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Rock toe stability of rubble mound breakwaters

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ABSTRACT

The toe structure of a breakwater provides support to the armour layer and protects the structure from damage due to scour at the toe. Often a toe structure consists of rock material. Several design formulae exist to predict the amount of damage to the toe structure under wave loading. These design formulae for the required rock size include effects of the wave height and the water depth above the toe structure. Here, rock toe stability has been studied by means of physical model tests to provide information on the required rock size in the toe structure. The tests and analysis are focussed not only on the influence of the wave height and the water depth above the toe structure, but also on the influence of the width of the toe structure, the thickness of the toe and the wave steepness. The wave steepness, width of the toe and the thickness of the toe appear to affect the damage to the toe; these parameters need to be taken into account in order to derive accurate predictions of the damage to the toe structure. Based on the test results a prediction formula has been derived including these effects. The formula can be used to determine the required rock size in the toe of rubble mound breakwaters within the ranges of the performed tests.

1. Introduction

For rubble mound breakwaters and dikes the armour layer is an important part of the design. Also the toe of rubble mound structures and dikes are important since toe structures need to provide support to the armour layer and prevent scour to occur directly beneath the armour layer. Also because a number of rubble mound breakwaters have failed due to insufficient strength of the toe structure, this part of the structure should receive adequate attention. However, the number of available design formulae is low, the range of validity of these formulae is limited, and the accuracy of these formulae is relatively low compared to the accuracy of formulae for armour layers. A number of parameters that are expected to affect the damage to toe structures are not present in existing formulae. Therefore, new physical model tests have been performed in a wave flume. Fig. 1 shows a few pictures with wave action above a toe structure of rock.

The tests and analysis are focussed on providing a conceptual design formula for rock toe structures such that the required rock size can be determined. Tests have been performed for a rather wide range of toe configurations and wave conditions, not only on the influence of the wave height and the water depth above the toe structure, but also on the influence of the width of the toe structure, the thickness of the toe and the wave steepness. As Fig. 1 shows, the conditions include wave breaking above the toe. The test programme is limited to a configuration with a 1:2 armour slope and a 1:30 foreshore.

Several formulae exist for the prediction of damage to toe structures. Here the expressions that have been published by Gerding (1993), Burcharth and Liu (1995), Van der Meer (1998) and Muttray (2013) are referred to. The data and analysis by Gerding (1993) are also published in Van der Meer et al. (1995). Although these expressions use the parameter N_{OD} (i.e. the number of displaced stones per stone diameter width of structure) to characterise damage, the definition is not always the same. In the tests presented here all stones that have been displaced over more than one rock diameter have been considered as damage. Gerding (1993), however, only counted the stones that disappeared from the toe, thus not including the stones that are displaced over more than one diameter but remain in the toe. The expression by Van der Meer (1998) is based on the data by Gerding (1993) thus with the same definition of the N_{OD}. Muttray (2013) also used the data by Gerding (1993), thus applied the same definition of the N_{OD}. Burcharth and Liu (1995) adapted the formula by Gerding (1993) for applications with concrete cubes in the toe. Although not specifically mentioned, it is assumed that Burcharth and Liu (1995) use the same definition as applied here (which is a common approach, also for armour slopes), namely all units that are displaced over more than one diameter.

Gerding (1993) performed tests on a structure with a 1:1.5 slope. A large portion of these tests had a high toe structure compared to the water depth, for instance toe structures with a height that is once or twice the water depth above the toe structure (thus a thickness of 50% to 70% of the water depth in front of the structure). Such very thick toe structures compared to the local water depth can actually be seen as a berm that is part of the armour layer. For such structure reference is made to Van Gent (2013) where prediction formulae are given for

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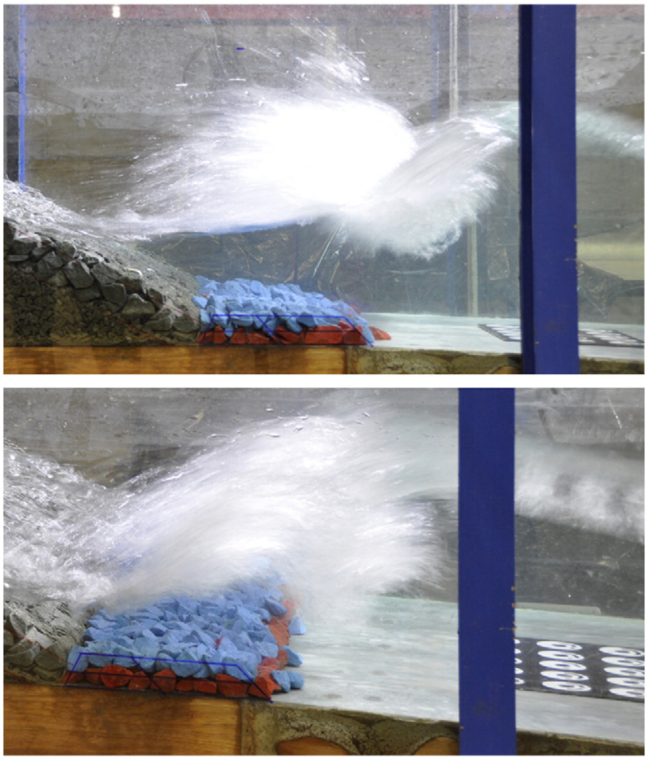

Fig. 1. Pictures of rock toe structures under wave loading in a wave flume.

rubble mound structures with a berm. The present study is focussed on toe structures that have a thickness of 10% to 30% of the water depth in front of the structure. Out of the total of 57 test series by Gerding (1993), 26 test series were for toe structures with a thickness of more than 30% of the water depth. Nevertheless, Gerding (1993) performed also tests within a range of 16% to 30% and therefore his study is considered relevant for comparison, despite that the resulting formula may be affected by a large number of tests for different types of toe structures.

