STABILITY OF A LOW-COST SHORE PROTECTION DEVICE

by

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ABSTRACT

Low-cost shore protection devices provide coastal communities with a worthy alternative to conventional shore protection. An experimental study to investigate the stability of one of these low-cost shore protection devices, the mortar-filled fabric bag, was conducted. Results indicate that the stability of mortar-filled fabric bags is comparable to that of quarrystone for at least one breakwater configuration. More laboratory tests along with monitoring of existing low-cost shore protection projects are necessary to accurately determine design criteria for low-cost alternatives.

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INTRODUCTION

The use of low-cost shore protection devices in the United States has been limited although a well-maintained low-cost shore protection device has the potential to provide a coastal community with the same degree of protection as a conventional shore protection device. A shore protection device can be defined as any device which will reduce the loss of beach material from the coast. One major reason for the limited use of low-cost shore protection devices is that design specifications for low-cost alternatives have not yet been determined. Therefore, before low-cost devices will be widely used, laboratory experimentation and on-site inspection of low-cost structures must be carried out. The intent of my research was to determine the stability of one low-cost shore protection device, the mortar-filled fabric bag.

CONVENTIONAL OR LOW-COST SHORE PROTECTION

Definitions

In the United States the majority of shore protection projects utilize conventional devices, whereas, in many of the developing countries the majority of shore protection projects are constructed with low-cost devices. The difference between these two approaches to shore protection is illustrated in Fig. 1. Low-cost devices require

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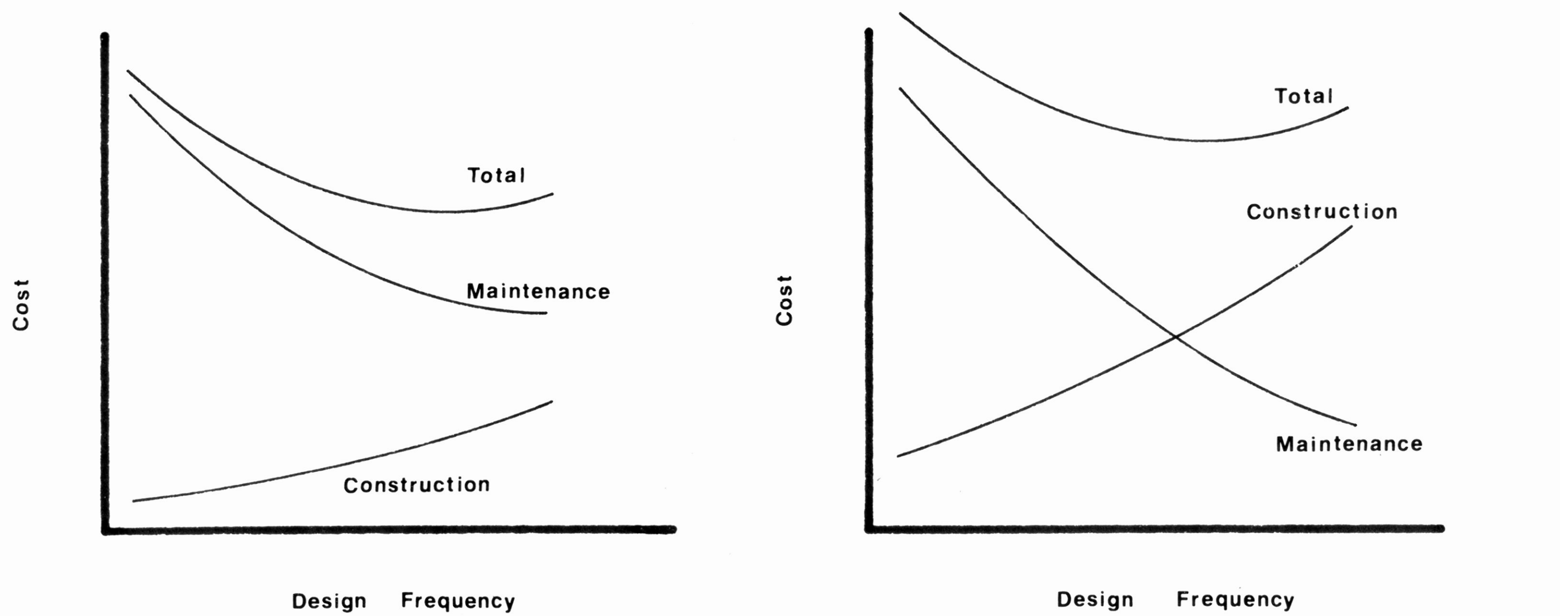


