CHAPTER 113

RUN-UP DUE TO BREAKING AND NON-BREAKING WAVES

by

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ABSTRACT

This study was conducted to investigate the effect of incident wave parameters on run-up for both a smooth-faced structure and a structure armored with quarry-stone. The ratio of the depth-to-wave length (the relative depth) was found to be important in defining wave run-up for both cases. The relative run-up (expressed as the ratio of the run-up elevation above still water level to the incident wave height) for waves which break at the toe of the structure was less than the maximum relative run-up for non-breaking waves for the same relative depth. For both structures, the maximum relative run-up for experiments with long waves occurred at a value of the modified Ursell number, $(1/2\pi)^2(HL^2/h^3)$, of order unity which indicates that the nonlinear and linear effects are approximately equal in the incident wave.

INTRODUCTION

The objective of the present fundamental study, some results of which are presented herein, is to investigate the effect on run-up of certain basic parameters which describe the incident wave system. In addition, two structures with the same slope were used in the laboratory to evaluate several effects of the characteristics of the structure on run-up: a smooth slope and a slope armored with a quarry-stone. Relative to the present investigation it is of interest to discuss briefly three previous run-up studies which are also fundamental in emphasis.

Saville (1956) investigated run-up on smooth-faced structures of various slopes with a beach (slope of 1:10) constructed seaward of the toe of the structure. All run-up data were normalized with respect to the unrefracted deep water wave height and plotted as a function of the ratio of this wave height to the square of the wave period without detailed attention to the ratio of the depth to the wave length. (The ratio of the wave height to the square of the wave period is directly proportional to the wave steepness.) As noted by Saville the data exhibited scatter (less than $\pm 20\%$), and the experimental curves which were presented were fitted through the average of these data.

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Hudson (1959) presents experimental results for run-up on rubble-mound structures of various slopes constructed in a constant depth wave tank and exposed to periodic non-breaking waves. The experiments were conducted for variable wave heights and with constant values of the ratio of depth-to-wave length varying from 0.1 to 0.5. The run-up data were presented as the relative run-up, i.e., the run-up normalized by the wave height without the structure in place, plotted as a function of the wave steepness (wave height divided by wave length) for constant depth-to-wave length ratios. Since it was difficult to find trends of data for given values of depth-to-wave length, experimental curves were presented which were envelopes of the data. (The experimental curves obtained from these data and from Saville (1956) are also presented by CERC (1966).)

Grantham (1953) shows limited data which demonstrate the importance of structure slope, wave steepness, depth-to-wave length, and porosity of the structure on run-up. With relation to the present study, Grantham found that for the three structures studied, there was an increase in relative run-up as the ratio of depth-to-wave length decreased for a given wave steepness.

ANALYTICAL CONSIDERATIONS

A definition sketch describing the problem under consideration is presented in Figure 1 showing the run-up defined as the maximum vertical distance from the still-water surface to the position on the face of the structure to which the water surface rises during wave attack. For convenience the wave height, H, is indicated on the figure; however, the wave height which is used in describing the incident wave system is that which exists at the location of the toe of the slope without the structure in place. (This will be discussed later in connection with the experimental procedure.) For both the smooth-slope and the rubble-mound structure no over-topping is permitted. In fact, the latter structure is constructed on an impermeable base; hence, there is only run-up on the structure without associated transmission of wave energy either through or over the structure.

The problem of run-up has not been amenable to a complete theoretical treatment due to the difficulty in analytically describing the many factors involved. Therefore, dimensional analysis will be employed (similar to Hudson (1958)) to define the important non-dimensional parameters which describe the problem. Referring to Figure 1, if a functional relationship exists between the run-up and the description of the incident wave system and the structure, for regular waves impinging on a breakwater face at normal incidence to the structure, this relationship can be expressed as:

$$f(H,\ T,\ h,\ 1/m,\ R,\ \rho_w,\ \mu,\ g,\ \alpha,\psi) = 0 \tag{1}$$

where f( ) indicates a "function of".

The first four variables of Equation (1) define the incident wave characteristics, R is the run-up on the structure face measured vertically from the still water level, ρ_w is the fluid density, μ is the dynamic fluid viscosity, g is the acceleration of gravity, and α is the slope of the face of the structure with respect to the bed. The last variable, ψ, (assumed to be non-dimensional) describes the physical characteristics of the breakwater

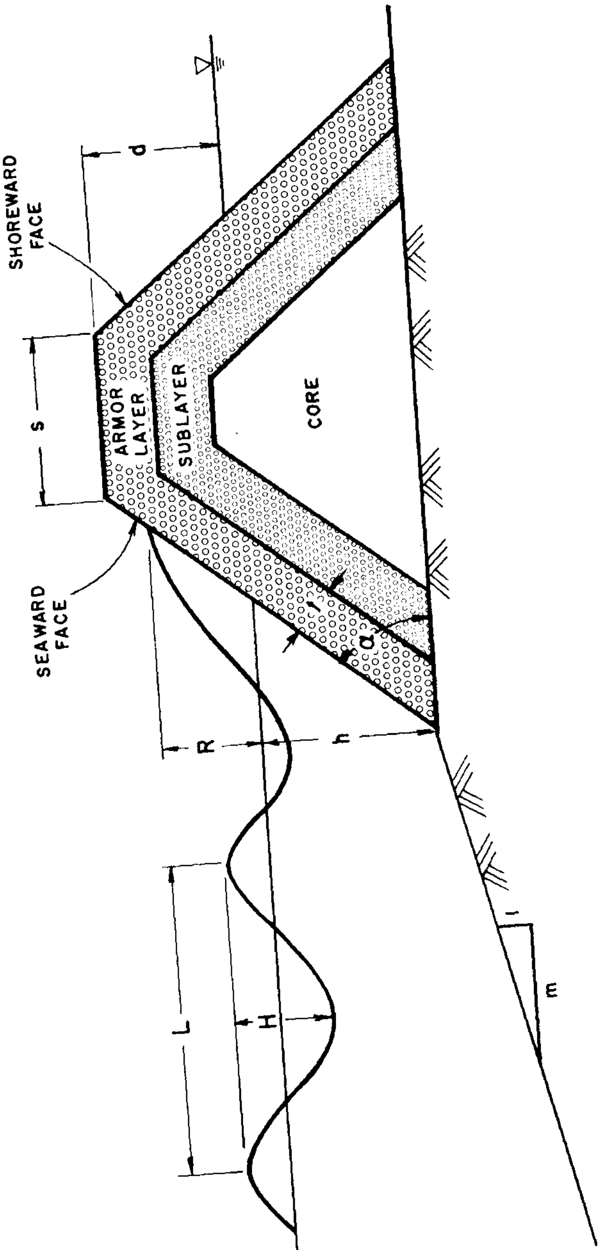

Figure 1. Definition Sketch of a Rubble-Mound Structure.

