KAHALUU BEACH PARK,
KONA-HAWAII

COASTAL ENGINEERING EVALUATION

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INTRODUCTION

PROJECT LOCATION AND GENERAL DESCRIPTION

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

The park is owned and operated by the County of Hawaii, and is heavily utilized by both residents and tourists. Sandy beach areas are relatively limited on the Big Island, and thus Kahaluu Beach Park is a valuable recreational resource. The park is about 5.4 acres in size, with about 700 feet of shoreline and typically less than 0.5 acres of sandy beach area. Approximately one-third of the park shoreline is protected by grouted rock and concrete-rubble-masonry (CRM) walls, and there are backshore walls protecting park facilities and landscaping. Park facilities include pavilions, restrooms, showers, picnic areas, a basketball court and parking area. The existing park facilities are shown on Figure 2. The nearshore waters in the vicinity of the park are very popular for swimming and snorkeling.

The bay shoreline is fronted by a shallow basalt rock shelf, typically 1 to 4 feet deep at mean lower low water (mllw) on the southern half and 3 to 5 feet deep on the northern half. The seaward side of the southern portion is elevated slightly with respect to the inner portion of the shelf, and is lined with large boulders which protrude above the water surface (the "menehune breakwater"). The origin of these boulders is unknown. The boulders and shallow water on the south side dissipate incident wave energy before it reaches shore, while the relatively deeper water on the north side of the bay permits more wave energy to enter the bay on this side.
STUDY PURPOSE AND OBJECTIVES

The R.M. Towill Corporation, under contract to the Hawaii County Department of Parks and Recreation, is preparing a master plan for possible beach park improvements. The purpose of this coastal engineering evaluation is to assist with master plan preparation by:

- analyzing existing shoreline and oceanographic conditions;
- assessing shoreline problems and needs, and desired improvements; and
- evaluating possible alternatives for beach and park improvement.

The study objective is to provide the County with coastal engineering information and recommendations to aid in selecting an alternative (or alternatives) for further detailed study.

PREVIOUS STUDIES

The U.S. Army Corps of Engineers, Honolulu District (USACOE), at the request of the County of Hawaii, conducted a study in 1981 to identify and evaluate beach erosion and related problems at Kahaluu Beach Park. Their report Kahaluu Beach Park Erosion Control - Summary Report, December 1981 presents the results of site investigations, topographic and bathymetric surveys, archaeological studies, geologic investigations, oceanographic and beach processes evaluations, engineering design and environmental assessment.

GENERAL PROBLEMS AND NEEDS

Kahaluu Beach is a relatively stable pocket beach, which has shown little change over the years. However, the beach is small, the sand is relatively coarse with a significant percentage of gravel and cobble sized material, and there is very little sand in the littoral environment of the bay. There is very little sand offshore within the bay, with the exception of small isolated sand patches in depressions on the northern part of the bay. There is virtually no longshore transport of sand, and thus no sand to nourish the beach.
Kahaluu Bay is well sheltered from typically prevailing wind and wave conditions by the island itself, however the bay and shoreline are directly exposed to winter Kona storm waves and other westerly approaching waves. Westerly approaching storm waves result in a water level rise in the bay, and storm waves which overtop the low elevation beach crest, resulting in flooding of the park and the movement of sand into backshore areas. This not only results in damage to park facilities, but also can considerably reduce the beach area. A severe Kona storm in January 1980 resulted in waves and high water levels which destroyed park facilities, damaged the seawalls, and deposited up to a two-foot thick layer of sand in portions of the backshore area.

Site investigations conducted for this study showed typical beach conditions, with a narrow strip of sand along the southern half of the park and a small pocket beach at the north end of the park bounded by the highway and the CRM wall which protects the north comfort station, and continued deterioration of the seawalls and backshore CRM walls. The vertical CRM seawall which protects an existing pavilion seaward of the fish pond is badly undermined and has a large hole completely through the foundation through which backshore material is being eroded.

General problems and needs of the park which this study addresses include:

- beach improvements to provide more recreational sandy beach area and increased protection for the backshore park facilities,
- necessary seawall repairs and improvements, and
- coastal engineering recommendations to guide planning of backshore park improvements.

