. As material adjacent to the tube is lost, the anchor tubes and mats settle to form a scour apron, thus providing addition scour protection to the structure.

The tubes are filled on site via pump with sand/water slurry (a new technology is being investigated that is reported to be able to blow wet or dry sand into a tube). Ports constructed into the material allow hose connection. The fine weave of the geotextile tubes allows the slurry to be dewatered by the bag itself, limiting the amount of fines released to the environment and reducing the size of the onshore staging area. The ends of the MacTubes are sealed with a proprietary technology, which the manufacturer claims to result in a 33% stronger seal than competitors' seals. In the event of a tear in the bag, the most common field repair method involves sewing the material closed on site. An additional method involves securing an HDPE patch to the structure with adhesive and bolts.

Figure 5-1 Oblique view of T-groin constructed of MacTubes (source: Maccaferri USA)

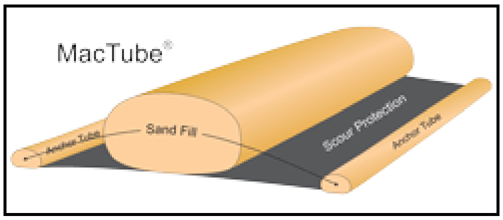


Figure 5-2 Schematic of MacTube (source: Maccaferri USA)

Examples of geotextile tubes in use

Geotextile tubes were used to form five T-groins on Upham Beach in Pinellas County, Florida. The tubes were used as part of a beach stabilization project on the downdrift side of a tidal inlet. The objective of the project was to extend the lifetime of nourishment projects, while still allowing sand to be transported to the neighboring beaches, albeit at a lesser rate. Each groin consists of six tubes in a pyramid stack as shown in Figure 5-3. The tubes were installed in 2005 at a cost of $1.5 Million and were shown by Wang and Roberts (2007) to be functioning as designed two years later.

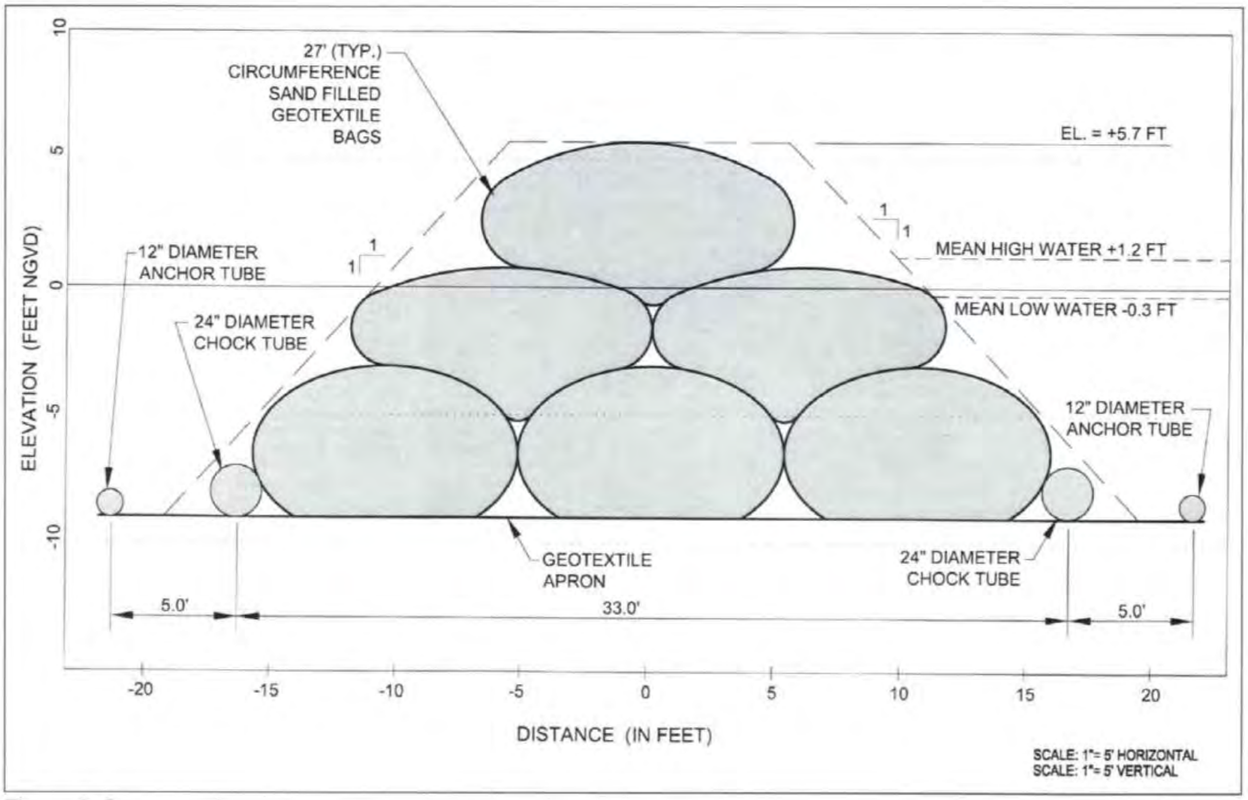


Figure 5-3 Cross-sectional view of typical groin used in Upham Beach construction; cross-sectional area = 52 sq. ft. each (source: Elko and Mann, 2007)

Installation

The installation of the geotextile tubes in Florida presented a challenge to the contractor. Construction of the first groin was attempted in open water conditions following the installation of temporary cofferdams offshore to reduce wave exposure. This installation took more than four months, longer than the initial schedule for the entire project, primarily due to waves overtopping the cofferdam and continuously carrying sand into the area. In order to construct the remaining groins, the beach was first nourished. The groins were then constructed in excavation pits in the sand. The pits, however, extended below sea level and would fill with seawater, requiring divers to fill the bags in low or no visibility. The construction process is shown in Figure 5-4.

Typical installation requires an excavator, diesel generator, sand pump, small boat, and several hundred feet of hose. The sand is required to be in slurry form; this allows direct filling from an offshore sand source, if available. Filling directly from an offshore source could therefore be accomplished with a minimum 1,900 ft^2 onshore staging area. If the tubes are filled from land, water would have to be added to the sand to produce slurry and a much larger staging area would be required.

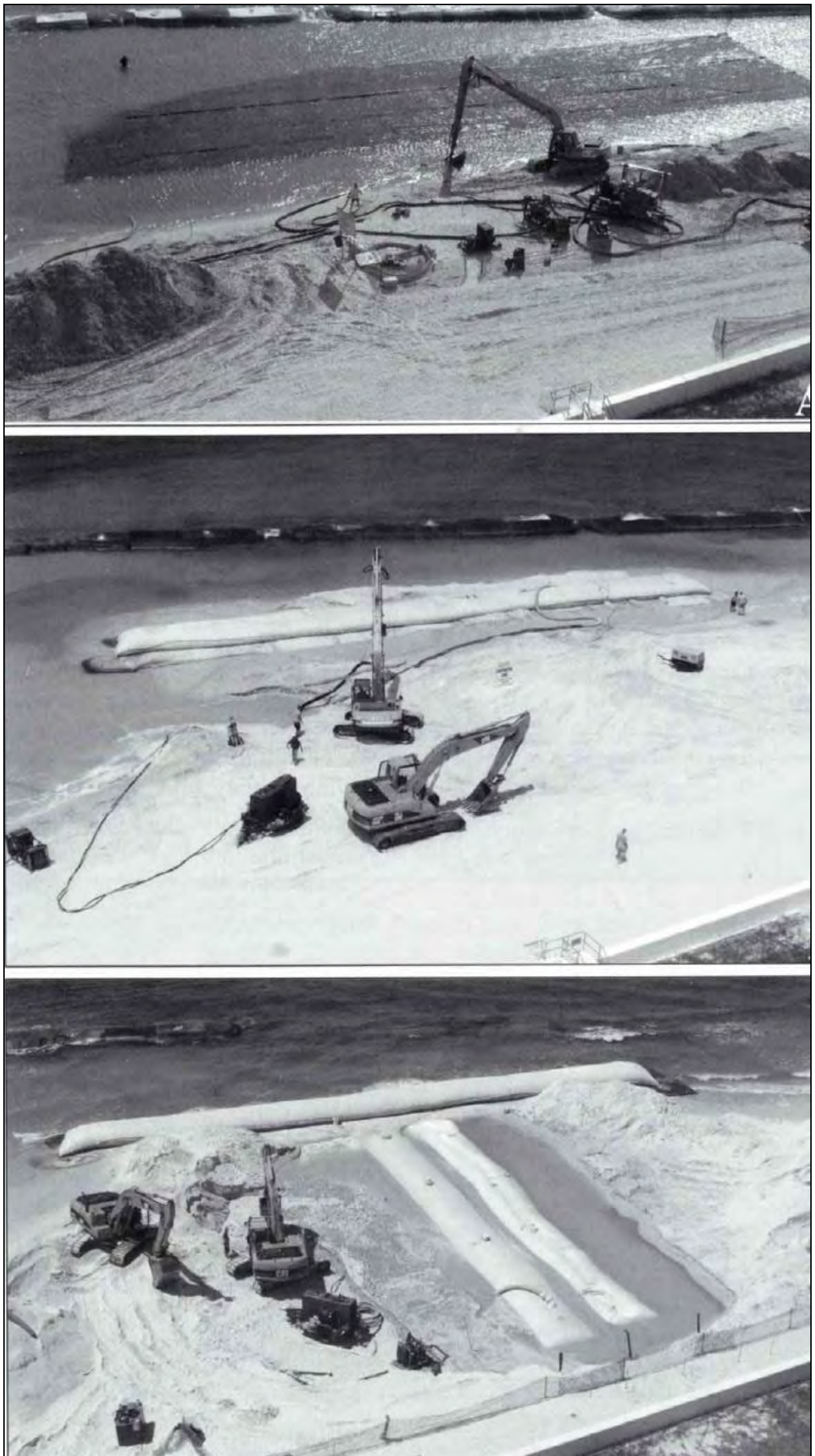


Figure 5-4 T-head groin construction process, Upham Beach: A) April 19, 2005; B) June 16, 2005; and C) September 13, 2005. (source: Elko and Mann, 2007).

Site-specific design

A. Single-tube cross section

The manufacturer has recommended a single-tube cross section design for the present project, using a total of three tubes to form each groin (one stem and two heads). Each tube has a nominal circumference of 60 feet. Bag deformation results in an oblong cross section, and the resulting tube dimensions are expected to be approximately25.5 feet across and 10.5 feet high. The manufacturer also recommends that the tubes be manufactured with a non-woven cushion layer to add protection to the tubes, allowing them to be placed directly on the hard bottom foundation that was found by jet probing. The total sand required to fill the two groins would be about 6,500 cubic yards of beach-quality sand. A typical cross section is shown in Figure 5-5.