face which affect run-up, and would be a function of such factors as: the shape of the armor units, the roughness of the face, the permeability of the armor layer, the characteristics of the underlying material, the method and type of construction, etc. Hence, ψ describes features of the breakwater, relevant to run-up, which demonstrate the difficulty of developing universal run-up characteristics for rubble-mounds. (In the case of a structure with a smooth impermeable face, the parameter ψ becomes relatively unimportant in defining run-up.) The variables ρ_w, g, and h are used in non-dimensionalizing Equation (1) which results in the following:

$$f\left(\frac{H}{h}, \frac{T^2 g}{h}, \frac{1}{m}, \frac{R}{h}, \frac{\mu/\rho_w}{\sqrt{gh}\ h}, \alpha, \psi\right) = 0. \tag{2}$$

The second term in Equation (2) is proportional to the ratio of the wave length to the depth, L/h; the fifth term is an inverse Reynolds number with the velocity equal to the shallow water wave celerity. By a suitable combination of terms in Equation (2) it can be shown that the length dimension in the Reynolds number can be expressed as the characteristic dimension of an armor unit and the velocity can be expressed as the water particle velocity. Therefore Equation (2) can be rewritten as:

$$\frac{R}{H} = \phi\left(\frac{H}{h}, \frac{L}{h}, \frac{1}{m}, R_e, \alpha, \psi\right) \tag{3}$$

where as mentioned, the Reynolds number, R_e, is a function of the water particle velocity and a linear dimension of the armor. In Equation (3) the relative run-up (expressed as the run-up elevation divided by the wave height) is considered a function of the wave characteristics as embodied in the ratio of wave height-to-depth and the ratio of the wave length-to-depth as well as the off-shore slope, 1/m. It is noted that these three parameters define the characteristics of long-waves and the latter becomes most important in defining the shape of the breaking or incipient breaking wave. As mentioned previously, when comparing experiments for smooth-faced structures the variable ψ can be eliminated; of course, this parameter is less important when investigating the action of various incident wave systems on a particular rubble-mound slope. (The distance of the structure from the wave source may be important in laboratory tests due to the effect of distance on the evolution of nonlinear waves. This distance is not included in the present discussion.)

The breakwater face armored with quarry-stone represents a hydrodynamically rough surface; hence, for sufficiently large Reynolds number, the dependence of run-up on the Reynolds number as shown in Equation (3) should be minimal. Therefore, the list of variables shown in Equation (3) can be reduced to:

$$\frac{R}{H} = \phi\left(\frac{H}{h}, \frac{L}{h}, \frac{1}{m}, \alpha, \psi\right) \tag{4}$$

Corrections for the effect of Reynolds number must be applied to the results of laboratory experiments to investigate run-up on prototype structures, if the scale of the experiments is not large enough.

In the case of deep water waves Equation (4) can be rewritten in terms of the incident wave steepness as:

$$\frac{R}{H} = \phi(\frac{H}{L}, \frac{L}{h}, \frac{1}{m}, \alpha, \psi) \quad (5a)$$

Since neither the relative depth, h/L, nor the bottom slope, 1/m, are important when the wave is not affected by the bottom (the condition of deep water waves), the non-dimensional variables in Equation (5a) can be further reduced to:

$$\frac{R}{H} = \phi(\frac{H}{L}, \alpha, \psi) \quad (5b)$$

Equation (5b) is most applicable for steep structures; for structures with relatively small slopes (say 1:10 and less) the waves would in all probability shoal on the structure leading to an influence of water depth above the face of the structure.

Still a third way of presenting the non-dimensional variables which affect run-up is to combine the first two terms in Equation (4) such that the following expression is obtained:

$$\frac{R}{H} = \phi(\frac{HL^2}{h^3}, \frac{L}{h}, \frac{1}{m}, \alpha, \psi) \quad (6a)$$

The first term on the right-hand side of Equation (6a) is commonly referred to as the Ursell number which describes the relative importance of nonlinear and linear effects in an incident long wave. This expression results since nonlinear effects are proportional to the ratio of the amplitude to the depth and the linear effects of frequency dispersion are proportional to the square of the ratio of the depth to a characteristic length in the direction of propagation. The characteristic length shown in Equation (6a) is the wave length; however, the characteristic length should be associated with a local property of the wave, since the effect of frequency dispersion would be greatest in regions of large curvature of the water surface. Therefore, an appropriate horizontal length is: $\ell = (\eta_o/\eta_x)$ where η_o is the total change of wave amplitude in a region along the wave, e.g., between two points of zero slope, and η_x is the maximum slope of the water surface in that region. (See Hammack (1973) for a more detailed discussion of local length scales for irregular waves.) If, for convenience, the small amplitude wave theory is used, the characteristic length dimension corresponding to this definition is: $\ell = L/2\pi$. Therefore, a modified Ursell number may be given as: $(1/2\pi)^2(HL^2/h^3)$. For solitary waves where nonlinear and linear effects are balanced, the same type of definition of the horizontal length yields a numerical value of the Ursell number of approximately 2.3; hence, when the modified Ursell number is about 2 to 3, nonlinear effects should be approximately equal to linear effects. To comply with this definition of the Ursell number Equation (6a) can be rewritten as:

$$\frac{R}{H} = \phi\left\{\left[\frac{1}{2\pi}\right]^2 \frac{HL^2}{h^3}, \frac{L}{h}, \frac{1}{m}, \alpha, \psi\right\} \quad (6b)$$

It should be emphasized that Equation (6b) like Equation (4) is applicable for long waves and the use of these equations to describe run-up of short waves is not strictly correct. On the other hand, the non-dimensional variable $(1/2\pi)^2$ (HL^2/h^3) in Equation (6b) can be used to separate the data and to demonstrate the importance of h/L in the description of the problem. These features will be explored in detail in a later section.