Low - Cost

Maintenance — High
Initial Cost — Low

Total = Less

Conventional

Maintenance — Low
Initial Cost — High

Total = More

Fig. 1. Conventional Versus Low-Cost Shore Protection.

smaller construction costs than conventional structures because low-cost devices are constructed directly on the beach in their final position using materials which are locally available. The construction of low-cost structures usually involves the pumping of sand, mortar or concrete into large fabric bags using a portable cement mixer and a pump. Conventional shore protection structures are much more expensive to build because the large units of stone or concrete must be first shipped to the site then placed using heavy construction equipment, such as cranes or barges. A comparison of maintenance costs indicate a higher maintenance cost for low-cost devices. The low-cost structure usually has a shorter design life and therefore must be rebuilt more frequently. The costs associated with this increased maintenance are usually small enough to keep the total cost of a low-cost design below the total cost of a conventional design.

Shore Protection in the United States

Conventional shore protection projects are the most common form of shore protection in the United States today. The U. S. Army Corps of Engineers, the government agency responsible for the majority of coastal engineering projects in the United States, has followed this conventional approach with good results for many years. The Corps' adherence to this plan has helped those communities where the economic benefits of a project have exceeded the costs required to build the structure. On the other hand, many communities have been denied a project because the proposed cost exceeded the benefits gained by the project. In other words, the highly-developed tourist beaches will receive governmental help while the less-developed coastal communities and the private landowners receive nothing.

A U. S. Coastal Community

The city of Surfside, a community located forty miles southwest of Galveston and on the Texas Gulf Coast, has an erosion problem (Barnett, 1975; Dawson, 1976; Riley, 1977). The beach at Surfside has experienced rapid erosion in the last ten years resulting in the loss of several beach homes as seen in Figs. 2 and 3. The citizens of Surfside are demanding that some action be taken to prevent any further deterioration. The Galveston branch of the Corps of Engineers has looked at the Surfside erosion and determined that there is no chance of an expensive conventional solution to the dilemma there. The reason is simple, the loss of several more beach homes will not cause a large enough overall loss to justify the expenses involved in protecting the shore with some type of structure. Since a conventional structure is not feasible due to low benefits, Surfside has three alternatives. The simplest alternative is to do nothing, letting nature take its course (Brazosport Facts, 1977). The next easiest solution is to have the Federal government pay the residents for their homes and relocate the families. The only alternative which might satisfy both the citizens of Surfside and the Corps of Engineers would involve the use of a low-cost shore protection device.

Fig. 2. Surfside Beach Looking Eastward From the Jetty

Fig. 3. Close up of Surfside Beach Home

Low-Cost Shore Protection in Developing Countries

The use of low-cost structures is not a new idea but is, in fact, a way of life in many of the developing countries (Porraz, 1977). These countries typically have a large unskilled labor force and a lack of working capital. The low-cost solution therefore appeals greatly to these countries because this type of project will employ many people. In Mexico, for instance, several different and innovative devices have been developed and used. Sand-filled nylon units, concrete-filled nylon bags and several types of slope stability mats have been used in Mexico for seven years with good results (Porraz, 1977). The future looks bright for low-cost shore protection as several international organizations and the industrial nations are studying the "Mexican approach" for application throughout the world.

Corps of Engineers Shoreline Demonstration Project

In the United States, the Corps of Engineers has embarked on a 5-year demonstration project, the National Shoreline Erosion Control Program, which is looking strictly at low-cost shore protection (Edge, 1977). Sixteen demonstration sites spread along the Great Lakes, Atlantic, Gulf of Mexico, and Pacific shores have been designated and are in varying stages of construction (Combes, 1977). A wide range of devices from seawalls and breakwaters to vegetation and institutional solutions will be tested. The goal of the program is to provide the private landowner with the latest in low-cost erosion control. All of the demonstration sites are in sheltered coastal waters where the significant wave height is less than six feet. Laboratory wave tank studies of low-cost structures seem to be the next logical step for the Corps of Engineers to take in order to determine design and construction criteria for low-cost devices.