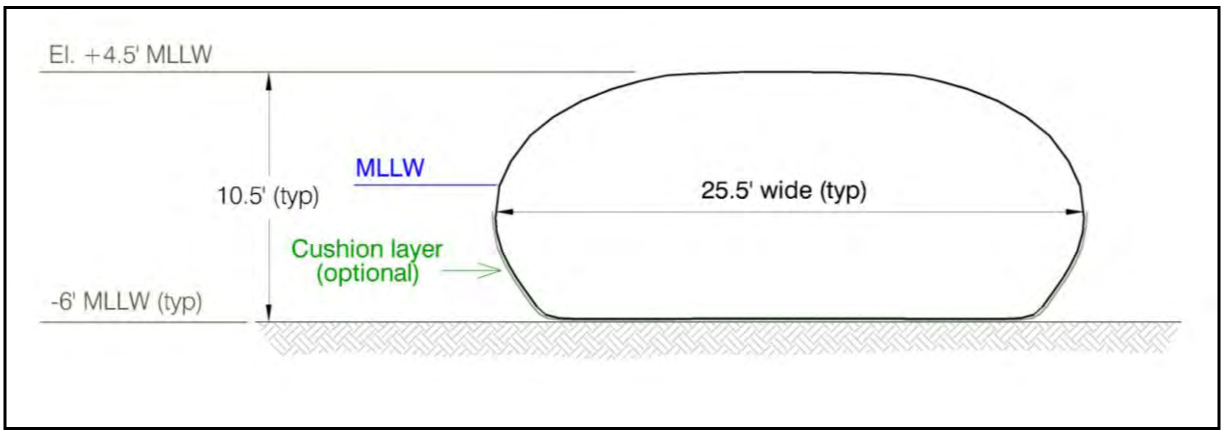


Figure 5-5 Cross-sectional view of single-tube geotextile groin

The material for the two groins in this configuration is estimated to cost $110,000. The total estimated cost of the groins (sand and material, exclusive of construction) is $1,085,000.

B. Pyramid structure

Based on the Upham Beach installation, another configuration could use a similar cross-section, building the groins in a 3-2-1 pyramid (Figure 5-6). A 48-foot wide geotextile apron with three-foot circumference anchor tubes would first be installed. The geotextile tubes would then be installed starting with the three base tubes. Construction using this method results in a 1:1 slope on the face of the groin.

The groins at LBRPP would be slightly smaller than those in Figure 5-3, ranging in elevation from -6 ft to +5 ft MLLW. The nominal diameter of each tube is 80 inches; the resulting cross-sectional area of a single tube would be about 31 sq. ft. with an approximate circumference of 21 feet. The total sand volume required to fill the two groins would be about 4,700 cubic yards of beach-quality sand. The material cost for the tubes is estimated to be $250,000. The total estimated cost of the groins (sand and material, exclusive of construction) is $955,000.

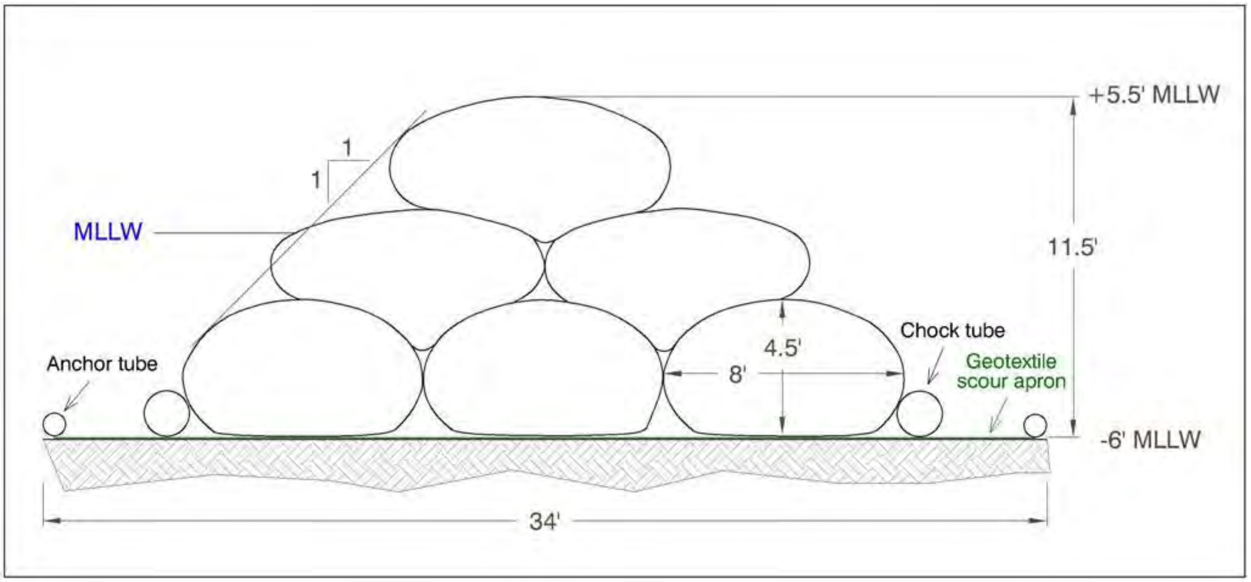


Figure 5-6 Cross-sectional view of MacTube pyramid

5.3 Geotextile Sand Bags

<u>Description</u>

Geotextile sand bags are smaller than geotextile tubes and generally made from the same woven-type of material. Sand bags are commonly used to combat coastal erosion or flooding. An advantage to using sand bag is the ease of installation; filling and placing typically require no special equipment. A disadvantage is that the bags may not be stable under wave attack and the material may degrade over time.

ELCORock Bags and Mega Containers are manufactured by ELCO Solutions Pty Ltd., an Australia-based geotextile manufacturer of a series of robust sand bags. ELCORock Bags and Mega Containers are constructed of a Polyester staple fiber non-woven needle-punched geotextile. This type of geotextile provides protection against the harsh marine environment, allows the Bags and Mega Containers to deform to uneven base conditions, and provides the necessary friction angle for stacking. The material is vandal-resistant and an optional ultraviolet-resistant outer lining can be added for longer life.

ELCORock Bags and Mega Containers have been used for a variety of shore protection structures, including seawalls, revetments, groins, jetties, and artificial reefs. The Bags come in various sizes, including standard 0.35 m^3, 0.75 m^3, and 2.5 m^3; larger custom sizes can be made (see Table 1-1 for conversions between Metric and English units). When full of sand, the 0.75 m^3 and 2.5 m^3 containers weigh approximately 3,300 lbs and 10,000 lbs respectively. ELCORock Mega Containers come in four standard sizes, ranging from 80 m^3 to 210 m^3. Typical cross-sections and placement schematics are shown in Figure 5-7 through Figure 5-9. The standard length of a Mega Container is approximately 60 feet; however, customized sizes are also available.

Table 5-1 ELCORock Bag and Container sizes and filling options

Metric (m^3)	English (cu. yd.)	Filling options
0.35	0.46	Wet/dry/slurry
0.75	0.98	Wet/dry/slurry
2.5	3.3	Wet/dry/slurry
80	105	Slurry
110	144	Slurry
170	222	Slurry
210	275	Slurry

ELCORock Bags offer the option of being filled with dry sand, wet sand, or slurry. For filling, a 0.35 m^3 or a 0.75 m^3 bag is placed in a hopper, filled with dry sand, and sewn shut. A photo of the filling process is shown in Figure 5-10. The 2.5-m^3 bags are placed in a specially-constructed hopper which also provides support for the bags, filled with either dry sand, wet sand, or slurry and then sewn shut. ELCORock Mega Containers have only the option of being filled with slurry.

A repair system is available consisting of an adhesive which is applied to the fabric and then further attached with screws.

Studies of the stability of ELCORock bags under wave attack have recently been completed. These results will be included when obtained.

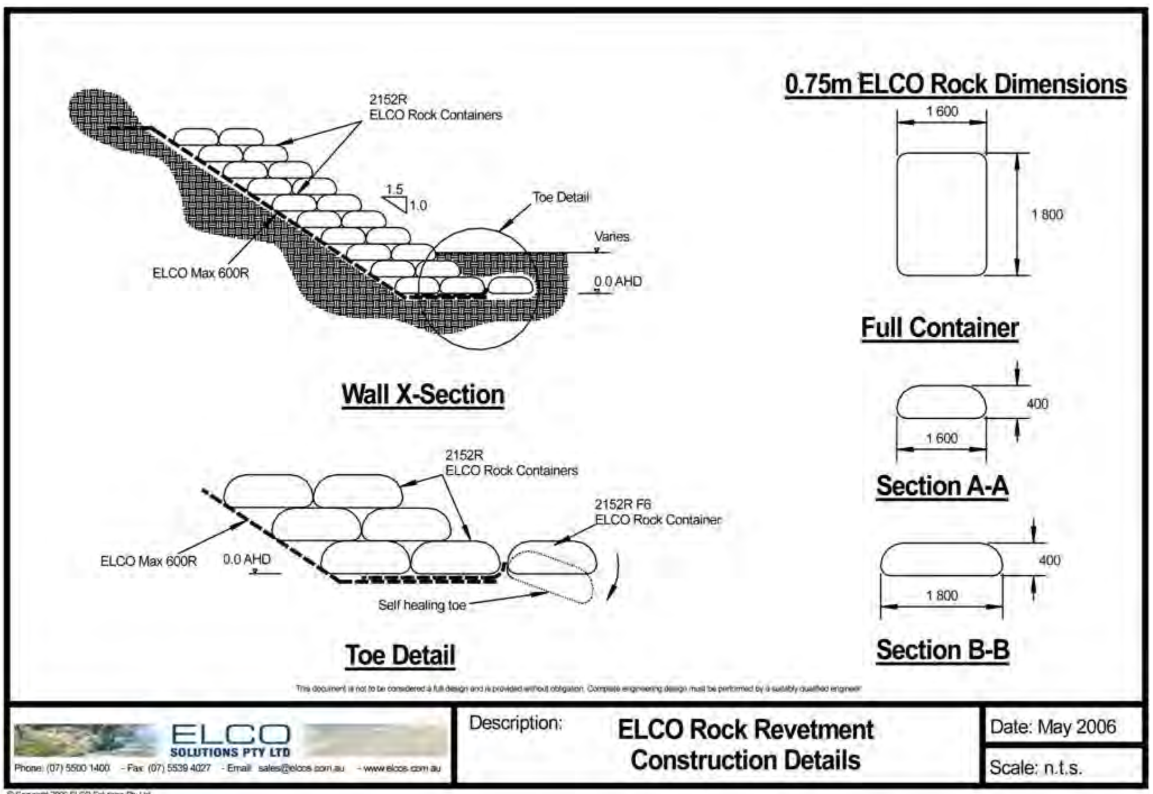


Figure 5-7 ELCORock dimensions for 0.75 m^3 bag and revetment cross section (source: ECLO Solutions)

