A number of equations has been developed for calculation overtopping quantities over breakwaters, as well as equations to describe the stability of the inner slope. However, not all the water which overtops the outer boundary of the breakwater will reach the inner slope; part of the water is infiltrated in the crest. In this paper data are presented on the percentage of water infiltrating in the crest of the breakwater. The results are compared with the equation presented in the UK overtopping manual.

1. Introduction

Under design conditions breakwaters of intermediate height will be subject to overtopping. Part of this overtopping water will pass the crest of the breakwater and will reach the inner slope, an other part will infiltrate in the crest of the breakwater, and may flow out of the inner slope, causing some additional damage.
In previous research [Verhagen et al., 2003] the stability of blocks on an inner slope has been studied as a function of the amount of water passing the inner crest line of the breakwater (q3, see figure 1). This paper focuses on the differences between the amount of water passing the outer crest line and the inner crest line of the breakwater (q1-q3) as a function of the crest width.

2. Summary of the previous work
As reported in Verhagen et al. [2003] the stability of stones on the inner slope of a revetment depends on the overtopping discharge. In order to quantify the discharge in such a way that a direct relation between the wave and damage can be described, a characteristic velocity has been defined. The characteristic velocity is the maximum discharge during overtopping divided by the maximum layer thickness (and of course by the flume width). The maximum discharge is indicated in Figure 2.

![Figure 2. Characteristic velocity is maximum discharge divided by maximum layer thickness](image)

From the analysis a stability relation can be derived. This relation is worked out in Verhagen et al. [2003] is as follows:

\[
\Theta_{u_{\text{char}}, R, \alpha, I} = \frac{(u_{\text{char}} \cos(\beta - \alpha))^2}{\Delta g D_{n50}} \frac{R}{D_{n50}} \frac{\sin(\alpha) \sqrt{f}}{i}
\]

(1)

in which:
- \(u_{\text{char}}\) characteristic velocity, as defined above
- \(\alpha\) inner slope of the breakwater
- \(\beta\) angle of attack of the wave plunge
From this investigation followed that $u_{char}$ is a very important parameter. The next step in research is to find out what values for $u_{char}$ can be expected in real cases. When overtopping experiments are conducted, the layer thickness can be measured with simple devices, like a capacity wave gage. However, because such a meter is also hampering the current itself, the output of such a meter is not very reliable. An example of the output of such a wave gage is given in figure 4. On the vertical axis the output in volt is given, to stress that the quantitative output is not reliable.
In order to use this information, one should first define a time condensing factor $F_{cd} = T/T_{ovt}$, in which $T$ is the wave period and $T_{ovt}$ is the time the wave is overtopping the breakwater. This value can be measured quite accurately with the wave gage. During overtopping tests one can measure the total overtopping. The amount of overtopping per unit of time ($q$, in m³/s per meter crest) can be easily computed by dividing the total overtopping by the duration of the test. From simple geometric relations follows then that the maximum discharge during overtopping is:

$$q_{\text{max}} = 2F_{cd}q$$

(2)

The value of $F_{cd}$ can be related to the geometrical properties of the slope (like slope angle), by using the structure of the Van der Meer overtopping formula [TAW, 2002]. This leads to

$$\frac{1}{F_{cd}^{\alpha}} \left( \frac{s_m}{\tan \alpha} \right) = a_1 \exp \left[ a_2 \sqrt{\frac{s_m}{\tan \alpha}} \left( \frac{R_c}{H_{m0}} \right)^{\frac{3}{2}} \right]$$

(3)

in which:

- $s_m$ wave steepness
- $H_{m0}$ zeroth moment wave height

From evaluation of flume tests followed $a_1 = 0.0032$ and $a_2 = -1.96$ (in the analysis for the period the value $T_{-1.0}$ was used).

![Figure 5. relation between time averaged overtopping and maximum overtopping for all the tests](image-url)

Analysis of the measured wave data shows that the relation between the time averaged overtopping $q$ and the maximum overtopping $q_{\text{max}}$ is not completely linear.
3. The overtopping and infiltration tests
In order to investigate how much infiltration can be expected in the crest of the breakwater, an experimental set-up has been made in the Laboratory of Fluid Mechanics. We created a simple slope, 1:2 with a rough rock slope. The rock was placed on fine gravel; the only purpose of the rock was to create a correct roughness. Because stability of the rock slope was not a subject of investigation, there were no secondary filter layers. The core was made up of fine gravel. During the tests wave height, wave period, crest width and freeboard were varied.

Figure 6. Overview of the set-up

The crest consisted of a wire mesh with on top a layer of course material (identical to the material on the slope). The size of the stones was (on a model scale) comparable to normal, stable armour rock for these conditions. See for details also figure 7.

From some intermediate tests it was found that the permeability of normal filter layers and core is sufficiently high to allow all water to pass through with such a speed that there will be no storage of water inside the crest armour layer. This allowed us not to include the secondary layer and core in the model. Practically this means that all water with is captured by the crest will flow vertically through the crest and secondary armour into the core layer. This water will not flow out of the breakwater at the inner slope (above the water line). However, this is only true for normal, permeable layers. In practise often a construction road is built on top of either the core or the secondary armour. When such a road is not removed, the infiltrated water will flow over that layer to the inner slope of the breakwater, and cause a destabilising outflow (flow q6 in figure 1). But even in case of a road between the secondary layer and the armour, the storage capacity of the armour is sufficiently large to prevent that a water layer is built up inside the armour in such a way that the top of the armour becomes flooded and infiltration is hampered.

