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Hydraulic stability of rock armors in breaking wave conditions

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ABSTRACT

Armor layers of mound breakwaters are usually designed with empirical formulas based on small-scale tests in non-breaking wave conditions. However, most rubble mound breakwaters are constructed in the depth-induced breaking zone, where they must withstand design storms having some percentage of large waves breaking before reaching the structure; in these cases, the design formulas for non-breaking wave conditions are not fully valid. To characterize double-layer rock armor damage in breaking wave conditions, 2D physical model tests were carried out with a bottom slope $m = 1/50$. In order to develop a simple method to determine the wave parameters in the depth-induced breaking zone, experimental wave measurements were compared to the numerical estimations given by the SwanOne model. An analysis was conducted to select the best characteristic wave height to estimate rock armor damage when dealing with depth-induced breaking waves; the spectral significant wave height, H_{m0}, estimated at a distance of $3h_s$ seaward from the structure toe, was found to be the most adequate. A new hydraulic stability formula is proposed for double-layer rock armors in breaking wave conditions, considering the observed potential 6-power relationship between the equivalent dimensionless armor damage and the H_{m0} at $3h_s$ seaward distance from the structure toe.

1. Introduction

Iribarren (1938, 1965), Hudson (1959), USACE (1975, 1984); Van der Meer (1988a), Melby and Kobayashi (1998), Van Gent et al. (2003) and others have reported different hydraulic stability formulas to design rock armors. Most of these formulas are based on tests carried out in non-breaking wave conditions. Some empirical modifications have been proposed to estimate rock armor damage in breaking wave conditions in which the largest waves break before reaching the structure due to the depth effect; nevertheless, very few physical model tests have been conducted to validate formulas in breaking wave conditions.

Armor design in breaking wave conditions usually involves estimating an incident characteristic wave height at the toe of the structure, typically the significant wave height $H_s = H_{1/3}$ (average of one-third highest waves) or a wave height with a prescribed low exceedance probability ($H_{1\%}$, $H_{2\%}$, etc.). When wave heights approximately follow the Rayleigh distribution (deep water), $H_{1\%}$ and $H_{2\%}$ are highly correlated to H_s; however, this does not occur when the wave height distribution is affected by wave breaking. Numerous attempts have been made to find a distribution for wave heights in the depth-induced wave breaking zone. Glukhovsky (1966) proposed using a Weibull distribution; Hughes and Borgman (1987) proposed a Beta-Rayleigh distribution; Battjes and Groenendijk (2000) used a composite Weibull distribution (CWD); Mendez et al. (2004) proposed a modified Rayleigh distribution, while Mendez and Castanedo (2007) and others recommended a distribution for the maximum wave height. Nonetheless, the hydraulic stability formulas found in the literature rarely take into consideration the relevant change of wave height distribution in the depth-induced wave breaking zone.

This study focuses attention on the hydraulic stability of double-layer rock armors in breaking wave conditions. To this end, new physical model tests were carried out at the wave flume of the Laboratory of Ports and Coasts at the *Universitat Politècnica de València* (LPC-UPV) with a $m = 1/50$ bottom slope. Experimental wave measurements were compared to the estimations provided by the SwanOne numerical model (see Verhagen et al. (2008)) in order to establish a rational procedure to determine the wave characteristics in the depth-induced breaking zone. Using both experimental results and SwanOne estimations, an analysis was conducted to identify which characteristic wave height and distance from the toe structure best determine armor damage evolution in breaking wave conditions. In this paper, existing formulas to design rock armor layers in breaking wave conditions are first compared. Secondly, the experimental setup and SwanOne simulations are described. Thirdly, results are analyzed. Fourthly, a new hydraulic stability formula for rock armors in breaking wave conditions is given and a comparison with existing formulas is provided. Finally, conclusions are drawn.

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2. Hydraulic stability of rock armors in breaking wave conditions

This section describes the most commonly used hydraulic stability formulas to design rock armors in breaking wave conditions. The stability number, $N_s = H/(\Delta D_{n50})$, is normally used to characterize hydraulic stability, where $D_{n50} = (M_{50}/\rho_r)^{1/3}$ is the nominal diameter of the rocks in the armor, M_{50} is the median rock mass, ρ_r is the mass density of the rocks, $\Delta = (\rho_r - \rho_w)/\rho_w$ is the relative submerged mass density, ρ_w is the mass density of the sea water, and H is a characteristic wave height.