EXPERIMENTAL EQUIPMENT AND PROCEDURES

Experiments were conducted in a wave tank 36.5 meters long, 91.5 cm wide, and 91.5 cm deep. The walls of the wave tank are constructed of glass throughout and the bottom is constructed of stainless steel built to within $\pm$ 0.8 mm of a plane surface. A periodic wave generator is mounted to the wave tank at one end and the tank is supported by a center hinge and motorized jacks upstream and downstream of the hinge point. Through this arrangement the slope of the wave tank can be changed from horizontal to a maximum of 1 vertical for 50 horizontal. Since the wave machine is attached to the wave tank, waves can be generated which propagate into water with a decreasing depth as the tank is tilted. Hence, breaking waves can be produced at a particular location in the wave tank for a given wave period simply by increasing the stroke of the wave machine incrementally.

In these experiments the slope of the wave tank was maintained at 1 vertical for 200 horizontal and the incident wave characteristics were measured using a parallel wire resistance wave gage at the location where the toe of the structure would be placed. The water surface variations were recorded until reflections from the end of the tank returned to the gage location. From an average of approximately five waves, the stroke setting of the wave machine was determined which would generate the desired incident wave height at that location and wave tank slope without the structure in place.

The wave run-up on two structures was investigated. Both structures had a slope of 1 vertical to 2 horizontal, and in the case of the smooth and impermeable face, the slope was constructed of plywood treated with an epoxy paint. The second structure (also with a slope of 1 vertical to 2 horizontal) had a fitted rock face with two underlayers of material placed on the same plywood face. In both cases the toe of the structure was located approximately 20 m from the wave generator in a water depth of 25.8 cm.

A schematic drawing of the rubble-mound structure used in this study and the size distributions of the armor layer material ("A" rock) and the sublayer material ("B" rock) are presented in Figure 2; the ordinate is the weight of the individual rock and the abscissa is the percent by weight of the sample which is finer than the indicated ordinate value. The mean weight of the armor material is approximately 460 grams and the sublayer material has a mean weight of approximately 50 grams. (The rock utilized was "block-like" and obtained from the Fisher Quarry of the Umpqua River & Navigation Company, Camus, Oregon.)

This structure was built as a model of a breakwater to a scale of 1/40 in three layers starting with a crushed rock layer (0.8 mm mean size) approximately 15.2 cm thick placed directly on the plywood slope described previously with a sublayer of "B" rock 6.9 cm thick placed on top of this and followed by the armor layer of "A" rock one layer thick (approximately 8.6 cm). Between the underlayer and the sand a thin filter layer of gravel with

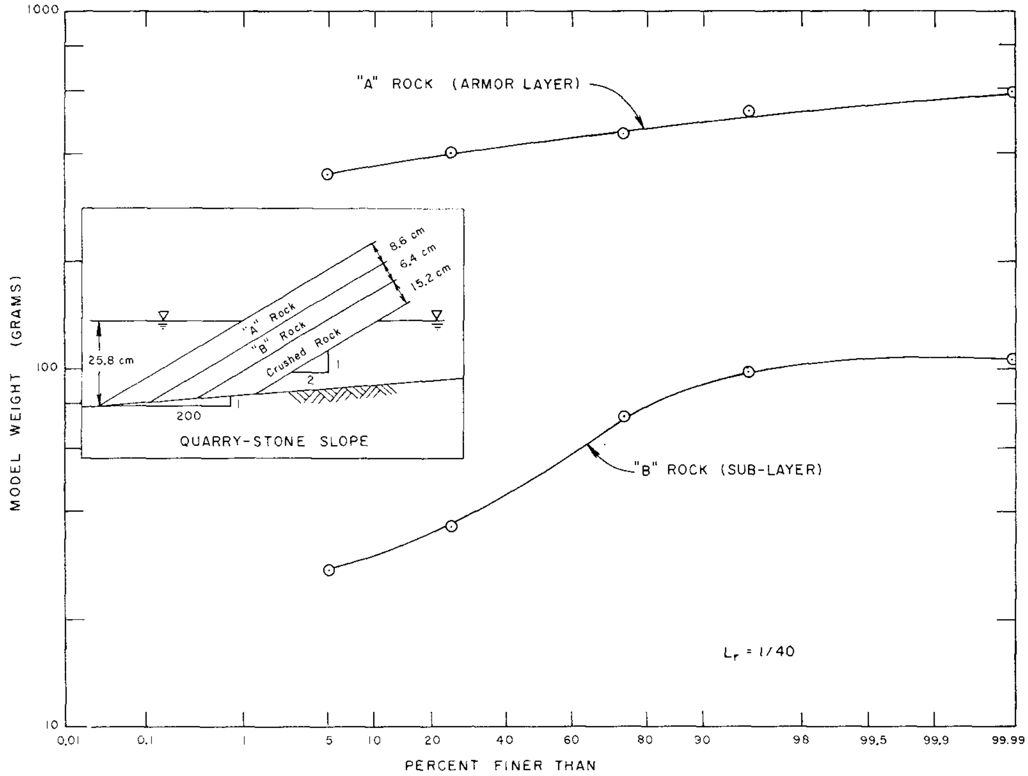

Figure 2. Rock Size Distribution and Schematic Drawing of Rubble-Mound Structure.

a 6 mm nominal diameter was used to prevent the sand from migrating through the rock layers. The armor rock was fitted from the bottom of the flume through the maximum elevation with the long axis of the rock perpendicular to the slope. Hence, this is not a true scale model since the rock below the mean water level ordinarily would be randomly placed with fitted rock placed only above mean water level. However, this was considered a satisfactory method of construction, since it was of interest to provide a rubble-mound with a surface roughness and permeability vastly different from the smooth slope yet relatively uniform throughout. A photograph of the face of the structure is presented in Figure 3 which shows the fitted nature of the surface.

