Stability Aspects of Nonreshaped Berm Breakwaters with Reduced Armor Weight

Subba Rao[1]; K. Subrahmanya[2]; Balakrishna K. Rao[3]; and V. R. Chandramohan[4]

Abstract: The present work involves the investigation of the influence of wave height, wave period, water depth, and sea-ward slope on the stability, wave runup, and wave rundown of statically stable rubble-mound berm breakwater. The weight of armor stones used in the present study is 20% lighter than the weight that is required for a conventional breakwater, designed using Hudson formula for a wave height of 0.1 m in the model. In the present work models with a berm width of 0.6 m, at constant depth of 0.32 m from the seabed were tested. The damage to the breakwater model with the berm was compared with the results on a model without the berm using different armor weights. The variation of relative runup and rundown was found for different values of wave steepness and water depths in front of the structure. The damage to the breakwater, wave runup, and rundown for the structure with seaward slope 1:2 and 1:1.5 were compared. The investigation was carried out in the Marine Structures Laboratory, Department of Applied Mechanics and Hydraulics, National Institute of Technology Karnataka, Surathkal.

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CE Database subject headings: Breakwaters; Wave runup; Berms; Model studies; Armor units; Damage assessment.

Introduction

Priest et al. (1964), Bruunn and Johannesson (1976), Baird and Hall (1984), and Ergin et al. (1989), showed that *S*-shaped rubble-mound breakwaters (berm breakwaters) are more stable than conventional breakwaters under certain conditions. In *S*-shaped breakwaters, the breaking wave usually does not strike the exposed breakwater slope, but plunges into the gentle (horizontal) part of the breakwater and dissipates their energy over a large area within the berm. This concept allows the designer not only to reduce the armor stone weight, but also to use a wide range of armor stones. The grading of armor stones is usually represented by the ratio D_{85}/D_{15}. Normally, a grading of $D_{85}/D_{15}=1.5$ is used in berm breakwater. Hall and Kao (1991) concluded that the influence of grading on reshaping is less if $D_{85}/D_{15}<3$. A similar conclusion was drawn by Van der Meer (1992) based on the tests conducted on dynamically stable breakwaters with $D_{85}/D_{15}=2.25$.

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The berm breakwaters may be divided into three categories, namely: statically stable nonreshaped, statically stable reshaped, and dynamically stable reshaped (PIANC MarCom W.G. 2003). Berm breakwaters have been adopted at several locations as an economic solution when large cover blocks of natural stone are not available (PIANC MarCom W.G. 2003). It might also be an economical solution even when large cover blocks are available for a conventional breakwater. The uncertainty in wave climate favors a breakwater design that is not too sensitive to the wave height with respect to stability. Hall and Kao (1991) performed basic tests on berm breakwaters, and studied the influence of wave height, period, spectral shape, number of waves, grading, and rock shape on the profile reshaping. Van der Meer (1992) developed a computational model for the profile development of rock slopes and gravel beaches and this can be used in the design of berm breakwaters. Torum et al. (1999, 2003) developed an equation to calculate the recession of the berm of berm breakwaters based on the wave height, period, nominal diameter of stones, gradation factor, and depth factor.

Objective of Investigation

Torum et al. (1999) have found that, in recent years, there has been a drive to design the berm breakwaters in such a way that they will not reshape at all, because the reshaping process may lead to excessive crushing and abrasion of individual stones in the berm breakwaters. Hence the characteristics of statically stable, nonreshaped, breakwaters play an important role in the design of berm breakwaters. In statically stable nonreshaped type breakwaters only a few stones are allowed to move, similar to the condition for a conventional rubble mound breakwater. The acceptable damage criteria similar to those for conventional breakwaters are assumed. There are no specific design criteria available for statically stable berm breakwaters as far as armor stone weight, berm width, location of berm, wave runup, and rundown are concerned. In the present investigation, statically stable nonreshaped two-