In the tests by Gerding (1993) also the width of the toe structures and the wave steepness were varied. It was concluded that the width of the toe and the wave steepness based on the wave height at the toe and the peak wave period at deep water have no significant influence although further research was recommended. Gerding (1993) developed two formulae; in one of the proposed formulae the H_s is used and in another one the $H_{2\%}$. The one using H_s is used here and this formula can be written as follows:

$$N_{OD} = \left(\frac{H_s}{\Delta D_{n50}} / \left(0.24\frac{h_t}{D_{n50}} + 1.6\right)\right)^{6.67} \tag{1}$$

where h_t is the water depth above the toe, H_s is the significant wave height obtained from time-domain analysis ($H_s = H_{1/3}$), D_{n50} is the nominal rock diameter, and Δ is the relative density of the rock material.

Based on the data by Gerding (1993), Van der Meer (1998) published another formula by Gerding (1993, Fig. 43):

$$N_{OD} = \left(\frac{H_s}{\Delta D_{n50}} / \left(6.2\left(\frac{h_t}{h}\right)^{2.7} + 2\right)\right)^{6.67} \tag{2}$$

where h is the water depth just in front of the toe.

Burcharth and Liu (1995) proposed a formula for concrete cubes in the toe. Their formula can be rewritten as follows:

$$N_{OD} = \left(\left(0.63\frac{H_s}{\Delta D_{n50}}\right)^{-1} + 0.4\left(\frac{h_t}{H_s}\right)\right)^{-6.67}. \tag{3}$$

Based on a re-analysis of existing data Muttray (2013) proposed another formula for rock toe structures. This formula can be rewritten as follows:

$$N_{OD} = \left(\frac{H_s}{\Delta D_{n50}}\left(0.58 - 0.17\frac{h_t}{H_s}\right)\right)^3. \tag{4}$$

The Rock Manual (2007) refers to Eqs. (1) and 2 and states that these equations can be considered valid for depth-limited situations only. Using the formulae by Gerding (1993) and Van der Meer (1998) for conditions that are not depth-limited may lead to rather large deviations between predictions and the actual situations. Therefore, it is relevant to focus more in detail on the stability of toe structures that are not necessarily in depth-limited conditions. Since Burcharth and Liu (1995) refer to toe structures with cube differences are to be expected with results from rock toe structures. The comparisons of test results are made with formulae for rock toe structures only. In the following the new physical model tests are presented, the analysis of the data, and a new prediction method for toe structures.

2. Physical model tests

Physical model tests were performed in a wave flume (width 1 m, height 1.2 m, length 110 m of which 55 m was used here) at Deltares, Delft. The wave board is equipped with active reflection compensation. This means that the motion of the wave board compensates for the waves reflected by the structure preventing them to re-reflect at the wave board and propagate towards the model. The wave board is equipped with second-order wave steering. This means that the second order effects of the first higher and the first lower harmonics of the wave field are taken into account in the wave board motion, which ensures that the generated waves resemble waves that occur in nature.

2.1. Test set-up

Wave conditions were measured at deep water and in front of the toe structure. The analysis was based on the time series of the incident waves at the toe. These signals, without reflected waves, were obtained using the method by Mansard and Funke (1980). The spectral significant wave height H_s (in this paper: $H_s = H_{m0} = 4(m_0)^{0.5}$) and the wave period $T_{m-1,0}$ ($T_{m-1,0} = m_{-1}/m_0$) were obtained from the measured wave energy spectra. In Van Gent (2001) the wave period $T_{m-1,0}$ was found to appropriately describe the influence of wave energy spectra on wave run-up, while in later studies this wave period was found to be the most appropriate wave period for wave overtopping, wave reflection, dune erosion, and the stability of rock slopes (e.g. Van Gent et al., 2003). In all tests a Jonswap wave spectrum has been applied. Each configuration was tested with two values for the wave steepness, referred to as the low wave steepness and the high wave steepness: $s_p = 0.015$ and $s_p = 0.04$ with $s_p = 2\pi\, H_s/gT_p^2$ and $H_s = H_{m0}$. This corresponds to $s_{m-1,0} = 0.018$ and $s_{m-1,0} = 0.048$ respectively, with $s_{m-1,0} = 2\pi\, H_s/gT^2_{m-1,0}$ and $H_s = H_{m0}$.

The basic configuration consists of a non-overtopped 1:2 rock armour layer ($D_{n50} = 27$ mm) on top of a permeable core ($D_{n50} =$ 8 mm), see Fig. 2. In all tests the foreshore slope was 1:30. The foreshore was fixed (no mobile bed) such that no toe scour could occur. On this foreshore most conditions were such that no severe wave breaking occurred before the waves reached the structure. The configuration of the toe structure was varied, see also Fig. 3. The rock diameter, width and thickness of the toe structure were varied. Two rock diameters were applied: $D_{n50} = 14.6$ mm and $D_{n50} = 23.3$ mm. Structures with a toe width (at the crest of the toe structure) of three and nine times the rock diameter were applied, leading to width of $B_t = 0.044$ m, 0.070 m, 0.131 m and 0.210 m. Toe structures with a thickness of two and four times the rock diameter were applied. For the largest rock size only the thickness of two times the rock diameter was applied.