By using this model the amount of water infiltrating in the crest as well as the amount of water flowing over the crest can be measured. Because the tests were done with regular waves the amount of infiltration and overtopping per
wave can be determined easily by dividing through the number of waves in the experiment.

Figure 7. Top view of the model

The test resulted in graphs like figure 8, where the volume of overtopping and infiltrated water are plotted as a function of the wave height.

Figure 8. Overtopping and infiltration as a function of the wave height for a crest of 4.2 cm

Also a number of tests were done using an impermeable layer between the armour units on the crest and the first filter layer. Because of the high
permeability of the top layer the result was that all "infiltration" water immediately flew through the top armour towards the inner slope. This amount of water will flow out of the inner slope relatively fast, and decrease the stability of the inner slopes.

4. Analysis of the results

A crest width and overtopping parameter has been defined:

\[ Q_{\text{tot}}^* = \frac{q_{\text{tot}}}{\sqrt{gB^3}} \]  

in which \( q_{\text{tot}} \) is the overtopping volume in \( \text{m}^3/\text{wave} \) per meter of crest length at the outer crest line, and \( B \) is the crest width. As output parameter is selected the percentage of water passing over the full crest, \( q_3/q_{\text{tot}} \).

Figure 9. All overtopping and infiltration results

All measurements resulted in figure 9. From the figure follows that with an increase of the crest width (i.e. when \( Q^* \) decreases) the percentage of water not able to reach the inner crest line is considerable. A more detailed look to the lower left corner of the figure reveals that there is a threshold value

\[ Q_3^* = 8.1 \cdot 10^{-3} \]

The dimensionless overtopping over the inner crest line can consequently be defined as:
\[
\frac{q_3}{q_{tot}} = \begin{cases} 
\frac{Q_{tot}' - Q_t^*}{Q_{tot}' + 7.0 \cdot 10^{-2}} & Q_{tot}' < Q_t^* \\
0 & Q_{tot}' < Q_t^* 
\end{cases}
\]

in which:
- \(q_3\) overtopping at rear side of crest
- \(q_{tot}\) overtopping at front side of crest
- \(Q_{tot}'\) dimensionless crest width
- \(Q_t^*\) threshold value for \(Q_{tot}'\)

With the above equation the real amount of water per wave on the inner slope can be computed, which can be used in the stability calculations, presented by Verhagen et al. [2003].

Small tests with a separation between the armour on the crest and the armour of the inner slope, using a rather impermeable layer between armour and crest resulted in an immediate filling of the "basin" on top of the breakwater. As a result the discharge on the inner crest and the outer crest became equal.

By using the distribution of overtopping quantities (e.g. as defined by Van der Meer [TAW, 2002]), which is in fact a Rayleigh distribution, one can determine the total amount of water passing the inner crest line of the breakwater. In this way one can adapt the common equations for overtopping quantities over a breakwater for the effect of a crest with of more than 2 units.

5 Comparison with other work
In the UK overtopping manual (Wallingford, 1999) a formula is presented by Besley which also includes the crest width. This formula is given as:

\[
\frac{q_3}{q_{tot}} = 3.06 \exp\left(-1.5 \frac{B}{H_s}\right)
\]

Because here the crest reduction is given as a function of the significant wave height, a direct comparison with the formula presented in this paper is not possible. In order to make a good comparison of the results, three graphs are presented (figure 10). In the left graph the observed values of \(q_3/q_{tot}\) are plotted vs. the results of equation 5. As expected, this gives a good correlation.
figure 10. Comparison of measured data with direct fit, fit with Van der Meer overtopping formula and with Besley overtopping formula
In the middle graph the overtopping is calculated using the Van der Meer equation as presented in TAW [1999], but corrected for the fact that in this tests the waves are regular. On the vertical axis the value of $q_3/q_{tot}$ is given used both the computed overtopping with the Van de Meer equation and equation (5). In the right graph the observed value of $q_3/q_{tot}$ is plotted vs. the formula of Besley from Wallingford [1999].

6. Conclusions
A formula for overtopping discharge has been developed in which the crest width has been included. In case of a wide crest, a substantial part of the water flows into the crest and does not damage the inner slope.

Economically it is not attractive to try to lower the crest by making the crest wider; especially not for larger waves and high overtopping quantities.

By using he time condensing factor it is easy to translate average overflow into instantaneous (maximum) overflow.

Detailed insight in the layer thickness during overflow is essential to improve the calculations.

Acknowledgements
This work has been done in the Laboratory of Fluid Mechanics of Delft University of Technology; the work of the third author has been funded from a cooperation project between the Dutch government and the Hanoi Water resources University

References
KEYWORDS – ICCE 2004

INFILTRATION OF OVERTOPPING WATER IN A BREAKWATER CREST
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Abstract number 072

Breakwaters
Infiltration
Overtopping
Crest