A continuous record of run-up as a function of time was obtained in these studies using two miniaturized staff gages mounted parallel to the slope and to each other for both the smooth slope and the quarry-stone slope experiments. The gages mounted approximately 15 cm apart above the quarry stone are shown in Figure 3. The instrument operates as a step gage with 56 equally spaced pins giving a step voltage output which is linearly proportional to the wetted length of the gage. A drawing of the gage is presented in Figure 4a; the main body of the gage which supports the pins is approximately 1 cm square and contains 56 conductors, one for each pin. The pins are 1.38 cm apart and each pin is approximately 1.6 cm long; the innermost 1.1 cm electrically insulated with only the outer 5 mm exposed. Mounted parallel to the rod which supports the pins is a stainless steel conductor which forms a reference ground for the system. Essentially the pins with the associated electronics work as 56 switches, each switch actuated when a threshold limit of conductivity is reached between the pins and the conductor rod which is caused when the water immerses the pin and the conductor simultaneously. To reduce the effect of the splash on the response of the instrument, the electronic logic insures that before a voltage level corresponding to the highest wetted pin is indicated, the two next lower pins must be wetted simultaneously.

In the case of the smooth slope the gage was mounted so that the centerline of the pins was approximately 5 mm above the surface of the structure. For the quarry-stone slope the run-up gage was placed parallel to the slope and as close as possible to the slope (see Figure 3). An obvious disadvantage in measuring the run-up in this manner is that the run-up is measured a fixed distance away from the slope. Thus, the gage location will tend to give a run-up which may be somewhat less than the intersection of the run-up tongue with the slope. Nevertheless, all measurements will be consistent, assuming the shape of the run-up tongue is similar for the experiments, and a comparison of the results for different value of h/L and between the two types of slope should be valid. The major advantage of such a gage is that a real time record of run-up and run-down can be obtained.

The gages were placed on the structure so that the pins on one gage faced those on the other (see Figure 3) with the gages staggered in the direction of wave propagation such that a pin on one gage was located midway between the opposite pins on the other gage. Since these gages were only approximately 15 cm apart, it was possible to average the run-up spacially between these two gages with an accuracy of $\pm$ one-half the gage pin spacing. (Of course, to obtain a better spacial average more run-up gages of this type placed across the slope would be preferable.)

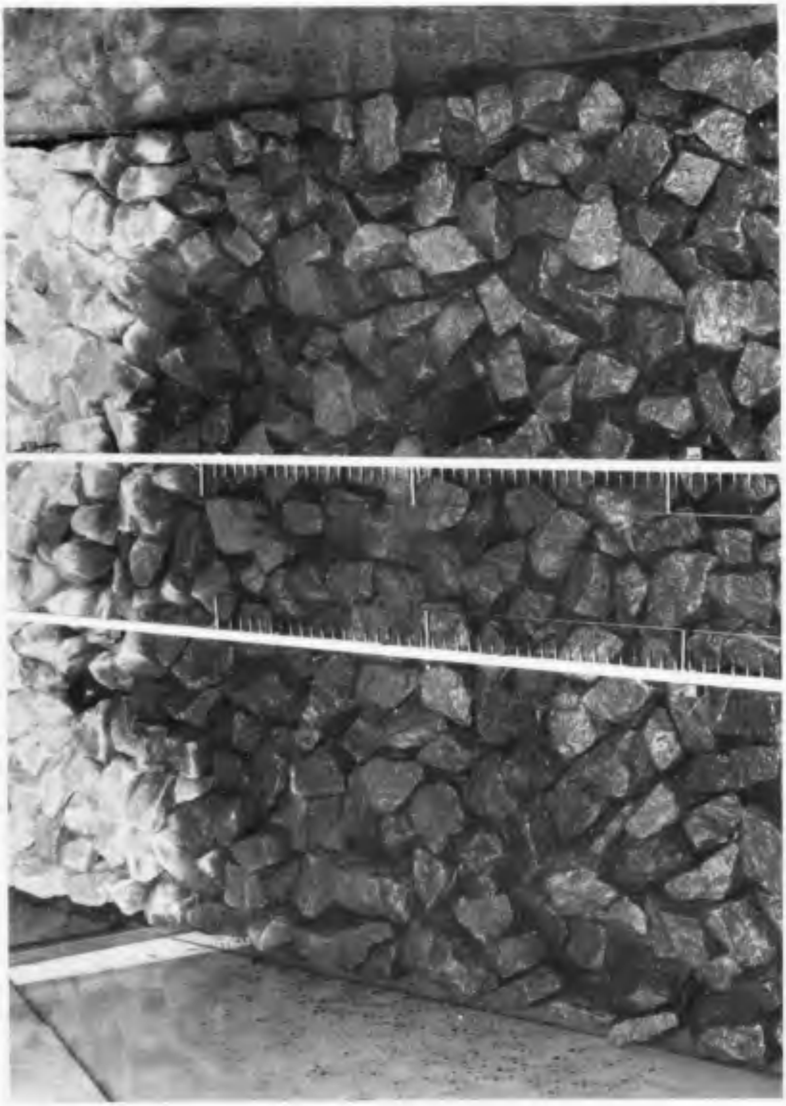

Figure 3. Photograph of Face of Rubble-Mound Structure Showing Run-Up Gages in Place.

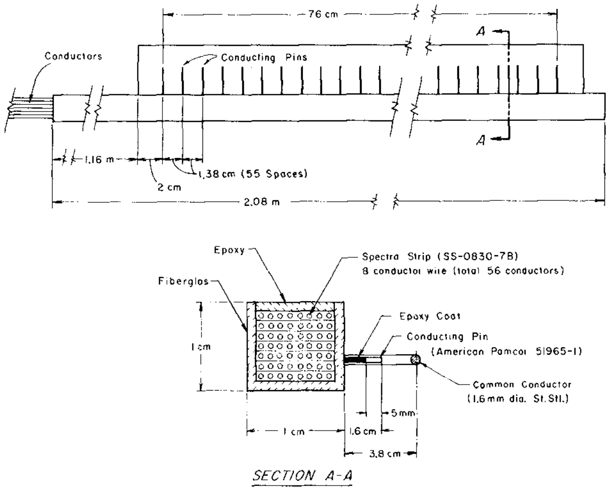

Figure 4a. Drawing of Run-Up Gage.