A primary consideration guiding the planning and design of park shoreline improvements is the need to protect the bay's environment, and to result in no adverse impact to the water quality, flora and fauna which provide a very valuable and much utilized coastal resource.
OCEANOGRAPHIC DESIGN PARAMETERS

WIND

Prevailing Winds

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

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

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

Storms

There are many recorded tropical storms or hurricanes which have approached the Hawaiian Islands. Hurricanes form near the equator, and in the central North Pacific typically move toward the west or northwest. These tropical storms or hurricanes usually pass south of the islands, and generally stay far enough offshore to only cause high surf and heavy rainfall as they pass. For example, Hurricane Susan (1978) with sustained wind speeds of 120 knots was pointed directly at the Big Island but died before coming within 200 miles of the island. The frequency of occurrence of severe winds at Keahole Point, 13 miles north of the study area, was statistically estimated by Rocheleau (1977) based on the records of severe storms which affected the west coast of the Big Island, as follows:
Sustained Wind

<table>
<thead>
<tr>
<th>Return Period, years</th>
<th>Speed, knots</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>40</td>
</tr>
<tr>
<td>25</td>
<td>46</td>
</tr>
<tr>
<td>50</td>
<td>51</td>
</tr>
<tr>
<td>100</td>
<td>56</td>
</tr>
</tbody>
</table>

The hypothetical model hurricane wind speed for the Hawaiian Islands as developed by Haraguchi (1984) is 67 knots.

WAVES

Prevailing Deepwater Waves

Kahaluu Bay is completely sheltered from the prevailing tradewind seas approaching the Hawaiian Islands from the northwest, and well sheltered from winter North Pacific swell. Kona storm waves and South Pacific swell, on the other hand, approach the coast in the project area directly.

Deepwater wave data in the Hawaiian Islands is collected by the Coastal Data Information Program (CDIP), with wave height and period data measured by a network of Waverider buoys connected to a central data acquisition and analysis station at Scripps Institution of Oceanography in California. Data from a buoy at Barbers Point on Oahu is considered reasonable for application to the project site because the wave exposure at this location (sheltered from tradewind seas and partially sheltered from north swell) is similar to that of Kahaluu Bay. Wave height and period data from this gauge is shown on Table 1, unfortunately wave direction is not measured. As can be seen on Table 1, over 85% of the waves are less than about three feet high, with heights ranging from 1.3 feet to 9.5 feet and periods from 5 to 20 seconds. Wave height exceedances at Barbers Point for a 50%, 10% and 1% frequency of occurrence are 3 feet, 4.6 feet and 7.2 feet, respectively, with typical periods of 6 to 16 seconds. These wave heights are likely over-estimates of the typical wave climate at Kahaluu because it is such a sheltered location, however they do illustrate that typically prevailing conditions are not severe. A 7-foot deepwater wave with a period of 16 seconds is considered reasonably representative of an annual worst case wave event.
Table 1.
CDIP Program Deepwater Wave Statistics for Barbers Point, Oahu

<table>
<thead>
<tr>
<th>HEIGHT (FEET)</th>
<th>WAVE PERIOD (SEC.)</th>
<th>PERCENT FREQUENCY (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0-6.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>6.0-8.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>8.0-10.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>10.0-12.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>12.0-14.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>14.0-16.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>16.0-18.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>18.0-20.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>20.0-22.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>TOTAL</td>
<td>2.3</td>
<td>19.8</td>
</tr>
<tr>
<td></td>
<td>15.0</td>
<td>15.0</td>
</tr>
<tr>
<td></td>
<td>16.5</td>
<td>24.9</td>
</tr>
<tr>
<td></td>
<td>15.8</td>
<td>4.7</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>100.0</td>
</tr>
</tbody>
</table>

THE TOTAL NUMBER OF DATA = 1239
THE RANGE OF WAVE HEIGHTS (FEET) = 1.3 - 9.5
THE RANGE OF WAVE PERIODS (SEC.) = 5.0 - 20.0

THE WAVE HEIGHT IS THE SPECTRALLY BASED SIGNIFICANT WAVE HEIGHT.
THE WAVE PERIOD IS THE PERIOD ASSOCIATED WITH THE SPECTRAL PEAK.