Eq. (1) is equivalent to Hudson's (1959) formula, which was popularized by USACE (1975, 1984) and based on the pioneering work of Iribarren (1938). Eq. (1) was validated with regular wave tests in non-breaking wave conditions. USACE (1975, 1984) recommended a change in the stability coefficient (K_D) to use Eq. (1) in breaking wave conditions. K_D takes into account the geometry of the armor unit, number of layers, breakwater section (trunk or head) and an implicit safety factor for design; cotα is the armor slope.

$$\frac{H}{\Delta D_{n50}} = (K_D \cot\alpha)^{1/3} \tag{1}$$

USACE (1975, 1984) proposed using $H = H_s = H_{1/3}$ and $H = H_{1/10}$ at the toe of the structure, respectively. According to USACE (1975), using $H = H_s$, the stability coefficient for two-layer randomly-placed rough-angular rock armor was $K_D = 3.5$ for breaking waves and $K_D = 4.0$ for non-breaking waves. According to USACE (1984), using $H = H_{1/10}$, the stability coefficient for two-layer randomly-placed rough-angular rock armor was $K_D = 2.0$ for breaking waves ($H_{1/10} < 1.27H_s$) and $K_D = 4.0$ for non-breaking waves ($H_{1/10} \approx 1.27H_s$). Compared to USACE (1975, 1984) significantly increased the implicit safety factor of rock armors.

Feuillet et al. (1987) suggested a method to use Eq. (1) in breaking wave conditions taking into account the influence of shoaling and wave capping. This method provided the design wave height, H, to be used in Eq. (1) for $m = 1/100$, 1/20 and 1/10 bottom slopes, as a function of the wave steepness, the water depth at the toe, and the offshore $H_{1/10}$. Jensen et al. (1996) recommended $H_{1/20}$ to characterize the irregular waves attacking the structure.

Eq. (1) refers to no-damage (0–5% of the volume of armor units displaced from the armor active zone). To estimate higher damage levels, the armor damage results corresponding to rough quarrystones provided by USACE (1975) can be used. Using the dimensionless armor damage parameter $S = A_e/D_{n50}^2$, suggested by Broderick (1983) and popularized by Van der Meer (1988b), where A_e is the average eroded area in the breakwater's section, Medina et al. (1994) reported a 5-power relationship between H and armor damage for non-breaking wave conditions. Appendix A adapts the methodology given in Medina et al. (1994) to be used in this study.

Van der Meer (1988a) proposed Eqs. (2a) and (2b) to predict rock hydraulic stability under wave attack, based on irregular laboratory tests performed at Delft Hydraulics and the previous work conducted by Thompson and Shuttler (1975). Most of the tests were carried out in non-breaking wave conditions covering a wide range of armor slopes (cotα = 1.5, 2.0, 3.0, 4.0 and 6.0), stability numbers ($1 \le H_s/\Delta D_{n50} \le 4$) and core permeability. Eqs. (2a) and (2b) are applicable to "plunging" and "surging" waves, which refer to the type of wave breaking on the armor slope.

$$\frac{H_s}{\Delta D_{n50}} = 6.2 S^{0.2} P^{0.18} N_z^{-0.1} \xi_m^{-0.5} \quad \text{for} \quad \xi_m < \xi_{mc} (\text{plunging waves}) \tag{2a}$$

$$\frac{H_s}{\Delta D_{n50}} = 1.0 S^{0.2} P^{-0.13} N_z^{-0.1} (\cot\alpha)^{0.5} \xi_m P \quad \text{for} \quad \xi_m > \xi_{mc} (\text{surging waves}) \tag{2b}$$

in which $\xi_{mc} = 6.2P^{0.31}(\tan\alpha)^{0.5})^{1/(P+0.5)}$ is the critical breaker parameter, $0.1 \le P \le 0.6$ is a parameter which considers the permeability of the structure, N_z is the number of waves, and $\xi_m = \tan\alpha/(2\pi H_s/(gT_m^2))^{0.5}$ is the surf similarity parameter based on the mean period, T_m.

Additionally, Van der Meer (1988a) conducted 16 physical tests in breaking wave conditions with a $m = 1/30$ bottom slope and a permeable structure with nominal diameter of rocks D_{n50}(cm) = 3.6, armor slope cotα = 2.0, $1.6 \le H_s/\Delta D_{n50} \le 2.5$ and $3.3 \le h_s/\Delta D_{n50} \le 6.5$. For breaking wave conditions, Van der Meer (1988a) replaced H_s in Eqs. (2a) and (2b) by $H_{2\%}/1.4$; the factor 1.4 corresponds to the ratio $H_{2\%}/H_s$ in the Rayleigh distribution (deep water).