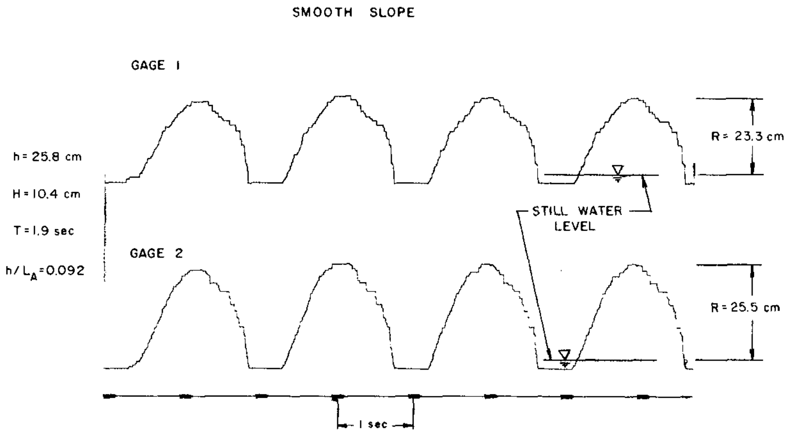

Figure 4b. Typical Record of Run-Up as a Function of Time.

A typical record of run-up as a function of time is shown in Figure 4b for the indicated wave conditions. These measurements were made on the smooth slope, and it is seen that due to the length of the run-up gage it was not possible to measure the run-down with this gage location; the minimum level indicated is the voltage level with the lowest pin out of the water.

The experimental procedure for both the smooth and the quarry-stone slope was as follows. Waves were generated at the desired wave period and wave height and the run-up recorded continuously. The wave generator was turned off before the first wave which had been generated and reflected from the structure reached the wave machine. Hence, depending upon the wave period, from five to eleven run-up maxima were averaged to give the time averaged run-up for each gage. As mentioned previously, the time averaged values of maximum run-up obtained with each of the two gages were then averaged to give a temporal and spacial average of the run-up on the structure for the given wave conditions. A deviation of $\pm$ one-half a pin spacing (measured vertically) was then assigned to the average run-up readings; hence, the run-up elevation could be measured accurately to within $\pm$ 3.1 mm. Since the run-up gages consist essentially of 56 switches which can only record an "on-off" condition they were calibrated by electronically connecting the pins in increments of 6, 12, 18, ..., 48, 54 pins. Point gage measurements were used to locate the pins on the slope both vertically and horizontally; hence, the voltage level associated with each group of six pins could be related to a run-up elevation. Such calibrations were conducted before and after experimental series corresponding to each wave period, and the gages were found to be reproducible and stable over the duration of an experiment.

PRESENTATION AND DISCUSSION OF RESULTS

Experiments have been conducted at six different depth-to-wave-length ratios for the smooth and quarry-stone slopes and at wave heights which include incipient breaking waves at the toe of the structure. (The wave length used in the definition of relative depth has been determined from the linear dispersion relation based on the incident wave period and the water depth at the toe of the structure and is denoted hereafter as L_A.) For all cases the slope of the wave tank was maintained at 1 vertical to 200 horizontal; hence, considering Equations (4), (5) and (6) the non-dimensional parameters which describe run-up in the experiments with the smooth or the quarry-stone slopes are: L_A/h and H/h <u>or</u> H/L_A <u>or</u> HL_A^2/h^3. In this way the effect on the relative run-up of two parameters dependent on the incident wave system is investigated.

The results for the smooth slope are presented in Figure 5 where the abscissa is the modified Ursell number, $(1/2\pi)^2(HL_A^2/h^3)$, and the ordinate is the relative run-up, R/H. The waves represented by the data in Figure 5 range from deep water to shallow water waves. The limits of the accuracy of the run-up gage ($\pm$ 3.1 mm) are indicated on each data point in terms of the relative run-up; hence, as the absolute run-up increases the accuracy of the gage expressed in a relative sense improves. Experimental curves have been fitted to the data corresponding to each relative depth using a second-order curve-fitting procedure tempered by judgment.

