CHAPTER ONE HUNDRED EIGHTY SIX

BREAKWATER ARMOR DISPLACEMENT THRESHOLDS: A POSSIBLE CORRELATION WITH CUMULATIVE WAVE ENERGY

Daniel L. Behnke, A.M. ASCE[1]
Fredric Raichlen, F. ASCE[2]

Abstract

An extensive program of stability experiments in a highly detailed three-dimensional model has recently been completed to define a reconstruction technique for a damaged breakwater (Lillevang, Raichlen, Cox, and Behnke, 1984). Tests were conducted with both regular waves and irregular waves from various directions incident upon the breakwater. In comparison of the results of the regular wave tests to those of the irregular wave tests, a relation appeared to exist between breakwater damage and the accumulated energy to which the structure had been exposed. The energy delivered per wave is defined, as an approximation, as relating to the product of H^2 and L, where H is the significant height of a train of irregular waves and L is the wave length at a selected depth, calculated according to small amplitude wave theory using a wave period corresponding to the peak energy of the spectrum. As applied in regular wave testing, H is the uniform wave height and L is that associated with the period of the simple wave train. The damage in the model due to regular waves and that caused by irregular waves has been related through the use of the cumulative wave energy contained in those waves which have an energy greater than a threshold value for the breakwater.

Introduction

To properly design a breakwater and its armor and to evaluate any damage that a structure has received, it is necessary to characterize the wave conditions in a meaningful way. Most often breakwater armor stability has been evaluated using hydraulic models exposed to regular waves. More recently irregular waves have been used in the laboratory to simulate the actual wave climate to which the breakwater would be exposed. Usually the irregular waves which are used for design purposes are characterized by an energy density spectrum. The spectrum may be described in general terms by using: a statistical measure of the wave height, the period of the maximum energy density of the spectrum, and a description of the shape of the spectrum. There are other

[1] Consulting Engineer, ARCTEC, Incorporated, Columbia, MD, U.S.A.
[2] Professor of Civil Engineering, W. M. Keck Laboratory of Hydraulics and Water Resources, California Institute of Technology, Pasadena, CA, U.S.A.

variables which may be important for a design, such as the wave direction, the storm duration, the stillwater level, and the wave groupiness. Each of these descriptors of the irregular wave train can change with time during a particular storm, and they can also occur in different combination over the lifetime of a structure. Therefore, the design wave conditions for a structure must be defined by a combination of wave characteristics which will be the most serious to the particular breakwater under consideration.

For breakwater armor experiments which are conducted with regular waves, the degree of damage of the breakwater is usually a function of the number of waves of a given wave height to which the breakwater is exposed. In other words, a group of waves of a given height and duration may slightly damage a structure whereas an increase in duration may destroy it. In most tests which are conducted, the experimental conditions include a range of wave periods and stillwater levels to more realistically account for changes in the prototype wave climate in addition to wave duration. The design wave height which is used in regular wave tests often is defined in terms of a statistical height related to the ocean waves such as the significant wave, i.e., the average of the highest one-third waves in a wave train, or some other higher wave average; the latter might be used to incorporate a higher degree of conservatism into the design. The shape of the armor pieces, their weight, and the seaward and landward slopes of the breakwater are chosen based on the model results so that under exposure to regular waves of a given height an acceptable amount of damage occurs. This may range from no movement of units to a significant displacement of armor units depending upon economics and the function of the structure.

In any event, the height of the regular waves which would be used in laboratory model tests generally is "numerically equal" to a statistically defined design wave height. (The term "numerically equal" is used to emphasize the difference between a wave height which characterizes a train of irregular waves and the height of waves in a periodic train). One has to be careful in this definition because, for example, a group of regular waves with height H has twice the energy of a group of irregular waves where heights follow a Rayleigh distribution with a significant wave height equal to H. In addition, it is recalled that the irregular wave train could include waves more than twice the height of the significant wave as well as waves which are much smaller. Therefore, a priori, it is not clear how the results of breakwater model tests with regular waves can be used to define the design where the actual breakwater will be exposed to a train of complex waves.