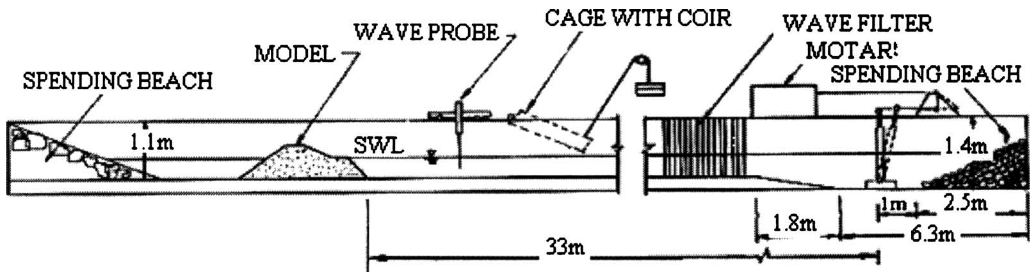

Fig. 1. Details of experimental setup

layer rubble-mound breakwater models with a wide berm were tested. The weights of armor stones used in the present study are compared with armor stone weights required for a conventional breakwater for a wave height of 0.1 m (1:30 model scale). The earlier experiments conducted in the same laboratory on a conventional breakwater model with a 0.6-m-wide berm and 30% lighter armor stones [armor weight, $W_{50}=52$ g m (Subba Rao et al. (2004))], have not shown stable profile for the design wave height of 0.1 m (1:30 scale). The further reduction in armor weight may require an increase in the armor layer thickness and the initial reshaping of the breakwater in order to achieve a stable breakwater. The weight of armor stones used in the present study is 20% lighter than the weight that is required for a conventional breakwater designed using the Hudson formula for a wave height of 0.1 m in the model. The present work investigates the efficiency of the berm on the stability, wave runup, and wave rundown for varying water depths. The stability of the breakwater with respect to the seaward slope is also tested for slopes of 1:2 and 1:1.5

Test Setup

Wave Flume

The wave flume is 50 m long, 0.71 m wide, 1.1 m deep, and has a 42 m long smooth concrete bed. Fig. 1 shows a sketch of the wave flume used in the present work. A bottom-hinged flap generates waves at one end of the deep chamber which is 6.3 m long, 1.5 m wide, and 1.4 m deep. About 15 m length of the flume is provided with glass panels on one side. The flap is controlled by an induction motor of 11 kW and 1,450 rpm. This motor is regulated by an inverter drive (0–50 Hz) rotating with a speed range of 0–155 rpm. Regular waves of height from 0.02 to 0.24 m, and periods from 0.8 to 4 s can be generated with this facility.

Data Acquisition System

Capacitance type wave probes along with amplification units were used for acquiring the data. The probes, along with a computer data acquisition system, were used for acquiring water elevations. The regular waves were analyzed using the concept of "equivalent wave height," given by Darbyshire (1952) and Dattatri (1978). The equivalent wave height H_{eq} is the height of a regular sinusoidal wave, which has the same energy content as the irregular wave system. The total wave energy of the wave system E can be evaluated from the knowledge of $\eta(t)$, the displacement of water surface from the mean water level as

$$E=\frac{\rho g}{T^1}\left(\int_0^{T^1}\eta^2(t)dt\right) \tag{1}$$

in which T^1=time length of wave record. The equivalent wave height H_{eq} may be evaluated as

$$E=\frac{\rho g}{8}(H_{\mathrm{eq}}^2) \tag{2}$$

The digital signals were modified to get physical wave signals using the calibration constant of the wave probe. These modified signals were analyzed using the software in C-Graphics. From the digitized transmitted wave form, H_{eq} was calculated by using a software program. The probes used were calibrated before and after each session of each work day and the average of the probe constant was applied. Runup and rundown were measured manually by using well-marked strips of graph paper pasted along the seaward slope on the glass panel of the flume, both above the berm and below the berm, and then the vertical difference was calculated. This was observed for the initial 50 waves. The accuracy of the measurements was 1 mm.

Breakwater Structure

The typical cross-section details of the breakwater model are as shown in Fig. 2. The model was designed to suit the wave parameters of the Mangalore coast, near the laboratory. A geometrical similarity scale of 1:30 was selected for the present investigation. The weight of the armor stone used in the model is in scale with the weight of armor stones required for a conventional breakwater structure for a wave height of 0.10 m in the model. For this wave height, using the Hudson formula ($K_D=3.5$, $\cot\alpha=2$, $\rho_a=2.74$, $\rho_w=1.0$, two layer, and random placement), the weight of primary armor stone obtained was $W_{50}=73.2$ g m which is denoted as W in this paper. In the present study the weight of the armor stones used was $W_{50}=58.6$ g m, which is 20% less than 73.2 g m and varies in the range of $0.75W_{50}-1.25W_{50}$. The weight of armor stones used in the secondary layer was $W_{50}=5.85$ g m. A horizon-

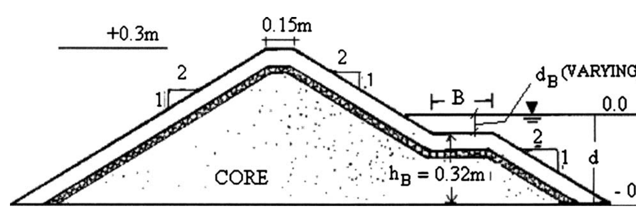

Fig. 2. Cross section of berm breakwater model

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Table 1. Range of Experimental Variables

