Stability of mound breakwaters: dependence on wave reflection.

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Abstract

Since the work by Ahrens and McCartney (1975) and Bruun and Johannesson (1976), it has been accepted that, for given slope, unit type and damage level, the stability number, N_s, and the so called K_d factor depend on Iribarren number, I_r, and that the worst stable conditions are related to collapsing wave breakers. The plotted values of N_s and K_d from laboratory tests, as a function of I_r, show a large scatter. This scatter is larger for large values of I_r. It is shown here that the scatter almost disappears once the data are plotted against kh, or I_{rt}, using the total wave height at the toe of the structure instead of the incident wave height. The total wave height, H_t, results from the interaction of the incident and reflected wave trains on the slope and depends not only on the magnitude of the reflection coefficient but also on its phase.

Introduction

The weight of the armor unit of a rubble mound breakwater depends on the geometry of the mound, the type of units, the characteristics of the incoming wave train represented by the wave height and period, and on the level of damage. One of the early formulas to calculate the weight of the armor units, W, of a mound breakwater was given by Castro (1933). Iribarren (1938, 1965) and Iribarren and Nogales (1950, 1954) based on this former work, proposed a more general formula, which was tested under monochromatic waves. Hudson (1959) proposed a simplified formula using the so called K_d factor or stability coefficient. These formulas based on monochromatic wave tests are still being used in many countries. All of them can be written as follows (Losada and

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Giménez-Curto, 1979):

$$W = \gamma_w \cdot \Psi \cdot S \cdot H^3 \tag{1}$$

with,

$$S = \frac{\dfrac{\gamma_s}{\gamma_w}}{\left(\dfrac{\gamma_s}{\gamma_w} - 1\right)^3} \tag{2}$$

where H is the incident wave height, H_i, which will be used later to differentiate H_i from H_t, γ_s and γ_w are the unit weight of armor unit and water, respectively. Ψ is the stability function, which is related to the K_d factor by,

$$K_d = \frac{1}{\Psi \cdot \cot\alpha} \tag{3}$$

where α is the angle of the structure slope. The stability number is given by

$$N_s = (K_d \cot\alpha)^{1/3} \tag{4}$$

For given slope, unit type and damage level, K_d, N_s and Ψ depend on Iribarren number, I_r, defined as follows:

$$I_r = \frac{\mathrm{tg}\,\alpha}{\sqrt{\dfrac{H}{L_0}}} \tag{5}$$

where L_0 is the linear wave length in deep water and H is the characteristic wave height, H_i or H_t.

For irregular waves there is no unique characteristic wave height. Van der Meer (1988) recommends to use the significant wave height, H_s; The Shore Protection Manual (1984) recommends to use higher wave heights such as $H_{1/10}$ or $H_{1/20}$. In order to consider the influence on the damage level of the number of waves in the train higher than a certain height, Vidal et al. (1995) examined characteristic wave height statistics and recommended $H_{1/20}$.

Moreover, K_d, N_s and Ψ, based on the experiments using any of those characteristic

wave heights, and plotted against Iribarren number, showed a considerable scatter. Figure 1 shows Ψ versus I_r obtained from the experiments carried out by Iribarren and Nogales (1965) for several slopes. Furthermore, the best fit curve to the data points was given by Losada and Giménez-Curto (1979). The data points scattered about the best fit curve by a factor of about 3. This empirical scatter exists for both monochromatic and irregular waves. Figure 2 shows the stability number versus I_r for quarry stones given by Van der Meer (1988). The data were plotted in terms of the nominal diameter of the unit, that is proportional to the 1/3 power of the weight. The scatter appears to be reduced because of the use of $W^{1/3}$, but is still apparent.

Figure 3 summarizes the scatter of the experimental results depending on Iribarren number, where $\Delta\Psi$ and Ψ_{min} are the range and minimum value of Ψ, respectively, in each range of I_r. The largest scatter occurs under collapsing and surging waves. For these types of waves, mound breakwaters reflect a considerable potion of the incident energy: more than 60% of the incoming wave energy may be reflected.

