

CHAPTER 99

HONOLULU REEF RUNWAY DIKE

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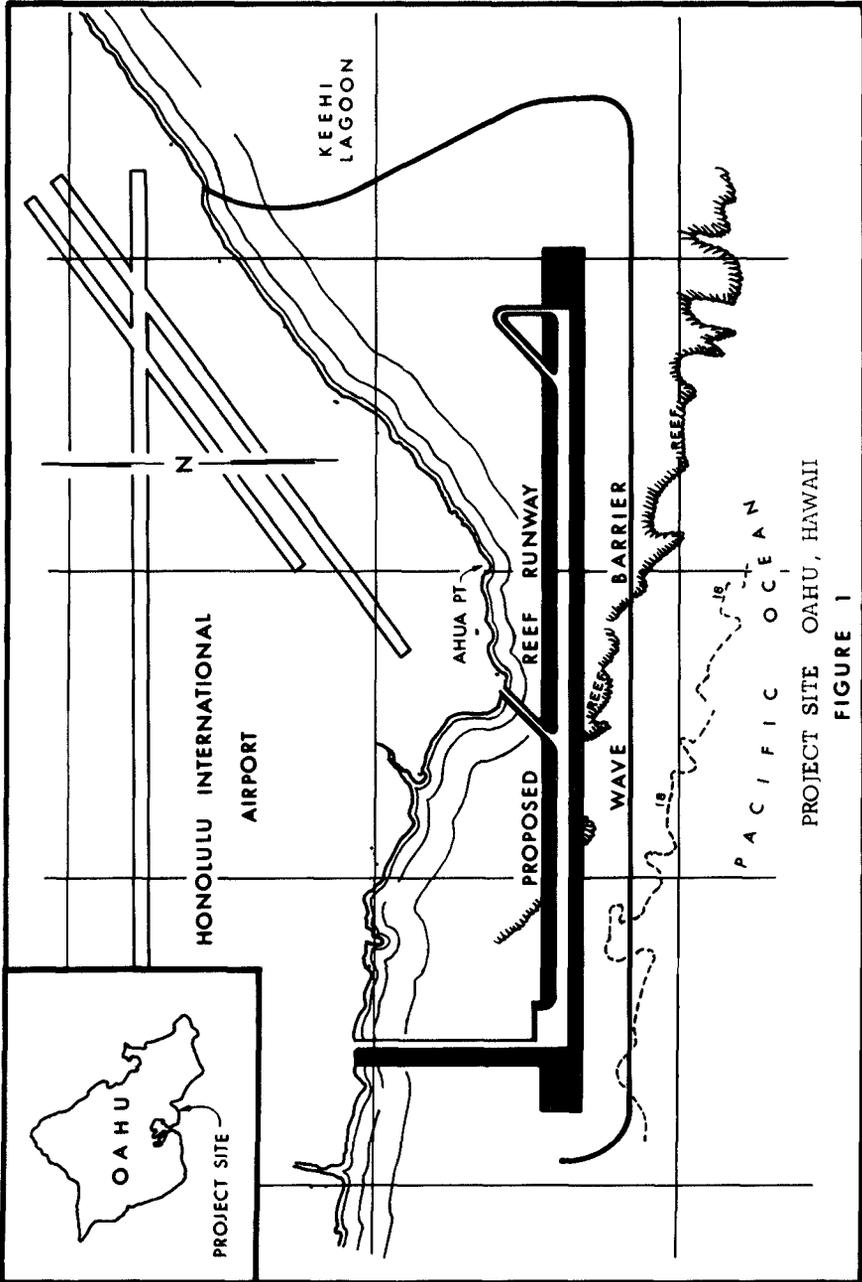
ABSTRACT

Criteria for design of a wave barrier to protect the proposed Honolulu International Airport Reef Runway from breaking waves were developed in wave flume model tests. Structures with tribar and quarry-stone armor units placed in single and multiple layers, on homogeneous and composite slopes, were subjected to both overtopping and non-overtopping breaking waves. Data on wave runup, armor unit stability, quantities of overtopping water, and transmitted wave heights were obtained using a 1:50 bottom slope, which modeled the irregular coral bathymetry seaward of the proposed structure. The model to prototype scale ratios ranged from 1:5 to 1:35.

Model Tests indicated that the weight of armor units placed below one-third the water depth may be three-fourths that of the units located near the water surface. It was noted that the maximum wave runup was 1.8 times the water depth fronting the structure. Data were obtained concerning quantities of overtopping water and transmitted wave heights over the low barriers. The study augments available criteria for economical design of structures subjected to breaking waves.

INTRODUCTION

The anticipated greater volume of future air traffic and larger aircraft imposes the need for an additional runway at the Honolulu International Airport, Oahu, Hawaii. The most practical site for the runway, shown in Figure 1, is on a coral reef, adjacent to the existing airport. The proposed runway location and alignment will not conflict with use of existing runways, but it will eliminate hazardous takeoffs and landings over the city, and will alleviate the present noise problem in urban areas.