Sl. number	Variable	Expression	Range
1	Wave height	H	0.10, 0.12, 0.14, 0.16 m
2	Wave period	T	1.6, 2.0, 2.6 s
3	Storm duration	N	3,000 waves
4	Angle of wave attack	θ	90°
5	Water depth above bed level	d	0.25, 0.30, 0.35, 0.40 m
6	Water depth above or below berm	d_B	+0.08, +0.03, −0.02, −0.07 m
7	Armor stone weight	W_{50}	58.56 g m (43.9–73.2)
8	Shape of the armor stone	—	Angular, rounded, flat
9	Crest height above seabed	—	0.70 m
10	Berm width	B	0.60 m
11	Berm position above seabed	h_B	0.32 m
12	Seaward slope		1:2, 1:1.5
13	Specific gravity of armor stone	S_r	2.74

tal berm was constructed at a constant depth of 0.32 m above the seabed. The slope shoreward of the berm and seaward of the berm was similar. The model was constructed at a distance of 33 m from the generator flap. Armor stones used in the present tests were taken from a local quarry. Hence the shape of the stones varied in the range of angular, rounded, and flat. Only very flat stones, where the length of any dimension is more than two times the thickness, is rejected. The core material composed of crushed stones was placed and formed to the required level, and after compacting, the secondary layer, which itself functions as a filter layer, was placed at two stone layer thickness. Primary armor layers, with armor weight $W_{50}=58.6$ g m, were placed over the secondary layer for a two-layer thickness, as mentioned in CEM (2002) for conventional breakwaters. Primary armor stones are placed casually to get a fitted surface. Table 1 shows the details of the range of experimental variables.

The paper also includes the results of earlier experiments conducted in the same laboratory by the writers: (1) on conventional breakwater with armor weight $W_{50}=73.2$ g m; and (2) on conventional breakwater with a wide berm of width 0.6 m and armor weights of $W_{50}=52$ g m, which is 30% less than 73.2 g m (Subba Rao et al., 2004). Hence in order to differentiate these test results, the armor weights used in the model are denoted as $W_{50}=1.0$ W for 73.2 g m, $W_{50}=0.8$ W for 58.6 g m, and $W_{50}=0.7$ W for 52 g m.

Test Procedure

Before starting the experiments the initial seaward profile of the breakwater was recorded using a surface profiler system. The system consisted of nine sounding rods, spaced at 7.5 cm center-to-center, fixed to a wooden frame. These rods could be moved up and down freely. The wooden frame is mounted on a trolley, which moved over the railings provided on the wave flume sidewalls along the length of the flume. The surface along the seaward side of the breakwater was measured at 0.05 m intervals. At every point, sounding was taken to an accuracy of 1 mm. The incident wave height was measured at a distance 2 m from the main breakwater. In order to minimize the reflection from the breakwater structures the experiments were conducted using a burst of a maximum of five waves at a time. The next burst was started after obtaining calm conditions in the flume. A minimum of 3,000 waves or the failure of the breakwater, whichever occurs earlier, was set as the limit for every test run. The damage criteria adopted by Van der Meer (1988) was used to define a no damage condition or a failure of the breakwater. With the exposure of the secondary layer, the breakwater section was considered to have failed.

Profiling of the damaged section was carried out. The damage to the breakwater was quantified by an equation $S=A_e/(D_{n50})^2$ (Van der Meer 1988), where A_e was the eroded area of the breakwater cross section, obtained by comparing the initial and final profiles of the breakwater. D_{n50} was the nominal diameter of the median armor stone and was calculated as $(W_{50}/\gamma_a)^{1/3}$, where W_{50}=50% value of the weight distribution curve of armor stones, and γ_a=specific weight of the armor stone. The influence of wave characteristics on the damage was explained by drawing a graph of damage (S) versus stability number (N_s), and damage versus wave height. With $N_s=H/(\Delta D_{n50})$ also known as the wave height parameter, H is the wave height in front of the structure and $\Delta=((\rho_a/\rho_w)-1)$ is the relative mass density with ρ_a and ρ_w the mass densities of armor stone and water, respectively. The model was rearranged after each completed run and the experiment was repeated for other wave parameters.

Test Conditions

The test conditions involved in the present investigation are as follows:

1. The seabed was horizontal and rigid;
2. Sediment motion was assumed not to interfere with the wave motion and does not affect the model performance;
3. The influence of secondary waves was considered to be negligible; and
4. The density difference between fresh water and seawater was not considered.