Many of the questions which have been raised could be avoided by the use of irregular waves in two-dimensional and three-dimensional models of coastal structures. This method of testing is becoming increasingly common and important since a variety of wave spectra can be used to simulate the design wave conditions at a site. The spectra can be varied throughout a test program to reproduce the growth and decay of waves from a storm, and the shape of a specturm and the frequency of the maximum energy can be varied also to examine these effects on the stability of the breakwater. A major difficulty with this technique is the number of experiments required to cover the possible combinations of wave heights, periods, and growth and decay characteristics which would be combined to give the worst conditions. In three-dimensional models additional experiments would be required to evaluate the effect of wave direction.

The resistance of breakwater armor to wave damage has been related to the wave height without regard for the wave period in the commonly used equations of Irribarren and Nogales (1953) and Hudson (1959). The basis for these equations has been described by Raichlen (1975) where the simplifications made in their developments are summarized. More recently experimental data have demonstrated the effect of wave period or wave length on armor stability. A summary paper published by the Permanent International Association of Navigation Congresses (1976) lists sixteen different formulae of which twelve use a term proportional to H^3, two use a term proportional to H^2L, and two use a term proportional to H^2T; where H is the design wave height, L is the wave length, and T is the wave period. Thus, the importance of wave length has been incorporated in several design approaches.

In this paper the results of a limited series of experiments with a three-dimensional hydraulic model of a breakwater armored with Tribars will be used to suggest a possible correlation of the stability of the breakwater to a measure of the energy in groups of regular and irregulars waves. If this is valid for a wider range of conditions than those tested, such a method may be able to resolve some of the problems of correlating the results of regular wave testing with those from irregular wave programs.

The Hydraulic Model

The hydraulic model from which these results were obtained was a three-dimensional model of the west and east breakwaters at the Diablo Canyon Nuclear Power Plant where an extremely complex offshore bathymetry was carefully reproduced in an undistorted model built to a scale of 1 foot in the model corresponding to 45 feet in the prototype. The breakwater is armored with a single layer of pattern place Tribars with weights of 21.6 tons (19.6 metric tons) and 36.8 tons (33.4 metric tons) and slopes of 1.5 horizontal to 1 vertical on the landward side of the trunk of the breakwater and 2.25 horizontal to 1 vertical on the seaward face of the trunk; the terminus cone has a slope of 3 horizontal to 1 vertical. The interested reader is referred to Lillevang, Raichlen, Cox and Behnke (1984) for the details of the breakwater cross-section, its construction, a complete description of the model, and the testing program. The model was needed for engineering design purposes because the west breakwater had been damaged by waves from a storm on January 28, 1981 and a reconstruction scheme was required so the structure could withstand future expected storm waves. The model was exposed to both regular waves and irregular waves representing the most severe storms in the past 85 years as determined by storm wave hindcasts and also from available buoy measurements. In addition, the directions of these waves were varied.

Since the model was not constructed for basic research on breakwater stability, the results presented herein are limited to a single unique breakwater design. However, as mentioned earlier, it is hoped that the concepts developed in this study may be useful evaluating other breakwater designs and testing programs.

Presentation and Discussion of Results

A series of regular wave experiments were conducted using the hydraulic model discussed by Lillevang et al. (1984) to examine the effect of the wave length on the stability of 36.8 ton (33.4 metric ton) Tribars which armored the terminus cone of the breakwater. The duration of the tests extended until damage began, with the initiation of damage for the structure defined as the removal of four contiguous Tribars from the armor blanket. This number was based on the observation that the removal of four adjacent Tribars generally led to rapid failure of the armored slope whereas the removal of less Tribars from an area did not appear to affect the armor stability.

The variation of the wave height necessary to initiate damage as a function of the number of waves to which the structure was exposed until that damage was observed is presented in Figure 1. It is apparent in Figure 1 that for a given wave height regular waves with larger wave periods are more damaging than those waves with the same wave height but smaller wave periods. In addition, for each wave period there is a threshold wave height below which no damage to the breakwater occurs regardless of the number of waves.

Considering the observed effect of wave period, or wave length, on the armor stability, the variation of the stability with the parameter H^2L was investigated. This parameter is of interest since it is proportional to the energy contained in a periodic wave with height H and wave length L. The data which were shown in Figure 1 are presented in Figure 2 in terms of the parameter H^2L as the ordinate with the number of waves to initiate damage as the abscissa. (The wave length has been calculated from small amplitude wave theory for a depth of about 75 feet (23 m) which represents the region just offshore of the west breakwater terminus.) The data appear to be less scattered when presented in this manner which indicates that the effect of wave length on Tribar stability may be related to the energy to which the structure is exposed. The solid curve shown in the figure was fitted to the data and the dashed curves represent a variation of plus or minus 10 percent from the solid curve. The data still exhibit scatter which may be due to the fact that each data point represents damage at a somewhat different location on the structure which is probably a function of the variation in the fitting of the Tribars in the armor layer. A similar correlation between stability and the energy, H^2L, has been shown for other breakwater armor types, including rubble mounds by Ahrens (1984) and Gravesen et al. (unpublished); these tests primarily used irregular waves.