PROJECT SITE OAHU, HAWAII
FIGURE 1

The proposed wave protection for the reef runway would be situated in water ranging in depth from about one to twenty-five feet below mean low lower water (MLLW). Approximately one-third of the site would be located on a wide, flat coral reef, one to five feet deep and only 200 feet of the deeper portion would be in water greater than 25 feet deep. The bottom slope is approximately 1:50 to seaward. The design water surface of +3.5 feet MLLW allows for concurrence of a fairly high tide and storm surge. Southern hemisphere storms generate waves that approach the site with deep-water wave heights up to 15 feet, with 16-second periods. On rare occasions, local storms generate about 25-foot waves with shorter periods. A refraction analysis and a three-dimensional hydraulic model study⁷ indicated that waves are sufficiently high to break seaward of all reaches of the site at least ten hours per year.

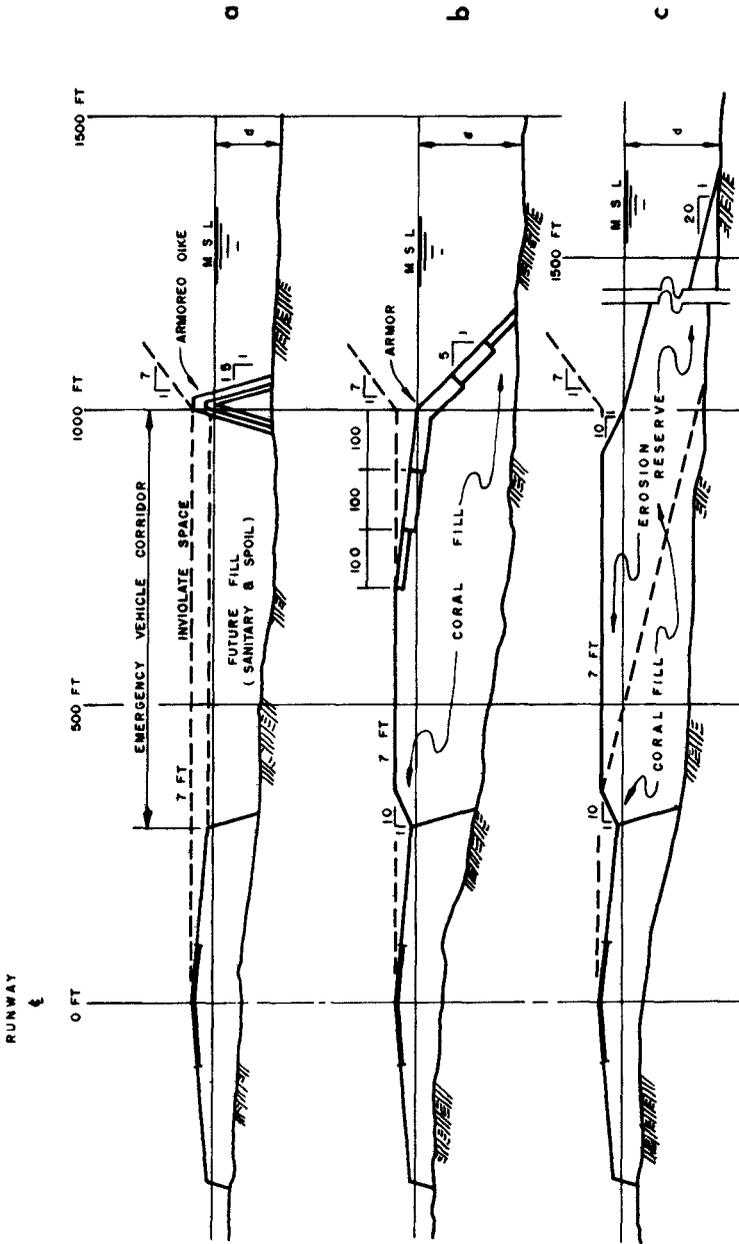
The purpose of this study was to develop the most economical system to protect the runway from storm waves. The runway, with 1,000-foot overruns, will be 14,000 feet long, 200 feet wide with 200-foot shoulders, and will have a centerline elevation of about seven feet above mean sea level (MSL). Air-space criteria require that there be no obstacles higher than the runway centerline for a distance of 1,000 feet from the centerline. Beyond 1,000 feet there is to be a side-clearance zone slope of one vertical to seven horizontal.

The apparent dearth of design data for structures subjected to breaking waves prompted the model investigation described herein. The testing objectives were divided into four categories, they were to determine wave runup, armor unit stability, quantities of overtopping water, and transmitted wave heights over low structures. In order to provide a basis for correlation of results with previous work, preliminary tests and procedures used by other investigators^{3, 8, 11} were made with non-breaking waves. Procedures were then modified as necessary to obtain the design breaking wave conditions.

The first three of six general concepts considered for the reef runway wave protection system are shown in Figure 2. These concepts are a) a seaward wave barrier which will contain future fill to provide access for emergency vehicles, b) a wide berm with flat armored slopes, and c) a wide berm with a flat beach slope similar to the Sand Dam of Europort in the Netherlands. Figure 3 shows the three other concepts which are d) a combination of a low seaward barrier and an artificial beach along the runway shoulder, e) a combination of a low seaward barrier and an armored levee along the runway, and f) a detached non-overtopped breakwater.