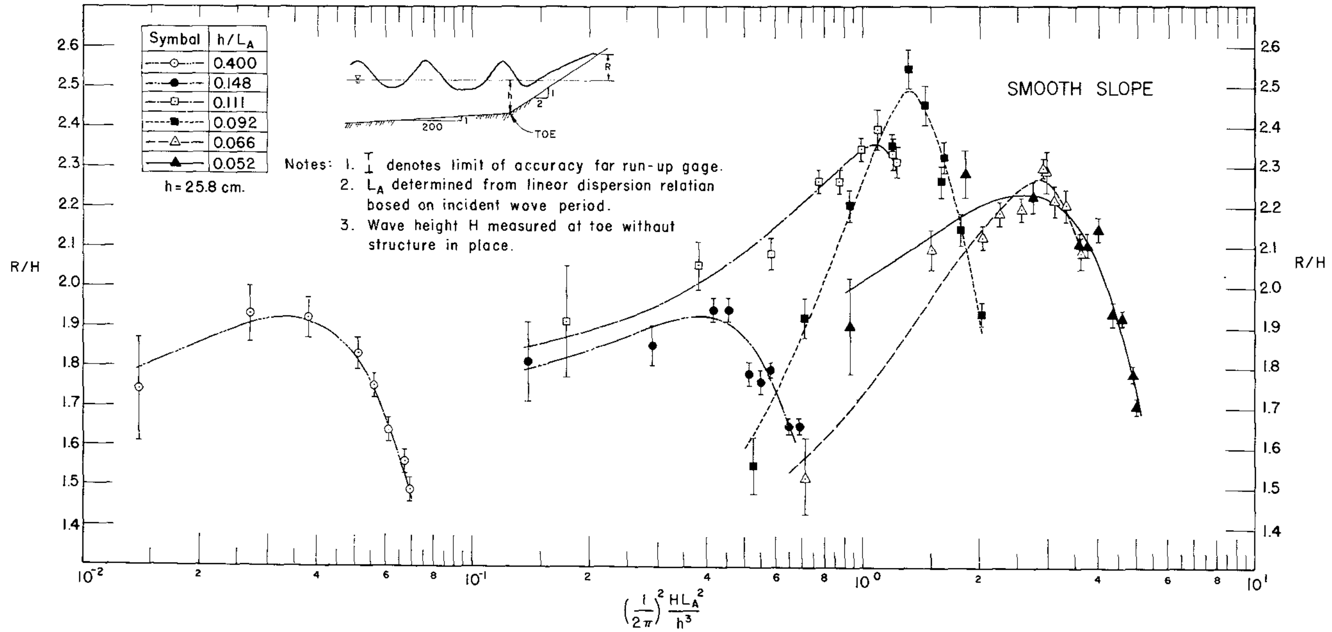

Figure 5. Relative Run-Up on a Smooth Slope for Various Ratios of Depth-to-Wavelength.

There are several features of the data of Figure 5 which should be emphasized. The range of the modified Ursell number which is covered by the experiments is from approximately 10^{-2} to 10; however, this parameter arises in connection with long wave (shallow water wave) theory so that strictly speaking its significance in describing the relative importance of nonlinear effects to linear effects in the incident wave is applicable to these data primarily for $h/L_A = 0.066$ and 0.052. Nevertheless, this parameter does tend to sort all the data in a rational manner. For each relative depth (h/L_A) the relative run-up increases to a maximum and then decreases with increasing Ursell number. Since the data point at the largest value of the modified Ursell number for each relative depth corresponds to an incipient breaking wave at the toe of the structure, it is evident that the relative run-up for incipient breaking waves on the smooth slope is considerably less than the maximum run-up for non-breaking waves.

As the relative depth, h/L_A, decreases the maximum relative run-up increases to a maximum and then decreases. The maximum relative run-up associated with the two shortest waves are approximately equal as are the maximum relative run-up for the two cases with shallow water waves (h/L_A = 0.066 and 0.052). In the latter case the maximum relative run-up occurs at a modified Ursell number of order unity, i.e., approximately 2.5; hence, it appears that the maximum run-up for long waves occurs when nonlinear and linear effects in the incident wave are approximately equal.

The relative run-up for the quarry-stone slope is presented in Figure 6 in an identical manner as the data in Figure 5 for the smooth slope. The first obvious difference between Figure 5 and Figure 6 is the reduction of the relative run-up by nearly a factor of two for identical waves incident upon the quarry-stone slope compared to the smooth slope. At first glance, although the data separate according to the value of the relative depth, some of the trends noted easily in Figure 5 are not immediately apparent. Nevertheless, when the data corresponding to each depth-to-wave length are analyzed separately, trends similar to the smooth slope case are observed. For example, at each relative depth, as the modified Ursell number increases the relative run-up reaches a maximum and then decreases to that corresponding to the incipient breaking wave; however, the variation is not as great as that observed for the experiments with the smooth slope. In the case of the quarry-stone slope, as the relative depth decreases the maximum relative run-up appears to increase monotonically. The maximum relative run-up for the two longest waves investigated (h/L_A = 0.066 and 0.052) is again approximately the same and occurs at a magnitude of the modified Ursell number of approximate 2.5.

In Figure 7 the relative run-up is presented as a function of the relative depth h/L_A for the smooth slope and the quarry-stone slope. The relative run-up shown for the non-breaking waves is the maximum run-up observed for the indicated h/L_A. For convenience of interpretation, the data corresponding to incipient breaking waves are shown in shaded areas. The effect of h/L_A is readily apparent in Figure 7; R/H reaches a maximum value of approximately 2.5 for the smooth slope at $h/L_A = 0.92$ for non-breaking waves. The relative run-up for the quarry-stone slope generally increases with decreasing h/L_A and is approximately one-half the corresponding maximum relative run-up for the smooth slope with non-breaking waves. When waves