Results and Discussion

Influence of Wave Period, Height, and Steepness on Stability

The variation of damage level (S) with wave height (H) for different wave periods is shown in Fig. 3. The graph shows the results of models tested with berm ($B=0.6$ m, $W_{50}=58.6$ g m), and models tested with straight seaward slope, 1:2 (without berm, $W_{50}=73.2$ g m), and under the same test conditions, $d=0.4$ m and $h_B=0.32$ m. From the figure, it is clear that damage is greater when the wave period is 1.6 s. For the design wave height of

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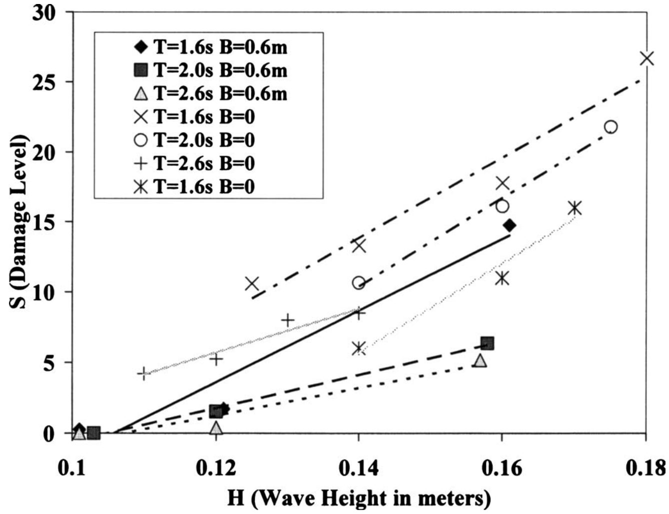

Fig. 3. Influence of wave period on damage level, $d=0.4$ m

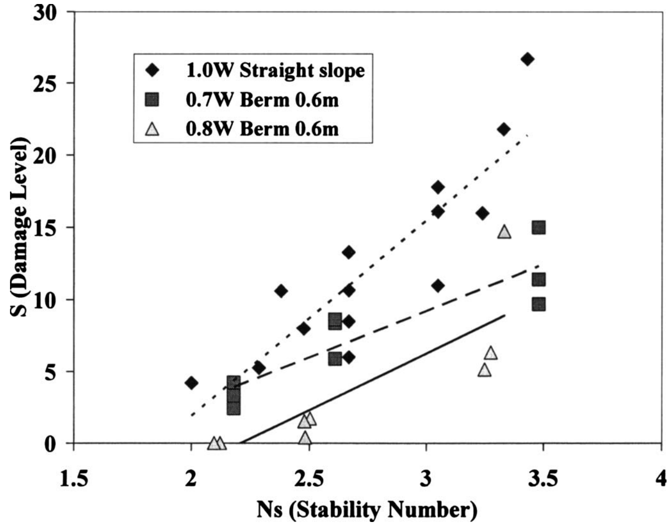

Fig. 4. Influence of armor weight on damage level, $d=0.4$ m

0.10 m, the structure with the berm was not susceptible to any damage under the entire range of wave periods studied. For increased wave height of 0.12 m, the damage was greater for $T=1.6$ s and less for $T=2.6$ s, and for the range of experimental variables studied, the damage level was less than 2. The model tested with straight seaward slope also showed greater damage when $T=1.6$ s and less damage when $T=1.2$ s. The damage was found to increase as the wave steepness increased from 0.015 to 0.05 ($T=2.0$ and 1.6 s), and further increase in wave steepness from 0.07 to 0.08 ($T=1.2$ s) showed the least damage compared to other values of wave steepness studied. In the range of wave steepness ($H_0 \times 2\pi/gT^2$) from 0.05 to 0.008 ($T=1.6$, 2.0, and 2.6 s) the damage was found to be relatively greater for shorter period waves in comparison with longer period waves. The possible reason for this was, shorter period waves in this range, which are of a collapsing nature, have a strong uprush and downrush, leading to larger damage to the structure as compared to surging type long period waves.

Influence of Variation of Water Depth on Stability

The stability of the breakwater was tested for different water depths (measured in front of the structure). The depths were 0.25, 0.3, 0.35, and 0.4 m. The position of the berm was fixed at 0.32 m above the bed level. The structure was found to be stable when the water depth was 0.07 and 0.02 m below the berm and 0.03 m above the berm, for wave heights of 0.10, 0.12, and 0.14 m and wave periods of 1.6, 2.0, and 2.6 s. When the water depth was 0.08 m above the berm, breakwater structures showed no damage for 0.10 m wave height, permissible damage for 0.12 m wave height, and severe damage for 0.16 m wave height (Fig. 3). This indicated that variations in depth of water above the berm influenced the stability of the berm breakwater.