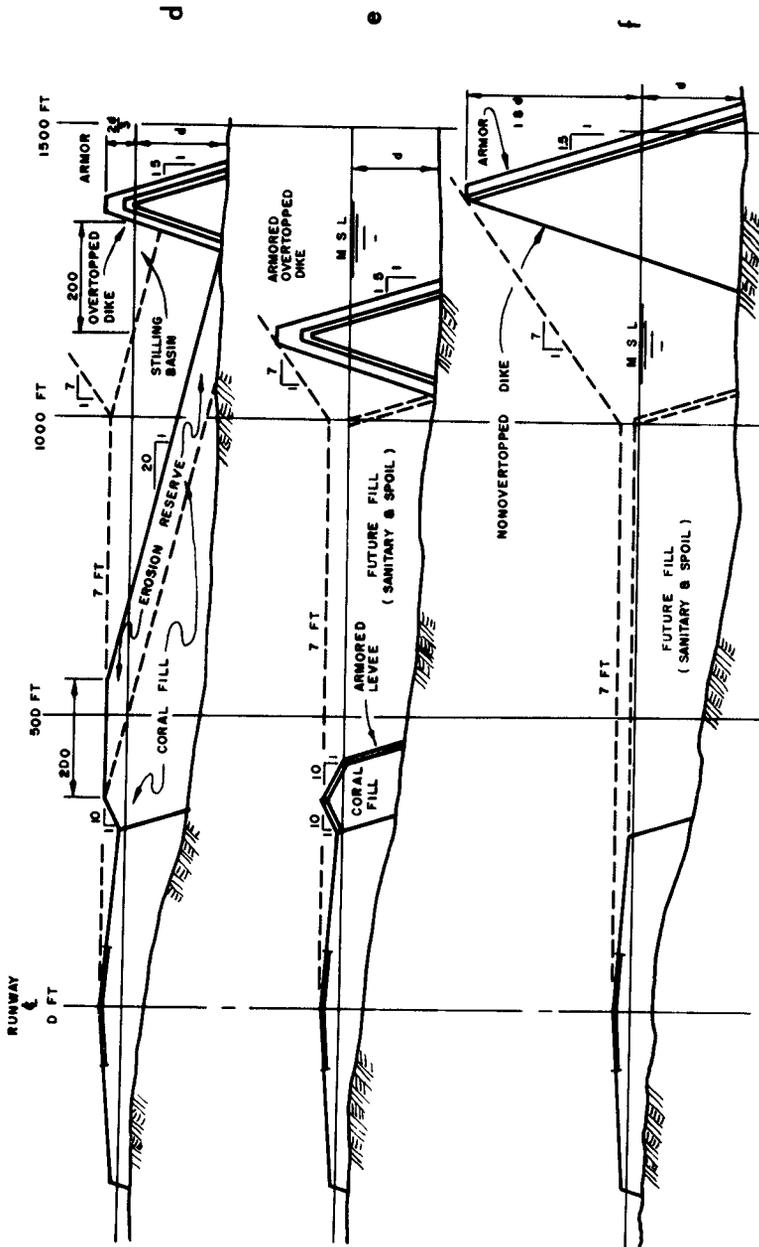
THE MODEL

The wave flume used for the two-dimensional tests was 180 feet long, four feet wide, and six feet deep. The walls were made of Transite and glass paneling. The glass panels were arranged to allow viewing



ALTERNATIVE WAVE PROTECTION CONCEPTS FOR HONOLULU REEF RUNWAY

FIGURE 2



ALTERNATIVE WAVE PROTECTION CONCEPTS FOR HONOLULU REEF RUNWAY
FIGURE 3

of the test section from one side of the flume. A plywood floor on a 1:50 slope simulated the offshore bathymetry. This slope extended approximately 65 feet seaward from the model test section.

The test sections were subjected to waves generated by a plunger-type wave generator. The wave generator was powered by a 20-HP electric motor coupled to a hydraulic pump. The hydraulic pump drove a high-torque, variable-speed hydraulic motor, which rotated an adjustable crank attached by a connecting rod to the plunger.

Wave heights were measured by resistance wave gauges, the signals were displayed on a light beam oscillograph recorder. Wave run-up elevations on structures were measured visually. Overtopping quantities were trapped in a one-foot wide box located in the center of the test section, at the core level, on the basin side of the structure. The trap door on top of the box opened and closed to capture the overtopping water of a selected series of waves.

The model breakwater sections had stone and tribar armor units. The armor stone was hand-shaped to simulate the type of stone that would be used in the prototype. The mean weight (saturated and surface-dried) of these units was 1.043 lb, with a standard deviation of 0.045 lb. The mean specific weight was 177.03 lb/cu ft, with a standard deviation of 2.83 lb/cu ft. However, the specific weight of the prototype stone may be as low as 165 lb/cu ft. The applicable model scales were determined by the WES (Hudson)¹ formula to range from 1.5 to 1.25 for these stones.

The model tribars were made of a mixture of concrete and barite sealed in a resin coating. The mean weight of these units was 2.10 lb, with a standard deviation of 0.023 lb. The specific weight of these units was 145.74 lb/cu ft, with a standard deviation of 1.05 lb/cu ft.

The armor stone and tribar underlayer stone mean weights were 0.055 lb and 0.189 lb, respectively, with specific gravities of 1.69 lb/cu ft. The impermeable core was made of a mixture of fines to gravel, 100 percent of the material passed a three-fourth inch sieve.

Test procedures were based upon the precedent set by related model studies^{3,8,9,11} and were modified as necessary to simulate prototype breaking waves. The incident wave heights were measured by a wave gauge located five feet seaward of the toe of the 1:50 bottom slope (70 feet from the test section). The incident wave heights were measured before reflections from the structure set up a standing wave. This wave height was used to estimate the deep water height, H_0 , by application of a shoaling coefficient. Another gauge was located one to five feet from the toe of the structure to measure the breaking-wave height. In order to generate a consistent wave of known height, the wave generator was stopped before wave reflections from the structure returned to the generator. The water was allowed to still before the generator was restarted.

