

# WAVE OVERTOPPING OF RUBBLE MOUND BREAKWATERS

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## Abstract

*A general expression for the overtopping discharge of a rubble mound breakwater has been derived utilising the general experience on this subject found in the bibliography and from the results of comprehensive series of model tests carried out at Danish Hydraulic Institute (DHI). The expression includes the important breakwater parameters (geometry of the breakwater profile) and the environmental parameters, and has been derived for applications with non-breaking waves in front of the structure. In research studies, model test series were carried out on pure rubble mound breakwater profiles with quarry rock as armour layer. Through results from projects carried out at DHI, the expression was extended to include different armour types and to describe the influence of a superstructure.*

*The aim has been to set-up a reliable expression, which is simple, general and easy to use. Previously, predictions of overtopping discharges have been based either on expressions including empirical constants for different shapes of the breakwater profile, on very complicated expressions or on diagrams giving the overtopping discharges as function of the layout of the breakwater profile and the environmental conditions.*

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## Introduction

Research on overtopping of rubble mound breakwaters has been undertaken at DHI during the last couple of decades. Since 1993, a large number of tests have been carried out as part of a research study on overtopping with the aim to determine the overtopping discharge as a function of the geometry of the breakwater profile and of the environmental parameters. On the basis of the test results, an expression on the functional relationship between the overtopping discharge and the breakwater parameters and the environmental parameters has been derived.

All research tests were carried out with quarry rock as armour, but the derived expression has been extended also to include artificial blocks as armour layer by using the test results from projects carried out in the past.

The overtopping discharges measured in the model have been transformed to prototype values in order to get the results as easily accessible and understandable as possible. It has been the intention to focus on overtopping discharges in the range where there will be a risk of damage to structures, vehicles, installations and persons behind the breakwater. According to for instance reference 1, limited damage may occur for an average overtopping discharge of  $10^{-6}$  m<sup>3</sup>/s per metre run of the breakwater, but serious damage may take place if the average discharge exceeds  $10^{-5}$  m<sup>3</sup>/s per meter run. If the average overtopping discharge exceeds  $10^{-3}$  to  $10^{-2}$  m<sup>3</sup>/ms, the discharge will be so large that the damage to possible installations behind the breakwater will be severe. In case of such large overtopping discharges, the exact discharge is not interesting, but only the stability of the crest and the rear side. Accordingly, the present paper has focused on average overtopping discharges from  $10^{-6}$  m<sup>3</sup>/ms to  $10^{-2}$  m<sup>3</sup>/ms.

## Model Set-up and Test Programme

Physical model tests have been carried out partly in a wave flume partly in a wave basin at DHI with the purpose to measure the average overtopping per metre run of the breakwater. All tests were carried out with long-crested waves generated on the basis of a Pierson-Moscovitz wave spectrum. The modelled profile and the definition of the breakwater parameters are shown in Figure 1.

The investigated profiles were traditional rubble mound profiles with core, filter and armour layer and without superstructure. The armour layer was quarry rock. The size of the rocks was so large that significant damage to the structure for the investigated wave conditions could be avoided during testing. The tests were all carried out with horizontal seabed in front of the profile.

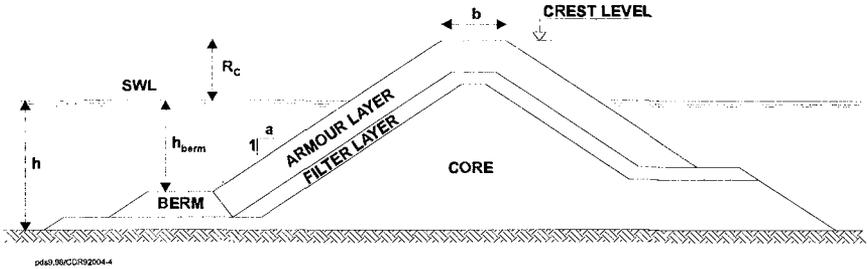


Figure 1 Definition of breakwater parameters

The overtopping discharge was defined as the average amount of water passing the rear side edge of the breakwater crest.

The tests were carried out with different wave conditions and different breakwater parameters. The following parameters were varied.