Influence of Armor Stone Weight on Stability

The influence of armor stone weight on the stability is shown in Fig. 4. The graph shows the results of models tested with three different armor weights. $W_{50}=1.0$ W represented the model with straight seaward slope and armor stone weight, $W_{50}=73.2$ g m. $W_{50}=0.7$ W represented the model with a 0.6-m-wide berm and 30% reduction in armor stone weight ($W_{50}=52$ g m). $W_{50}=0.8$ W represented the model with a 0.6-m-wide berm and 20% reduction in armor stone weight ($W_{50}=58.6$ g m). All the models were studied for $d=0.4$ m, $h_B=0.32$ m, and $T=1.6$, 2.0, and 2.6 s. From the graph it is clear that provision of a 0.6-m-wide berm increased the stability of the breakwater to a large extent. The structure with straight seaward slope and $W_{50}=73.2$ g m, was found unstable for a wave height of 0.12 m ($N_s=2.3$) and the structure with 20% reduction in armor stone weight ($W_{50}=58.6$ g m), with the berm, showed a stable profile for the same wave height ($N_s=2.5$) and for all the wave periods tested. The model with 30% reduced armor stone weight showed damage greater than 2, even for the design wave height of 0.10 m ($N_s=2.2$), whereas the model with 20% reduced armor stone weight, the 0.6-m-wide berm, showed zero damage for design wave height of 0.10 m, and damage less than 2 for 0.12 m wave height, for the range of wave periods studied. The damage to the breakwater model tested with $W_{50}=0.8$ W (58.6 g m) is 100% less than that of the model studied with $W_{50}=0.7$ W (52 g m), for wave heights of 0.10 m ($N_s=2.1$ and 2.18) and 75–95% less for wave heights of 0.12 m ($N_s=2.5$ and 2.6) for all the wave periods studied. For the 0.10 m wave height, the model with 30% reduction in armor stone weight showed damages between 2 and 4, indicating that a further increase in berm width may produce a stable structure for the same wave height.

A wide scatter of points can be observed in the graph, especially for models tested with 1.0 and 0.7 W and for N_s values greater than 2.5. In the present work, the influence of each wave height for different wave periods was studied. Hence for a single stability number, the graph shows different damage levels corresponding to different wave periods. As the wave height increases the stability of the breakwater decreases and the influence of wave period on damage becomes predominant. It is observed from the graph that the variation in damage values for the different wave periods is less for lower wave heights. The scatter of points indicated that the structure was no longer statically stable and the influence of wave period was significant in this case.

Influence of Seaward Structure Slope on Damage Level

The variation of damage level (S) with wave height (H), for initial seaward slope 1:2 and 1:1.5 of the breakwater, is plotted in

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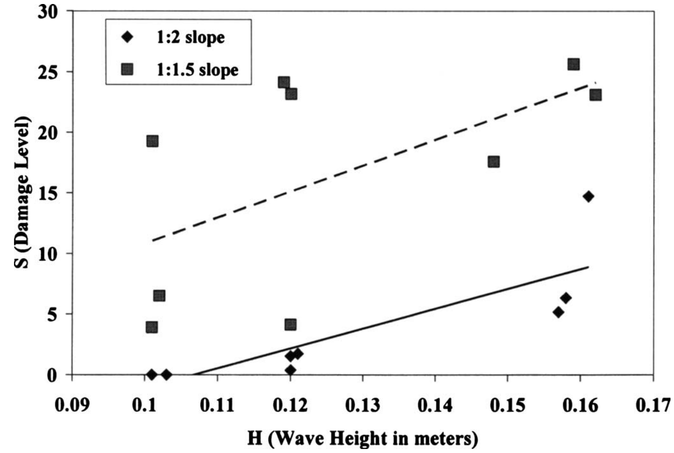

Fig. 5. Influence of seaward slope on damage level, $d=0.4$ m

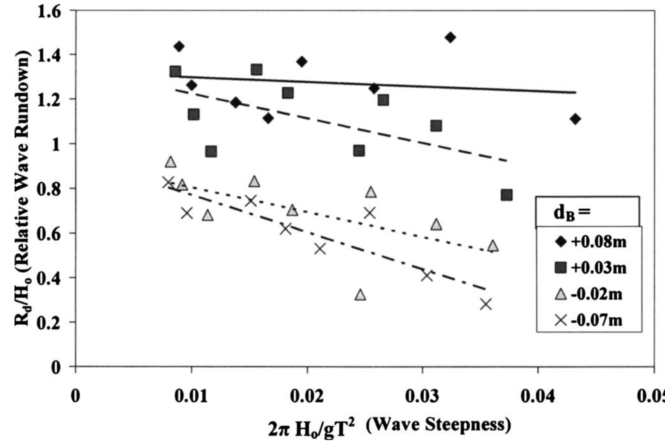

Fig. 7. Influence of variation in water depth on rundown

Fig. 5. The percentage of variation of damage level for the slope 1:2 was significantly less than that for slope 1:1.5. In the case of the steeper slope of 1:1.5, the reflected waves of a wave group lead to turbulence above the berm, which produced greater damage. A greater scatter of points was observed for models studied with a seaward slope of 1:1.5 when compared with a seaward slope of 1:2 for all the wave periods studied. This is an indication of uncertainty of the stability of the steeper slope for different wave periods when compared to the flatter slopes studied. As the slope changes from 1:1.5 to 1:2 the damage decreased to zero for 0.10 m wave height, decreased by 90% for 0.12 m wave height, and 36–75% for 0.16 m wave height.