Test sections were subjected to breaking waves to determine their stability under design conditions. The general procedure was to subject the structure to a series of small, non-breaking waves for a short period of time to allow the units to settle. Then the structure was subjected to the highest wave obtainable with the given depth to toe. The duration represented five to six hours of prototype wave attack. If the structure remained stable, the water depth was increased and a larger wave was allowed to attack the structure. This was done to determine if failure would occur with a slightly larger wave. Thus, a degree of safety was indicated.

Two types of armor stone placement were used in the models: in a single layer with the long axis perpendicular to the slope, and in multiple layers in a random fashion. Tribar armor units were either placed in a single layer arrangement or in multiple, random layers. Placement in the model was done to simulate, as closely as feasible, the placement in the prototype. For the majority of the tests, tribars were randomly placed in two layers below minus ten foot elevation. This placement was adopted in the model because of difficulties anticipated in prototype placement in murky water on an irregular bottom. Above minus ten feet, the units were placed upright with bars in contact with adjacent units.

The wave runup, R_u , as a function of deep-water wave height, H_o , wave period, T , and water depth d , was determined by visual observations. Runup, the elevation above the still-water surface to which a wave rises on a structure, was tested by increasing by increments wave height and period until the range of prototype conditions was covered. For a constant period, the wave height was increased from a small, non-breaking wave to a wave that broke sufficiently seaward of the structure to ensure that the maximum runup was observed. The wave attack was allowed to stabilize on the structure, and then the average runup of the next six to 15 waves was recorded. The number of waves used in the average runup was limited to the time it took for the first wave to reflect off the generator and return to the test section. When waves broke on or seaward of the structure, there was a large variation in the runup, therefore, the maximum runup value was recorded. In order to obtain a clear definition of the effect of location of the breaking wave, the generated wave height was increased by small increments when the wave started to break at the toe of the structure.

The quantity of water overtopping structures was trapped to determine the rate of over-swash. The procedure was to allow the wave attack to stabilize, then the over-swash of several waves was trapped and the quantity was measured. The procedures followed in obtaining the design wave in the overtopping tests were the same as described in the runup tests.

The water swashing over the top of the structure generated a transmitted wave height, H_T . This wave was measured by a gauge on the basin side, located five feet from the heel of the structure. The levees protecting the runway, as shown in Figures 3d and 3e, were modeled and also acted as wave absorbers for the transmitted waves.