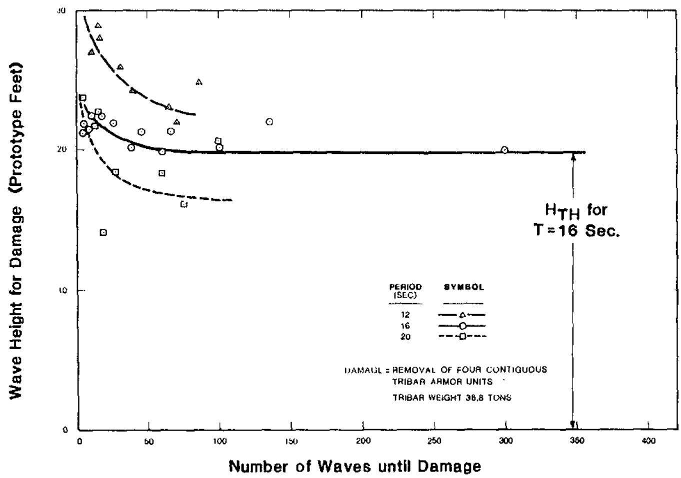


Figure 1. Wave Height to Initiate Damage as a Function of Period and Number of Waves

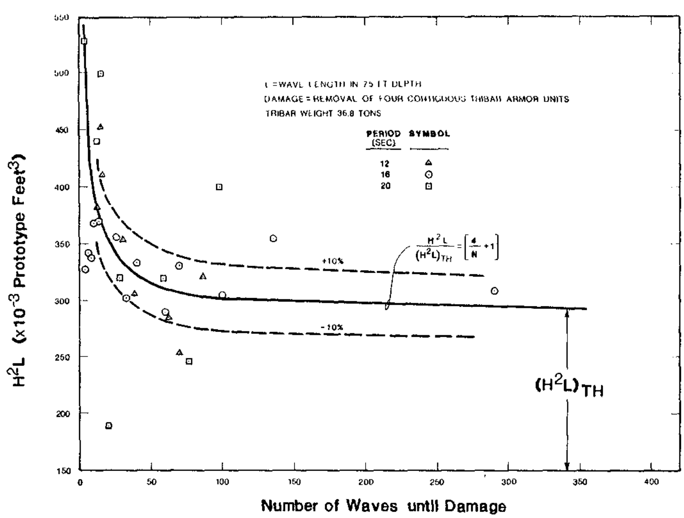


Figure 2. Total Energy, H^2L, Versus Number of Waves to Initiate Damage

The energy parameter, H^2L, appears to account for the effect of wave length (wave period) and wave height on the stability of the breakwater armor. However, as such, it does not provide a direct means of comparison of the results of regular wave tests to those from irregular wave experiments where the structure is exposed to a simulated storm wave spectrum. In addition, this does not appear to provide a means for comparing the damage caused by waves from a storm composed of a small number of large waves to the damage due to the waves from a less intense storm with a larger number of small waves. A method is proposed herein to extend the concepts just presented to that situation.

In developing these relationships use will be made of the concept of a "zero damage wave height". This wave height, also denoted as the threshold wave height, was mentioned earlier and refers to the wave height below which no damage occurs for a given breakwater armor and geometry. As could be seen in Figure 2, the threshold also applies to the wave energy parameter, H^2L, i.e., there is a value of the wave energy parameter below which no damage occurs regardless of the number of waves to which the structure is exposed. This threshold value is denoted as $(H^2L)_{TH}$. The assumption is made in this development that the damage due to a single wave greater than this threshold value is proportional to the magnitude of H^2L for that wave. Furthermore, it is assumed that the total damage to a structure is a result of the cumulative effect of all waves which have a total energy which exceeds the threshold value. Therefore, for a train of N regular waves with height H this cumulative energy would be equal to NH^2L. Note that this implies that waves of different wave height and different wave periods can have the same potential for damage to the same structure.