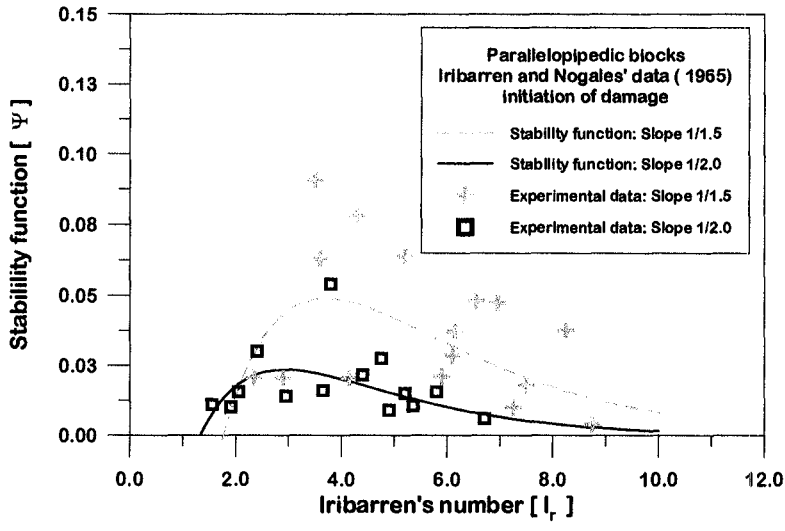

Figure 1. Stability function of parallelopipedic blocks versus I_r for monochromatic wave test.(Losada and Giménez_Curto, 1979)

The interaction of the incident and reflected wave trains leads to a partial standing wave in front of the structure. Flow kinematics and dynamics on the slope depends on this partial standing wave pattern.

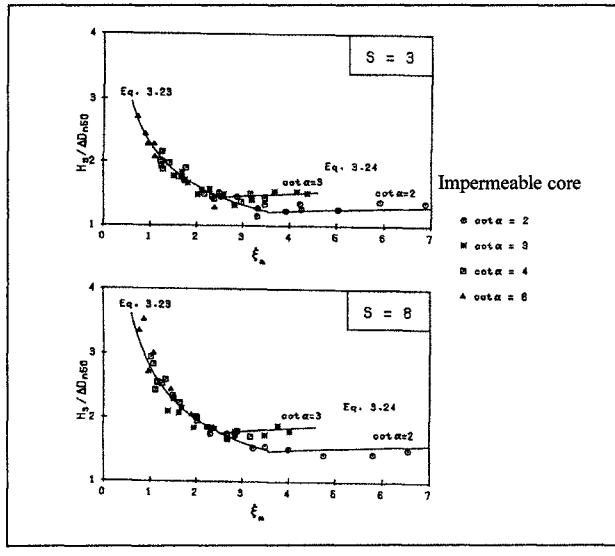

Figure 2. Stability results for irregular waves. (Van der Meer, 1988)

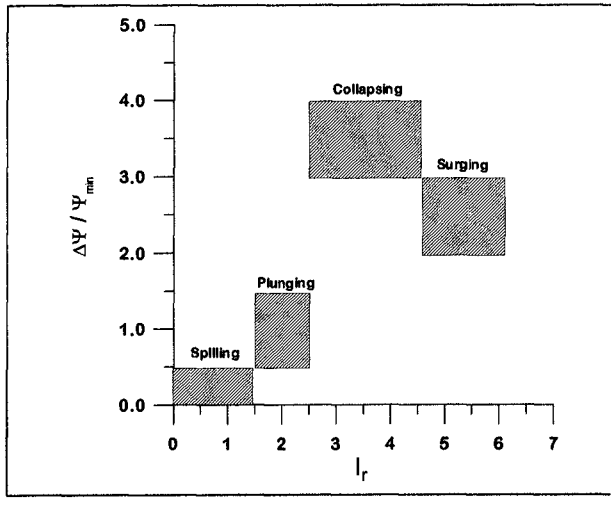

Figure 3. Scatter of monochromatic wave data depending on Iribarren number.

In this paper, the connection between the partial standing wave and the stability of the units of the cover layer is analyzed. First, the reflection process is considered and the magnitude and phase of the reflection coefficient are evaluated. Next, the total wave height of the incident and reflected wave trains at the toe of the structure is calculated. Finally, this total wave height is used to calculate the stability function, Ψ_t, for the experiments by Iribarren and Nogales (1965) and the new tests conducted in the flume of the University of Cantabria. It is concluded that, for plunging-collapsing, collapsing and surging waves, the stability function, Ψ_t, does not depend on Iribarren number, but only on the breakwater slope, unit type and damage level. Furthermore, the use of Ψ_t based on the total wave height at the structure toe is shown to reduce the data scatter.