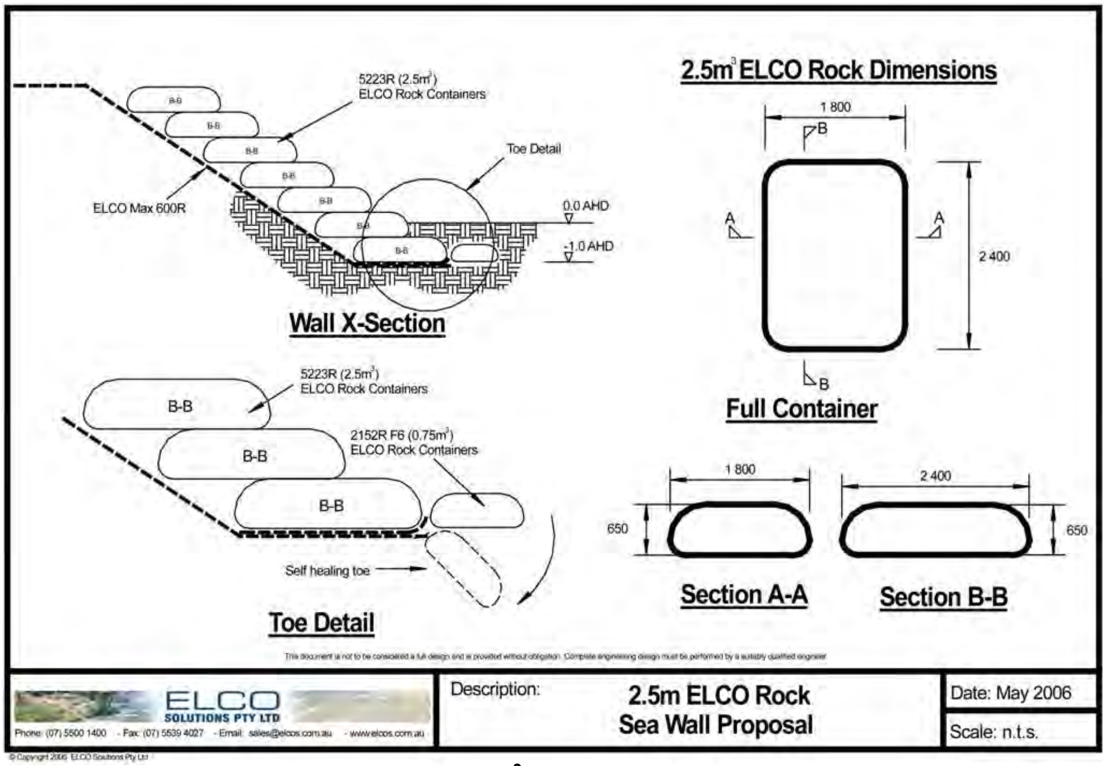


Figure 5-8 ELCORock dimensions for 2.5 m^3 bag and revetment cross section (source: ECLO Solutions)

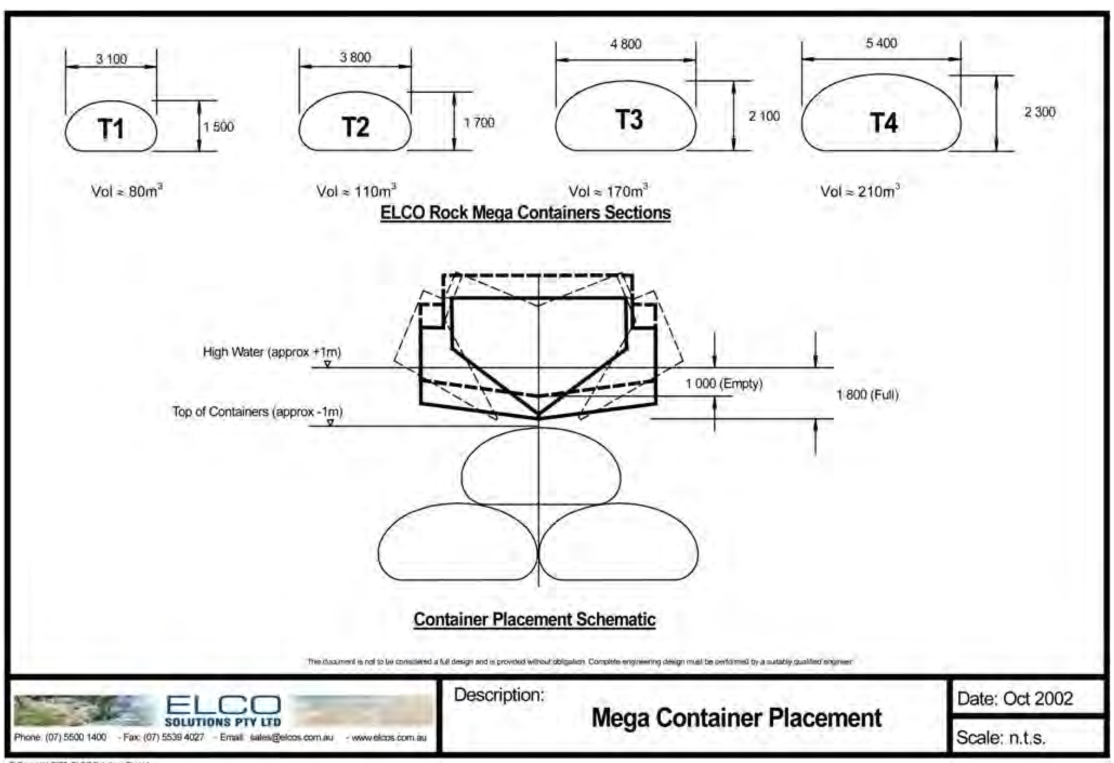


Figure 5-9 ELCORock Mega Container dimensions and placement schematic (source: ELCO Solutions)

(Note: Placement technique not recommended at Lanikai)

Figure 5-10 Filling procedure for a 0.35 m^3 ELCORock Bag (source: Geofabrics Australasia)

Figure 5-11 Filling and placement apparatus being transported to site

Figure 5-12 Assembled apparatus on site

Figure 5-13 Filling of 2.5 m^3 bags

Examples of ELCORock in use

ELCORock Bags and Mega Containers have been used to construct revetments, groins, breakwaters, and offshore reefs. An ELCORock Mega Container groin was constructed at Sonaisali Resort in Fiji in 1998. The groin is composed of three 50-meter long Mega Containers stacked in a pyramid. The container diameters are 1.2 meters. The containers took approximately 2.5 hours each to fill using a 6 inch pump. The project has resulted in significant widening of the beach, and the groin withstood a typhoon attack in 2000.

In 2001, a 2.5-meter high by 100-meter long groin was constructed from 650 2.5-m^3 ELCORock Bags on Maroochydore Beach in Queensland, Australia. The groin was designed to withstand a 10-foot wave height, as well as to provide some vandal resistance. The success of the first groin led to the construction of three additional groins in the area in 2003. A total of 2,000 Bags were used to construct the three additional groins. The original groin was reported to still be in good condition as of 2007. One of the groins is shown in Figure 5-14.

Three 25-m long groins were constructed in Milang, Australia, in 2003, as part of a shoreline restoration project. A total of 410 of the 0.75-m^3 bags were used to construct the groins, which were completed in one week. The groin and nourished beach provide improved recreational value and safe access to the water versus the previous concrete revetment. The old and new shore protection are shown in Figure 5-15.

In 2004, a groin was constructed at Clifton Springs Harbor in Australia to block the transport from depositing sediment in the harbor. The local officials were concerned that a groin constructed from rock would contaminate the neighboring beach. The groin was therefore constructed from 2.5 m^3 ELCORock Bags stacked in a 3-2-1 pyramid. The 100-meter long groin is approximately 2 meters high. The completed groin is shown in Figure 5-16. The installation was so successful and accepted that two smaller groins were installed nearby to improve the neighboring beach.

Figure 5-14 ELCORock Groin, Maroochydore, Australia (source: Geofabrics Australasia)

Figure 5-15 Milang, Australia, shore protection, old (foreground) and new (background); (source: ELCO Solutions)


Figure 5-16 ELCORock groin, Clifton Springs, Australia (source: ELCO Solutions)

Installation

The ELCORock Bags are filled with dry sand, wet sand, or slurry. The bags are held upright with a filling apparatus and loaded with sand using any available means (shovel, backhoe, etc.). The seams are then sewn shut. Placement of the bags is done with an excavator with a smooth cradle on the end of the boom as shown in Figure 5-17. The blue cradle shown carrying the bag was used to hold the bag during filling in Figure 5-13. The groin can be constructed using the excavator shown, which uses the placed bags to drive on while placing new bags (Figure 5-18). Smaller bags may be placed at the toe of the groin to assist in stabilizing the toe in the event of scour. These bags are referred to as a “self-healing toe” and are shown in Figure 5-8. A length of fabric attached to the bag is tucked beneath the adjacent bag, thereby securing the toe in place.

Figure 5-17 Excavator carrying a 2.5 m^3 ELCORock bag

Figure 5-18 Placing of bag

Site-specific design

Two ELCORock configurations were considered in this study. The first concept involves building the groins entirely of 2.5 m^3 bags. Each bag has dimensions 7.9 ft long by 5.9 ft wide by 2.1 ft high. To achieve the desired crest elevation of +4.5 ft MLLW, a pyramid with 5 bags at the base would be necessary (15 bags in the pyramid). This would present a groin base 29.5 feet wide; the single-bag groin crest would be 5.9 feet wide. A groin face slope of 1V:1.4H would be achieved with these bags. A toe protection row of 0.75 m^3 bags is included in the design. These bags would have a tail of material that would be buried under the main bags to limit their movement. A cross-sectional view of this configuration is shown in Figure 5-19.

A total of 720 bags for the north groin and 580 bags for the south groin would be required. At a cost of $1,000 per bag, the total material cost for the bags would be $1,300,000. The toe protection layer consists of 230 of the 0.75 m^3 bags at a cost of $375 per bag. A total of 3,400 m^3 (4,475 cu. yd.) of sand would be required to construct the groins. The total estimated cost of sand and material is $2,060,000.

It may also be possible to fill the bags elsewhere and transport them to the site, reducing the size of the staging area and the on-site equipment. Lifting loops could be manufactured into each bag to lift the bags from the transport vehicle without tearing. Transport of the bags from the road to the beach could be accomplished via a conveyor belt-type of assembly. Regardless of the filling location, an excavator is required to place the bags.

The second conceptual design involves using a combination of ELCORock Mega Containers, similar to the installation at Sonaisali, Fiji. The groins would be built of a base layer of two T2 series containers and a top layer of a single T2 container. The bag dimensions were shown previously in Figure 5-9. A groin face slope of about 1V:1H could be attained with this configuration. A cross-sectional view of this configuration is shown in Figure 5-20. This configuration uses 5,100 cubic yards of sand and is 26 feet wide. The total estimated cost of sand and material is $1,310,000. A scour protection layer such as shown in Figure 5-6 could be included and would raise the price to about $1,500,000.

A groin made of smaller bags has several advantages. Damage occurring to one bag would result in a maximum loss of 2.5 m^3. Damage to a monolithic container, even if compartmentalized, could result in significantly more sand loss and potential loss of effectiveness of the groin. Additionally, the tuned T-head groin guidance in the literature recommends rubble mound structures to dissipate return currents along the groins. Geotextile tubes are advised against; the 2.5 m^3 ELCORock bags possess roughness and porosity to better dissipate energy as compared to monolithic containers.