Applying this concept to irregular waves, the cumulative magnitude of H^2L for all waves with "energy" greater than the threshold value, $(H^2L)_{TH}$, is dependent on the joint probability distribution of wave height and wave period. To simplify the analysis it is assumed that the frequency distribution of wave heights follows a Rayleigh distribution and that the wave length is independent of wave height. Hence, the cumulative "energy" can be written as:

$$\Sigma(H^2L)_{TH} = NL \int_{H_{TH}}^{\infty} H^2P(H)dH \tag{1}$$

where the representative wave length for the storm waves is taken as that defined by the period of the peak of the energy spectrum at the depth of interest, and the probability distribution of wave height is denoted as P(H). As mentioned, the Rayleigh distribution is used as a representative probability distribution:

$$P(H) = 4.01 \frac{H}{H^2_{33\%}} \exp\left[-2.01 \left(\frac{H_{TH}}{H_{33\%}}\right)^2\right] \tag{2}$$

Substituting Equation 2 into Equation 1 and performing the integration one obtains the following:

$$\Sigma(H^2L)_{TH} = \frac{1}{2}\ NLH^2_{33\%}\ [2.01\ (\frac{H_{TH}}{H_{33\%}})^2 + 1]\ \exp\ [-2.01\ (\frac{H_{TH}}{H_{33\%}})^2] \qquad (3)$$

Equation 3 provides a measure of the cumulative total energy incident on a breakwater which is contained in those waves whose height is greater than the threshold height H_{TH}. It should be noted that the number of waves N used is based on the interval of time for which the particular significant wave height applies and the period of the peak of the energy spectrum. Thus, the assumption is made that the spectrum is a narrow band spectrum as is indicated by the use of the Rayleigh distribution.

A series of breakwater stability tests that were conducted in connection with the reconstruction efforts of the Diablo Canyon breakwater have been analyzed using the concept of the cumulative energy, $\Sigma\ (H^2L)_{TH}$. These tests were conducted with both regular and irregular waves so that there was an opportunity to compare stability for a particular structure with these different waves. For these experiments in the Diablo Canyon breakwater model, the breakwater was armored with 21.6 ton (19.6 metric ton) Tribars. The regular waves had a wave period of 16 seconds and the peak of the energy spectrum for the irregular waves was 16 seconds also. The geometry of the breakwater was identical in all experiments. To have a basis for comparison, the damage to the breakwater armor was defined in terms of the total number of Tribars displaced, since for a pattern placed armor consisting of a single layer the number of units which were displaced could be evaluated easily. Another criterion for comparison of the effect of regular and irregular waves is the displacement of the cap-blocks which form the crest of the breakwater (see Lillevang et al., 1984). The concrete cap-blocks consisted of individual cast in place concrete blocks, each approximately 30 feet (9.1 m) long, 21 feet (6.4 m) wide, and 7 feet (2.1 m) thick with a weight of about 300 tons (270 metric tons) In the prototype, a number of these cap-blocks were displaced in a storm which occurred in January 1981. In the model the displacement of the blocks was useful in comparing the results of regular and irregular wave tests.

The damage to the breakwater is shown in Figure 3 in terms of the number of dislodged Tribars as a function of the cumulative energy. For 16 second waves a threshold wave height of 17.5 feet (5.3 m) was determined from the experiments and this was used in Equation 3. It should be noted that the threshold wave height is less than that shown in Figure 2 because 36.8 ton (33.4 metric ton) Tribars had ben used in the former case and the smaller 21.6 ton (19.6 metric ton) Tribars were used for these data. Two forms of testing were used for the periodic waves: either exposing the structure to bursts of 9 waves with a 16 second period or to a continuous train of these waves. There does not appear to be a significant difference in the results from these two different types of regular wave tests. The irregular and regular wave tests are specifically denoted in Figure 3. It can be seen that the results of these two types of experiments are similar. An important fact related to the experiments is that a greater number of waves (and greater total cumulative wave energy) impacted the breakwater during the irregular wave tests. However, because only the energy in those waves greater than a threshold limit was summed, the damage potential for the irregular waves was accounted for in a way similar to that for the regular wave experiments.