Extreme Deepwater Waves

The project area is also exposed to severe wave attack from passing tropical storms and hurricanes. The US Army Corps of Engineers (1967) hindcasted wave heights generated by 17 severe storms during the period 1947 to 1965, seven of which affected the south and/or west shores of the islands. Marine Advisors (1963) also hindcasted deepwater wave conditions off the west coasts of Lanai and Molokai produced by the ten worst storms during the 15-year period from 1947 to 1961. Sea Engineering, Inc. hindcasted the deepwater wave characteristics for the Kona storm of January 1980, and the Hurricane Iwa (1982) deepwater wave heights near the project area. Storm wave data from these sources is summarized in Table 2.
Table 2.
Historic Storm Wave Characteristics

<table>
<thead>
<tr>
<th>Date</th>
<th>Deepwater Wave Height (feet)</th>
<th>Deepwater Wave Period (seconds)</th>
<th>Approach Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 DEC 1955</td>
<td>14.8</td>
<td>11</td>
<td>West</td>
</tr>
<tr>
<td>5 SEP 1957 (Della)</td>
<td>18.9</td>
<td>21</td>
<td>West</td>
</tr>
<tr>
<td>2 DEC 1957 (Nina)</td>
<td>20.0</td>
<td>13</td>
<td>South &amp; West</td>
</tr>
<tr>
<td>18 JAN 1959</td>
<td>14.0</td>
<td>10</td>
<td>South &amp; West</td>
</tr>
<tr>
<td>6 AUG 1959 (Dor)</td>
<td>22.5</td>
<td>12</td>
<td>South &amp; West</td>
</tr>
<tr>
<td>7 JAN 1962</td>
<td>13.6</td>
<td>11</td>
<td>South &amp; West</td>
</tr>
<tr>
<td>17 JAN 1963</td>
<td>12.0</td>
<td>10</td>
<td>Southwest</td>
</tr>
<tr>
<td>11 JAN 1980</td>
<td>17.0</td>
<td>9</td>
<td>Southwest</td>
</tr>
<tr>
<td>23 NOV 1982 (Iwa)</td>
<td>14.0</td>
<td>14</td>
<td>Southwest</td>
</tr>
</tbody>
</table>

Based on the characteristics of the model hurricane developed by Haraguchi (1984), hypothetical hurricane wave conditions can be determined as follows:

\[
\begin{align*}
H_o &= 16.5 \exp \left( \frac{R_{AP}}{100} \right) \left( 1 + 0.208 \frac{V_F}{V_R} \right) \\
T &= 8.6 \exp \left( \frac{R_{AP}}{200} \right) \left( 1 + 0.104 \frac{V_F}{V_R} \right)
\end{align*}
\]

where,

- \( U_R \) = the maximum sustained wind speed in knots (67 knots)
- \( \Delta P \) = \( P_n - P_o \) in inches of mercury and \( P_n \) is the normal pressure of 29.92 inches of mercury and \( P_o \) is the central pressure of the hurricane (\( P_o = 28.8 \) inches of mercury)
- \( V_F \) = hurricane forward speed in knots (\( V_F = 20 \) knots)
- \( R \) = radius of maximum wind in nautical miles (\( R = 19 \) n.m.)
- \( \alpha \) = 1

The wave height and period associated with the model hurricane are 31 feet and 12 seconds.
A summary of extreme deepwater wave conditions applicable to the assessment of design parameters for Kahaluu Beach Park are as follows:

<table>
<thead>
<tr>
<th>Hindcasted Storm Waves</th>
<th>Wave Height, feet</th>
<th>Wave Period, sec.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>12 to 23</td>
<td>9 to 21</td>
</tr>
<tr>
<td>Model Hurricane</td>
<td>31</td>
<td>12</td>
</tr>
</tbody>
</table>

For design purposes, the January 1980 Kona storm wave height of 17 feet and period of 9 seconds is considered reasonable. This type of event has an estimated return period of 25 years or more, and can reasonably be expected to again occur at the project site. A hurricane occurrence would be considerably more severe, however direct hurricane attack is considered an extremely unlikely event, and too severe for design of park improvements.

