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Toe berm design for very shallow waters on steep sea bottoms

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ABSTRACT

The toe berm is a relevant design element when rubble mound breakwaters are built on steep sea bottoms in breaking conditions. Different design formulas can be found in the literature to predict the damage caused to submerged toe berms placed on gentle bottom slopes. However, these formulas are not valid for very shallow waters in combination with steep sea bottoms where toe berms receive the full force of breaking waves. To guarantee breakwater stability in these conditions, new design formulas are needed for toe berms. To this end, physical model tests were carried out and data were analyzed to characterize rock toe berm stability in very shallow water and with a bottom slope $m = 1/10$. Based on test results, a new formula was developed with three parameters to estimate the nominal diameter (D_{n50}) of the toe berm rocks: water depth at the toe (h_s), deep water significant wave height (H_{s0}), and deep water wave length (L_{0p}).

1. Introduction

Rubble mound breakwaters are usually protected by a toe berm when concrete armor units are used for the armor layer. This toe berm is placed on the seafloor or a bed layer, providing support to the concrete armor units which are placed later on the structure slope (USACE, 2006). Fig. 1 shows a typical cross-section for a conventional mound breakwater with a toe berm placed on a steep seafloor, where h_s is the sea bottom water depth at the toe, h_t is the water depth above the toe berm, B_t is the toe berm width, and t_t is the toe berm thickness.

Many rubble mound breakwaters are constructed in breaking conditions and in shallow waters on steep sea bottoms. In these conditions, the highest waves start breaking on the sea bottom and impact the toe berm directly. This is particularly common for rocky sea bottoms with $m = 1/10$ or higher slopes; in this case, the toe berm must be designed to guarantee armor stability. In very shallow waters combined with steep seafloors, the stone size required for the toe berm may significantly exceed the armor unit size.

Several empirical formulas have been developed to predict damage to rock toe berms in depth-limited conditions. Most were obtained from laboratory tests with gentle bottom slopes and are only valid for submerged toe berms ($h_t >> 0$); however, when constructed in very shallow waters on rocky coasts and steep seafloors, seawalls may require emerged toe berms ($h_t < 0$) built with large rocks.

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This research focuses on the design of toe berms placed in very shallow waters ($-0.15 < h_s/H_{s0} < 1.5$) in combination with steep seafloors ($m = 1/10$) since these conditions have not yet received sufficient attention in the literature. New physical model tests were carried out in the wave flume at the Universitat Politècnica de València (Spain), and data were analyzed to determine the influence of shallow waters and steep seafloors on toe berm stability. In this paper, existing formulas to design toe berms are first compared. The experimental setup is then described, test results are analyzed, and a new design formula with confidence intervals is provided. Finally, conclusions are drawn.

2. Design formulas for toe berms

In this section, the most relevant formulas to design quarrystone toe berms are examined. The stability number, $N_s = H_{st}/(\Delta D_{n50})$, is used to characterize hydraulic stability, where D_{n50} is the nominal diameter of the rocks in the toe berm, $\Delta = (\rho_r - \rho_w)/\rho_w$ is the relative submerged mass density, ρ_r is the mass density of the rocks, ρ_w is the mass density of the sea water, and H_{st} is the significant wave height at the toe of the structure.

Markle (1989) performed physical tests in breaking conditions with a bottom slope $m = 1/10$. Regular waves were generated with increasing wave heights ($9.1 < H_{mt}(\text{cm}) < 22.9$) and wave periods ($1.32 < T_m(\text{s}) < 2.82$) for a given water depth at the toe ($h_s(\text{cm}) = 12.2, 15.2, 18.3, 21.3, 24.4, 27.4$), where H_{mt} is the average wave height at the toe of the structure and T_m is the mean wave period. Four rock nominal diameters were used ($D_{n50}(\text{cm}) = 2.58, 2.95, 3.30, 4.06$) for toe berms with $t_t = 2 \cdot D_{n50}$ and $B_t = 3 \cdot D_{n50}$. Eq. (1) is the lower bound formula obtained from Markle's data