#### Wave Conditions:

- Significant Wave Height  $H_s$  (m)
- Peak Wave Period  $T_p$  (s)
- Wave Steepness  $s_p (= H_s \cdot 2 \cdot \pi / g \cdot T_p^2)$
- Wave Direction  $\beta$  ( $^\circ$ )

#### Breakwater Parameters:

- Water Depth  $h$  (m)
- Crest Freeboard  $R_c$  (m)
- Crest Width  $b$  (m)
- Seaside Slope Angle  $a$
- Type of Armour

The following ranges of parameters were investigated in the model studies. The values are given in model measures. The values in brackets being the model values interpreted at a linear scale of 1:40.

#### Investigated Wave Conditions:

- $H_s = 0.05$  to  $0.11$  m (2.0 to 4.4 m)
- $T_p = 1.0$  to  $2.0$  s (6.3 to 12.6 s)
- $s_p = 0.018, 0.025, 0.030$  and  $0.045$
- $\beta = 0^\circ, 10^\circ, 20^\circ, 30^\circ, 40^\circ$  and  $50^\circ$

All tests were carried out with irregular, long-crested waves. Most of the tests were carried out with wave steepness of  $s_p=0.018$  and  $0.030$  and with perpendicular wave attack ( $\beta=0^\circ$ ).

#### Investigated Breakwater Parameters:

- $h = 0.350, 0.375$  and  $0.400$  m (h = 14.0, 15.0 and 16.0 m)
- $R_c = 0.100, 0.075$  and  $0.050$  m ( $R_c = 4.0, 3.0$  and  $2.0$  m)
- $b = 0.16, 0.21$  and  $0.26$  m (b = 6.4, 8.4 and 10.4 m)
- $a = 1.5, 2.0$  and  $2.5$
- Quarry rocks (100 - 180 g) (6400 – 11500 kg)

### Formulation of the Overtopping Expression

The expression for the average overtopping of rubble mound breakwaters given in this paper is derived from the results of the model tests described above, but the evaluation of the expression has been inspired by previously developed overtopping formulae given in the literature. Owen, 1980 (see for instance references 1, 3, 5 and 7), has formulated one of the best known expressions. Various authors have elaborated on the formula to include different types of armour, different roughness of the layer, etc.

Owens formula expresses the overtopping  $Q$  ( $m^3/ms$ ) as:

$$Q = Q^* \cdot (g \cdot T_m \cdot H_s) \quad [1]$$

where

$Q^*$  is a dimensionless expression for the overtopping discharge

$$Q^* = A \cdot \exp(-B \cdot R^* / r)$$

A and B are constants depending on the geometry of the profile such as the slope of the seaside armour and elevation and width of the berm

$R^*$  is a dimensionless expression for the freeboard, ie the vertical distance between the still water level and the crest level

$$R^* = \frac{R_c}{T_m \sqrt{H_s \cdot g}}$$

$R_c$  is the freeboard of the breakwater (m)

$T_m$  is the mean wave period (s) of the incoming wave train

$H_s$  is the significant wave height (m) of the incoming wave train

$g$  is the acceleration of gravity ( $m/s^2$ )

$r$  is a 'run-up reduction factor' or a description of the roughness of the armour layer

The formula includes the effect of different slopes of the seaside armour through the constants A and B, which however, are difficult to understand physically. The run-up reduction factor is a measure of the run-up level relative to a smooth impermeable slope.

Goda presents design diagrams for the overtopping discharges at block mounded seawalls covering different wave steepness and seabed slopes in front of the seawall (reference 2).

Jensen et al tried to include the geometry of the profile through a representative 'width' of the breakwater (reference 10). This 'width' was defined as the distance from the point where the profile intersects with the still water level to the position from where the overtopping was measured. In this way, both the slope of the breakwater and the crest width were taken into consideration.

Van der Meer et al give formulae which can be used to determine overtopping discharges at dikes, sloping revetments and seawalls (reference 4).

Juhl and Sloth present an expression for estimating the overtopping discharges of breakwater profiles armoured with quarry rocks (reference 9). The expression was based on some of the model test results, included in this present paper, and considers both the geometry of the profile, the wave height and wave period.

$$Q = Q^* \sqrt{g \cdot H_s^3}$$

$$Q^* = \exp \left[ (-17.505 - 4.20 \ln(s_p)) + (1.869 + 1.198 \ln(s_p)) \left( \frac{a^{0.3} (2R_c + 0.35b)}{H_s} \right) \right] \quad [3]$$

- a is the slope of the armour layer  
 b is the width of the crest (m)

The expression includes the actual shape of the breakwater, ie the slope of the armour layer, the width of the crest and the freeboard, but does not include, for instance, the type of armour.