In a similar manner the number of dislodged cap-blocks versus the cumulative energy is shown in Figure 4. The cap-blocks were cast for each separate experiment and were not reused in a subsequent experiment once they had been displaced. Generally there is considerably more scatter of the data in Figure 4 compared to Figure 3, although the irregular wave damage results agree well with the regular wave damage for Tests 1 and 4. There appears to be a much greater resistance of the cap-blocks to motion for Test 3 which was conducted using regular waves. It is believed that this was a result of the underside of the cap-blocks for these experiments being much rougher than in other tests, which resulted in a greater interlocking with the underlying intermediate stone layers and thus an increased resistance to sliding. This increased roughness was due to a different method of casting the cap-blocks for this particular test. Also shown in the figure is the best estimate of displacement of the prototype cap-blocks which occurred on January 28, 1981 (see Lillevang et al., 1984). The data shown are based on observations of the time of cap-block motion and estimates of storm wave conditions based on wave measurements from a National Oceanographic and Atmospheric Administration (NOAA) buoy located 18 miles from the site. It is postulated that the slower rate of damage of the prototype cap-blocks as compared to those in the model tests may be due to differences between the roughness of the underside of the blocks in the prototype and the model. Since there were no direct measurements possible of the actual prototype cap-block roughness, it cannot be determined whether this was accurately reproduced in the model. Of course, there are certainly other difference which could contribute to the differences. Not the least of these is the water surface time history used in the model, which had to be synthesized from a measured spectrum without wave phase information. The results shown in Figure 4 indicate that design conclusions for the rate of damage of the cap-blocks based on this model are probably conservative, leading to a factor of safety in the interpretation of the model results. Even though the rate of damage in the model was in some cases different, the mode of failure of the breakwater was accurately described by the model.

The relation of the breakwater armor displacement to cumulative wave energy has been developed based on limited data from experiments on this particular breakwater design. However, it appears that this approach has potential for providing a means of characterizing storm waves for breakwater design and model testing which could be much more generally applicable.

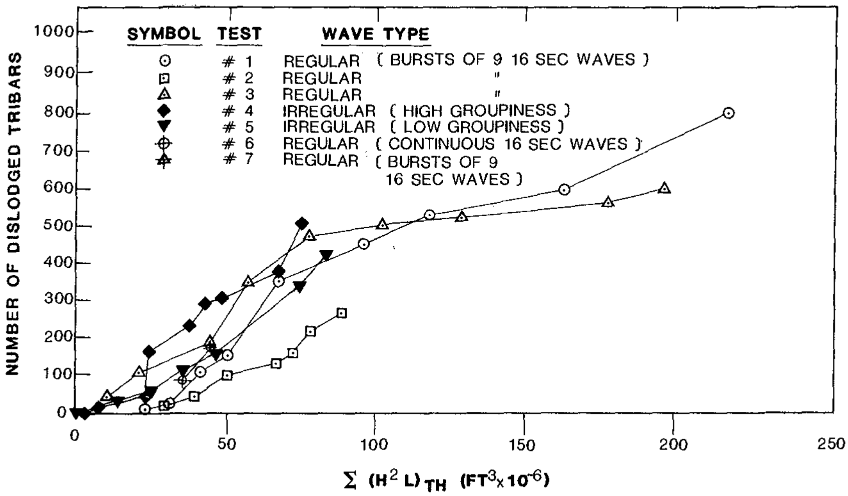


Figure 3. Tribar Displacement Versus Cumulative Wave Energy. Based on $(H^2L)_{TH}$ of 230,000 ft^3. Each Tribar Weighs 21.6 tons.