Wave Reflection from Porous Structures

Wave reflection from permeable rubble mound breakwaters were examinated by Ahrens and McCartney (1975), Losada and Giménez-Curto (1979), van der Meer (1988), Wurjanto and Kobayashi (1992) and Seelig and Ahrens (1995), who showed the dependence of the reflection coefficient on Iribarren number. More recently, Hughes and Fowler (1995), and Sutherland and O'Donoghue (1998), analyzed the phase shift of the reflected wave train at the toe of coastal structures. The modulus, R, and the phase, ε, of the reflection coefficient as a function of the structure geometry and the hydraulic properties of the porous medium were discussed only partially.

For vertical porous medium the dependence of the reflection coefficient on those factors can be analyzed theoretically following the work by Dalrymple et al. (1991) and Méndez (1997). Figure 4 shows the modulus and phase of the reflection coefficient versus kh, were k is the wave number in front of the structure and h is the water depth, for a breakwater located at a certain distance from the flume end wall. Several values of the breakwater width, b, are considered, where the porosity $P = 0.45$ and the median diameter $D_{50} = 0.03$ m. for the porous material. This figure clearly shows that the reflection coefficient modulus and phase depend strongly on the experimental setup.

If the breakwater is sloped, waves breaking may occur, but similar patterns may be found. Figure 5 shows the modulus, R, of the reflection coefficient versus the width and porosity of the cover layer, and Figure 6 shows the dependence of R on the slope angle, α. Both figures are obtained by the numerical computation using PBREAK (Wurjanto and Kobayashi, 1992).

Figure 7 shows the envelope of the wave crest on the slope and in front of the breakwater. The data points were recorded by Iribarren and Nogales (1965), and the curve was obtained by the best fit of the wave envelope to the experimental data points. Two different cases of kh for $\cot \alpha = 2.0$ are shown, where h is the water depth at the structure toe. In Figure 7a, for kh=0.47, the underwater slope is under the node of the standing wave. For the second case, kh=1.26 in Figure 7b, the underwater slope is under the antinode of the standing wave.