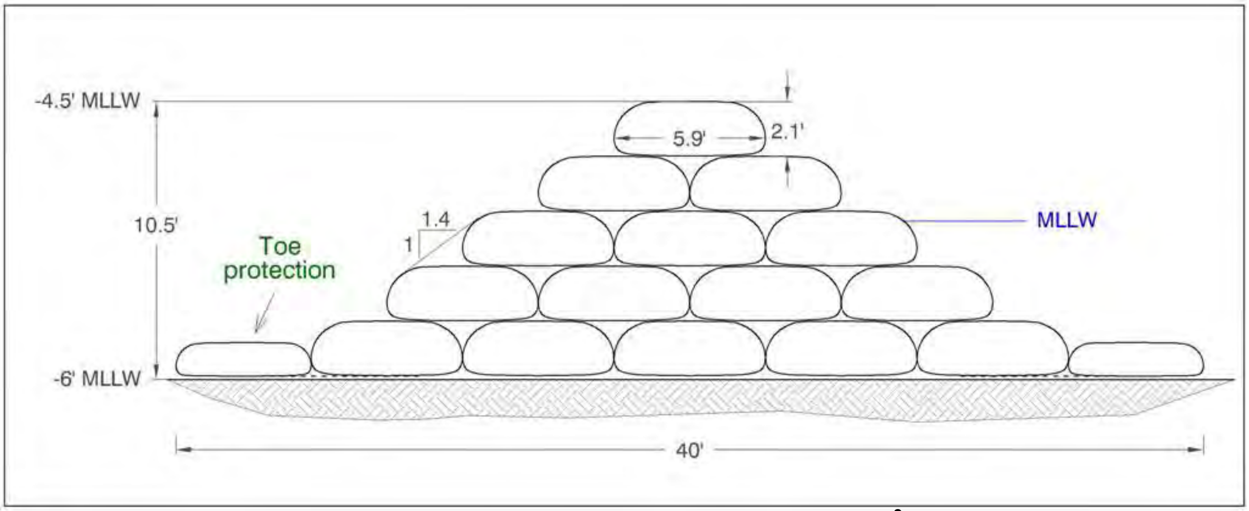


Figure 5-19 Cross-sectional view of ELCORock 2.5-m^3 bag groin

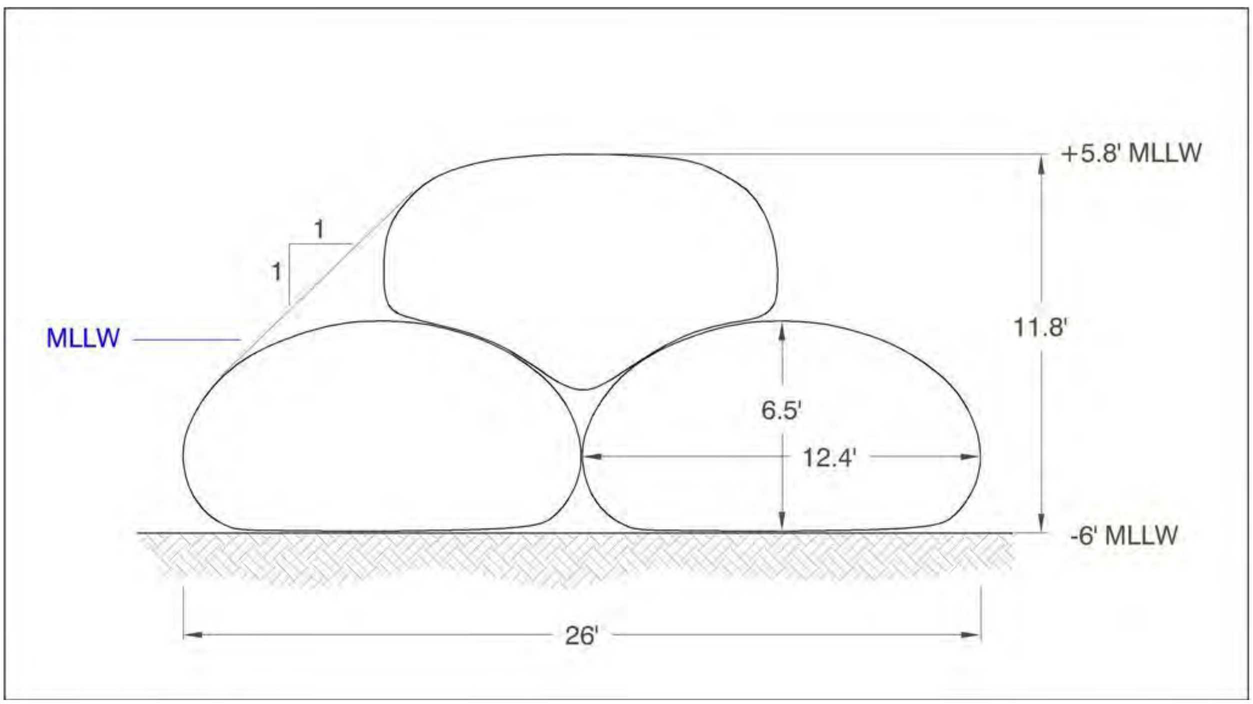


Figure 5-20 Cross-sectional view of ELCORock Mega Container groin

5.4 Triton Marine Mattress

Description

Triton Marine Mattresses are rock-filled containers constructed of a polypropylene geogrid manufactured by Tensar International Corporation, Inc. The openings in the geogrid can range from one to several inches, depending on the application and the filler material size. The mattress shape is formed by lacing geogrid panels together. The mattresses are then filled with rock to form a high mass, high porosity, flexible, and durable monolithic unit. Figure 5-21 shows a filled mattress ready for deployment.

Marine mattresses are typically filled with stone and the mattress dimensions, geogrid size, and stone sizes are selected based on project-specific information. This produces a somewhat permeable structure that has mass as well as energy dissipative characteristics. This makes marine mattresses particularly effective in salt water environments, on irregular sub-grade surfaces, and in wave conditions.

Applications include revetments, dune stabilization, foundations for breakwaters and groins, and scour mats. As seawalls and revetments, rock-filled marine mattresses may compose the main part of the structure. In groins and breakwaters, marine mattresses tend to serve as the foundation for armor stone. The U.S. Army Corps of Engineers (2006) produced a technical note outlining the basic information and potential applications for marine mattress use in coastal engineering. Triton Marine Mattresses have the added benefit of rapid installation for emergency repairs.

Repair to damaged mattresses can be accomplished if the damage covers a relatively small area. If the mattress has already been filled and compacted, a geogrid patch can be stitched over the damaged area. If the mattresses have not been filled, it is best to simply replace the damaged panel.

Figure 5-21 View of Triton Marine Mattress (source: US Army Corps of Engineers)

<u>Examples of Triton Marine Mattresses in use</u>

The Town of Palm Beach, Florida, installed eleven groins in 1995 to combat erosion along a 3,000-foot long shoreline reach. Each groin was constructed of pre-cast concrete modules placed on a base of Triton Marine Mattresses. The project included a beach nourishment phase prior to construction of the groins. The groins were constructed in pits in the beach to provide a calm environment. As the beach equilibrated, the groins became exposed and stabilized the beach as designed. A photo of the project site, one year post construction, is shown in Figure 5-22.

Figure 5-22 Town of Palm Beach, one year post construction (source: NOAA Coastal Services Center)

Also in Florida, the South Amelia Island Shore Stabilization Project was completed in January of 2005. The project, sponsored by the U.S. Army Corps of Engineers, consisted of installation of a 1,500-foot long terminal groin and a 300-foot long detached breakwater. Structure installation was the final phase of a project that included sand nourishment along 3.5 miles of shoreline. Strong currents at the site made scour protection necessary. Marine mattresses were selected to serve as the foundations of the structures, providing stability and reducing the amount of armor stone necessary. The marine mattresses were filled off site with a two to four-inch aggregate. Approximate mattress dimensions were five feet by 38 feet, with a thickness of about 12 inches. A temporary trestle was constructed and a crane on the trestle placed the marine mattresses and armor stone. A view of the project site following construction is shown as Figure 5-23.

Figure 5-23 Amelia Island Shoreline Restoration, post construction (source: US Army Corps of Engineers)

The Hololani Resort in West Maui installed rock-filled Triton Marine Mattresses as part of a shore protection project designed by Sea Engineering. The marine mattresses served as the foundation for three to five layers of sand bags. A top layer of mattresses was also installed to provide erosion control to the soil. The performance of the shore protection structures can be seen in Figure 5-24 and Figure 5-25, which span a period of nine months. The crest of the revetment was filled with beach-quality sand and planted with grass and Beach Morning Glory, helping to stabilize the backshore while providing an aesthetic benefit to the project.

Figure 5-24 Hololani shore protection, November 27, 2007 (Sea Engineering photo)

Figure 5-25 Hololani shore protection, August 25, 2008 (Sea Engineering photo)

Installation

Tensar Corporation provides a detailed installation guide for constructing, filling, and placing of the Triton Marine Mattresses. Figure 5-26, from the installation guide, shows the various components of a mattress. Filling the mattresses is completed on land, with the use of a filling frame and a front-end loader (Figure 5-27). Compartmentalization within each mattress prevents shifting of the fill. Each compartment is partially-filled, then the rock is manually packed, and the compartment is filled the rest of the way, packed again, and topped off, if necessary. A team of four can fill and close a 30-ft mattress in approximately 40 minutes. The required rock size ranges from 2 to 6-inches, with an average size of 4-inches and the recommended specific gravity of the stones is 2.5. The mattresses must be filled according to specifications to minimize interior stone movement under wave action. Deployment requires three workers and a large backhoe or a crane, either land-based or barge-based, and a spreader bar. Figure 5-28 through Figure 5-30 show deployment of a marine mattress.

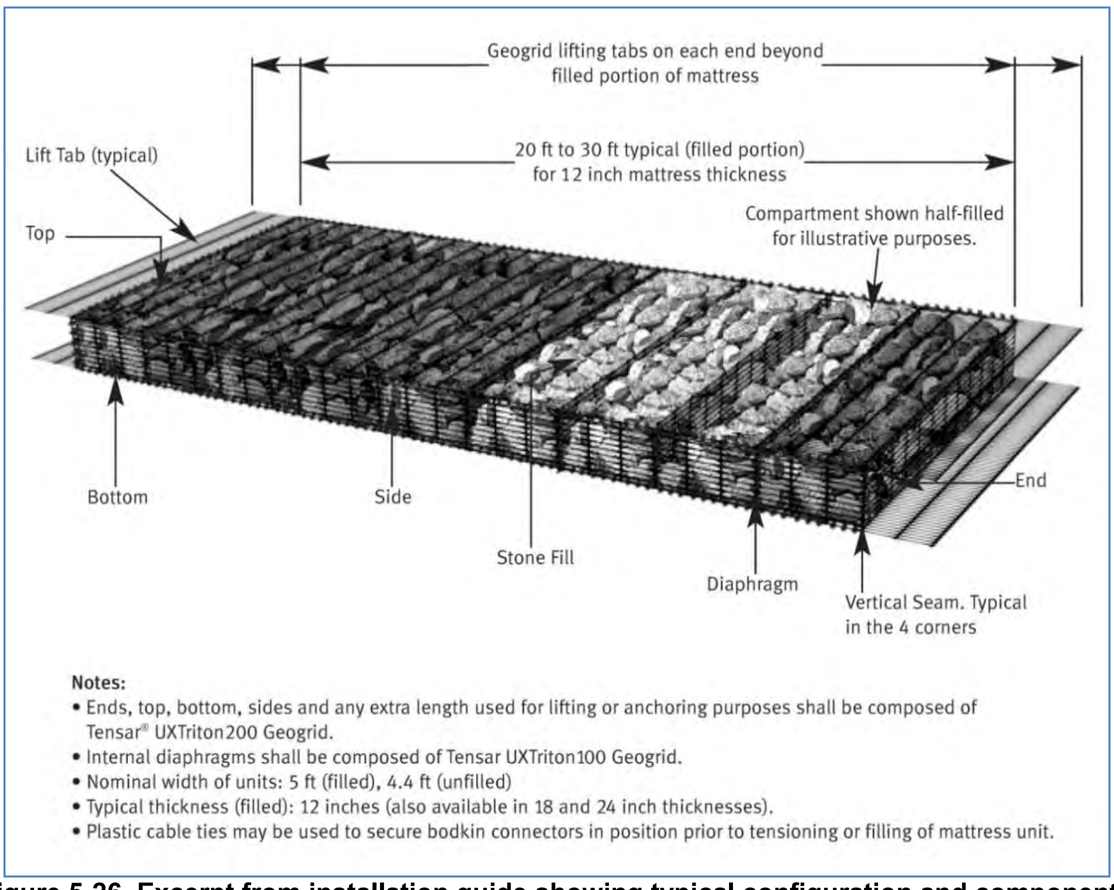


Figure 5-26 Excerpt from installation guide showing typical configuration and components (source: Tensar International)