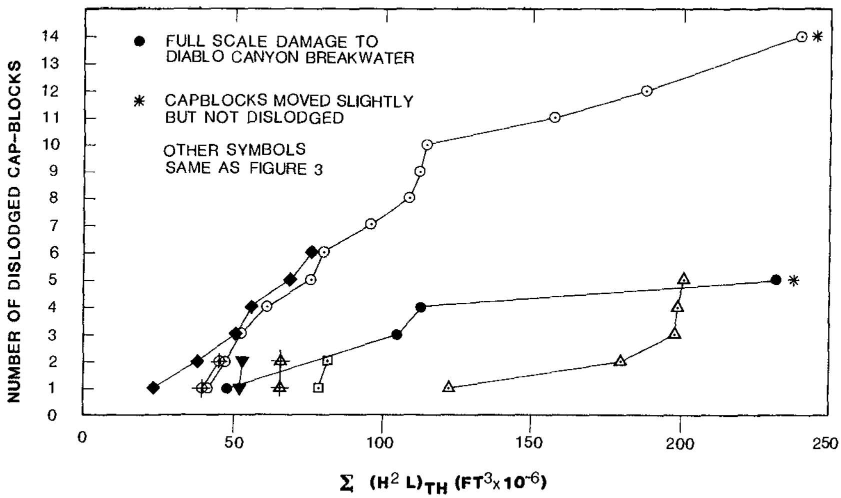
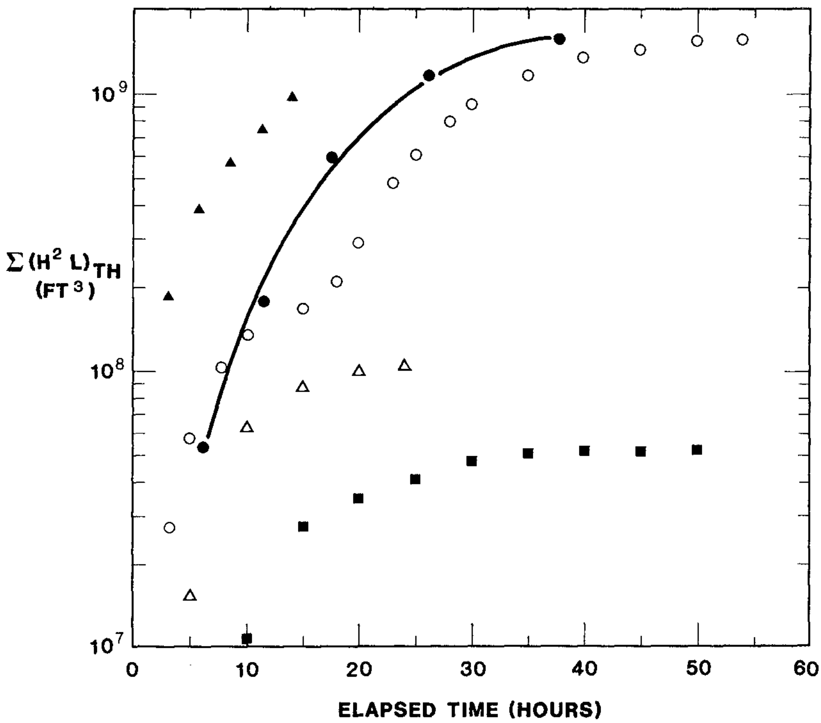


Figure 4. Cap-block Displacement Versus Cumulative Wave Energy. Based on $(H^2L)_{TH}$ of 230,000 ft^3. Each Tribar weighs 21.6 tons.

One question which occurred in this investigation related to storm events which took place after the model tests were completed. A series of storms occurred in the winter and spring of 1983 offshore of the west coast of the United States which caused significant damage along the California coast. Since the model tests were completed and there was not opportunity to expose the hydraulic model to a spectrum representing the waves from those storms, the question was raised as to what the effect of recent storms on the reconstructed breakwater could have been, compared to the results of the storm waves to which the breakwater model had been exposed during the extensive model testing program. The characteristics of the most serious irregular wave system the breakwater had been exposed to in the testing program was obtained from hindcasts of a storm which occurred in March 12 and 13, 1905. This storm was hindcast to have had a maximum significant wave height of about 31 feet (9.4 m) with the period of the peak of the spectrum of about 15 seconds. The storm of March 1983 had considerably larger wave periods which varied from about 13 seconds to 25 seconds and about the same maximum significant wave height. The wave data from this recent event were obtained from the NOAA buoy mentioned earlier, so that the variation of the period of the peak energy of the spectrum as the storm grew and subsided could be incorporated in computations. The proposed method was applied to this storm (and several others measured with the NOAA buoy) and compared to the hindcast 1905 storm waves. These results are shown in Figure 5 where the cumulative total energy has been plotted as a function of the elapsed time of the storm as the abscissa. The cumulated energy from the model experiments using the 1905 storm waves are shown along with that for the 1983 storm. It is seen that the storms to which the model was exposed had about the same total maximum cumulative energy as the recent (1983) storm waves, but the former occurred in a shorter elapsed time. The major reason for this is that the storm to which the model was exposed grew and decayed more rapidly than the 1983 storm did. Thus, if the concept of cumulative energy is correct, it would be expected that the effect of the 1983 storm on the breakwaters would not be more serious than the storms to which the model had been exposed in the testing program. Several other storms which are of lesser consequence are also shown in the figure.