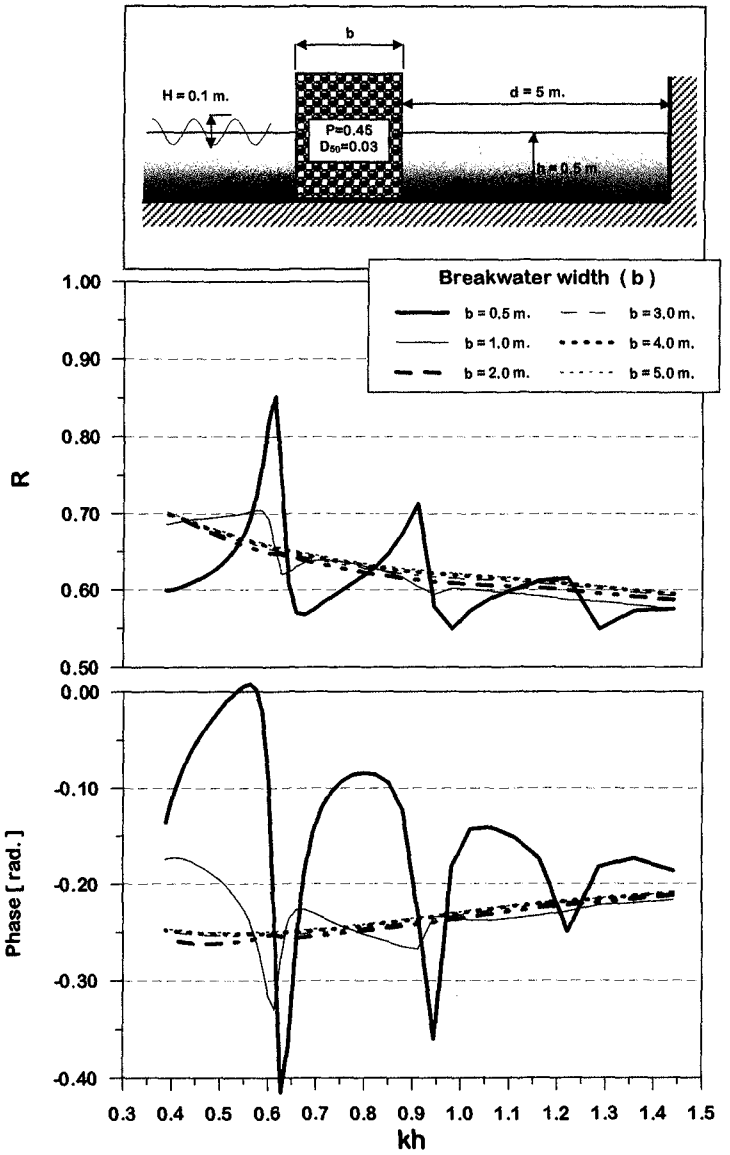

Figure 4. Modulus and phase of reflection coefficient versus *kh* for breakwater located at 5 m. from the flume end wall.

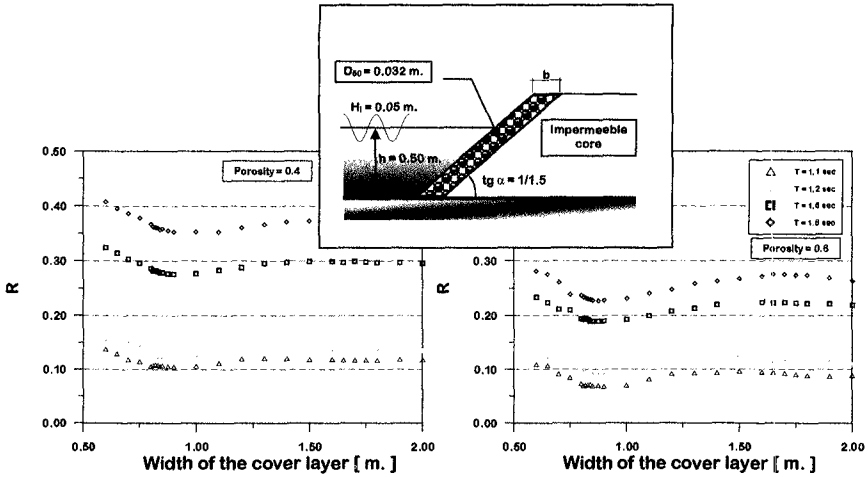

Figure 5. Modulus, *R,* of the reflection coefficient, computed using PBREAK, versus the width and the porosity of the cover layer.

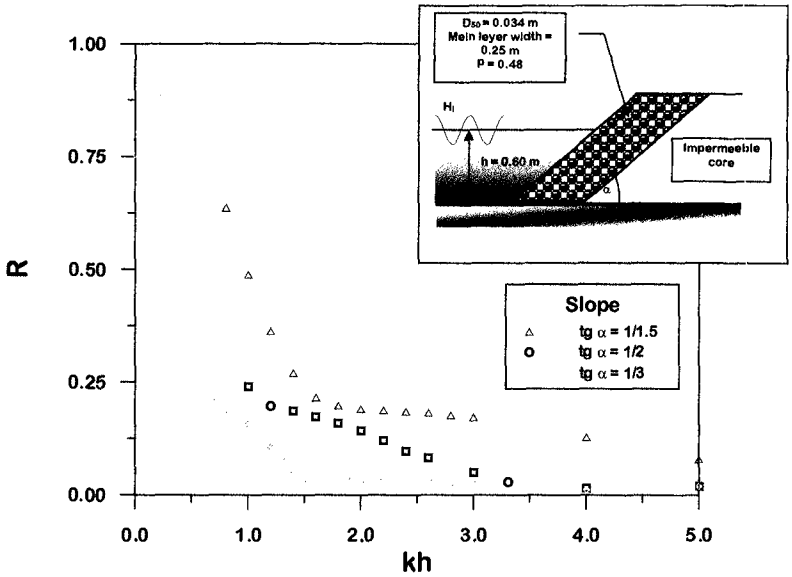

Figure 6. Modulus, *R*, of the reflection coefficient, computed using PBREAK, versus *kh* for different slopes.