Figure 5-27 Workers filling a Triton Marine Mattress (source: Tensar International)

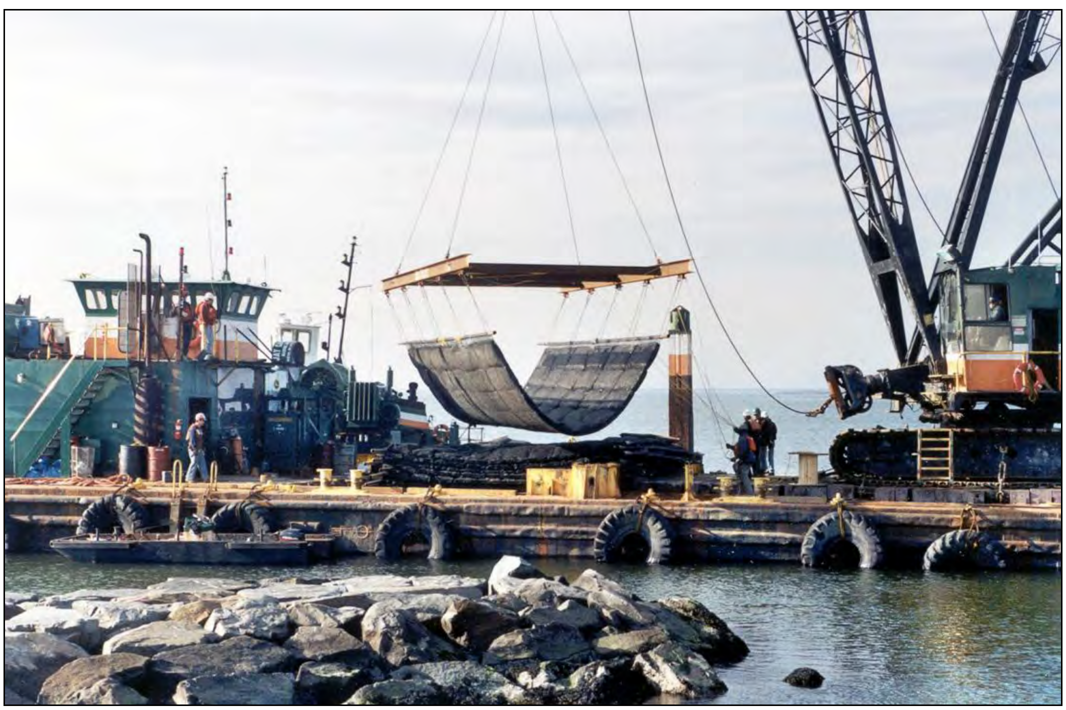


Figure 5-28 Deployment of Triton Marine Mattress—three units wide (source: Tensar International)

Figure 5-29 Deployment of Triton Marine Mattress (source: Tensar International)

Figure 5-30 Deployment of a Triton Marine Mattress (source: Tensar International)

Site-specific design

A typical set of dimensions for a marine mattress is 25 x 5 x 1 (length x width x height; feet). Since the geogrid is supplied in a roll, custom lengths can be produced on site. This allows a great deal of flexibility in designing the structures. At a thickness of about 12 inches, approximately 10 layers would be necessary to construct the groins at Lanikai. Constructing the layers two thick would produce an overall groin face slope of 1V:1.25H. A cross section is shown in Figure 5-31. This configuration would require 822 mattresses and 3,800 cubic yards of rock. The material cost for the mattresses is $720,000 and the total estimated cost of the groins (rock and mattress material, exclusive of construction cost) is $910,000.

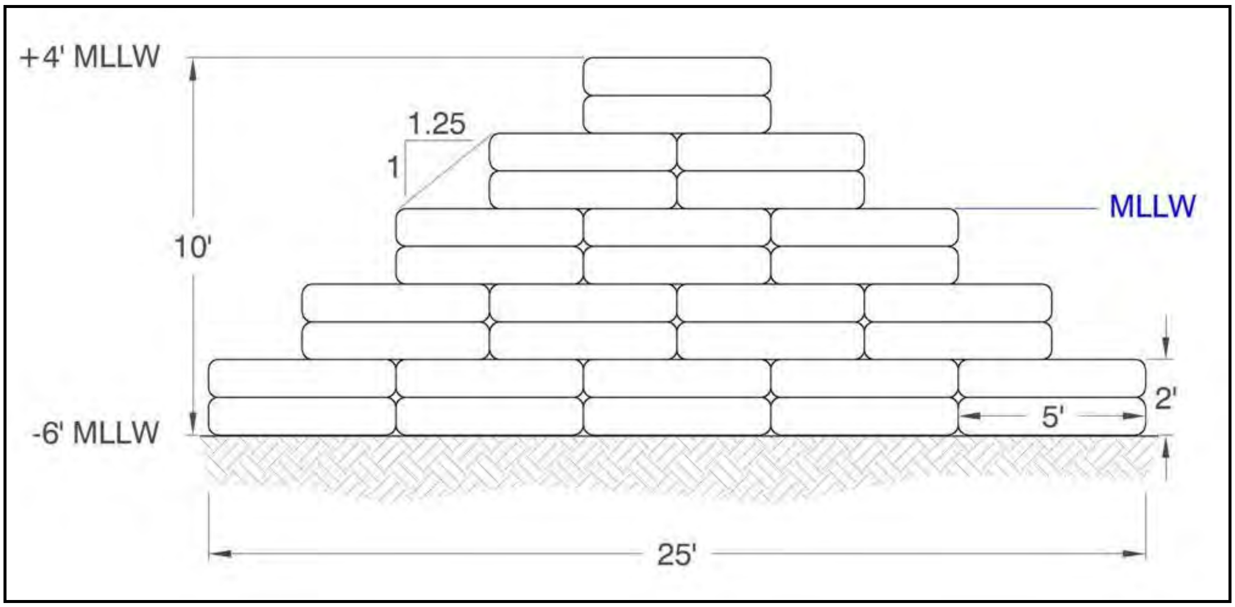


Figure 5-31 Cross-sectional view of Triton Marine Mattress groin

5.5 Coir—coconut fiber

Description

Coir is the coarse fiber from the coconut husk. During processing, the fibrous husk is soaked for many days to soften the fibers, at which time the fibers are removed from the husk and dried. The fibers may then be packed into bales for shipping. Common uses of the fibers are mattresses, logs, bags, rope, netting, and doormats.

Mats and logs are manufactured by mechanical compression the fibers. Once compressed, the logs are wrapped in a coir or synthetic mesh to maintain the shape of the log. Typical densities of the logs are 5 lbs/cf and 9 lbs/cf, with diameters ranging from 8 to 16 inches. Deliverable lengths in Hawaii are limited to no more than 20 feet. Figure 5-32 shows rolls of coir geotextile mats and Figure 5-33 shows coir logs. The mats can be cut and sewn to produce bags that can be filled with sand.

Coir mats and logs can be used in bank stabilization projects. The coir mat is laid along the bank where it serves as a temporary medium in which bank-stabilizing vegetation can grow. The vegetation quickly becomes established in the coir as the roots grow through the coir into the soil. Over time the coir degrades, leaving a vegetation-stabilized bank. Figure 5-34 shows coir geotextile deployed along a stream bank.

Figure 5-32 Rolls of coir geotextile (source: www.unitedcoir.com)

Figure 5-33 Coir logs (source: www.unitedcoir.com)

Figure 5-34 Stream bank protection using coir geotextile (source: www.unitedcoir.com)

Similarly, coir logs can be installed and tied-in to the bank to stabilize the vegetation line. The fibers that compose the log provide ideal growing conditions for bank-stabilizing vegetation such as Naupaka. Figure 5-35 and Figure 5-36 show installations of coir logs for bank protection.

While coir fibers can be packed tightly together, the durability of the products is limited. Installations in wave environments are recommended to be limited to the upper shoreline above the reach of waves, since repeated wave action can separate the fibers and compromise the integrity of the geotextile. The installation in Figure 5-36 shows the use of gravel-sized rock as protection against wave energy. The local Hawaii distributor ranks coir products as being less durable than woven geotextile.

Figure 5-35 Coir logs stabilizing the vegetation line at an East Oahu location

Figure 5-36 Coir log stabilizing bank (source: Geotech Solutions Hawaii)

Examples of coir products in use

While Figure 5-35 shows coir logs as shoreline bank protection, installations of this type are mainly above the reach of waves. The local distributor of coir products, Geotech Solutions Hawaii, knows of no installations where coir products are used as groins or breakwaters.

Sea Engineering constructed a temporary experimental revetment at the Maui Marriott in 2003. The 10-foot section of revetment consisted of sand-filled bags within tubes made of coir rope. A photo of the revetment is shown in Figure 5-37. The coir tubes were tied together and helped maintain the sand bags as a single unit. The tubes were removed a few weeks later.

Figure 5-37 Coir and sand bag revetment in place

Site-specific design

Custom-sized coir sand bags can be manufactured by cutting and sewing the coir mats. For comparison with the ELCORock products presented earlier, the use of coir-filled sand bags assumes the same dimensions as the ELCORock 2.5-m^3 bag. The cross-section would thus look identical to the one presented in Figure 5-19.

Filling the bags could best be accomplished using a filling apparatus similar to the one shown in Figure 5-12 and Figure 5-13. The bucket holding the bag during filling could also be used to place the bag. Placement would require care to not damage the bag, and at present, no placement technique has been developed. Coir bags are not sufficiently durable that an excavator could drive on them during construction, as is the technique with the ELCORock bags. Placement might require use of a land-based crane.

The attractiveness of sand-filled coir bags as the groin material is in the degradable nature of the coir. The groins would be considered temporary structures, with an estimated lifespan of up to five years. As the material degrades, the beach-quality sand would be released to the environment, where it could help nourish the beach.

The unpredictability and non-uniformity of coir degradation, however, could limit the effectiveness of the project as a whole. It is unlikely that the structures would degrade uniformly, and since coir cannot withstand repeated wave attack, it is expected that the groin heads would fail first. This would compromise the success of project, as the gap width between groins would effectively increase, allowing greater energy to the beach, thereby creating an unstable condition that would lead to rapid erosion. Since no data exists on the ability of coir products to withstand wave forces, there is no way to quantify the durability of the products. As with the woven geotextile tubes, coir bags may require a protective layer to prevent the substrate from damaging the bags.