For  $\left[ a^{0.3} (2R_c + 0.35b) / H_s > \sim 4 \right]$ , ie for small values of the wave height compared to the freeboard, expression [3] gives larger overtopping for larger wave steepness. For smaller values of this factor, the overtopping discharge will increase with decreasing wave steepness. Whether this assumption is correct is difficult to tell from the test results, as no clear tendency was found.

In the evaluation of the new expression, both Owens formulae and Juhl and Sloth's formulation were considered. With the basis in the model tests, Juhl and Sloth found a representative dimension, which could describe the influence of the geometry of the profile [3],

$$C = a^{0.3}(2R_c + 0.35b)$$

This dimension, which can be taken as a representative 'width' and/or as a representative crest 'freeboard' of the breakwater, was included in the new expression. It was investigated if an expression in the following form would fit the model test data.

$$Q = Q^* \sqrt{g \cdot H_s^3}$$

$$Q^* = k_1 \cdot \ln(s_p)^{c1} \cdot \exp\left(\frac{k_2 \cdot C \cdot (s_p)^{c2}}{H_s}\right) \quad [4]$$

where  $k_1$ ,  $k_2$ ,  $c1$  and  $c2$  are constants.

The best fit to the data was obtained with  $k_1=-0.3$ ,  $k_2=-2.9$ ,  $c1=1$  and  $c2=0$ . The derived expression has then the following form

$$Q = Q^* \sqrt{g \cdot H_s^3}$$

$$Q^* = k_1 \cdot \ln(s_p) \cdot \exp\left(\frac{k_2 \cdot C}{H_s}\right) \quad [5]$$

with  $k_1=-0.3$  and  $k_2=-2.9$ . Applying a roughness factor or a wave run-up reduction factor for the armour layer of  $r=0.55$ , which is a recognised value for quarry rock slopes in two layers (see for instance reference 1), the expression will be as follows.

$$Q^* = k_1 \cdot \ln(s_p) \cdot \exp\left(\frac{k_2 \cdot C}{rH_s}\right) \quad [5a]$$

$$k_1 = -0.3, \quad k_2 = -1.6, \quad C = a^{0.3}(2R_c + 0.35b)$$

## Verification of the Expression, Quarry Rock Slope

Figure 2 shows the results of all tests, which were carried out in the wave flume compared to the results obtained by using expression [5].

As previously mentioned, the overtopping discharges are given in 'nature' values assuming a linear scale of 1:40. These tests were carried out with wave steepness of 0.018 and 0.030,  $R_c=2, 3$  and 4 m, and  $a = 1.5, 2.0$  and 2.5.

Figure 3 shows the influence of the crest freeboard for an armour layer slope of 1:2, Figure 4 shows the influence of the slope (all tests) and Figure 5 shows the influence of the wave steepness (all tests).

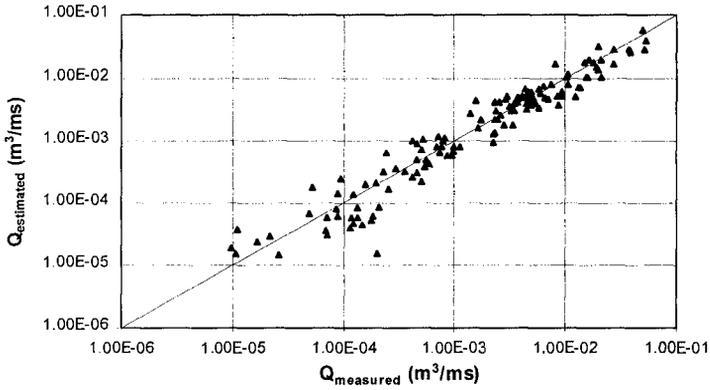


Figure 2 Comparison of measured and computed overtopping discharges, all data from flume tests

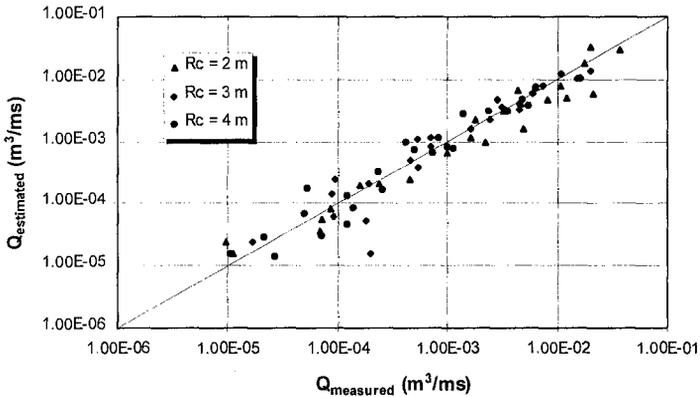


Figure 3 Comparison of measured and computed overtopping discharges, influence of crest freeboard,  $a=2$