TIDE

The nearest tidal bench mark is located at Kailua Bay, approximately 4.5 miles north of the project site. Tide data based on US Coast and Geodetic Survey measurements is as follows:

- Mean higher high water (MHHW) 2.1 feet
- Mean high water (MHW) 1.6 feet
- Half tide level (approx. MSL) 0.9 feet
- Mean low water (MLW) 0.2 feet
- Mean Lower low water (MLLW) 0.0 feet

All elevations in this report are referenced to the MLLW datum. The MLLW elevations can be converted to mean sea level (MSL) by subtracting 0.9 feet from all positive elevations (elevations above MLLW).

DESIGN NEARSHORE WAVE AND WATER LEVEL CONDITIONS

Wave Transformation From Deepwater to Breaking

Deepwater waves propagating toward the coast are primarily altered by refraction, shoaling and ultimately by breaking. Because deep water extends very close to shore, refraction effects seaward of Kahaluu Bay can be considered negligible. The waves travel toward
shore, shoaling (increasing in height as the water depth decreases and the wave speed slows) until the water depth becomes shallow enough to initiate wave breaking. After enough energy is dissipated to stop the breaking action, the wave reforms and continues shoreward increasing in height until it breaks again. This sequence is repeated until the wave finally runs up on the shoreline.

**Nearshore Stillwater Level Rise**

The total water level rise along the shore above the MLLW datum is primarily the sum of four components: the astronomical tide, wave setup, and, during hurricane events, storm surge due to wind stress and a reduction of atmospheric pressure. It is not considered reasonable to design park improvements for hurricane conditions, thus the storm surge effects of a cyclonic storm will not be considered for this estimate of the appropriate design water level.

**Astronomical Tide** - An astronomical tide of 2.1 feet (mean higher high water) is considered appropriate for design purposes because of the reasonable likelihood that a severe winter Kona storm could occur concurrently with a high spring tide.

**Wave Setup** - Wave setup is the change in the mean water level caused by momentum flux changes in a train of waves of changing amplitude, and is directly influenced by the bottom profile in the nearshore area of the project site. It increases shoreward starting at the initial wave breaking point seaward of the shallow rock shelf of the bay. Wave setup can be estimated based on methodology in the *Shore Protection Manual* (US Army Corps of Engineers, 1984), and using a typical nearshore bottom profile as shown on Figure 3. Initial breaker heights and wave setup associated with the selected deepwater design wave is estimated as follows.

<table>
<thead>
<tr>
<th>Deepwater Wave Parameters</th>
<th>17 ft., 9 sec.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Breaker Height</td>
<td>20 ft.</td>
</tr>
<tr>
<td>Wave Setup</td>
<td>2 ft.</td>
</tr>
</tbody>
</table>

**Total Nearshore Design Water Level** - The total design water level on the nearshore rock shelf fronting the park is the sum of the tide and the wave setup, and is equal to 4.1 feet MLLW (3.2 feet MSL).
Nearshore Design Wave Height

The maximum wave height (breaker height) to which the shore and nearshore structures might be subjected is a function of the bottom slope and total water depth at the point of interest. The bottom slope on the rock shelf is essentially flat. The design stillwater level is 4.1 feet, and assuming a 1-foot water depth below the MLLW datum at the toe of the beach or a nearshore structure, the total design water depth is 5.1 feet. This would result in a design breaker height of 4 feet.