The cost of a double-lined coir sand bag was unavailable at the time of this report. If designed to be similar to the 2.5-m^3 ELCORock bags, the project would require 1,300 bags and 4,475 cu. yd. of sand. The cost of sand and materials is expected to be around $1,100,000. The design consisting of one cubic yard sized bags is expected to cost approximately $1,700,000.

5.6 Decision Matrix

Five configurations from three manufacturers were presented in Sections 5.2 through 5.4. To aid in selecting the most appropriate configuration for the LBRPP, a decision matrix was constructed and is shown as Table 5-3. The matrix considers the following general categories:

- Design
  - Effectiveness (energy dissipation of rip currents and waves, ability to retain sand)
  - Stability (resistance to moving)

- Construction
  - Time to construct (time required on site)
  - Ease of construction (amount of equipment, size of labor force, skill level)
  - Staging area (size of open area needed)

- Longevity:
  - Durability (resistance to UV degradation and vandalism)
  - Repairability (replace unit or repair on site)

- Public concerns
  - Public safety (hazard for slipping, cuts)
  - Visual effect (look of structure)
  - Footprint (bottom area covered

- Cost
  - Cost of materials including sand

The cost of labor and equipment has not been estimated for this report.

For each category, the proposed structure is given a value based on the following scale:

Table 5-2 Decision value descriptions

Value	Description
-2	Strongly negative
-1	Mildly negative
0	Neutral
1	Mildly positive
2	Strongly positive

Table 5-3 Materials Decision Matrix

	Geotextile tubes (single-tube cross section	Geotextile tubes (pyramid structure)	ELCORock 2.5-m^3 bags	ELCORock Mega Containers	Triton Marine Mattresses	Coir sand bags
Effectiveness	-2	-1	2	-2	2	2
Stability	2	1	1	1	2	1
Time to construct	2	1	0	1	-2	0
Ease of construction	2	1	0	2	-2	1
Staging area	2	2	1	2	0	1
Public safety	0	0	0	0	-1	0
Visual effect	0	0	1	0	-1	1
Footprint	2	1	0	1	1	0
Durability	-1	-1	2	2	2	-2
Repairability	0	0	2	2	2	-2
Volume of sand	0	-1	0	-2	2	0
Cost	2	1	-1	1	1	0
TOTALS	**9**	**4**	**8**	**8**	**2**	**2**

As can be seen in Table 5-3, the highest scores were obtained for the geotextile tubes (single-tube cross section), the ELCORock 2.5-m^3 Bags, and the ELCORock Mega Containers. The ELCORock 2.5-m^3 Bags offer the greatest effectiveness by most closely simulating the energy dissipation characteristics of rubblemound structures. The ELCORock products also provide the greatest durability. The geotextile tubes, however, were found to be the most cost-effective solution.

6. BEACH DESIGN ANALYSIS

The experience gained from studying natural crenulate-shaped bays provides a design tool for coastal engineers to produce stable shorelines. Silvester and Hsu (1993) present methods for determining the stable beach planform between headlands, thus facilitating the engineering use of headlands as shore stabilization structures. Whereas natural beaches obtain a stable shape in response to the wave climate and headland orientation, engineered beaches can be developed by selecting the headland orientation that produces the desired beach shape. Bodge (2003) proposed the use of T-head groins as artificial headlands to produce stable beaches. This approach has been used successfully on numerous beaches in Florida and the Caribbean (Bodge, 1998). Locally, this empirical database was recently used in designing proposed beach nourishment at Iroquois Point and the Waikiki Sheraton Hotel.

The heads of the T-groins can be aligned, or "tuned," according to the prevailing wave crest orientation, to produce the desired beach configuration. Rubblemound T-head groins are recommended to reduce rip currents and the subsequent offshore sand losses and the beach should be nourished with sand to achieve the predicted shoreline shape. According to Bodge (1998), tuned structures work well in the following situations:

- Erosion stress is so severe that renourishment would be too frequent to be economical or practical
- A wide beach would affect an environmentally-sensitive area
- The shoreline is no longer conducive to having beaches, such as at a hardened shoreline

These criteria are generally met in Lanikai.

The beach design process thus includes establishing the desired physical characteristics of the beach, and then applying coastal engineering analysis and tools to orient structures to achieve the desired beach planform.

6.1 Beach Physical Characteristics

Beach physical characteristics include crest height, dry beach width, beach slope, and sand grain size. Standard methodology typically involves trying to match adjacent beach characteristics both because this indicates what is naturally stable for local conditions and because it is aesthetically more pleasing to match the adjacent beach. Site observations at accreted parts of Lanikai Beach indicate a vegetation line elevation of approximately +6 feet above MLLW, a beach crest width of up to 50 feet, and beach face slopes varying from 1V:7H to 1V:17H (Vertical:Horizontal). Beach parameters selected in the Lanikai conceptual design were a beach crest elevation of +6 feet and a beach slope of 1V:12H. This beach slope was confirmed by recent measurement of the profile through the Pokole Way beach access. A goal of the design process is to achieve a minimum beach crest width of 30 feet. The sand fill characteristics will be designed to match the sand at the accreting part of the beach and were discussed in the Lanikai conceptual design report.

6.2 Structure Orientation and Beach Planform

The prevailing wave crest alignment is a key design parameter for tuned T-head groin alignment. To determine prevailing wave crest alignment, a numerical model was used to transform selected waves from deep water to shallow water at Lanikai.

6.2.1 Bathymetry

An important input for modeling is the site bathymetry. The high-resolution SHOALS data set was used to produce the bathymetric grid. The data set, however, has considerable gaps believed to be due to waves breaking over the shallow nearshore reef. These gaps in the data are particularly noticeable between the Mokulua Islands and Flat Island. These data gaps were confirmed with aerial photos to be in areas of shallow reef. Using the data “as is” results in erroneous, greater depth values being interpolated across these shallow reef areas, and consequently, errors in the modeling results.

The reef in this area is quite variable in depth and includes emergent outcrops at low tide and pockets and grooves in the reef several feet deep. Wave transformation is greatly affected by the large, shallow areas of reef, which were found to have typical depths of less than 3 feet MLLW, with substantial amounts near 0.0 feet MLLW. Proper representation of the reef in the bathymetric data is necessary for the modeling to produce meaningful results. Therefore, several site visits were conducted at low tide to confirm the shallow reef areas identified in the aerial photographs. Data points with depths of 0.7 feet (0.2 meters) MLLW were added to the LiDAR data to represent the reef in the known shallow areas. It is possible that the LiDAR data also misrepresents the smaller reef outcrops; however, these locations are difficult to identify. Misrepresentation of smaller reef outcrops may have less of an effect on the results than if the larger reef areas were not corrected.

6.2.2 REF/DIF Modeling

The selected representative deepwater wave conditions were entered into the numerical wave refraction-diffraction model REF/DIF1, version 2.5, (Kirby and Dalrymple, 1994) to simulate the wave transformation as the waves propagate shoreward. The model is steady-state and nonlinear, and the waves are transformed by the processes of wave shoaling, refraction, diffraction, and energy dissipation due to bottom friction and wave breaking. Model output includes wave heights, breaker zones, and a snapshot of the water surface elevation. The model input conditions are shown in Table 6-1.

REF/DIF model results were interpreted to provide wave crest orientations at Lanikai for structure design. Model runs were completed for each of the wave cases listed in Table 6-1, where water levels of 0.8 feet and 2.8 feet refer to MSL and MHHW + 1 foot, respectively.

Table 6-1 Model Input Information

Direction	Period (seconds)	Height (feet)	Water Level (feet)	Grid Spacing (feet)
N (0°)	14	6.0	0.8	20.0
	14	6.0	2.8	20.0
NE (45°)	8	6.0	0.8	20.0
	8	6.0	2.8	20.0
E (90°)	9	8.0	0.8	21.3
	9	8.0	2.8	21.3

The model results showed that the wave environment in the nearshore waters off Lanikai was very complex, due to wave transformation past the shallow reef and the Mokulua Islands. These complex wave patterns are instrumental in shaping the shoreline. The results also showed that significantly less wave energy propagated to shore at the MSL model water level, indicating that the beaches are more likely to react to waves during higher water levels.

6.2.3 BOUSS2D Wave Modeling

As mentioned previously, the REF/DIF results past the Mokulua Islands and across the shallow reef show that the nearshore wave field is very complex. This is verified by observations and aerial photos. To better analyze the nearshore waves, the wave model BOUSS2D was employed. BOUSS2D, a component of the Surface-water Modeling System suite of modeling products, is a shallow-water non-linear wave model that also includes the processes of wave shoaling, refraction, diffraction, and breaking. BOUSS2D is a time domain model and has been shown to be particularly useful in modeling wave/structure interaction. The model has the capability of outputting an animation of water surface elevations, which is very effective in showing wave propagation involving complex wave patterns.

Refraction causes wave crest orientation to approach parallel with the bathymetry as the waves propagate toward shore. This presents a limited range of wave crest orientation near shore at a given point. The wave crest orientation of the prevailing wave was included in determining the gap orientation. BOUSS2D modeling was performed for the prevailing ENE wave condition determined in Section 4 of *Dir* = 67.5°, *H* = 6 feet, *T* = 8 seconds, at a water level of +2.7 feet MLLW to account for tide, wave setup, and eddies. The gap orientation was determined as the average orientation of the NE, E, and ENE modeled wave crests. The incident wave crest orientation for each of the three modeled wave directions and the average wave crest orientation are shown in Figure 6-1