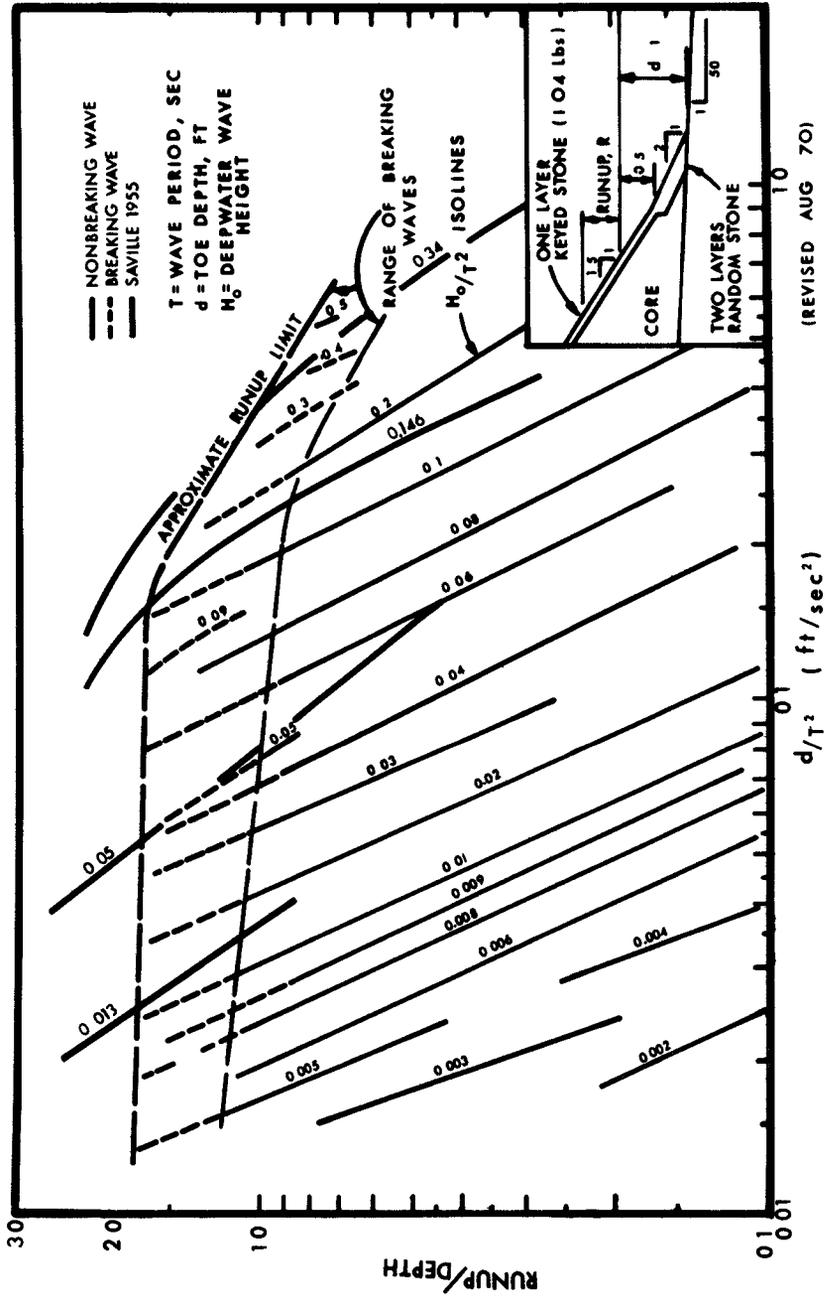
TESTS AND RESULTS

Runup on a structure subjected to breaking waves is a primary concern when considering a non-overtopping structure. For this reason, a typical dike section was designed and tested to determine the maximum runup. The inset in Figure 4 shows the test section. The core was impermeable and the material used has been described previously. Two layers of underlayer stone were randomly placed over the core, then a double layer of 1 04-lb stone was randomly placed on a 1 2 slope from the floor to -0 5 feet. From this point up to +3 5 feet, a single layer of fitted stone was placed on a 1 1 5 slope. The water depth was -1 0 foot at the toe of the structure.

One hundred and seventy-six tests were run on this structure to determine the maximum runup under prototype conditions. The results are shown in Figure 4 as a plot of the runup divided by depth, R_u/d vs d/T^2 for isolines of deep-water wave steepness, H_o/T^2 . These isolines are lines fitted through data points of non-breakers (solid line) and breakers (dashed line). The range of breaking waves is outlined by a line drawn approximately where the waves start to break at the toe of the structure. An increase in wave steepness approaches an upper limit of runup which is also drawn in Figure 4. This plot shows a rapid increase in runup as the wave period is increased until $d/T^2 = 0 15$ and $H_o/T^2 = 0 1$.

Saville⁸ previously conducted tests on a similar structure fronted by a 1 10 beach slope. Saville's results were plotted with the Reef Runway data for comparison. It was noted that the primary difference between the results of the two sets of data was that the runup Saville measured was about 45 percent greater than that of the Reef Runway data. Apparently, the steeper beach slope in Saville's experiment allowed a nearly 50 percent larger wave to break near the structure. This increase in breaker height, attributed to the difference in the bottom slope, was noted in the experimental work of Iversen² et al. The effect of the 1 2 slope at the toe of the Reef Runway test section does not account for such an increase in R/d when comparing the two structures using the composite slope method⁹. These runup data provide the designer of the Reef Runway project with an upper limit to which breaking waves will run up on a single-layer, stone-armor structure with a 1 1 5 slope.

Stability coefficients, K_D , were computed for each test section subjected to the breaking waves. When damage occurred to the armor, the degree was noted. The WES (Hudson)¹ formula was used in the analysis. The results indicate that a) published values of K_D for