Influence of Variation of Depth of Water above and below Berm on Runup

Fig. 6 shows the influence of variation in water depth in front of the structure on relative wave runup with wave steepness. The seaward slope above and below the berm was 1:2. The term d_B indicates the depth of water above and below the berm level. It is clear from the figure that as wave steepness increased, the relative wave runup (R_u/H_0) decreased for all the water depths considered in the present investigation. This trend of variation is similar to the trend given in CEM (2002) and Van der Meerand Stam (1992). The variation of nondimensional runup was 0.55–1.15 for a wave steepness variation of 0.008–0.043, with variation in water depths of +0.08 to −0.07 m above and below the berm, and B/d ratio of 2.4, 2.0, 1.71, and 1.5, where B was the berm width and d was the still water depth in front of the structure. The runup was greater for longer period waves in comparison with waves of a shorter period. Therefore the longer period waves were more damaging from the runup point of view. When the water depth was −0.07 m (below the berm), the wave runup was found to be more than the water depth at −0.02 m (below the berm). When the water depth was −0.02 m, where SWL was approximately the berm level, the waves attacked on the downward slope of the seaward side, and to some extent on the berm, and thus effectiveness of the berm resulted in minimum runup. The water depths at +0.03 and −0.07 m showed almost the same runup. It was observed that the extent of runup was higher for +0.08 m water depth. Even though the wave broke on the berm portion, the attack of waves onto the upward slope was strong enough to produce higher runup. From the above data it may be concluded that magnitude of wave runup is less when the berm is located close to the still water level.

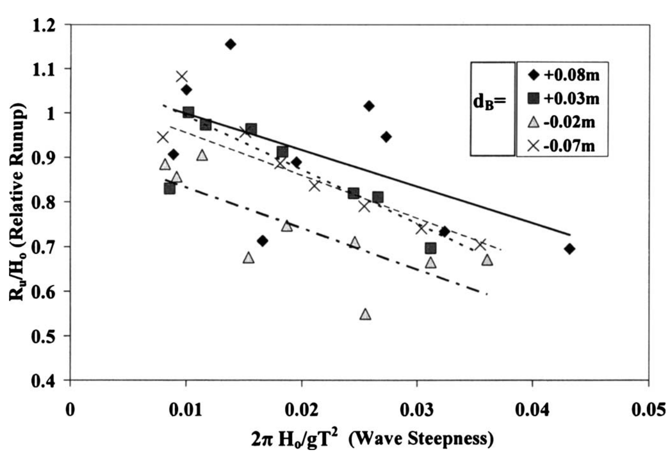

Fig. 6. Influence of variation in water depth on runup

Influence of Variation of Depth of Water above and below Berm on Rundown

The influence of variation in the water depth in front of the structure on relative wave rundown with wave steepness is given in Fig. 7. It is clear from the figure that as wave steepness increased, relative wave rundown (R_d/H_0) decreased. It may also be observed that the lines for −0.07 and −0.02 m, and for 0.03 and +0.08 m water depths, tend to converge at lower wave steepness with a diverging trend at higher wave steepness. As the depth of water decreased the rundown also decreased. The variation of nondimensional rundown (R_d/H_0) was 0.28–1.72 for a wave steepness of 0.008–0.043, for water depths of −0.07–+0.08 m below and above the berm. From Fig. 7 it may be observed that for water depth of +0.08 m, the trend line remains higher and almost constant for the entire range of wave steepness values. From Figs. 6 and 7, it is also observed that for water depth of +0.03 and +0.08 m, where SWL is above the berm, the rundown was more than runup, and for −0.07 and −0.02 m water depth, where SWL is below the berm, the rundown was less than runup.