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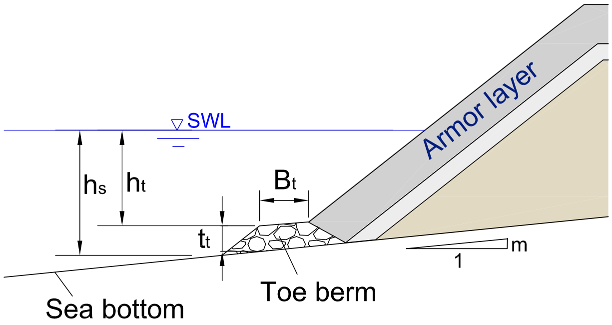

Fig. 1. Cross-section of a conventional mound breakwater with a toe berm.

(see Muttray, 2013); the water depth ratio (h_t/h_s) was identified as the determining parameter for toe berm stability. Eq. (1) refers to moderate damage.

$$N_s{}^* = \frac{H_{mt}}{\Delta D_{n50}} = 1.6 + 5.5 \cdot \left(\frac{h_t}{h_s}\right)^3 \tag{1}$$

where $N_s{}^* = H_{mt}/(\Delta D_{n50})$ is the stability number for regular waves.

Gerding (1993) measured toe berm damage in physical tests using runs of 1,000 random waves and a bottom slope $m = 1/20$. Tests were characterized by a constant wave steepness at the wave generating zone ($s_{gp} = 2\pi H_{sg}/gT_p^2 = 0.02$ and 0.04), an increasing significant wave height at the wave generator (H_{sg}(cm) = 15, 20, 25), and a fixed water depth at the toe (h_s(cm) = 30, 40, and 50). Four stone sizes were tested (D_{n50}(cm) = 1.7, 2.5, 3.5, or 4.0), varying the toe berm height (t_t(cm) = 8, 15, and 22), and the toe berm width (B_t(cm) = 12 and 20). Gerding (1993) also proposed using the damage number N_{od} to quantify the damage observed on the toe berm. N_{od} is defined as the number of displaced rocks in a strip as wide as D_{n50} of the toe berm. N_{od} is independent of the shape and volume of the toe berm; therefore, damage geometry may differ significantly from quantitative N_{od}.

$$N_{od} = \frac{N}{B/D_{n50}} \tag{2}$$

where N is the number of displaced rocks and B is the total width of the wave flume. After each test, the damage number N_{od} was calculated and the model was rebuilt. The formula given by Gerding (1993) can be re-written to estimate toe berm damage as a function of the stability number.

$$N_{od} = \frac{1}{\left(0.24 \cdot \left(\frac{h_t}{D_{n50}}\right) + 1.6\right)^{1/0.15}} \cdot (N_s)^{1/0.15} \tag{3}$$

Docters van Leeuwen (1996) conducted tests on a bottom slope $m = 1/50$ to analyze the influence of the relative submerged mass density ($\Delta = (\rho_r - \rho_w)/\rho_w$) on Gerding's formula, concluding that Δ was well reproduced since different stone mass densities gave similar results for $H_{st}/(\Delta D_{n50})$ as a function of h_t/D_{n50}.

Van der Meer (1998) re-analyzed the data given by Gerding (1993) for rock toe berms, using the water depth ratio (h_t/h_s) as the explanatory variable; the new Van der Meer formula can be re-written as follows:

$$N_{od} = \frac{1}{\left(6.2 \cdot \left(\frac{h_t}{h_s}\right)^{2.7} + 2.0\right)^{1/0.15}} \cdot (N_s)^{1/0.15} \tag{4}$$

CIRIA/CUR/CETMEF (2007) made reference to the formulas given by Gerding (1993) and Van der Meer (1998) to calculate the rock size for toe berms of rubble mound breakwaters. Gerding (1993) recommended using $N_{od} = 2.0$ for safe designs while Van der Meer (1998) recommended $N_{od} = 0.5$ for conservative designs. For a standard toe berm size of 3–5 rocks wide and a thickness of 2–3 rocks, CIRIA/CUR/CETMEF (2007) criteria indicated $N_{od} = 0.5$ for start of damage, $N_{od} = 2.0$ for moderate damage, and $N_{od} = 4.0$ for failure.