THE IMPORTANCE OF STABILITY IN THE DESIGN OF COASTAL STRUCTURES

A major problem facing the coastal engineer in the design of a breakwater is the determination of the armor unit weight. The weight of the armor units must be heavy enough for the structure to be stable in the wave environment along the coast. Several empirical equations have been developed which relate the optimum weight of the unit to the unit weight and specific gravity of the units, the breakwater slope, the design wave height and a dimensionless number which takes care of everything else affecting a breakwater. The most widely used equation was developed by Hudson (1959) after extensive wave tank tests. The Hudson Stability Equation has the following form :

$$W_r = \frac{\gamma_r H^3}{K_D (S_r - 1)^3 \cot \alpha}$$

in which

W_r = weight of individual armor units

γ_r = specific weight of the armor units

H = design wave height at breakwater site

K_D = experimentally determined damage coefficient

S_r = specific gravity of armor units relative to the water at the breakwater site

α = angle of the breakwater slope measured from the horizontal.

The equation is easily used if K_D is known for the armor unit used.

The value of the stability coefficient, K_D, or more appropriately called the damage coefficient, depends on the need of the coastal engineer. The engineer usually wants to know the K_D value associated with the maximum wave height measured at the breakwater site prior to construction that will not damage the armor layer. The removal of more than 1% of the total number of armor units is considered damage (Hudson, 1959). Once the "no damage" wave height has been determined the effects of higher waves on the structure is determined from a plot of percent damage versus wave height developed from model tests for each type of armor unit. This graph gives the coastal engineer an idea of the damage which will occur for a given wave height. The engineer also knows the frequency of occurrence of wave heights at the proposed site and therefore can determine the degree of damage to the breakwater for a known probability of occurrence of a wave height.

EXPERIMENTAL STUDY

Preliminary Considerations

The initial task in the study was to determine a weight of the individual armor units which would be suitable for testing in the wave tank. Hudson's equation was used with the following values: K_D = 2.1, H = 3 inches, slope = 1:1.5, γ_r = 127 pounds/cubic foot, and S_r = 1.98 to fresh water. The K_D value used is the one for smooth quarrystone armor units because it was thought that the value of K_D

for mortar-filled fabric bags would be similar. The design wave height was chosen from experience gained in previous studies in the wave tank (Dominguez, 1971). The weight of the armor unit came out to be .67 pounds. Once the weight was known a bag size which would yield this weight was required. Several fabric bags of varying dimensions were fabricated and a bag size of four inches by three inches provided the necessary weight.

Breakwater Cross Section

The wave tank used in the test measures 120 feet long, 2 feet wide, and 3 feet deep and is shown in Fig. 4. The breakwater was placed with the crown 95 feet in front of the wave generator and was constructed of three layers as shown in Fig. 5. The core layer consisted of limestone between 1/4 inch and 1/2 inch in diameter. The underlayer consisted of a wide variety of stones all between 3/4 inch and 1 inch in diameter. Two layers of armor units were planned up to the level shown in Fig. 5. Preliminary calculations indicated that less than 200 bags would be required to cover the underlayer up to the design height. In order to save time and insure a quality bag, 200 bags were ordered from a national company. The breakwater was designed to be non-overtopping using the design wave height of 3 inches.

Breakwater Construction

The breakwater was built on the solid horizontal bottom of the wave tank. The rocks were placed with a shovel and hand-shaped to fit