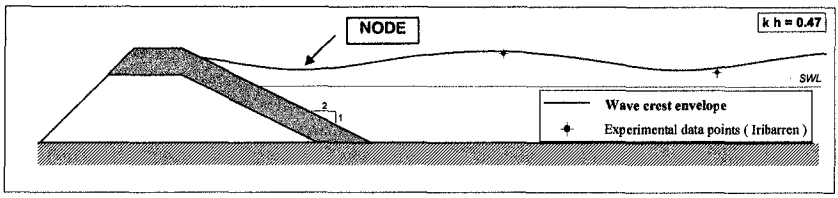

Figure 7a. Envelope of the wave crest on the slope and in front of the breakwater for kh=0.47 with node on underwater slope.

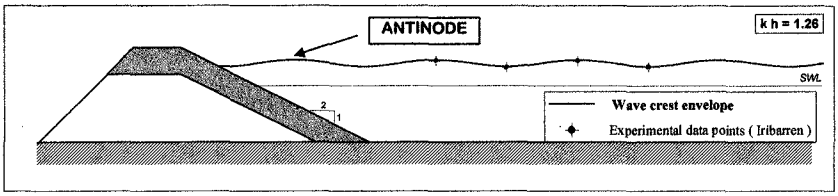

Figure 7b. Envelope of the wave crest on the slope and in front of the breakwater for kh=1.26 with antinode on underwater slope.

From these results it can be concluded that the characteristics of the reflected waves depend strongly on the structure geometry, materials properties and experimental setup. Once the modulus and phase of the reflection coefficient is known, the total wave height, H_t, at the toe of the structure can be evaluated as follows:

$$H_t = H_i\left(1 + R^2 + 2R\cos(2kx + \varepsilon)\right)^{1/2} \tag{6}$$

where H_i is the incident wave height and R is the reflection coefficient modulus

$$R = \frac{H_r}{H_i} \tag{7}$$

where H_r is the reflected wave height, ε is the phase shift obtained from the experimental data and x is the cross-shore coordinate with x=0 at the toe of the structure.

In the following, if the K_d factor, the stability number, N_s or the stability function, Ψ, are calculated using H_t as the characteristics wave height, the empirical scatter shown in Figure 1 turns out to be reduced significantly.

Stability Analysis

The stability of armor units placed on the cover layer is analyzed using both incident and total wave heights. First, the experimental data of Iribarren and Nogales (1965) on the stability of uniform quarry stones for four slopes, *cot* α = 1.25, 1.50, 2.0 and 3.0 are re-analyzed. The K_d factor and the stability function, Ψ, are computed using the incident wave height, H_i, and the total wave height, H_t.

Figure 8 shows the values of the stability function,Ψ, given by (1), plotted as a function of *kh*, using the incident wave height, in Figure 8a, and the total wave height, in Figure 8b. Figure 8b, shows that the experimental points for each slope follow approximately a straight line. The value of the stability function decreases with the decrease of the slope, *tan* α. The dependence of the stability function on *kh* is very weak. For slopes *cot* $\alpha \geq 1.5$ and the range $kh < 2.5$, Ψ is practically constant for each slope and independent of the wave period.

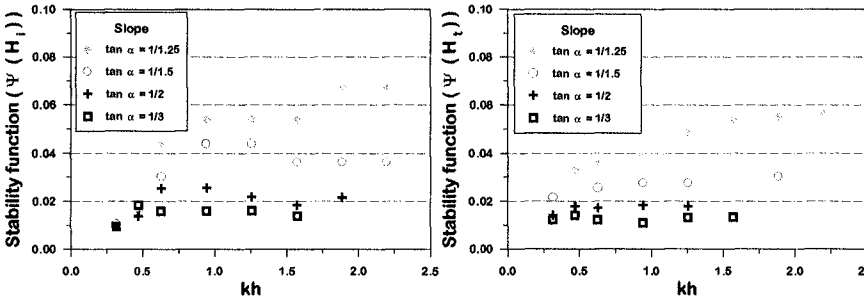

Figure 8. Stability function versus *kh* for the data of Iribarren and Nogales (1965): (a) using the incident wave height and (b) using the total wave height.