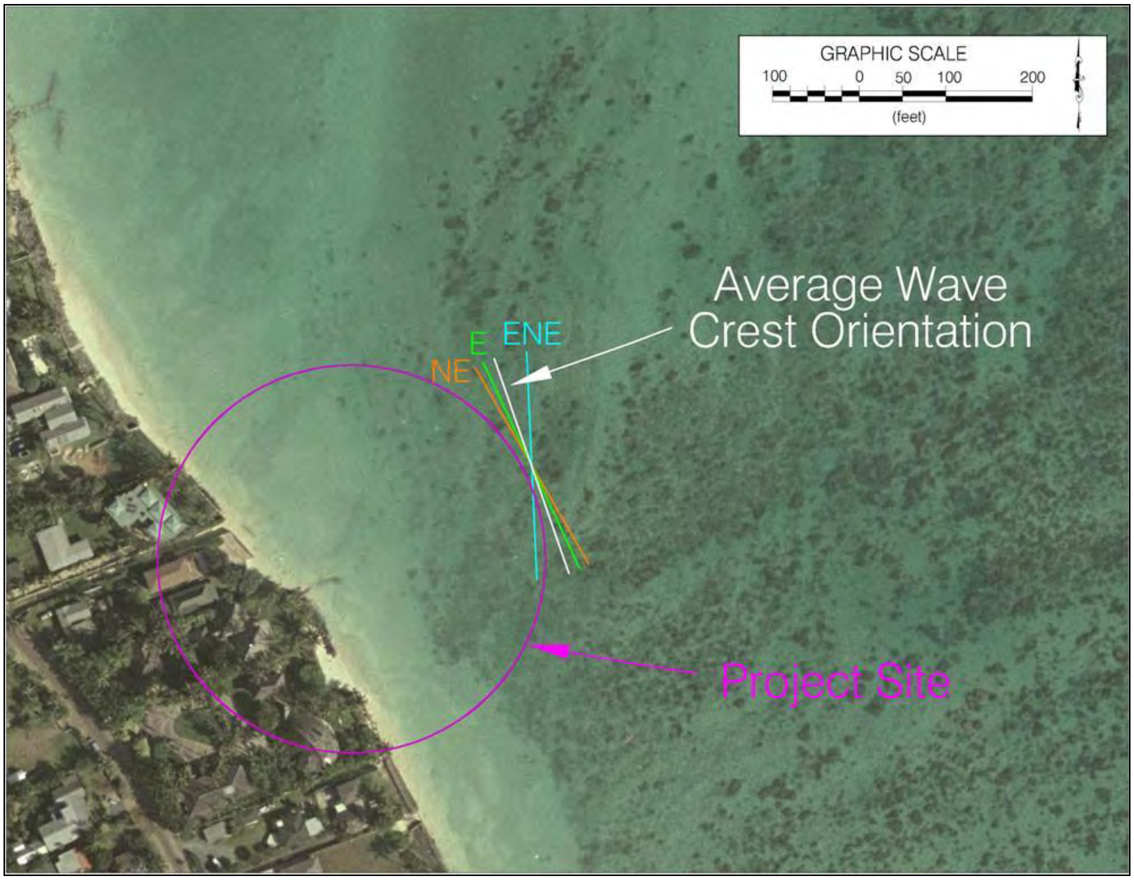


Figure 6-1 Incident and average wave crest orientation.
Wave crest orientations are shown for the northeast (yellow), east-northeast (cyan), and east (green) deepwater waves, as well as the average (white).

6.3 Beach Design

Tuned T-head groins simulate the general configuration of natural landforms that produce arcuate beaches. Key design parameters for T-head groins include groin length, head width and orientation, gap distance between the heads, and beach shape and width. In general, the beach shape responds more to the gap width (opening) between the groin heads than it does to the structure heads themselves. Thus, the stable beach is a function of the length and orientation of the gaps. Orientation of the gaps is primarily dictated by the shape of the shoreline and the prevailing wave approach direction.

For design, the empirical rule is that the mean low water (low tide) shoreline will be located between one-third and two-thirds of the gap length, *G*, behind the groin head, i.e., 0.35*G* to 0.65*G*. Larger values in the range are appropriate for 1) energetic open coasts directly exposed to wave action, 2) larger gap openings, 3) large angles between the wave approach and the gap orientation, 4) poor beach fill sand compatibility, and 5) a greater desired level of conservatism. The groin head length should be long enough so that the mean low water shoreline approaches the head, while maintaining a minimum ratio of gap width to head width of about 60:40 for aesthetic reasons so that the groins do not appear to dominate the viewscape. A schematic of the tuned T-head groin system is presented as Figure 6-2.

The groin layout and head angle should be oriented such that the gap opening is as parallel as possible with the average prevailing wave crest approach. This “tuning” of the heads to minimize the angle between the prevailing wave approach and the gap opening helps ensure the predictability of the beach shape and yields potentially greater shoreline stability within each cell. The groin stem should extend landward to the design beach crest to eliminate flanking and loss of sand from the cell around the back of the groin. The crest elevation should be above the high tide elevation and high enough to prevent significant overtopping during typically prevailing (non-storm) water level and wave conditions. A crest elevation of +4.5 feet is recommended, as discussed in Section 4.1.

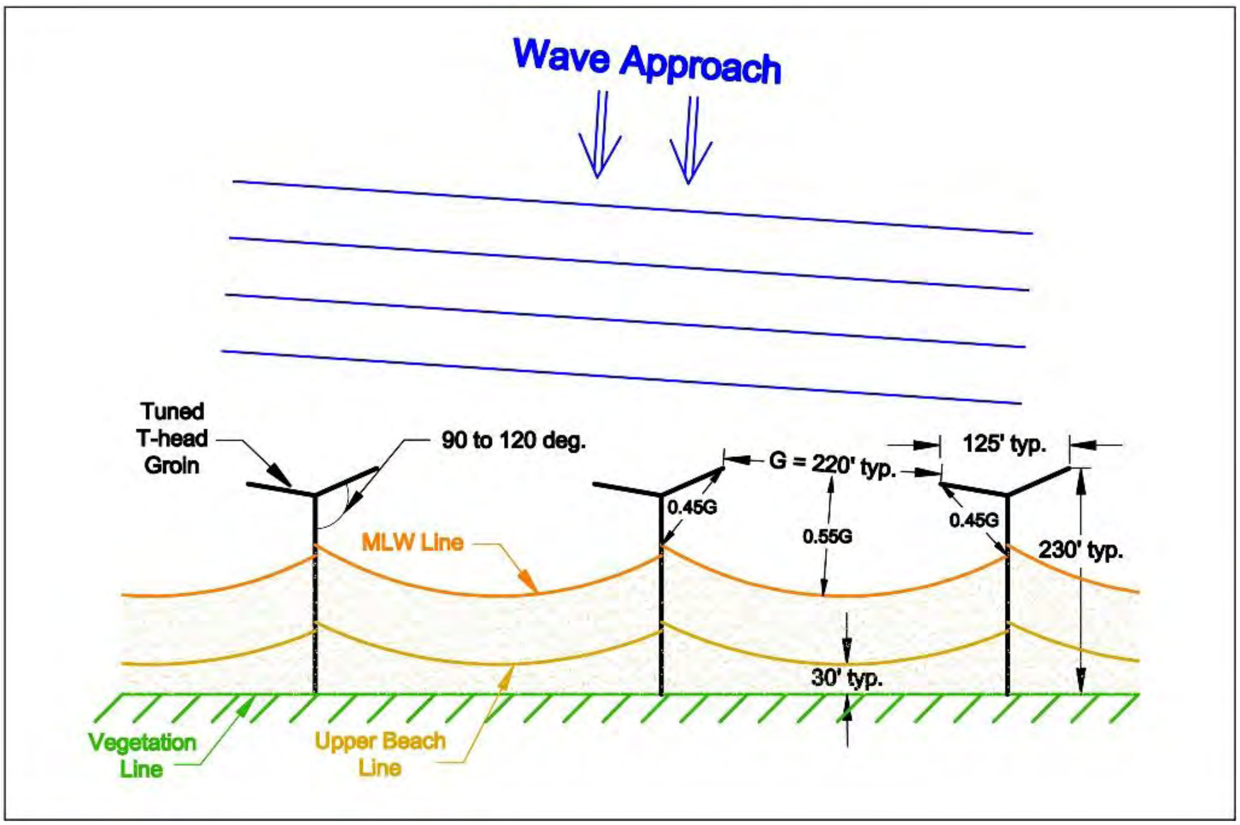


Figure 6-2 Schematic of tuned T-head groin system

The location of the mean low water (MLW) beach for Lanikai is determined to be 0.45 times the gap width measured from the ends of the groin heads. Experience has shown that the center of the beach should be located about 0.55 times the gap width measured from the center of the gap to produce an arc-shaped beach. The beach is expected to take a 1V:12H slope and the beach toe and crest lines are drawn at elevations -6 feet and +6 feet MLLW respectively, based on findings in Section 3.3.5.

The length of the groins is determined by the desired beach crest width. The schematic shown in Figure 6-2 indicates a minimum beach crest width of 30 feet. The groin system can be moved offshore to produce a wider beach; in areas where sand supplies are scarce, the groins can be moved landward to reduce the required fill material. The actual minimum beach width varies with the wave crest orientation and the shape of the shoreline. The volume of sand necessary to construct the beach was calculated using Terramodel, which uses the differences between the available LiDAR data and the designed beach to calculate the fill needed to achieve the desired profile.

The U.S. Army Corps of Engineers has consulted with the members of the Lanikai Homeowners Association and they have decided that the most appropriate groin configuration would be a single geotextile tube such as the MacTube presented in Section 5.2 and the conceptual design is shown in Figure 6-3. Wave crest orientations are shown in the following figures for the northeast (yellow), east-northeast (cyan), and east (green) deepwater waves. The gap orientation is the same as the average of the three colored wave crest orientations, shown as the white line. The minimum dry beach width of 30 feet is measured from the 2005 vegetation line to the +6 feet MLLW contour line and is generally found opposite the center of the gap. The irregularity of the vegetation line causes this point to move slightly south of center. The maximum dry beach width of 120 feet is found along the northern groin.

The gap between the groin heads is 200 ft. The groins are designed to extend vertically to an elevation of +4.5 feet MLLW. Groin dimensions are presented in Table 6-2. Figure 6-5 and Figure 6-6 show the shorelines at the termination of the groins.

Table 6-2 Groin Dimensions

	North Groin	South Groin
Stem length	249 ft	173 ft
North head length	67 ft	67 ft
South head length	67 ft	67 ft
North angle	110°	110°
South angle	110°	110°

An overfill ratio of 1.1 was found during the Lanikai conceptual study. The same overfill ratio is used in the present report, assuming that the borrow sand will have the same characteristics as the offshore sand analyzed in the conceptual study (Sea Engineering, 2008). Applying this overfill ratio, the required sand fill to produce the beach between the groins is 14,200 cubic yards. A small amount of sand, 1,400 cubic yards), would also be necessary to produce the sand fillets on the outside of the groins.