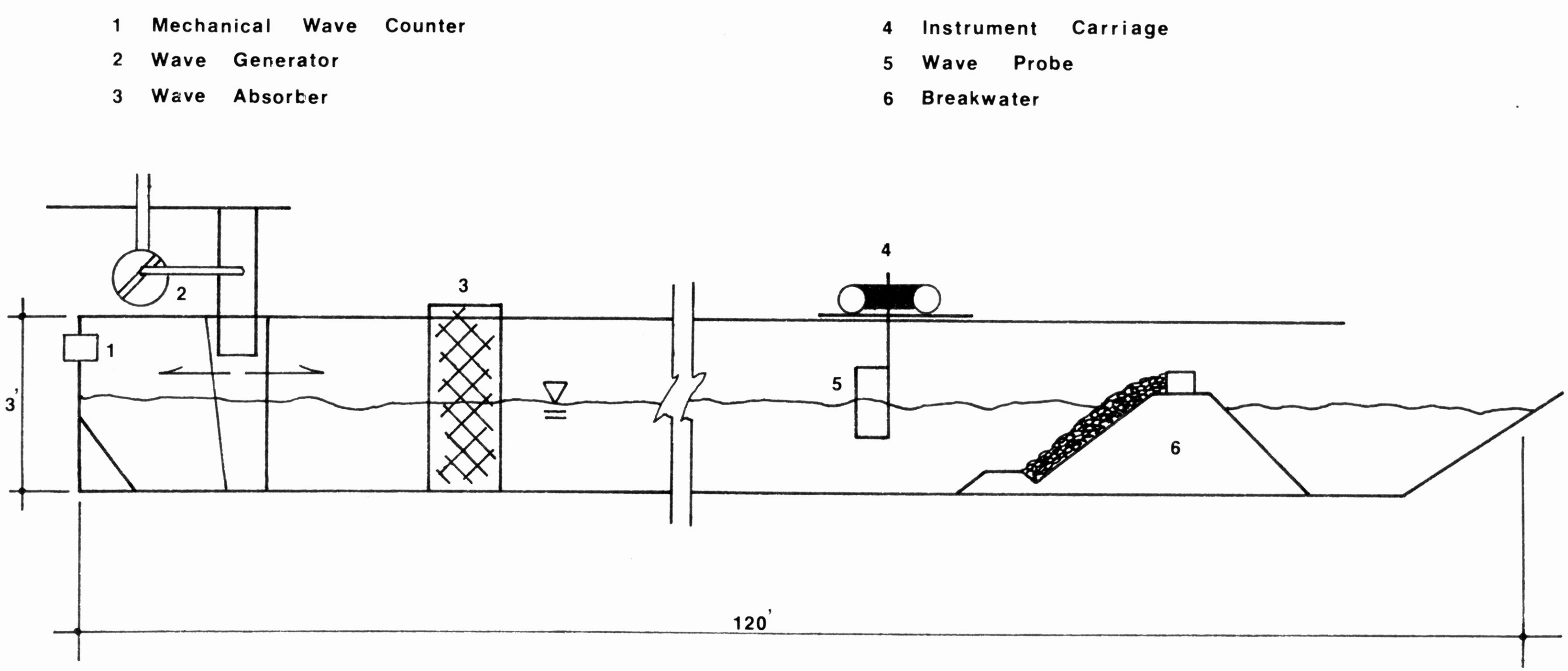


Fig. 4. Schematic of Test Facility

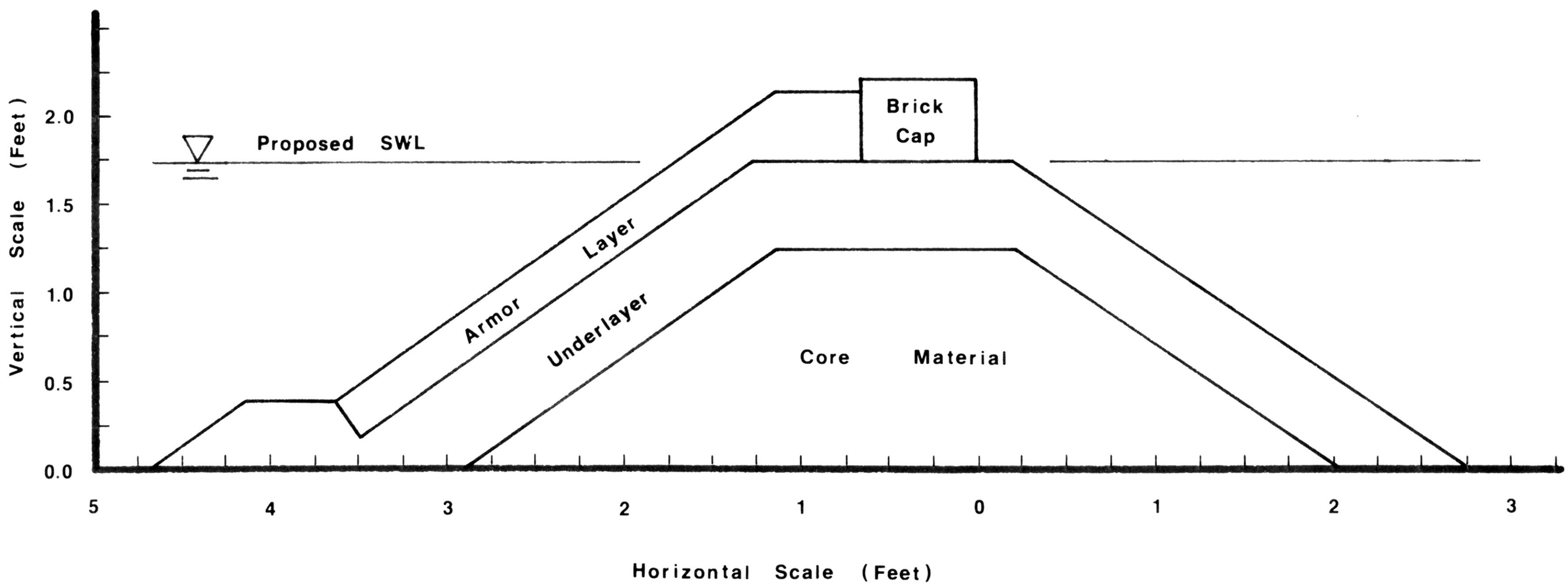


Fig. 5. Cross Section of Proposed Breakwater.

the guiding lines drawn on the side of the tank. The armor units were filled with mortar made using a sand:cement:water ratio of 3:1:1 in a portable cement mixer. The sand had a specific gravity of 2.60, the cement was Portland cement with a specific gravity of 3.15, and the water was fresh tap water. The bags were filled one at a time, the drawstring was tied closed and each bag placed immediately in the tank. The armor layer was built from the toe upward one row at a time with the long axis of the bags placed perpendicular to the length of the tank as shown in Figs. 6 and 7.

The placement of the bags while soft is probably the most important element in the stability of the breakwater as the units harden together and are difficult to pry apart. All 200 of the bags were put in place as more could have been used. However, since no more bags were readily available the tests were run with the cross section as shown in Figs. 6 and 7. The flume was then filled to the desired depth and the experiment begun.

Test Apparatus

The wave tank is equipped with a mechanical, pendulum-type wave generator as shown in Fig. 4. A wave absorber, consisting of wire mesh, was placed 15 feet in front of the wave generator in order to insure uniform waves in the flume. Instrumentation consisted of a mechanical wave counter, movable instrument carriage and a capacitance-type wave probe and strip recorder.

Fig. 6. Breakwater Prior to Testing

Fig. 7. Breakwater Prior to Testing

TEST CONDITIONS AND PROCEDURES

Test Procedure

The purpose of the test was to determine the stability coefficient for the mortar-filled fabric bag armor units. At the outset of the testing, only the breakwater slope was known. The weight, unit weight and specific gravity of the armor units were not measured until completion of the testing since the units were placed in the breakwater while soft. The determination of the wave height corresponding to the "no damage" condition involved increasing the wave heights in the tank until the maximum wave height which damaged less than 1% of the armor units was determined and exceeded. Each test was run at a given wave generator setting and water depth. The duration of each test depended on the response of the