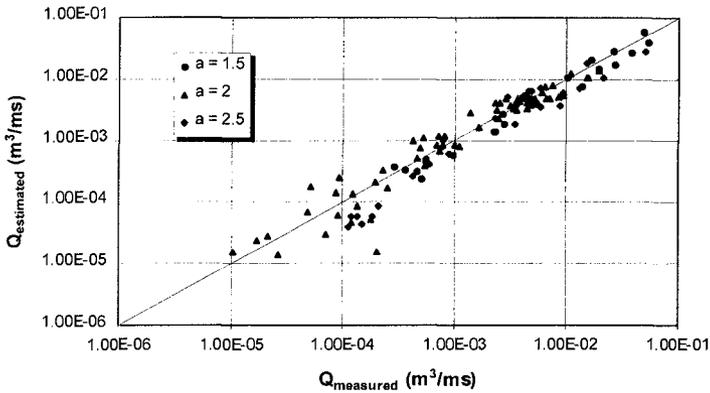


Figure 4 Comparison of measured and computed overtopping discharges, influence of seaside slope

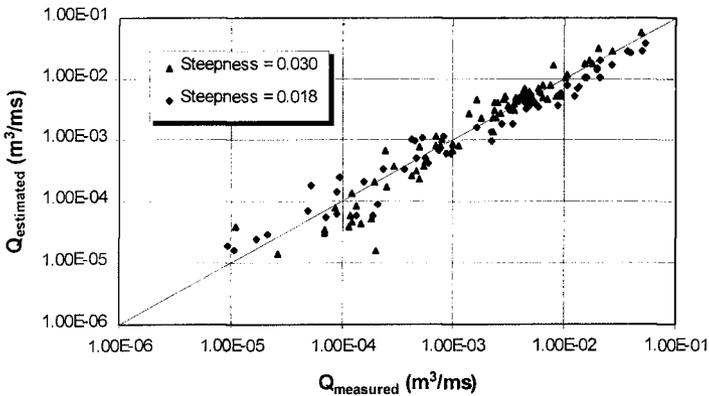


Figure 5 Comparison of measured and computed overtopping discharges, influence of wave steepness

It is seen that the measured and the computed overtopping discharges in most cases are in good agreement with each other.

### Overtopping under Oblique Wave Attack, Quarry Rock Slope

The tests carried out with different wave directions relative to the breakwater alignment have been analysed, and the influence of wave direction on the overtopping discharge has been fitted into the formula [5]. Generally, the tests showed that the overtopping discharges for a wave obliqueness of 10° are almost the same as for head-on waves, and that the discharges are reduced significantly for wave obliqueness larger than 20°. The influence of the wave obliqueness has been included in the overtopping expression as shown in [6].

$$Q^* = k_1 \cdot \ln(s_p) \cdot \exp\left(\frac{k_2 \cdot C}{rH_s \sqrt{\cos\theta}}\right) \tag{6}$$

where

$$k_1 = -0.3, \quad k_2 = -1.6, \quad C = a^{0.3} (2R_c + 0.35 \cdot b)$$

$\theta$  is the oblique angle (relative to head-on wave direction)

Figures 6 and 7 show the results of all tests carried out in the wave basin. Figure 6 shows the results of the tests carried out with a crest freeboard of 2 m, and Figure 7 shows the results of the tests carried out with crest freeboards of 3 and 4 m.

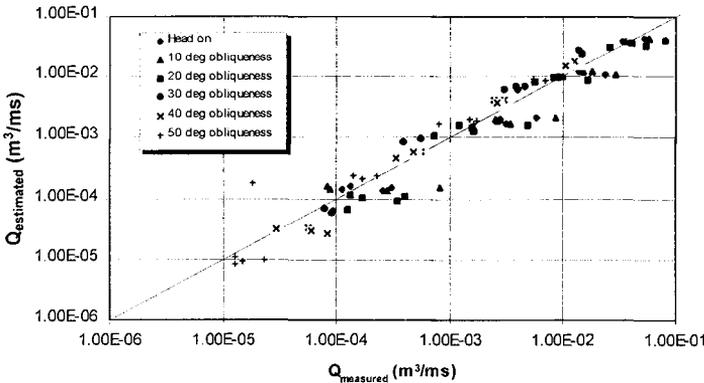


Figure 6 Comparison of measured and computed overtopping discharges, influence of wave direction,  $R_c = 2$  m,  $s_p = 0.018, 0.025, 0.030$  and  $0.045$

From Figure 6, it is seen that the expression gives a very good description of the overtopping discharges for the lowest crest freeboard, whereas the results presented in Figure 7 for the two other investigated freeboards give too low, respectively too high, estimated overtopping discharges.