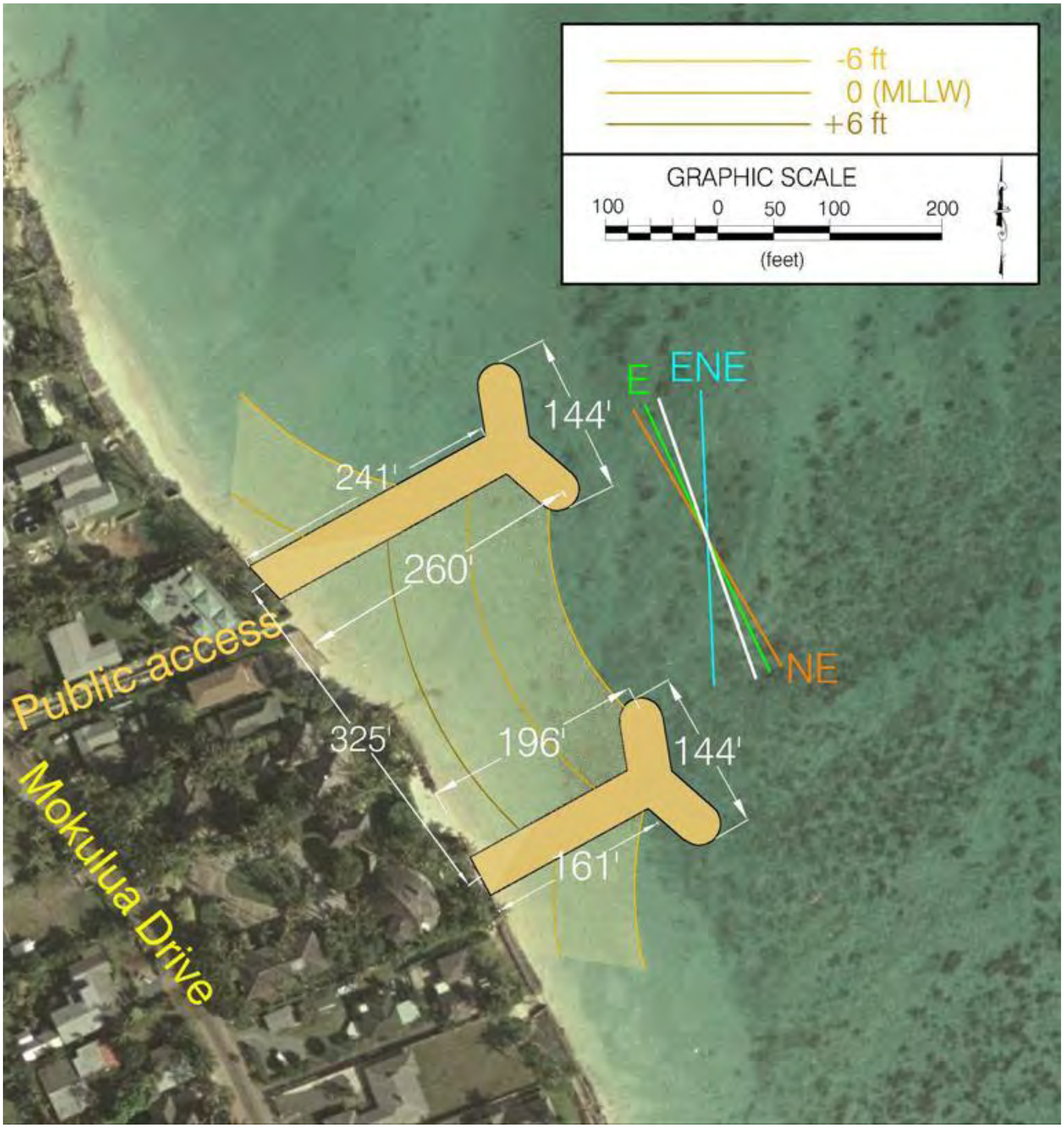


Figure 6-3 Geotextile tubes with beach fill

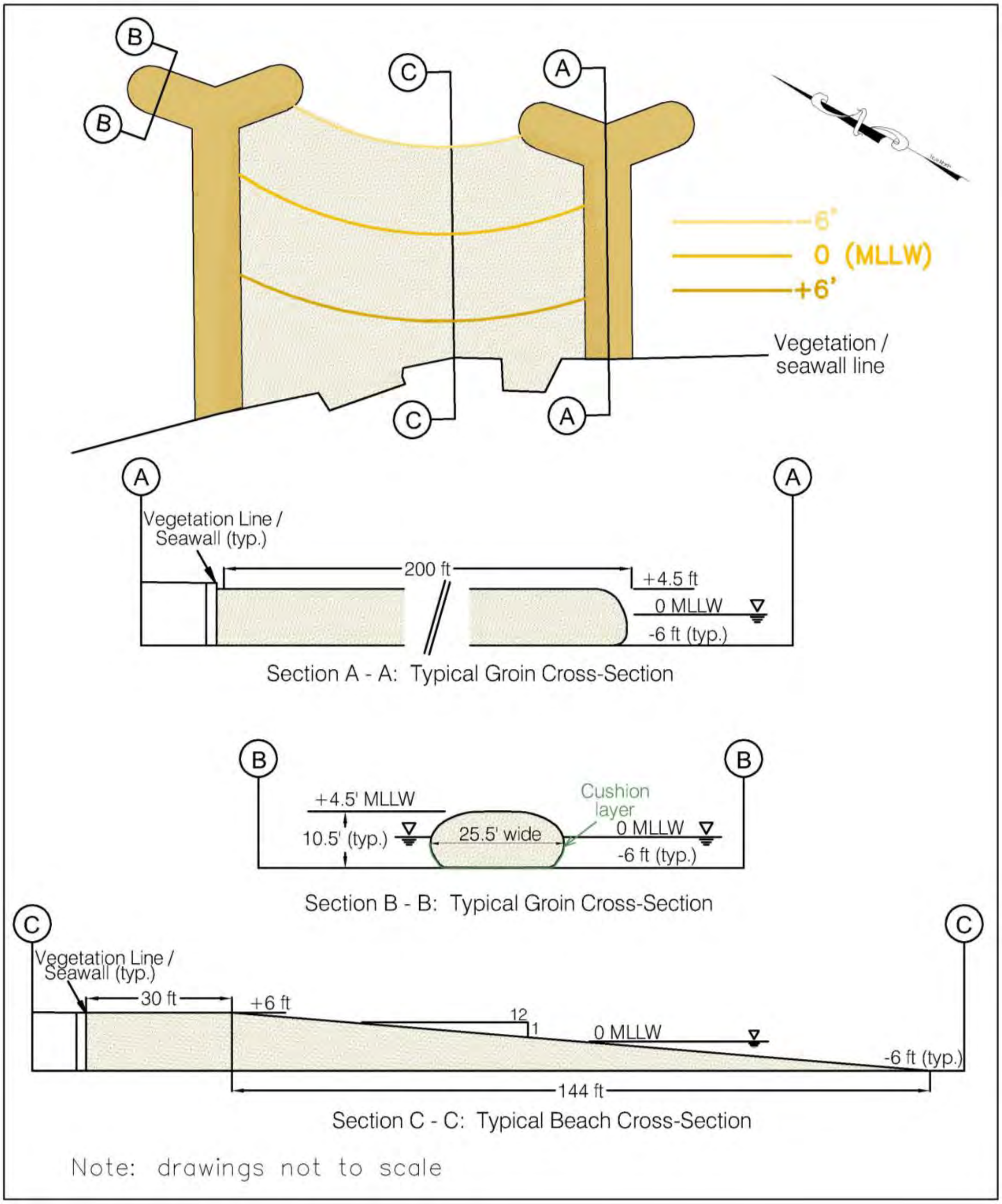


Figure 6-4 Typical groin and beach cross-sections

Figure 6-5 Photo of shoreline at termination of south groin

Figure 6-6 Photo of shoreline at termination of north groin

7. CONCLUSIONS AND RECOMMENDATIONS

The Honolulu District of the U.S. Army Corps of Engineers recently completed a Regional Sediment Management (RSM) plan on the southeast shores of Oahu. That study provided guidance on solving sediment problems in the region using a systems approach that considers the entire region from the mountains to the sea. The Southeast Oahu RSM culminated with the 2009 “Southeast Oahu Regional Sediment Management Plan” report. The Lanikai conceptual study within the RSM plan focused on beach restoration options for Lanikai Beach, including beach nourishment with and without structures.

The RSM project has been extended to a design for a pilot beach restoration project on the southern Lanikai shoreline. The pilot project is located at the Pokole Way beach access along a shoreline reach that has been hardened with seawalls and revetments. There is presently no dry beach along this reach due to chronic erosion.

A beach cell stabilized by two T-head groins was designed at the Pokole Way beach access. The groin heads were aligned, or tuned, based on the incident wave crests as determined by numerical modeling. The stem of the north groin is 249 feet long, while the stem of the south groin is 173 feet long. Each groin has a head composed of two 67-foot long units. The groins are designed to be constructed of geotextile tubes. Attached cushion layers protect the geotextile from the underlying hard bottom.

The design configuration produced a groin gap width of 200 feet and a distance of 325 feet between groin stems. The design produces a beach with a minimum 30-foot wide beach crest, measured from the vegetation line to the +6 foot contour. The design beach crest is 120 feet wide along the north groin and 50 feet wide along the south groin. Filling of the geotextile tubes and nourishment of the beach require a total of 22,100 cubic yards of beach-quality sand.

A construction cost estimate is included as Appendix A. The costs presented assume that there is a suitable sand source available. The cost of $150 per cubic yard of sand utilized in this estimate was developed based on cost estimates for offshore sand recovery at other locations in Hawaii. The attached cost estimate also assumes that there is a satisfactory staging area near the project site. The total cost of the project is estimated to be $4,750,000. A more detailed site-specific estimate from a dredging contractor could result in a different cost.

REFERENCES

Bodge, K. 1998. *Beach Fill Stabilization with Tuned Structures: Experience in the Southeastern U.S.A. and Caribbean. Proc. Coastlines, Structures, and Breakwaters '98*. Thomas Telford Publishing, London, p. 82-93.

Bodge, K. 2000. *Independent Evaluation Study of the Proposed Kuhio Beach Improvements.* Olsen Associates, Inc., Jacksonville, FL.

Bodge, K. 2003. *Design Aspects of Groins and Jetties. Advances in Coastal Structure Design*, ASCE, p. 181-199.

Elko, N.A., and D.W. Mann. 2007. "Implementation of Geotextile T-Groins in Pinellas County, Florida." *Shore & Beach*, vol. 75, no. 2, pp. 2-10.

Lipp, D.G. 1995. *Changes in Beach Profiles due to Wave Reflections off Seawalls at Lanikai, Hawaii*. Master's Thesis, Department of Ocean Engineering, University of Hawaii at Manoa, Honolulu, HI.

Sea Engineering, Inc. 2008. *Lanikai Beach Restoration Study Conceptual Design Report*. Prepared for the U.S. Army Corps of Engineers, Honolulu District.

Sea Engineering, Inc. 1990. *Windward Oahu Hurricane Vulnerability Study: Determination of Coastal Inundation Limits*. Prepared for U.S. Army Corps of Engineers, Pacific Ocean Division. SEI Report 90-02.

Silvester, R. and J.R.C. Hsu. 1993. *Coastal Stabilization, Innovative Concepts.* Prentice Hall, New Jersey. 578 p.

U.S. Army Corps of Engineers. 2007. Wave Information Studies (WIS). *Coastal and Hydraulics Laboratory.* http://frf.usace.army.mil/cgi-bin/wis/pac/pac_main.html

U.S. Army Corps of Engineers. 2006. *Coastal Engineering Manual*.

U.S. Army Corps of Engineers. 1984. *Shore Protection Manual*.

U.S. Army Corps of Engineers. 2006. *Uses for Marine Mattresses in Coastal Engineering*. CHETN-III-72.

Vituosek, S., and C.H. Fletcher. 2008. "Maximum Annually Recurring Wave Heights in Hawaii." *Pacific Science*, vol. 62(4:541-553)

Wang, P., and T.M. Roberts. 2007. "Performance of Upham Beach T-Groin Project and Its Impact to the Downdrift Beach." Progress report submitted to Coastal Coordinator, Pinellas County, Florida.

APPENDIX A. CONSTRUCTION COST ESTIMATE

Item	Quantity	Unit	Unit cost	Total
Geotextiles	1	set	$110,000	$110,000
Sand (for groins)	6,500	cy	150	975,000
Sand (for beach)	15,600	cy	150	2,340,000
Base Yard	1	ea	25,000	25,000
Labor (4-6 workers)	45	days	5,000	225,000
Heavy equipment	45	days	2,000	90,000
Sand blowing	15,600	cy	20	312,000
Environmental controls and monitoring	1	ea	50,000	50,000
			Subtotal	$4,127,000
			Contingency (15%)	619,050
			Total	**$4,746,050**