The wave runup on the existing beach slope for the design wave conditions would be 3 to 4 feet above the stillwater level, or 7 to 8 feet above MLLW. Thus, the design wave would overtop the existing top of beach ground elevation of about 6 feet, which was the case during the January 1980 storm which serves as the model for the design wave conditions.
COASTAL PROCESSES

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

Although Kahaluu Beach can be seen in aerial photographs dating back to 1950, and is a reasonably stable pocket beach under typically prevailing conditions, there is no source of sand or littoral transport of sand along this coast to nourish or replenish the beach. Occasional storms damage the beach by pushing sand onto the backshore area, and by removing some sand to the offshore area, and, during the course of the storm, likely transporting it away from the beach and out of the bay where it is lost to the system. The condition of the beach has in recent history has been primarily a function of maintenance and sand replenishment following storm damage.
SHORELINE IMPROVEMENT ALTERNATIVES

INTRODUCTION

Based on the general coastal engineering considerations and design parameters associated with improvements to Kahaluu Beach Park, various alternatives are possible and offer a range of benefits and impacts. These alternatives range from no action, or basically continuing the status quo, to the construction of structural measures to protect, maintain and improve the park facilities. The US Army Corps of Engineers 1981 report discusses the general range of alternatives, and is used along with other information for this discussion of possible alternatives. This report is primarily concerned with the conceptual design and evaluation of alternatives with respect to coastal engineering aspects of the plans, and does not address all the social, historical, and marine environmental aspects of possible project impacts. These aspects are being evaluated by other study team members, and will be incorporated along with this report in the final overall study report.

NON-STRUCTURAL MEASURES

No Action

The no-action alternative is simply to maintain the park in the same manner as has been done, with good likelihood of continued periodic episodes of storm damage and the need for maintenance of both park facilities and the beach. It would be possible to clean and repair the park following storm damage, and to replace sand periodically if it is lost. This would require a certain maintenance budget basically forever, rather than a relatively large up-front capital expenditure to construct improvements, and would result in no environmental change from existing conditions. The down-side is that park facilities would continue to be highly stressed and inadequate for the users, with the very likely risk of greatly curtailed useability following a storm until maintenance and repair could be accomplished. Even the no-action alternative, however, would require some immediate repair and maintenance, such as repair of the CRM seawall seaward of the fishpond location.
Shoreline Management

Shoreline management basically consists of planning for park facilities and beach uses which are compatible with the known problems and risks associated with storm occurrences. Improvements could consist of relocating park facilities such as pavilions and comfort stations to backshore areas away from possible storm wave damage and sand movement, or raising the ground elevation at the top of the beach to prevent overtopping. This alternative would result in continuing damage to the shoreline and need for periodic beach maintenance, continued reduction in beach area following major storms, and would not protect nearshore features which cannot be relocated (eg. historic sites).

STRUCTURAL MEASURES

The construction of structural measures, including revetments and seawalls, and breakwaters, groins or other beach stabilization methods, is necessary if protection from damages to the shoreline and the backshore area during design storm conditions are desired goals for park improvements. These structural measures can be divided into two general categories - structures designed for shore protection and those designed for erosion control and beach stabilization.

Shore Protection

Shore protection is primarily defined as measures to protect a shoreline from direct erosion by waves, and is typically accomplished either by revetments or seawalls.

Seawall - Seawalls are vertical or steeply sloping concrete or grouted masonry (CRM) walls used to protect the land from wave damage, with use as a retaining wall a secondary consideration. The north end of the park is presently protected by a vertical CRM seawall. A seawall, if properly designed and constructed, is a proven, relatively low maintenance shore protection method requiring limited horizontal space along the shoreline. The near vertical seaward face of seawalls can cause two problems, however. Wave energy is deflected both upward and downward, and also a large amount of energy is reflected seaward. Upward deflected energy can result in overtopping by incident waves, downward deflection can cause scour at the base of the wall, and the reflected waves can carry sand and eroded material seaward, increasing the potential for erosion in front of the wall.
Revetment - A revetment is a facing of erosion-resistant material whose primary purpose is to protect a shoreline from direct erosion by waves, and is one of the surest time-proven shore protection measures. The most common method of revetment construction, and generally the most satisfactory, is to place an armor layer of stone or concrete units sized according to the design wave height over an underlayer and/or bedding layer designed to distribute the weight of the armor layer and to prevent loss of shoreline material through voids in the revetment. Properly designed revetments are durable, flexible and highly resistant to wave damage, and can often still function to a certain extent if damaged by storm waves which exceed their design criteria. The rough and porous surface and flatter slope absorb more wave energy than smooth vertical walls, thus reducing wave reflection, runup and overtopping.