breakwater armor units to the wave environment. If no damage was detected or expected for a given wave height the test was stopped and the wave generator adjusted to yield higher wave heights in the next test. The incremental change in wave height between tests was intended to be small in order to insure accurate determination of the "no damage" wave height.

At the outset of each test various parameters were recorded in order to insure reproducibility in the future. The stroke-length and the dial reading of the wave generator, the initial count of the mechanical wave counter, and the water depth were recorded. Each test was timed coincident with the start of the wave generator using a digital stopwatch. The capacitance-type wave recorder was turned on prior to the passage of the first wave and left on for approximately five minutes. A test was terminated in most cases at a point when the breakwater and wave environment stabilized to a position where there existed little chance of further damage. The duration of the test was determined at the stopping point in the test. The number of waves hitting the breakwater was also known from the mechanical wave counter. The wave period for each test was determined by dividing the duration by the number of waves.

Determination of Wave Heights

The accurate determination of wave height is essential if the stability coefficient determined from this investigation is to be compared with known values from previous tests. The first difference between earlier studies (Hudson, 1959) and this one involves the duration and method of testing. Hudson ran each test for a cumulative period of 30 minutes stopping the wave generator when the first reflected wave from the breakwater reached the generator. The tank was then allowed to stabilize prior to continuation of the test. This procedure eliminated the reflection and beating interactions which were immediately evident in my study.

The wave heights used in the present experiment were measured after the breakwater had been removed from the tank. The wave probe was placed at the breakwater site in the same depth of water as in the original test. The stroke-length and dial reading of the wave generator were then set, the wave generator turned on and the wave heights measured. The wave height was determined from the wave record and used to develop the percent damage versus wave height plot.