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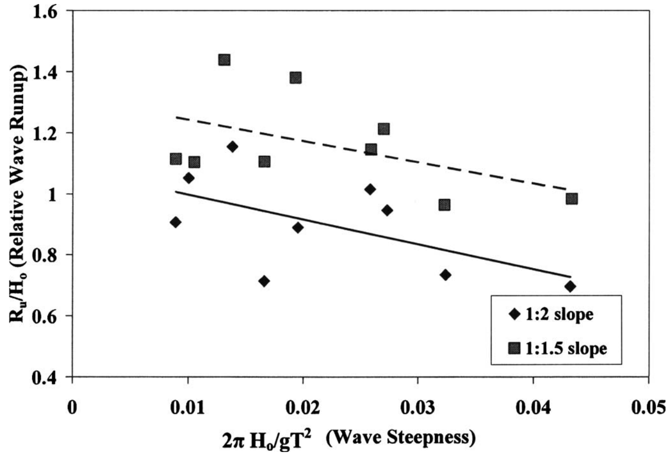

Fig. 8. Influence of seaward slope on runup, $d=0.4$ m

Influence of Seaward Slope on Wave Runup and Rundown

The variation of relative runup and rundown versus wave steepness $(2\pi H_0/gT^2)$ for initial slopes of 1:1.5 and 1:2 with water depth of +0.08 m above the berm is shown in Figs. 8 and 9 respectively. Fig. 8 shows that runup decreases as the slope decreases. Percentage decrease in wave runup by 4.78–35.63% has been observed for structures with an initial slope of 1:2 in comparison with that for structures with an initial slope 1:1.5. A reverse trend is observed in Fig. 9 for rundown, which decreases as slope changes from 1:2 to 1:1.5 for higher wave steepness values studied.

Uncertainty Analysis

Hydrodynamic testing facilities differ from one another with regard to size, instrumentation, experimental procedures, and scale. Hence, it becomes necessary for a test facility to provide possible lower and upper margins, which can be adopted with a fair degree of confidence. The width of confidence intervals is a measure of the overall quality of the regression line. The 95% confidence interval limits must always be estimated and this concept of confidence level is fundamental to uncertainty analysis. A confidence

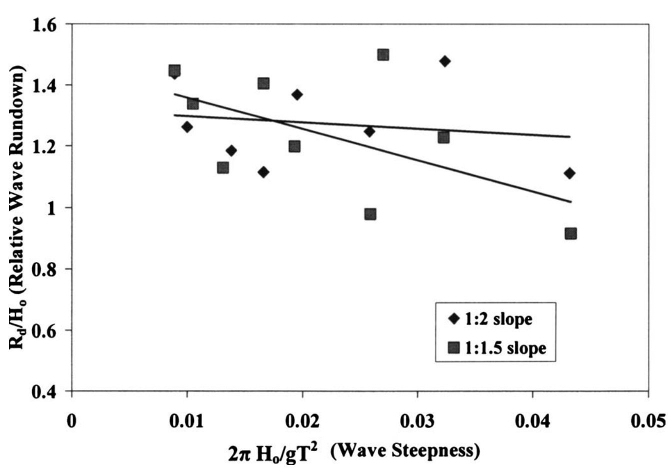

Fig. 9. Influence of seaward slope on rundown, $d=0.4$ m

Table 2. Results of Uncertainty Analysis (95% Confidence Bands) for Variation of Damage Level with Stability Number

x_0 (stability number)	Y_0 (damage level)	Upper limit	Lower limit
2.2	0.1872	0.6378	−0.2994
2.4	1.1963	1.6031	0.7895
2.6	2.2053	2.5979	1.8127
2.8	3.2144	3.6646	2.7642
3.0	4.2235	4.7812	3.6658
3.2	5.2326	5.9249	4.5403
3.4	6.2417	7.0828	5.4006

interval for variation of damage level with stability number for $W_{50}=0.8$ W and $d=0.4$ m is shown in Table 2, calculated using the method suggested by Montgomery and Runger (1999). The slope and intercept of the regression line (regression coefficients) for the experimental data are obtained by the method of least squares. The fitted or estimated regression line is therefore, $Y_0=\beta_0+\beta_1 x_0$, where β_0 and β_1 are regression coefficients. Y_0 is the dependant or response variable, and x_0 is regression or predictor variable. A 100(1-α) percent confidence interval about the mean response at the value of $x=x_0$, say, then Y_0 is given by

$$Y_0 = Y_0 \pm t_{(\alpha/2,n-2)}\sqrt{\left(\sigma^2\left[\frac{1}{n}+\left(\frac{(x_0-\bar{x})^2}{S_{xx}}\right)\right]\right)} \tag{3}$$

in which α=significance level used to compute the confidence level; σ=variance; n=sample size; $\bar{x}$=sample mean; x=variable; S_{xx}=standard deviation; and $t_{(\alpha/2,n-2)}$=t-distribution values for (n-2) degrees of freedom. Fig. 10 shows the plot the 95% confidence bands for the variation of damage level with stability number. It was observed that 87% of the points lie within the 95% confidence bands.