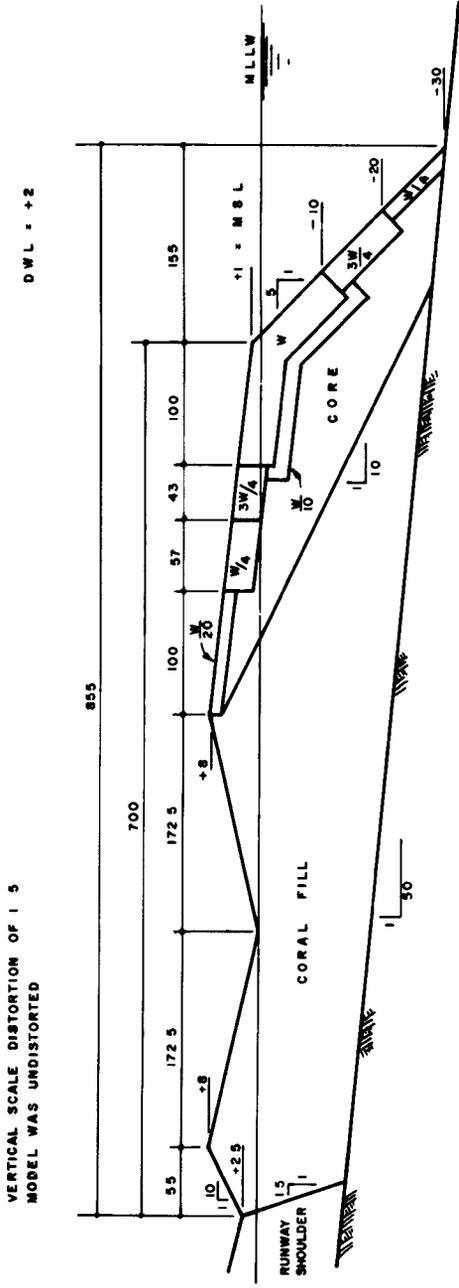


SHALLOW-WATER WAVE RUNUP ON A RUBBLE SLOPE
 FIGURE 4

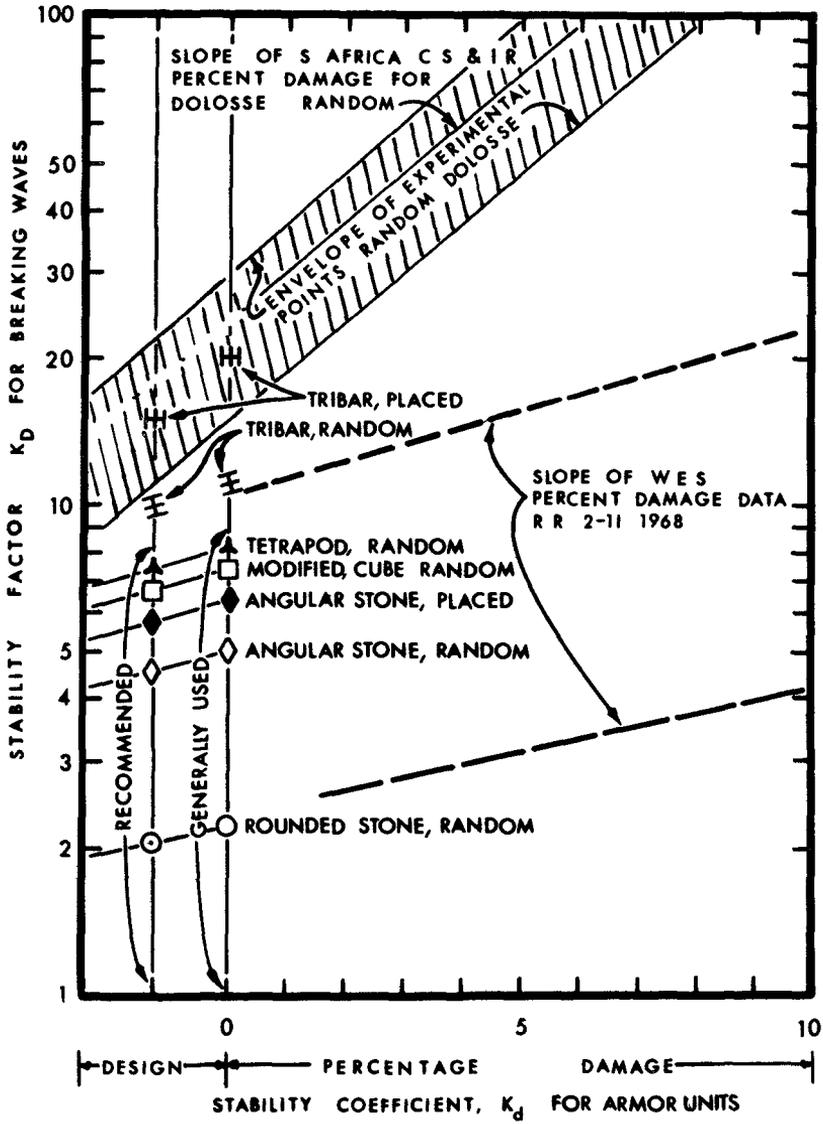
breaking waves are conservative, b) keyed and fitted armor is several times more stable than loosely-placed armor, c) displacement of one armor unit does not lead to sudden massive failure, in fact, the armor tends to heal unless it is grossly underweight, and d) the weight of armor may be reduced by one-fourth at depths below one-third of the toe depth and by three-fourths below the two-thirds depth as shown in Figure 5. A summation of stability coefficients for breaking waves is shown in Figure 6. The ordinate of the author's recommended design values are identified. However, it has been the general practice to design on the borderline of damage or even for a percentage of damage from infrequent waves.

Economically, the runway elevation should be kept as low as possible. A rise in water level behind the wave protection structure could affect the operation of the runway. In order to evaluate the effect of crest elevation upon the rate of overtopping water, series of tests were made on typical sections with various crest elevations. Results from two representative tests are given in Figures 7 and 8. The data is given in prototype dimensions and the test sections are shown in the insets. The linear model scale for these data was 1:15.2. The plots of these data are shown in Figure 7 for a crest elevation of 13.80 feet and in Figure 8 for a crest elevation of 9.35 feet. The rate of overtopping water per linear foot of crest, Q (cubic feet per second per foot of crest), is given as a function of deepwater wave height for isolines of wave periods. For a given wave period, the rate of overtopping rises rapidly with an increase in H_0 until the wave breaks at the toe of the structure. An increase in H_0 causes the wave to break seaward of the toe and the rate of overtopping approaches a maximum for a given wave period. The plots also indicate the effect of wave steepness. It was noted that it was inversely proportional to the amount of overtopping.