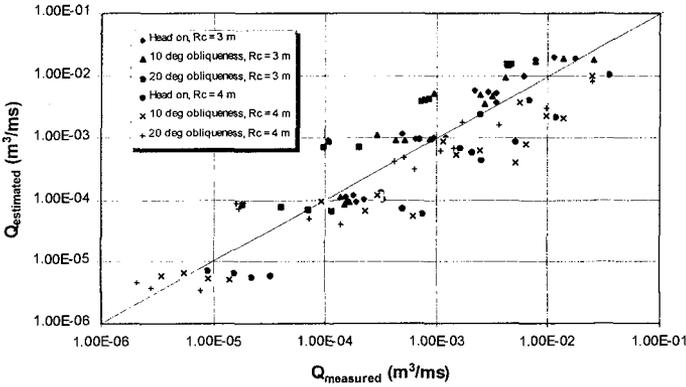


Figure 7 Comparison of measured and computed overtopping discharges, influence of wave direction,  $R_c = 3$  and  $4 m$ ,  $s_p = 0.018, 0.025, 0.030$  and  $0.045$

### Overtopping for Different Types of Armour

Results from overtopping tests, which have been carried out as part of projects at DHI, have been used in order to test the expression on different armour layer stones/blocks.

Results from four different projects with different armour types are shown in Figure 8. It should also be noted that the models in all four regarded projects were constructed with a sloping seabed in front of the breakwater.

The wave and breakwater parameters for the four projects are given in Table 1.

The results from the tests show that the run-up reduction coefficients ( $r$ ) to be applied should be  $\sim 0.65$  for grooved cubes,  $\sim 0.60$  for quarry rock/grooved cubes,  $\sim 0.65$  for rounded stones, and  $0.55$  for Accropodes in one layer.

Considering the deviations in test set-up for the different projects, it is found that there is rather good agreement between the measured and the estimated overtopping discharges. This indicates that the expression will give reasonably good results for other armour types when applying reasonable values of the run-up reduction factors.

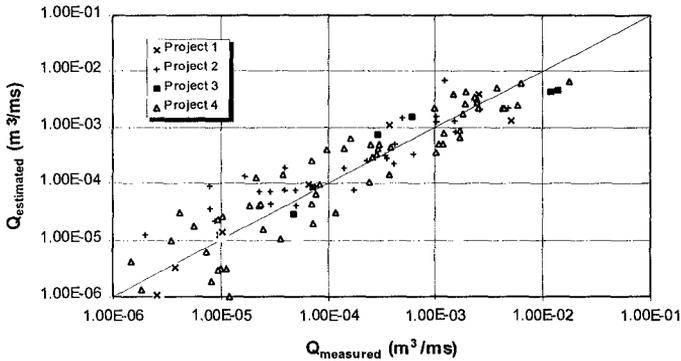


Figure 8 Comparison of measured and computed overtopping discharges, influence of armour layer type

Table 1 Wave and breakwater parameters, projects 1-4

Project	Armour	R <sub>c</sub> (m)	a	b (m)	H <sub>s</sub> (m)	s <sub>op</sub>
1	Rounded stones	3-5	2	4	1.3-2.8	0.009-0.025
2	Quarry rocks/ Grooved cubes *)	13-15	2	7	5.5-8.6	0.014-0.036
3	Grooved cubes	6.5-7.5	2	12	4.1-6.2	0.021-0.026
4	Accropodes	2-5	1.33	6.6	1.2-4.0	0.010-0.045

\*) Quarry rocks at the crest and down to 3-3.5 m below the crest at the seaside

### Influence of Superstructure at the Crest

On the basis of the results from projects carried out at DHI, expression [6] has been extended to include breakwater profiles with a superstructure at the crest. Two different layouts were included in the measured data, a ‘low’ respectively ‘high’ crested structure, see Figure 9.