Wave Overtopping and Water Depth

The original breakwater cross-section had been designed to be a non-overtopping breakwater with a design wave height of 3 inches. The design crown height of the breakwater was never reached as the number of armor units was not sufficient to build it this high. For the new crown height of 23 inches and a water depth of 18 inches, a wave height of 3 inches would not overtop the structure. Once the tests began, however, it was quickly evident that a wave height significantly higher than 3 inches would be necessary to damage the armor layer. As the wave heights were increased, overtopping did occur with subsequent movement of the brick cap and a partial destruction of the backslope. The armor layer was undamaged by this overtopping but it was decided to lower the water level in order to minimize overtopping and maximize the wave energy to the front slope of the breakwater. The duration of several tests was cut short because significant damage to the backslope was occurring and structural failure of the entire breakwater was feared.

TEST RESULTS

Summary of Tests

Tests were conducted until the armor layer had been damaged to the point where rebuilding would be pointless. Once a unit had fallen it could be returned to its original position but not to its original strength and stability due to the placement of the bags while soft. The replaced armor units were almost sure of failing again in a subsequent test because of the difficulty in replacing the units in exactly the same orientation as before. The study was terminated after test 13, in which 17% of the armor units had failed, with a breakwater configuration as shown in Figs. 8 and 9.

The basic philosophy followed in the experimentation was to increase the wave height at a given wave period until damage to the armor layer occurred. Table 1 contains the summary of test data and it can be seen that the wave period was not significantly changed until test 9. A change was made to a shorter period in order to prevent overtopping and to maximize the wave energy to the cover layer. The decrease in water depth in tests 11, 12, and 13 was made for the same reason. Two wave height measurements were recorded in the test data; H corresponds to the wave height measured at the breakwater after it had been removed from the tank; H' is the wave height measured while the tests were in progress.

After the completion of the tests, the 34 bags which had been moved were weighed in their saturated test state both out of and submerged in water. Table 2 contains the results of these measurements which yielded an average weight of .88 pounds, a specific gravity of 2.18 and a unit weight of 136 pounds per cubic foot. The average weight of .88 pounds is significantly larger than the assumed preliminary value of .67 pounds which helps justify why higher wave

Fig. 8. Breakwater After Completion of Testing

Fig. 9. Breakwater After Completion of Testing

Table 1. Summary of Test Data

Test No.	Wave Period (Sec)	H (in)	% Failure	H' (in)	Length of Test (min:sec)	No. of Waves	Water Depth (in)
1.	2.42		0	.36	2:20.2	62	18
2.	2.54		0	.82	5:20.0	126	18
3.	2.49		0	1.39	3:29.2	84	18
4.	2.49		0	2.03	3:04.3	74	18
5.	2.54		0	2.78	10:29.7	248	18
6.	2.53		0	4.48	32:48.7	779	18
7.	2.55		0	4.78	8:21.3	197	18
8.	2.58		0	5.83	20:24.7	475	18
9.	1.73		0	2.71	7:23.5	257	18
10.	1.80	3.99	0	6.87	34:06.7	1139	18
11.	1.80	4.54	2	7.85	21:49.7	727	17
12.	1.85	7.00	6.5	9.48	5:46.5	187	17
13.	1.70	6.87	17	9.16	19:39.7	693	16

Table 2. Weight and Specific Gravity of Mortar-filled Armor Units

	Armor Unit Weight (Lbs)	Specific Gravity of Armor Units
Mean	.88	2.18
Standard Deviation	.08	.04
Median	.88	2.18
Range	.74-1.03	2.10-2.27

heights were needed to damage the armor layer.

Test Observations

Tests 1-5. Wave heights were of insufficient magnitude to damage the breakwater. A confused sea state due to wave reflections was noted in all of these tests. Waves were non-breaking and no overtopping occurred.

Test 6. Overtopping occurred throughout the test. No movement was detected in armor layer or underlayer stones in the toe.

Test 7. Overtopping continued with no damage to the armor units. Oscillations of underlayer stones detected in the toe.

Test 8. Extensive overtopping was observed. There were oscillations and movement of several underlayer toe stones. No motion was detected in armor layer.

Test 9. Wave period and stroke-length decreased in this test with no damage or overtopping.

Test 10. Wave overtopping caused brick movement and damage to the backslope. Slight oscillation of 2 armor units along the glass boundaries on each side of the tank at the downrush line. The backslope was rebuilt prior to the next test.

Test 11. Water depth decreased to 17 inches with an increase in stroke length. Failure of the four previously oscillating armor units next to the glass occurred. There was no motion in the main body of the armor layer indicating that boundary effects were probably responsible for the movement of the 4 bags. No motion was detected in the underlayer toe stones. The four armor units were numbered and replaced in their respective positions.