Lamberti et al. (1994) analyzed how the water depth reshaped rubble mound breakwaters in deep and shallow water conditions. They conducted physical model tests with an initial horizontal bottom slope for deep water conditions, then with a $m = 1/20$ bottom slope for intermediate water depths, and a $m = 1/100$ bottom slope to represent shallow water conditions. These authors concluded that $H_{1/50}$ at the toe of the structure was a good representative wave height to estimate armor damage.

Melby and Kobayashi (1998) studied the progression and variability of armor damage on rubble mound breakwaters with a bottom slope $m = 1/20$ and water depths at the toe h_s(cm) = 11.9 and 15.8. Damage to a double-layer rock armor with cotα = 2 and D_{n50}(cm) = 3.64 was measured after three test series of long duration using a profiler. Melby (2001) provided a method to compute damage using the empirical equation (Eq. (3)) proposed by Melby and Kobayashi (1998) to consider cumulative damage for the wave parameters varying in steps in the ranges $1.6 \le H_s/\Delta D_{n50} \le 2.5$ and $2.0 \le h_s/\Delta D_{n50} \le 2.6$.

$$S(t) = S(t_n) + 0.025 \left(\frac{H_s}{\Delta D_{n50}}\right)^5 \frac{(t^{0.25} - t_n^{0.25})}{(T_m)^{0.25}} \quad \text{for} \quad t_n \le t \le t_{n+1} \tag{3}$$

where $S(t)$ and $S(t_n)$ are the mean armor damage at times t and t_n, respectively, with $t > t_n$ ($t = T_m N_z$), T_m is the mean period and N_z is the number of waves. Incident and reflected waves were separated using three wave gauges placed close to the breakwater, and the methodology described by Kobayashi et al. (1990a, 1990b). Eq. (3) was calibrated with H_s obtained at the wave gauge closest to the structure (0.91 m from the toe), which corresponded approximately to distances between $5.7h_s$ and $7.6h_s$ from the toe.

Van Gent et al. (2003) modified Eqs. (2a) and (2b) based on results from tests carried out by Smith et al. (2002) and new 2D physical tests in breaking and non-breaking wave conditions. These new tests were conducted with two bottom slopes ($m = 1/30$ and 1/100), two armor slopes (cotα = 2 and 4), three rock nominal diameters (D_{n50}(cm) = 2.2, 2.6 and 3.5) with an aspect ratio $LT = 2.1$ (see CIRIA/CUR/CETMEF (CIRIA et al., 2007)) and different breakwater geometries. Eqs. (4a) and (4b) are the modified hydraulic stability formulas proposed by Van Gent et al. (2003) calibrated with tests in breaking and non-breaking wave conditions in the ranges $0.5 \le H_s/\Delta D_{n50} \le 4.5$ and $1.5 < h_s/\Delta D_{n50} < 11$. Eqs. (4a) and (4b) are similar to Eqs. (2a) and (2b) but refer the surf similarity parameter to $T_{m-1,0}$ rather than the mean period, T_m.

$$\frac{H_{2\%}}{\Delta D_{n50}} = 8.4 S^{0.2} P^{0.18} N_z^{-0.1} \xi_{s-1}^{-0.5} \quad \text{for} \quad \xi_{s-1} < \xi_{mc} (\text{plunging waves}) \tag{4a}$$

$$\frac{H_{2\%}}{\Delta D_{n50}} = 1.3 S^{0.2} P^{-0.13} N_z^{-0.1} (\cot\alpha)^{0.5} \xi_{s-1}^{P} \quad \text{for} \quad \xi_{s-1} > \xi_{mc} (\text{surging waves}) \tag{4b}$$

in which $\xi_{S-1} = \tan\alpha/(2\pi H_s/(gT_{m-1,0}{}^2))^{0.5}$ is the surf similarity parameter based on the spectral period $T_{m-1,0} = \frac{m_{-1}}{m_0}$, where m_i is the *i-th* spectral moment, $m_i = \int_0^\infty S(f) f^i df$, being $S(f)$ the wave spectrum. Incident wave parameters were obtained in the breaking zone from physical tests conducted without a structure. $H_{2\%}$ at the toe of the structure was selected as the characteristic wave height.