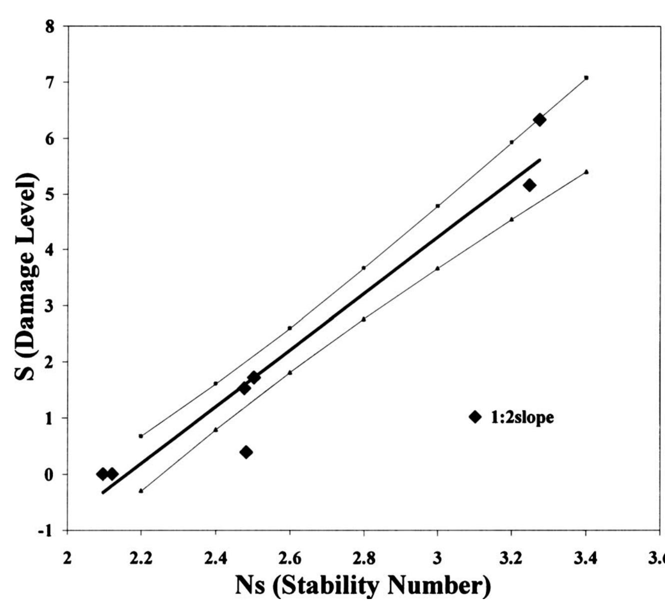

Fig. 10. 95% confidence band for variation of damage level (S) with stability number (N_S), $W_{50}=0.8$ W, $d=0.4$ m

Conclusions

Based on the present investigation, the following conclusions are drawn:

1. Damage was relatively significant for shorter period waves in comparison with longer period waves, in the range of wave steepness ($H_0 \times 2\pi / gT^2$), from 0.043 to 0.008;
2. Variations in water level above the berm influenced the stability of the berm. The structure is found to be more stable when water depth above the berm is smaller;
3. The structure with a straight seaward slope and $W_{50} = 73.2$ g m was found unstable for a wave height of 0.12 m ($N_s = 2.3$), and the structure with 20% reduction in armor weight ($W_{50} = 58.6$ g m) with berm showed a stable profile for the same wave height ($N_s = 2.5$) and for all the wave periods tested. The damage to the breakwater model with $W_{50} = 0.8$ W (58.6 g m) was 100% less than that for the model studied with $W_{50} = 0.7$ W (52 g m), for wave heights of 0.10 m, and 75–95% less for wave heights of 0.12 m for all the wave periods studied;
4. As the structure slope changed from 1:1.5 to 1:2, the damage decreased to zero for 0.10 m wave height; decreased by 90% for 0.12 m wave height; and 36–75% for 0.16 m wave height;
5. As wave steepness increased, the relative wave runup (R_u / H_0) and rundown (R_d / H_0) decreased for all the water depths considered in the present investigation;
6. The variation of relative runup (R_u / H_0) was 0.55–1.15, and relative rundown (R_d / H_0) was 0.28–1.72 for wave steepness variation of 0.008–0.043 for all the water depths considered. The magnitude of wave runup was less when the berm was located close to the still water level. The rundown was found to be more significant than the runup when SWL was above the berm and rundown was less than runup when SWL was below the berm; and
7. The change in seaward slope from 1:1.5 to 1:2 reduced the runup by 4.78–35.63% for the range of variables considered in the test, whereas rundown did not show any particular trend as the slope changed from 1:1.5 and 1:2.

Notation

The following symbols are used in this paper:

A_e = eroded area of sea-ward profile;
B = berm width;
D_{n50} = nominal diameter of median armor stone;
D_{15} = 15% of stones have diameter less than D_{15}, $D_{15} = (W_{15} / \rho_a)^{1/3}$;
D_{85} = 85% of stones have diameter less than D_{85}, $D_{85} = (W_{85} / \rho_a)^{1/3}$;
d = water depth in front of structure;
d_B = water depth above or below berm level ($\pm$);
g = gravitational acceleration;
H = wave height in front of structure;
H_0 = deep water wave height;
h_B = berm height above sea bed;
N_s = stability number;
n = sample size;
R_d = wave rundown;
R_u = wave runup;
S = damage level;
T = wave period;
t = t-distribution values from statistical table;
W = armor stone weight calculated using Hudson formula for 0.1 m wave height in model and $K_D = 3.5$;
W_{15} = 15% value of weight distribution curve of armor stones used in model;
W_{50} = 50% value of weight distribution curve of armor stones used in model;
W_{85} = 85% value of weight distribution curve of armor stones used in model;
x = variable;
$\bar{x}$ = sample mean;
α = significance level used to compute confidence level;
β_0 = regression coefficients—intercept;
β_1 = regression coefficients—slope;
γ_a = specific weight of armor stone;
Δ = relative mass density;
ρ_a = density of armor stone; and
ρ_w = density of water.

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