Test 12. Same test conditions as in test 11 except that the wave absorber was removed. Severe overtopping knocked bricks off crown and caused substantial damage to the backslope. Armor units on each side failed with a total number of 13 being displaced. Both layers of armor units displaced at one point with resulting extrusion of underlayer stones. Armor layer seemed to stabilize after a few minutes while the backslope was being severely damaged. The test was halted to avoid failure of the backslope. Replacement of the 13 displaced units was accomplished with a significant loss of stability. The backslope was rebuilt and the brick cap replaced.

Test 13. The stroke-length and the wave period were decreased in order to subject the breakwater to more of a deep water wave and to prevent the severe overtopping of the previous test. The water depth was also decreased to 16 inches. Significant damage to the armor layer occurred as 34 bags were displaced. Overtopping was still a problem causing damage to the backslope. Some of the waves broke on the structure. Failure of under-layer stones also detected.

Upon completion of this test the tank was drained and the damage was surveyed. No attempt was made to rebuild the breakwater because of the number of armor units displaced. Test sequence was complete.

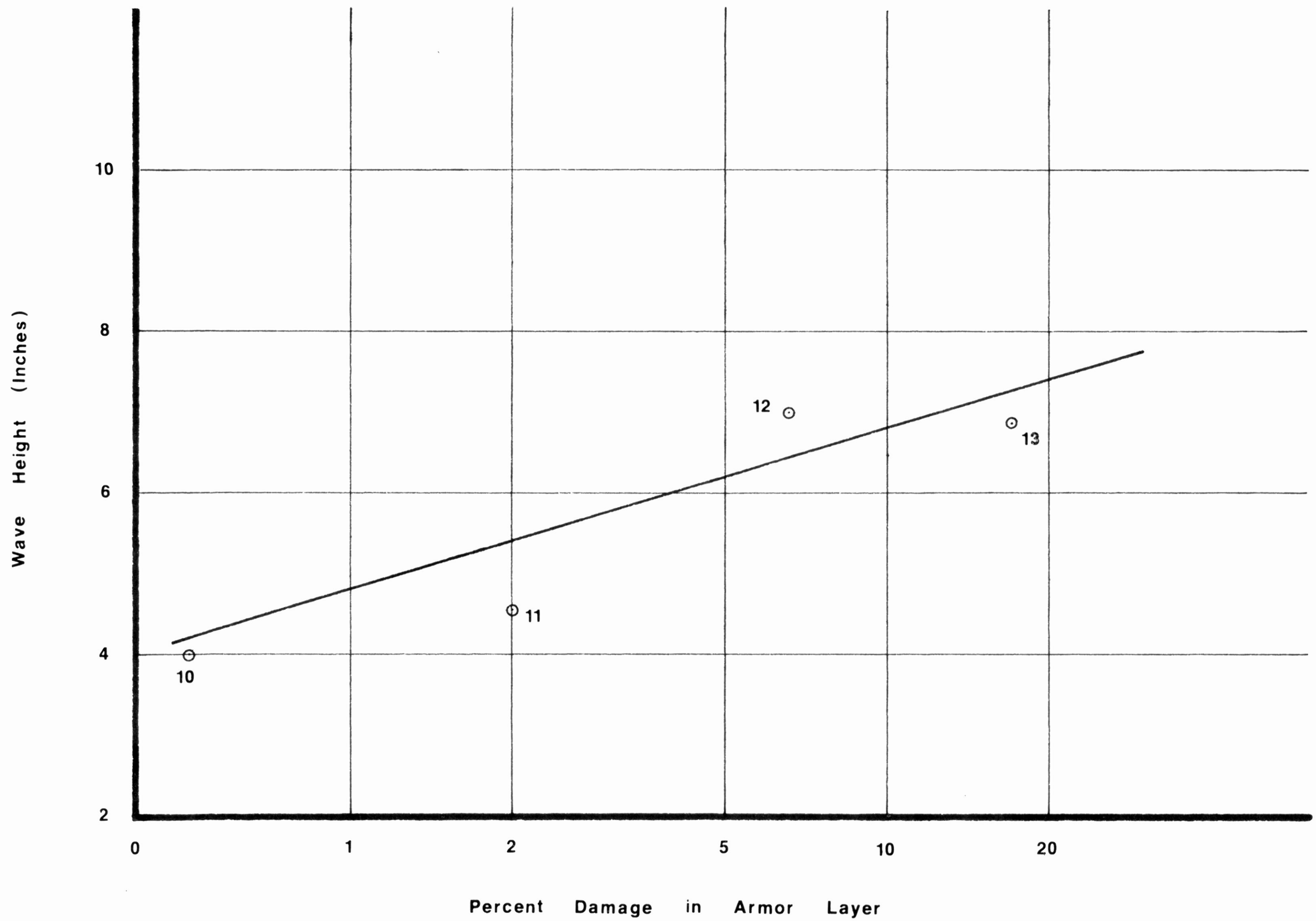
SUMMARY OF RESULTS

A plot of wave height versus percent damage of the armor units was developed from the test data and is shown in Fig. 10. The percent damage was calculated by dividing the number of units which failed in each test by the total number of units (200) and multiplying by 100. To determine the value of K_D associated with the "no damage" criteria the wave height at 1% damage was needed. The determination of the placement of the straight line on the graph was based on the following:

1. Actual wave heights (H') measured while the breakwater was in position are significantly higher than the wave heights measured after the breakwater was disassembled (H). The reason for this is due to the addition of reflected wave energy off the breakwater to the wave energy sup-plied by the wave generator. Hudson (1959) prevented the superposition of wave energies by stopping the wave generator when the first reflected wave reached it and the difference between wave heights in his testing must have been less.

2. The damage associated with test 12 would probably have been greater had the test been run longer. The higher

Fig. 10. Percent Damage in Armor Unit
Versus Wave Height.


Wave Height (Inches)
10
8
6
4
2
0
1
2
5
10
20
Percent Damage in Armor Layer
10
11
12
13

damage corresponding to the lower wave height in test 13 is also a reason for expecting a greater percent damage in test 12.