In their 1981 report, the USACOE suggested a rock revetment be placed against the damaged CRM seawall in the vicinity of the existing pavilion and restrooms at the north end of the park to provide a permanent repair.

Beach Replenishment with Stabilization

Constructing or nourishing a protective beach by placing sand of suitable grain size in a particular shape along a shoreline is an effective and attractive means of protecting a backshore area, as well as for providing recreational opportunities. A natural beach slope is very effective for dissipating wave energy. However, the capacity of the waves to remove sand may be so great that it is not economically feasible to restore or nourish the beach without structural measures to stabilize it.

A beach can be stabilized by constructing an offshore, or detached, breakwater to dissipate wave energy that would normally strike the shore. Their effectiveness for erosion control is directly proportional to their wave attenuation, however a detached breakwater need not act as a complete barrier; a low elevation and segmented breakwater may dissipate enough wave energy by causing the waves to break to function effectively. The USACOE (1981) suggested as an alternative erosion control plan that two 150-foot-long segments of offshore breakwater be constructed fronting the park beach 150 feet seaward of the shore to dissipate storm wave energy and minimize wave overtopping of the beach and flooding of the backshore area. This plan would have functioned well from an engineering standpoint. However, public concern was expressed about construction offshore where recreational swimming and snorkeling occur, and the project cost was not justified by the Federal benefit-cost analysis.
Where there is a predominant direction of sand movement, such as the project site where the incident waves tend to approach the shore so as to move sand from north to south, groins can be an effective method for stabilizing a beach. Groins are structures perpendicular to the shoreline, and sand accumulates on the updrift side. This assists the beach configuration to orient itself to conform to the predominant wave crests and thus reduce sand transport.

**Reconstruction of the "Menehune" Breakwater**

Randomly scattered boulders along the edge of the shallow rock shelf, extending some 500 +/- feet across the bay mouth on the south side, have the appearance of a planned breakwater structure and have thus been dubbed the "menehune breakwater." The real origin of these boulders, or even if they are natural or placed by man, is unknown. They do, however, act to help dissipate wave energy entering the bay. The possibility of turning the menehune breakwater into a properly designed protective structure so that it can more effectively protect the park has been raised. It is not considered practical or reasonable, however, to actually do this, for reasons including the following:

- the present location of the Menehune breakwater, near the bay mouth and edge of the shallow bay shelf, is exposed to large breaking wave heights, thus a properly designed structure would have to be big, and expensive to construct; and

- the existing boulders are located offshore, and work in this area would require a construction causeway from shore for access by heavy equipment, with the resultant potential for significant impacts to the marine environment.

**RECOMMENDED SHORELINE IMPROVEMENTS**

The recommended shoreline improvement plan consists of both shoreline management and structural measures as follows:

1. relocation of park facilities such as pavilions and restrooms to locations backshore of the beach as delineated by the interior CRM wall, and increasing the interior CRM wall elevation to prevent storm wave flooding;
2. construction of rock revetment seaward of the existing shoreline CRM wall at the north end of the park, to provide permanent shore protection for this area, and with a short extension of the revetment to act as a groin to stabilize the north end of the main beach; and

3. beach nourishment to provide shore protection and increased recreational area, and construction of a groin at the far south end of the beach to stabilize it and reduce future sand loss.

The conceptual design and layout of these recommendations is presented in the following report section.
CONCEPTUAL PLAN OF SHORELINE IMPROVEMENTS

GENERAL PLAN ELEMENTS

The general elements of the recommended shoreline improvements consist of the following:

- Rebuilding the backshore CRM wall and increasing its crest elevation to 8 feet above MLLW, in order for it to provide flood protection against storm waves which overtop the shoreline, and relocation of park facilities such as pavilions and restrooms to locations behind the wall;

- Construction of rock revetment seaward of the existing damaged shoreline CRM wall at the north end of the park to provide adequate shore protection for this area;

- Construction of a short stub groin extending from the new revetment at the north end of the park to stabilize the north end of the main beach;

- Construction of a groin at the south end of the park to stabilize the sand beach; and

- Placement of sand to increase the recreational beach area and improve its ability to dissipate wave energy.