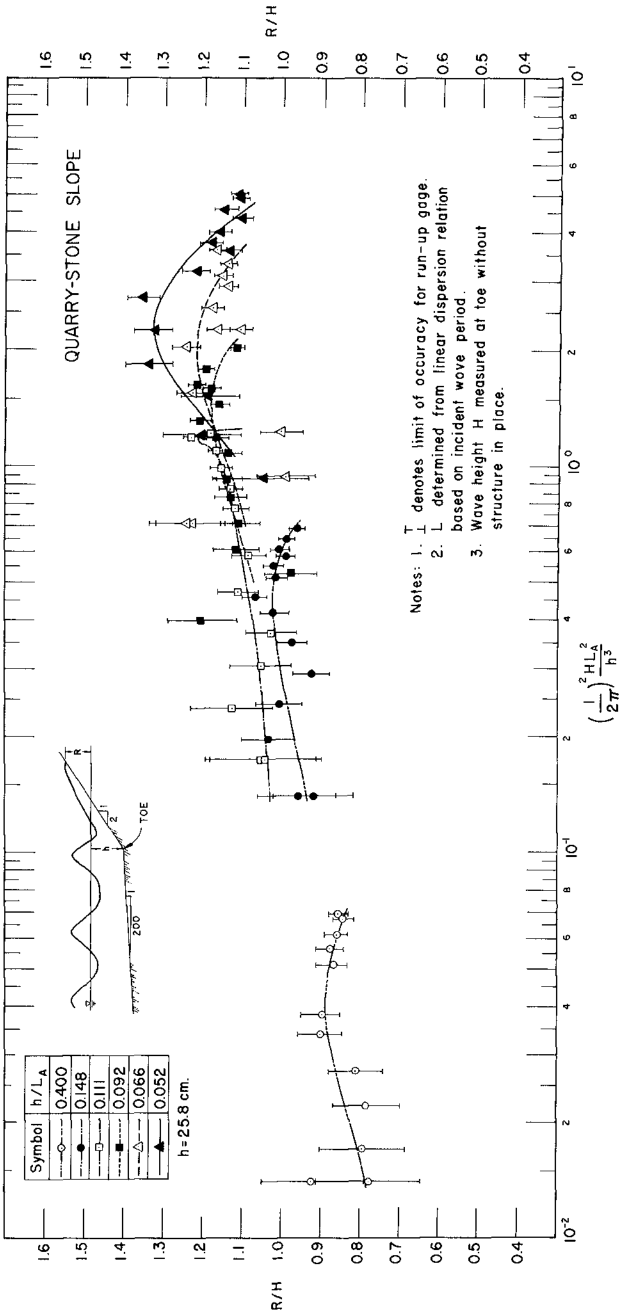

Figure 6. Relative Run-Up on a Quarry-Stone Slope for Various Ratios of Depth-to-Wavelength.

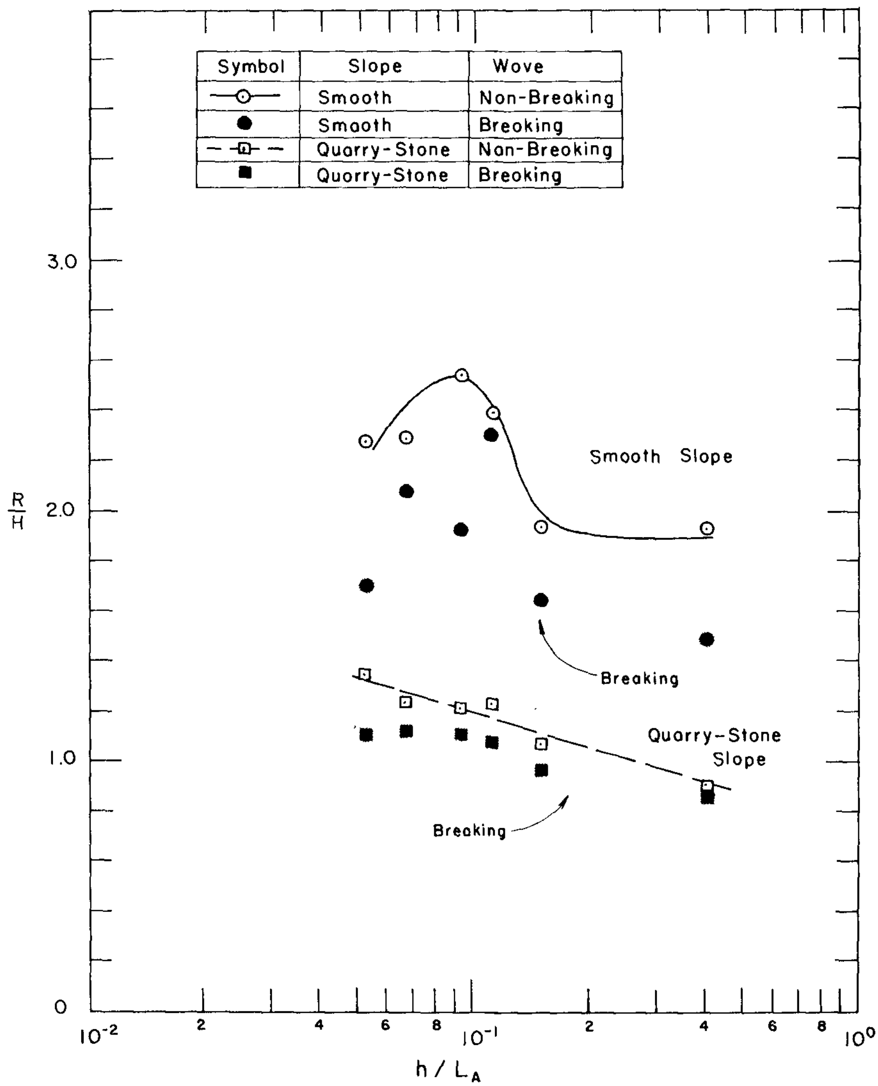

Figure 7. Maximum Relative Run-Up for Non-Breaking and Breaking Waves as a Function of Relative Depth.

break at the toe of the structure the relative run-up is always less than that corresponding to the maximum for non-breaking waves; the reduction is significantly greater on the smooth slope than on the quarry-stone slope. Therefore, it appears that the relative depth, h/L_A, is indeed an important parameter in defining run-up for these experimental conditions.

The same relative run-up data shown in Figure 7 have been replotted and are presented in Figure 8 as a function of the ratio of the incident wave height to the depth at the toe of the structure for each relative depth. (It should be realized that this type of presentation is more applicable for the long waves since the ratio of wave-height-to-depth is more important for these waves.) Again the breaking wave data are enclosed in shaded areas for each structure. A definite trend in the data is apparent if one progresses from data which correspond to the shortest wave length to those corresponding to the longest wave length; this can be seen from the data for both non-breaking and breaking waves. Hence, what first appears to be scatter of data is apparently an ordered behavior of the data when examined closely. This presentation emphasizes that the maximum run-up on both the smooth and the quarry-stone slopes for a given relative depth is greatest for non-breaking waves.

These data can be presented in a somewhat different manner as shown in Figure 9, where the maximum relative run-up is plotted as a function of the parameter H/gT^2 on the abscissa. Hence, the relative run-up is expressed as a function of the wave steepness, and is most applicable for waves which are nearly deep water waves. (As in Figures 7 and 8 the data which correspond to breaking waves are enclosed in shaded areas.) Included for convenience are the corresponding curves presented by CERC (1966) for smooth and quarry-stone structures with slopes of 1:2. It is seen for the deep water waves the data in both cases agree well with the curves. As the ratio of the depth-to-wave length decreases the data deviate from these experimental curves. Again, the importance of the ratio of depth-to-wave length can be seen by progressing with the data from the shortest to the longest waves, and for both the smooth slope and the quarry-stone slope, a trend appears to exist.