3. The wave height corresponding to zero damages must be higher than 4 inches which is the wave height corresponding to test 10, the last test in which no damage occurred. If possible boundary effects were ignored the zero damage wave height may exceed 4.5 inches.

The line was drawn as shown and the "no damage" wave height corresponding to 1% damage was 4.8 inches.

The stability coefficient was then calculated using Hudson's equation solved for K_D with H = 4.8 inches, W_r = .88 pounds, S_r = 2.18, γ_r = 136 pounds per cubic foot, and cot α = 1.5. Using these numbers, the stability coefficient is equal to 4.0. Suggested K_D values taken from the U. S. Army Coastal Engineering Research Center's _Shore Protection Manual_ for the no damage and minor overtopping criteria for several of the traditional armor units are shown in Table 3. The experimental K_D value obtained for the mortar-filled fabric bag armor units compares favorably with quarrystone. Much more research is necessary in order to more accurately determine the value of K_D.

Table 3. K_D Values for Conventional Armor Units

Armor Unit	n*	Placement	K_D**	Slope
Smooth rounded quarrystone	2	Random	2.4	1.5 - 3.0
Rough angular quarrystone	2	Random	4.0	1.5 - 3.0
Tetrapods	2	Random	8.3	1.5 - 3.0
Dolos	2	Random	25.0	2.0
Tribars	2	Random	10.4	1.5 - 3.0

Reference, U. S. Army Coastal Engineering Research Center, Shore Protection Manual, Vol. 2, 1975, pp. 1-170.

*n is the number of units comprising the thickness of the armor layer.

**K_D values are for the structural trunk of the breakwater for no damage and minor overtopping.

CONCLUSIONS AND RECOMMENDATIONS

Much more laboratory testing is required to accurately determine the stability of mortar-filled fabric bag armor units. This preliminary study has shown that the stability of mortar-filled fabric bags is comparable to quarrystone in at least one breakwater configuration. Future studies should investigate the effects on stability of varying the breakwater slope; using different sizes, shapes and placement techniques for the armor units; running the tests in a variety of wave tanks to determine the scale effects; and subjecting the breakwater to breaking waves as well as non-breaking waves.

It has been shown that low-cost shore protection devices provide coastal communities with an alternative which has previously been ignored. More widespread use of low-cost devices is dependent on the development of design specifications for each type of unit. The stability of one low-cost shore protection device, the mortar-filled fabric bag armor unit, was shown to be similar to that of quarrystone. More laboratory testing and on-site inspection of low-cost devices must be carried out. When this happens, communities such as Surfside will be waiting for a solution to their erosion problem.

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