As shown, the method of cumulative energy can be applied in assisting in the interpretation of hydraulic model results in light of prototype wave measurements. There are additional applications of this approach which can be mentioned.

The first might be the use of the term $H^2_{33\%}L$ to characterize the peak intensity of a storm, instead of $H_{33\%}$ as is commonly done. This would allow the effect of period on breakwater armor stabiltiy to be accounted for when selecting a design storm or storms for use in breakwater model testing. The second would be to characterize storms whose spectral characteristics vary throughout their duration into a single parameter, $\Sigma H^2_{33\%}L$, which accounts for variations in wave height, periods, and duration. Thus, a large number of storms could be evaluated as to their total potential for damage, instead of simply using the significant wave height associated with the peak of the



SYMBOL	DATE	COMMENT
—●—	3/12/05 - 3/13/05	USED IN MODEL (978A-984F)
○	3/1/83 - 3/3/83	FROM NOAA BUOY
△	2/21/83	FROM NOAA BUOY
▲	3/12/05 - 3/13/05	USED IN MODEL (989A-993A)
■	1/26/83 - 1/28/83	FROM NOAA BUOY

Figure 5. Comparison of Cumulative Wave Energy Used In Model Tests for March 1905 Storm to That Inferred From NOAA Buoy 46011 Data at A Nearshore Location. Based on a Threshold Wave Height of 17.5 feet.

storm, for instance, in the development of storm return periods. A third application of the proposed method would be in breakwater model testing for design purposes. Instead of testing the breakwater design over a wide range of spectral characteristics and durations, regular wave tests could be used to determine $(H^2L)_{TH}$. The potential of damage due to any storm of given wave characteristics and duration could then be compared to results of damage due to a limited number of regular and irregular wave tests which would characterize the response of the breakwater design to $\Sigma(H^2L)_{TH}$. This could reduce the number of tests required, and allow a wider range of potential storms to be evaluated.

Conclusions

A relationship between breakwater armor displacement and wave energy has been demonstrated for a single breakwater design. The armor consisted of both Tribar and crest cap-blocks. In addition, the damage in the model due to regular waves and that caused by irregular waves for the same structure has been related through the use of the cumulative total wave energy contained in those waves which have an energy greater than a threshold value for the breakwater. The threshold damage value is that magnitude of wave energy which will do no damage to the breakwater independent of the number of waves which impinge on the structure. For a given wave period, the threshold wave energy $(H^2L)_{TH}$ can be related to a threshold wave height H_{TH} for which there is no damage to the breakwater. The application of experiments with irregular waves to situations which have not been tested is described in these results. It should be emphasized that these results were for a specific breakwater model, so that additional studies would be necessary to demonstrate the applicability of this approach to a wider range of breakwater armor types and breakwater geometries.

Acknowledgments

The authors would like to express their appreciation to Mr. Omar J. Lillevang, the project engineer for the breakwater study. It was his initial comment regarding the possible importance of wave energy which led to the development described herein. The Pacific Gas and Electric Company is also acknowledged for their support of the project, as well as their permission to publish this paper.

Bibliography

Aherns, John P., "Reef Type Breakwaters," Proceedings of the 19th International Conference on Coastal Engineering, Houston, Texas, 1984.

Gravesen, H., O. J. Jensen, and T. Sorensen, "Stability of Rubble Mound Breakwaters II," Danish Hydraulic Institute, unpublished.

Hudson, R. Y., "Laboratory Investigations of Rubble-Mound Breakwaters", ASCE, Journal of Waterways and Harbors Division, Vol. 85, No. WW3, 1959.

Irribarren, C. R., and O. C. Nogales, "Report on Breakwaters," XVIII, Permanent International Association of Navigation Congresses, Section II, Subj. 3:119-39, 1953.

Lillevang, Omar J., Fredric Raichlen, Jack C. Cox, and Daniel L. Behnke, "A Detailed Model Study of Damage to a Large Breakwater and Model Verification of Concepts for Repair and Upgraded Strength," Proceedings of the 19th International Conference on Coastal Engineering, Houston, Texas, 1984.

Permanent International Association of Navigation Congresses, 1976. Report of the International Commission for the "Study of Waves". Annex to Bulletin No. 25, Vol. III, 1976.

Raichlen, F., "The Effect of Waves on Rubble-Mound Structures," Annual Review of Fluid Mechanics, Vol. 7, 1975.