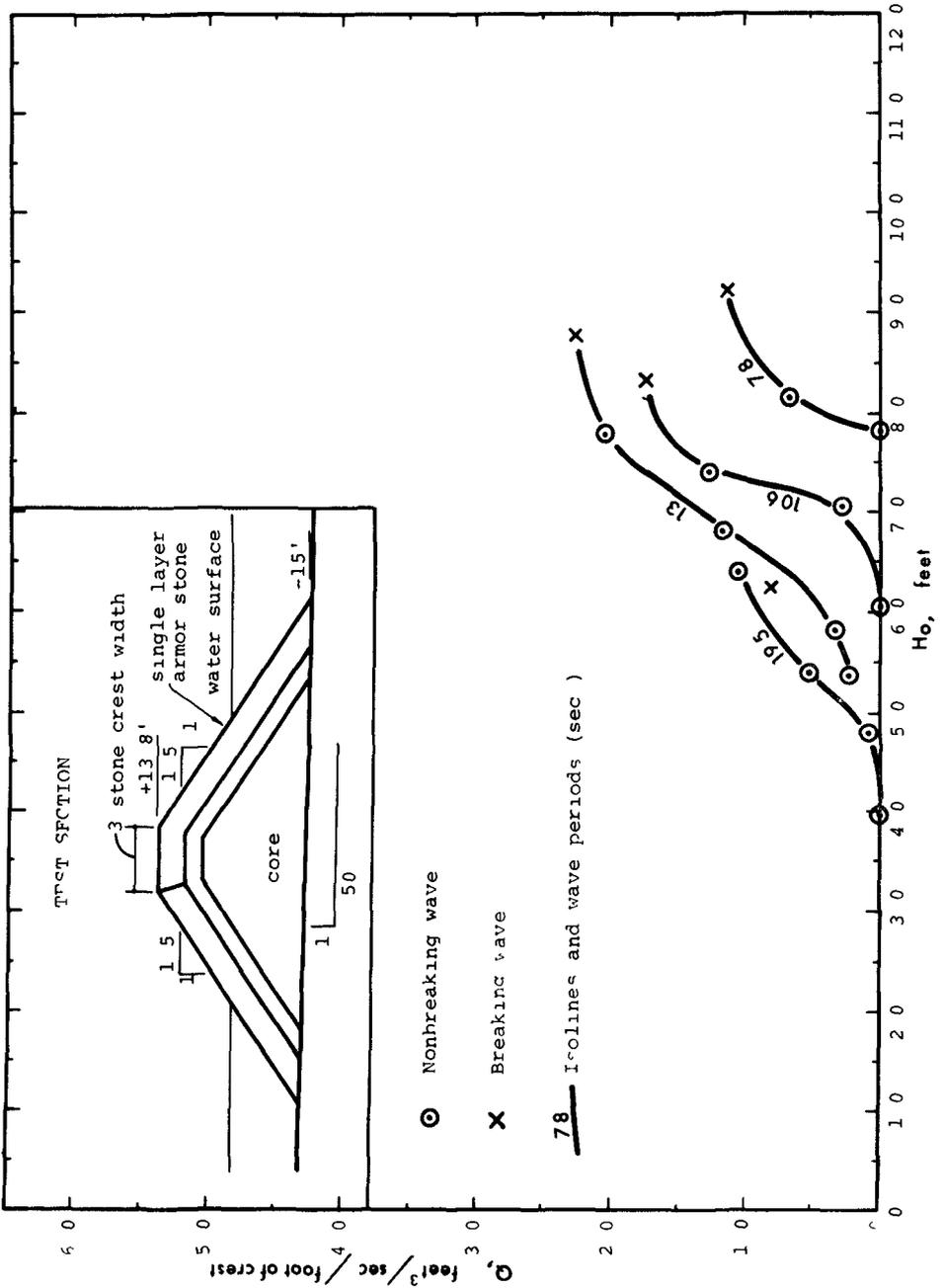
Wave swash over a structure generated waves in the basin behind the structure. When waves break just seaward of the structure, the maximum wave is generated in the basin. The relation of the effect of crest elevation is shown in Figure 9 as a plot of transmitted wave height, H_T , as a function of deep water wave height, H_0 , for two wave periods. The linear model scale was 1:15.2, and the test section in prototype dimensions is shown in Figure 9. The plots indicate that once the wave has broken, an increase in incident wave height does not produce an increase in transmitted wave height. An important observation noted during the testing program concerned the period of the transmitted wave, overswash of the incident wave generated a number of transmitted waves. This had a significant effect upon the runup on the runway levees. The size and depth of stilling basins also produced noticeable effects upon the transmitted wave. Due to these and other complicating factors, no general relations were developed for runup on the runway levees.