New experiments at the University of Cantabria

Additional experimental tests were performed in the flume of the University of Cantabria. The experimental setup is summarized in Table 1.

Table 1 – Characteristics of the flume.

Length	68.5 m.
Width	2 m.
Depth	2 m.
Wave board	Piston

- Experimental technique and damage criteria.

A burst of waves were generated, ensuring that the re-reflected waves from the paddle did not impinge on the slope. After the water surface calmed, a new burst of the same number of waves was generated. Damage was evaluated by counting the number of units displaced by a nominal diameter from its placed position. The test was repeated with the same burst until no unit was displaced by the burst. The slope was not repaired after the burst. Two levels of damage were considered: (1) *Iribarren damage*, defined by the number of displaced units at the time when the second layer of the cover layer was attacked by the waves, and (2) *initiation of damage*, defined as 5% of the number of units corresponding the Iribarren damage were displaced, as explained in detail by López (1998).

- Characteristic of tests.

The monochromatic waves characteristics are given in Table 2. The characteristics of the mound breakwater are summarized in Table 3.

Table 2 – Characteristics of monochromatic waves.

TEST	DEPTH (m.)	PERIOD (sec.)	*kh*
ES1	0.50	1.05	1.90
ES2	0.50	1.07	1.845
ES3	0.50	1.20	1.533
ES4	0.50	1.35	1.28
ES5	0.50	2.0	0.77

Table 3 – Characteristics of mound breakwater.

Slope (sea side)	Slope (lee side)	Height	Crown width	Main layer	Core (D_{50})
1/1.5	1/1.25	0.90 m	0.50 m.	Cubic units 3.2 x 3.2 cm 0.07 kg.	1-3 cm.

Results

Figure 9a shows the stability functions, Ψ_i and Ψ_t, versus *kh* for the new monochromatic wave tests using the incident wave height and the total wave height, respectively, for the initiation of damage. The values of Ψ_i are in the wide range 0.04-0.14, while Ψ_t is in the narrow range 0.06-0.075. The same trend is found for the stability functions plotted versus Iribarren numbers, $I_{r,i}$ and $I_{r,t}$, calculated using the incident and total wave heights as shown in Figure 9b.

For the Iribarren damage, as defined above, the values of Ψ_t plotted versus *kh* are in the narrower range 0.038-0.043.

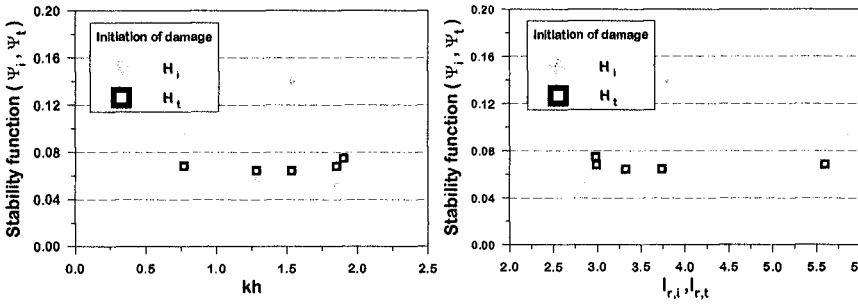

Figure 9. Stability function using the incident wave height and using the total wave height: (a) versus *kh* and (b) versus Iribarren number's. for new monochromatic wave test at the University of Cantabria.

Conclusions

From this research it may be concluded that:

(1) Wave reflection from the breakwater modifies the incident wave characteristics such that the flow velocities and accelerations on the slope are produced by the interaction of the incoming and reflected wave trains.

(2) The values of the K_d factor and the stability function, Ψ, depend on the total wave height, H_t. The scatter of the experimental data points can be reduced significantly by the use of H_t. The reduction is greater for larger damage.

(3) For the practical application of this method, the reflection coefficient modulus, *R*, and its phase shift, ε, produced by the reflection process on the breakwater need to be evaluated in advance. *R* and ε depend on the geometry and the hydraulic properties of the coastal structure as well as the boundary condictions on the landward side of the

structure.

(4) The extension of this method to irregular waves is being examined to clarify the connection between the irregular wave reflection and armor stability.

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