Ebbens (2009) conducted physical tests to analyze the influence of three bottom slopes ($m = 1/50$, $1/20$, and $1/10$). Random waves were generated with seven water levels varying in the range of $7.3 < h_s(\text{cm}) < 25.3$. The four lowest water levels (h_s(cm) = 7.3, 9.3, 11.3, and 13.3) were tested with two values for wave steepness at the wave generating zone ($s_{gp} = 2\pi H_{sg}/gT_p{}^2 = 0.04$ and 0.02). Tests with the three highest water levels (h_s(cm) = 15.3, 20.3, or 25.3) were only performed with $s_{gp} = 2\pi H_{sg}/gT_p^2 = 0.03$ for calibration. For each water level, wave runs were generated with four significant wave heights at the wave generator (H_{sg}(cm) = 6, 8, 10, or 12). Three rock sizes were tested (D_{n50}(cm) = 1.88, 2.15, and 2.68) with toe berm thickness t_t(cm) = 6 and toe berm width B_t(cm) = 10 (above a 2 cm-thick bed layer). Three rock porosities were used for each D_{n50} ($n = 0.36, 0.33, 0.32$). For the bottom slope $m = 1/10$, only D_{n50}(cm) = 2.15 and 2.68 were tested. To characterize toe berm damage, the damage parameter given by Eq. (5) was used.

$$N_{\%} = N \cdot \frac{D_{n50}{}^3}{(1-n) \cdot V_{\text{total}}} \tag{5}$$

where n is the void porosity and V_{total} is the apparent volume of the toe berm.

A difference in damage was observed when varying the wave steepness from $s_{gp} = 0.04$ to $s_{gp} = 0.02$. Steeper waves ($s_{gp} = 0.04$) led mainly to a downward movement of rocks, while longer waves ($s_{gp} = 0.02$) pushed rocks in an upward direction. Thus, for tests with $s_{gp} = 0.04$, only downward rock movements were considered to characterize toe berm damage. For tests with $s_{gp} = 0.02$, the number of displaced rocks was counted considering the number of stones moving downwards (away from the toe berm) and upwards.

Using $N_{\%}$, Ebbens (2009) proposed the following design equation for toe berm stability:

$$N_{\%} = 0.038 \cdot \left(\xi_{0p}{}^*\right)^{3/2} \cdot (N_s)^3 \tag{6}$$

where $\xi_{0p}^* = m/(H_{st}/L_{0p})^{1/2}$ is the surf similarity parameter in which $1/m$ is the bottom slope, and $L_{0p} = gT_p^2/2\pi$ is the deep water wave length. Although higher toe berm damage was measured during the tests, Eq. (6) only provides reliable values if $N_{\%} < 0.3$.

The toe berm was not rebuilt after each test but rather before each change in the water level. The cumulative toe berm damage did not

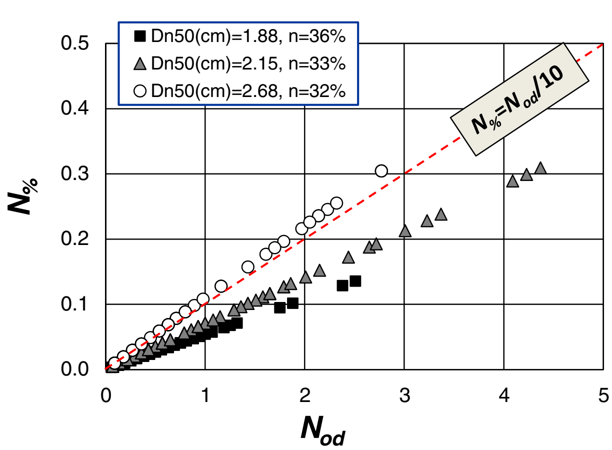

Fig. 2. Values of N_{od} corresponding to $N_{\%}$ measured by Ebbens (2009).