ARMOR WEIGHT RELATION & LOCATION ON STRUCTURE
FIGURE 5

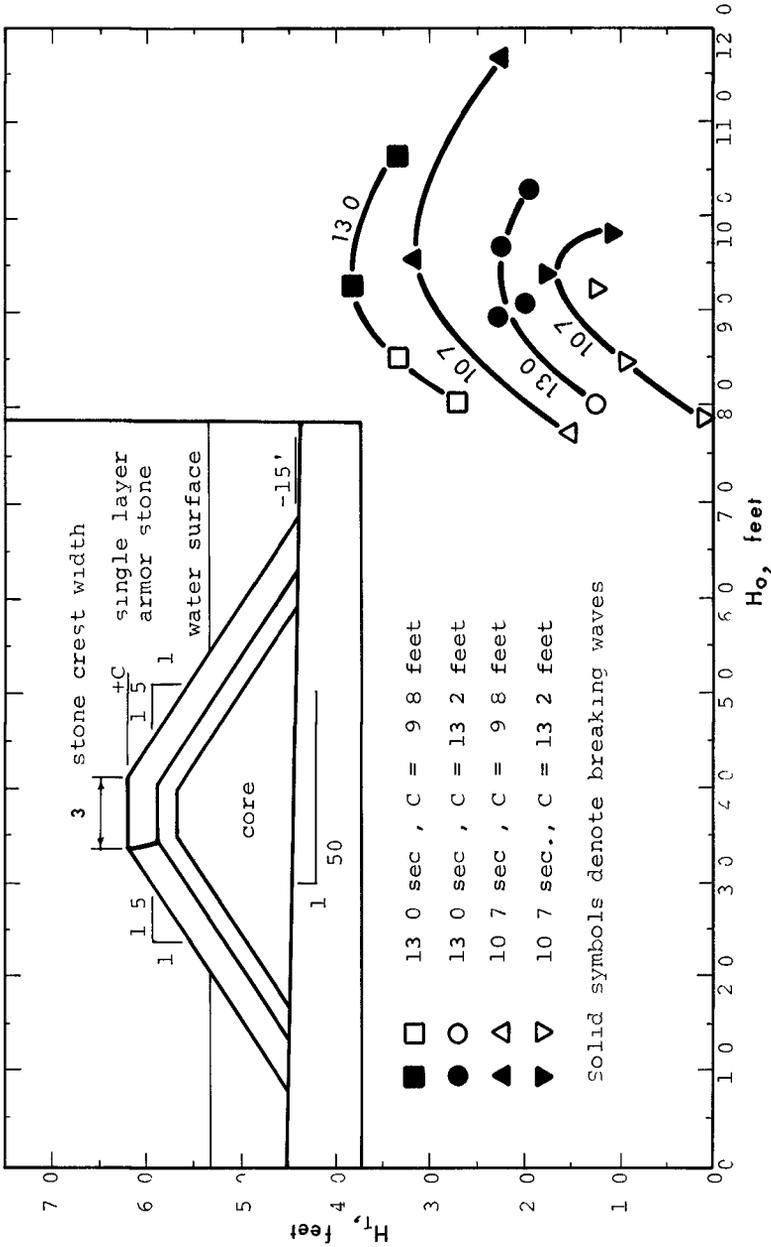


Note See references 4, 5, 9, 11



RATE OF OVERTOPPING WATER OVER LOW BARRIER

FIGURE 7



HEIGHT OF TRANSMITTED WAVE OVER LOW BARRIER

FIGURE 9

Forty-five sections were tested in this study. The results of these tests and comparative costs will be the subject of another paper. An example of one of the sections developed to attenuate runup is shown in Figure 10. Preliminary tests indicated that this high-void structure is stable and reduces runup.

CONCLUSIONS

A large scatter in the data was observed in the model testing of rubble structure with breaking waves. Even though a reasonably consistent wave train was generated, each wave broke in a different location, thus changing the effect of the wave upon the test. Since prototype waves are not consistent, the location of the breaking point varies even more. For this reason, maximum data points for the breaking waves were observed in the model tests.

Tests indicate that the maximum runup of a breaking wave for conditions shown on Figure 4 is 1.8 times the water depth fronting the structure. This was valid for the range of d/T^2 and H_o/T^2 tested.

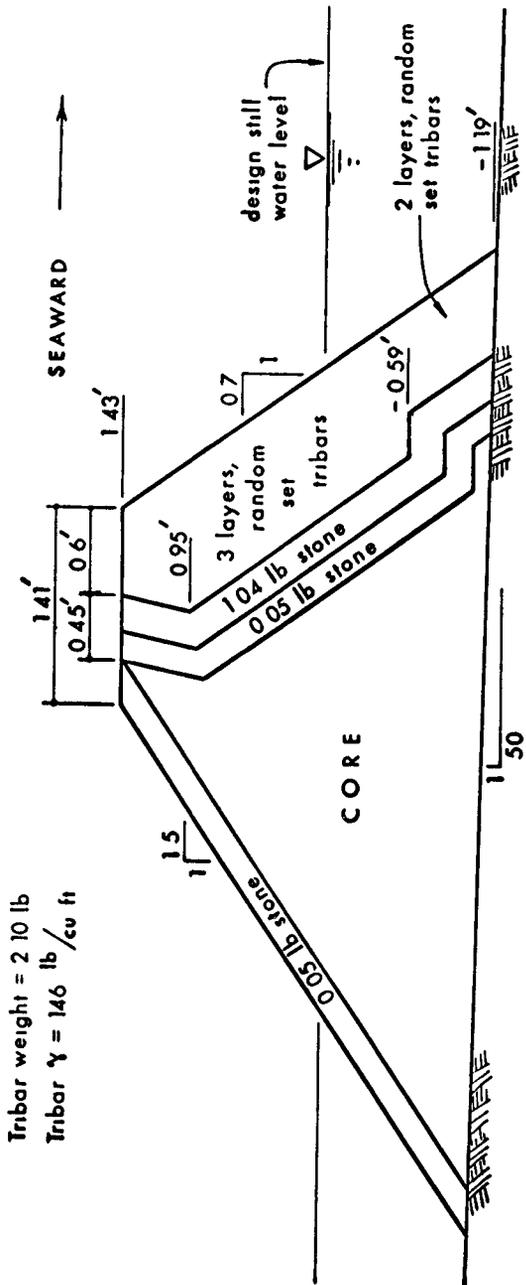
The tests confirmed the findings of previous investigations that the type of placement of armor is a major variable. Only skilled and experienced technicians can build rubble models which will yield fairly consistent results. Also, the designer and construction inspectors should be fully aware of the relationship between armor stability and placement. The tests confirmed that placement is as important a factor as the weight of the armor units. Specifically, the required weight of loosely placed stone may be twice that of a well-placed stone.

It is apparent that there is an overwhelming number of variables involved in analyzing breaking waves on complex rubble-mound structures. General design criteria can be developed for only a few unique conditions. It therefore appears that model tests should be made for all important rubble structures subject to breaking waves.

Scale effects¹⁰ should be considered when applying these data to prototype design.

ACKNOWLEDGEMENTS

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NONOVERTOPPING DIKE WITH SMALL CROSS-SECTIONAL AREA

FIGURE 10

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