CONCLUSIONS

The following major conclusions can be drawn from this study:

1. The ratio of depth-to-wave length is an important parameter in defining relative run-up, R/H. In fact, the scatter of data which is usually attributed to experimental error in some run-up studies may indeed represent the effect of relative depth.

2. The maximum relative run-up for both the smooth slope and the quarry-stone armored slope is always greater than the relative run-up associated with waves which break at the toe of the structure.

3. The maximum relative run-up for long waves occurs at a modified Ursell number of between two and three for both slopes. This Ursell number corresponds to an incident wave where non-linear and linear effects are approximately equal.

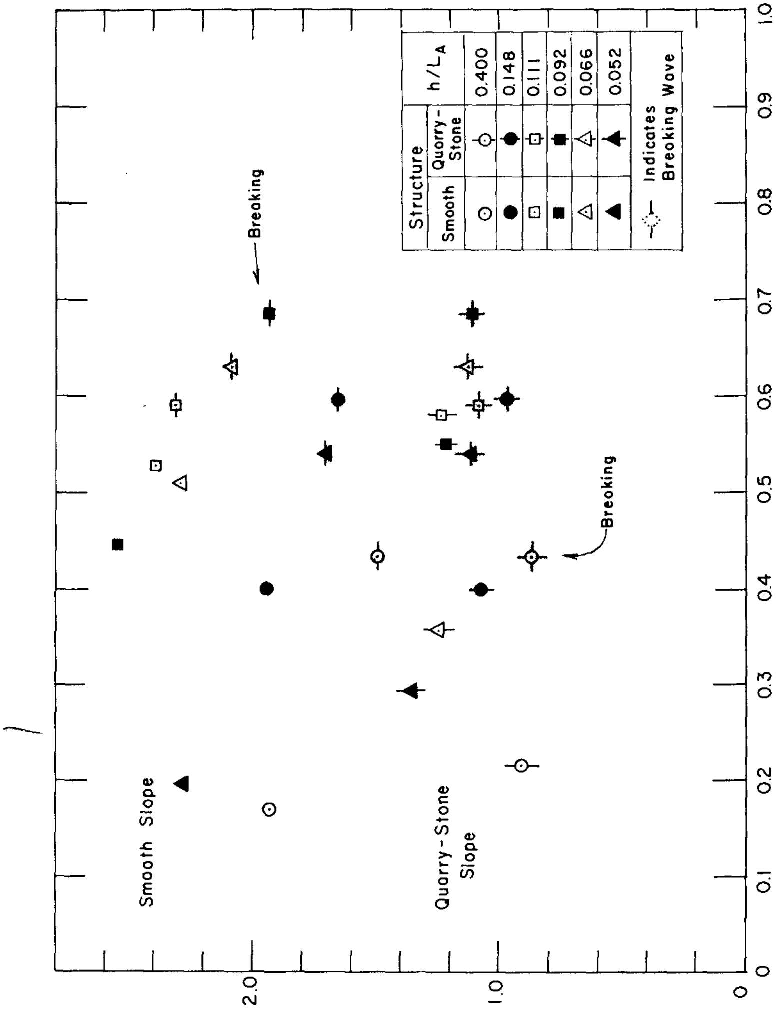

Figure 8. Maximum Relative Run-Up for Non-Breaking and Breaking Waves as a Function of H/h.

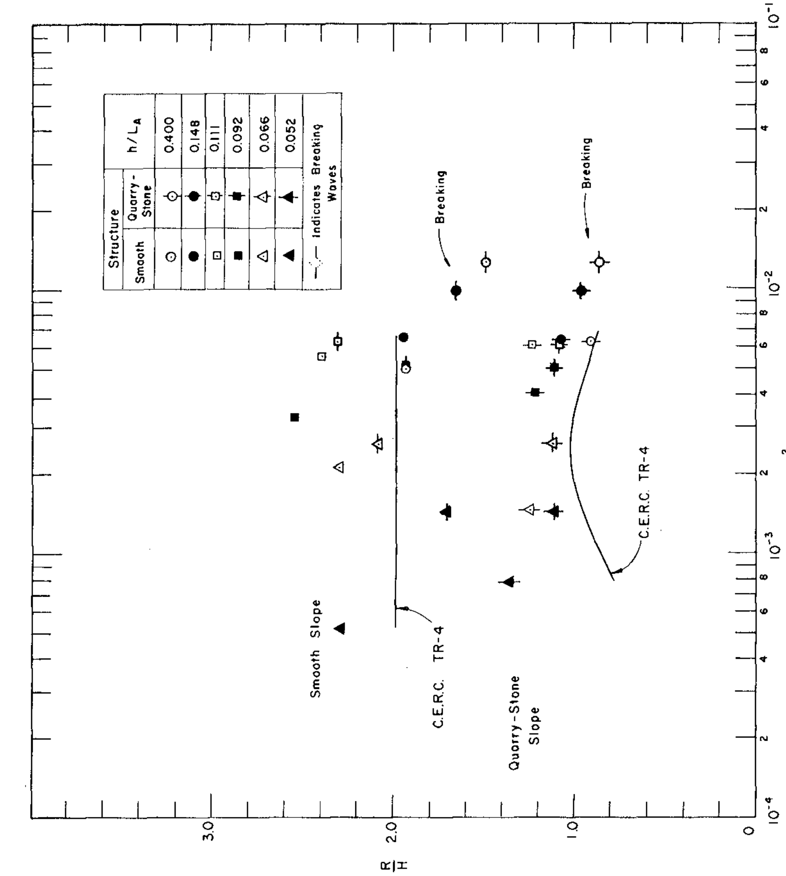

Figure 9. Maximum Relative Run-Up for Non-Breaking and Breaking Waves as a Function of H/gT^2.

4. The Experimental curves for run-up on smooth and quarry-stone slopes from CERC (1966) exhibit somewhat less relative run-up for the same wave steepness when compared to these results. Part of the reason for the difference for the quarry-stone slope may be attributed to differences in the method of construction and part may be the effect of relative depth described previously.

ACKNOWLEDGMENTS

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