It was found that the overtopping discharges for both ‘low’ and ‘high’ superstructures could be described by the expression

$$Q^* = k_1 \cdot \ln(s_p) \cdot \exp\left(\frac{k_2 \cdot C}{rH_s \sqrt{\cos\theta}}\right) \tag{7}$$

k<sub>1</sub> = -0.01,    k<sub>2</sub> = -1.0,    C = a<sup>0.3</sup>(2R<sub>c</sub>+0.35 b)  
 θ is the oblique angle (relative to head on wave direction)

The expression is apart from the constants  $k_1$  and  $k_2$  identical to the expression for pure rubble mound breakwaters.

Results from the tests with different types of armour layers show that the run-up reduction coefficient ( $r$ ) to be applied should be  $\sim 0.65$  for grooved cubes,  $\sim 0.55$  for quarry rock,  $\sim 0.65$  for rounded stones, and  $0.45$  for Dolos in two layers.

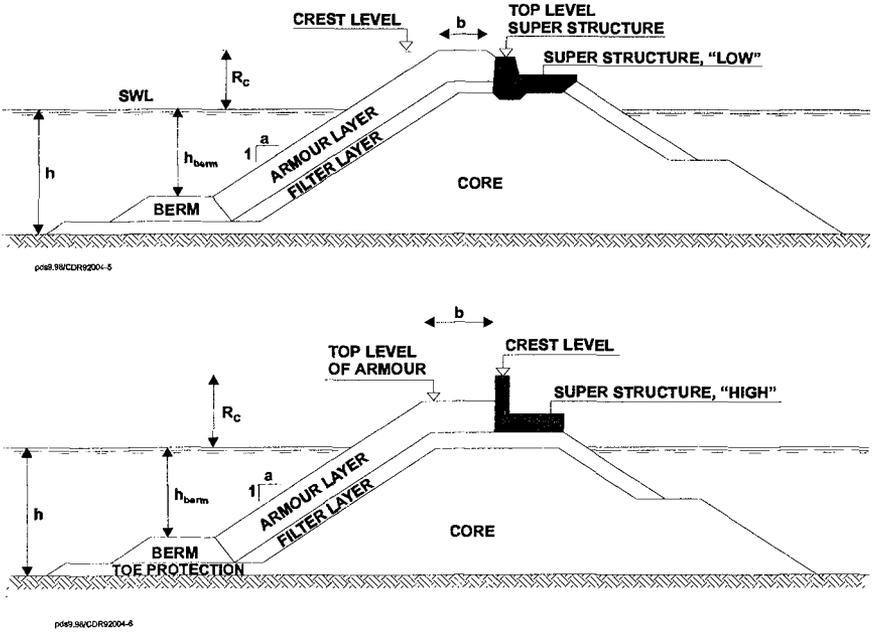


Figure 9 Layout of 'low' respectively 'high' superstructure

**Concluding Remarks**

A general expression for the overtopping discharge of rubble mound breakwaters has been proposed. It includes the influence of wave obliqueness and different types of armour. The expression is valid for rubble mound breakwaters both with and without superstructures.

$$\left. \begin{aligned} Q &= Q^* \sqrt{g \cdot H_s^3} \\ Q^* &= k_1 \cdot \ln(s_{op}) \cdot \exp\left(\frac{k_2 \cdot C}{r H_s \sqrt{\cos \theta}}\right) \\ C &= (a^{0.3} \cdot (2R_c + 0.35 \cdot b)) \end{aligned} \right\} \quad [8]$$

The following values for  $k_1$ ,  $k_2$  and  $r$  are recommended:

$k_1 = -0.3$ ,  $k_2 = -1.6$  for pure rubble mound structure

$k_1 = -0.01$ ,  $k_2 = -1.0$  for rubble mound structure with superstructure

$r = 0.65$  for an armour layer of rounded stones in two layers

$r = 0.65$  for an armour layer of grooved cubes (Antifer units) in two layers

$r = 0.55$  for an armour layer of quarry rock in two layers

$r = 0.55$  for an armour layer of Accropodes in one layer

$r = 0.45$  for an armour layer of Dolos units in two layers

It should be realised that the expression for other types of armour than quarry rock has been verified through model tests with different model set-ups, with sloping seabeds, different model scales, etc, than used in the research study. In spite of this, the results of the verifications show that [8] gives good estimates on the overtopping discharges.

It is, however, found that further research will be needed to obtain an understanding of the importance of

- Breaking waves in front of the structure
- Seabed slope in front of the structure
- Berm structures
- Influence of wind on the overtopping discharge.

## Acknowledgement

The Danish Technical Research Council (STVF) under the program Marine Technique financed the study.

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