The location and layout of these improvements is shown on Figure 4, and typical cross-sections are shown on Figure 5.

BACKSHORE CRM WALL IMPROVEMENTS

The existing backshore CRM wall has a crest elevation varying between about 6 feet and 8 feet, and is in poor condition, with gaps in the wall and portions that are simply piles of rock. Storm wave runup on the beach can reach elevations of 7 to 8 feet above MLLW, and it is not reasonable to build the beach high enough to prevent overtopping during extreme events, thus the backshore wall is necessary to prevent flooding by waves which overtop the beach crest. A minimum backshore wall elevation of 8 feet above MLLW is recommended.
ROCK REVETMENT

The existing vertical CRM seawall located at the north end of the park, seaward of the restrooms and fishpond, is in poor condition. Large holes near the toe of the wall extend all the way through to the backside, resulting in unsupported and unstable wall sections and erosion of park land through the voids in the wall. Rather than trying to repair the seawall as it is presently built, it is recommended that a rock revetment be constructed seaward of it to provide improved shore protection. The revetment would be constructed of 800 to 1,200 pound armor stone, two stones thick, over an underlayer of spalls to 200 pound stone. The side slope would be 1V:1.5H, and the crest elevation would be 6 feet MLLW. Total revetment length would be approximately 250 feet. The flatter slope and porous revetment will result in lower wave runup height and less splash over than presently occurs with the vertical seawall face.

NORTH STUB GROIN

A 60-foot-long stub groin would extend from the rock revetment at the north end of the park, to help stabilize the north end of the beach. The groin would be constructed of 1,000 to 2,000 pound armor stone, over a core of 50 to 200 pound stone. The crest elevation would be 6 feet MLLW, crest width about 6.3 feet (3 stones), and the side slopes would be 1V:1.5H.

SOUTH GROIN

A 160-foot-long groin would extend from the existing seawall at the south border of the park to act as a trap for the southerly transport of sand along the beach, and to help stabilize the beach and prevent the loss of sand. The groin would be constructed of 1,000 to 2,000 pound stone, over a core of 50 to 200 pound stone. The crest elevation would be 6 feet MLLW, crest width about 6.3 feet, and side slopes would be 1V:1.5H. This crest elevation may overtop during extreme wave conditions, as would the north stub groin, however it would still dissipate sufficient wave energy to function satisfactorily.

In addition to beach stabilization considerations, the location and alignment of the north and south groins was influenced by field observation of the nearshore marine environment.
The groins are located on very shallow (-1 foot MLLW) and smooth lava rock substrate, on which no significant coral or other marine flora and fauna was noted.

BEACH ENHANCEMENT

Beach sand can be placed between the two groins to augment the sand already present, to improve the recreational sandy beach area and to permit the beach to better dissipate wave energy. The tendency for incident wave energy to move sand toward the south will result in a crescent beach shape at the south end of the park held in place by the south groin. The sand would be placed with a crest elevation of 6 feet MLLW, and a slope of 1V:8H. The median grain size should be 0.5 mm to 1.0 mm, with the larger grain size being preferred for this location. The salt and pepper appearance of the existing sand at this site would permit the use of either or both white calcareous sand and black lava basalt sand as beach fill without changing the appearance of the park.

In actuality, the proposed plan will not increase the beach area so much as help stabilize the existing beach and protect the backshore area. The majority of the increase in useable beach area will result from relocating facilities such as the south pavilion to a backshore area. The actual new sand needed should not exceed 200 to 300 cubic yards.

ESTIMATED CONSTRUCTION COST

The following concept-level cost estimate is based on an update of cost data presented in the USACOE report of 1981, which presents unit costs for similar types of materials and construction. These estimates are intended only as approximate costs for preliminary planning purposes.

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REFERENCES


