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SINGLE WAVE OVERTOPPING VOLUMES AND THEIR TRAVEL DISTANCE FOR RUBBLE MOUND BREAKWATERS

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In the present paper small and large scale overtopping data for rubble mound structures have been analysed with respect to single wave overtopping volumes and their travel distance. The analysis has led to formulae for estimation of maximum single wave overtopping volumes and their travel distance. The estimation of the maximum single wave overtopping volume is based on the total overtopping volume and the number of overtopping waves within a given time (e.g. 1000 waves). The spatial distribution of the overtopping water in the wave giving the largest overtopping volume was found to be very similar to the spatial distribution of the total overtopping volume in a sea state.

INTRODUCTION

Overtopping discharges vary considerably from wave to wave and are very unevenly distributed in time and space. The major part of the overtopping water during a storm is due to a small fraction of the waves, basically those with the highest (fictitious) run-up levels. In fact, the local overtopping discharge from a single wave can be more than 100 times the average overtopping discharge. The volume, velocity and splash-down location (travel distance) of the individual overtopping waves is important for estimation of the damaging impacts on persons, vehicles and structures. Nevertheless, admissible overtopping is normally given as the time averaged overtopping discharge. This is partly due to limited information on maximum single wave overtopping volumes and their travel distance.

The paper presents formulae for these two parameters based on analysis of overtopping data from large scale and small scale 2-D model tests of rubble mound breakwaters. This includes a review of existing formulae for overtopping volume of single waves and number of overtopping waves.

It is expected that the velocity and the momentum of the overtopping water can be estimated from single wave overtopping volumes and their travel distance.

The procedure in estimating the maximum overtopping volume in a single wave, V_{max}, and its spatial distribution is as follows:

- The average overtopping discharge is estimated by use of existing formulae or the neural network developed under the EU-CLASH project, van Gent et al. (2007). From this is determined the total overtopping volume, V_{total}, in the sea state by multiplying with the duration;

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- The number of overtopping waves N_{ow} is estimated by existing formulae. An analysis shows that the formula by Besley (1999) seems to have the best performance;
- A new formula is presented which predicts V_{max} as function of V_{total} and N_{ow};
- A new formula is presented which predicts the spatial distribution of the falling water volume V_{max}.

The steps in the design procedure will be discussed in the following chapters.

MODEL TEST STUDY

Several physical model test programmes at Aalborg University have focused on the spatial distributing of overtopping volumes. This landward distribution of overtopping has in all tests been measured using overtopping trays as shown in Fig. 1. Each of the trays was equipped with a surface elevation gauge and a permanent pump controlled automatically by data acquisition software.

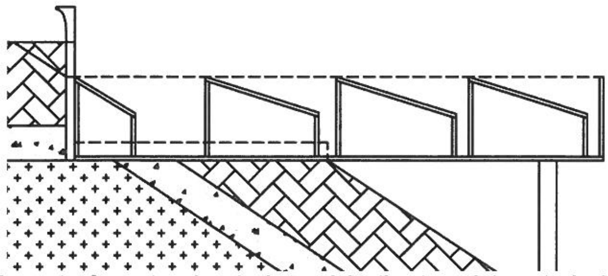

Figure 1. Layout of overtopping tank used in the two data sets included in the present analysis.

The data included in the present analysis are from the tests by Lykke Andersen (2006) and Burcharth et al. (2007). The tests by Lykke Andersen (2006) include 51 large scale tests and 170 small scale tests with geometrical similar cross-sections. The tested breakwaters are conventional rubble mound breakwaters with a 1:2 front slope and different top geometries, cf. Fig. 2. The small scale tests were conducted at Aalborg University and the large scale tests in the CIEM flume at Universitat Politécnica de Catalunya, Barcelona. Further details on these tests are given in Lykke Andersen (2006). The test by Burcharth et al. (2007) are performed with a cube armoured rubble mound breakwater having a 1:1.5 front slope and a wave return wall with a top level slightly above the armour crest level, cf. Fig. 3.

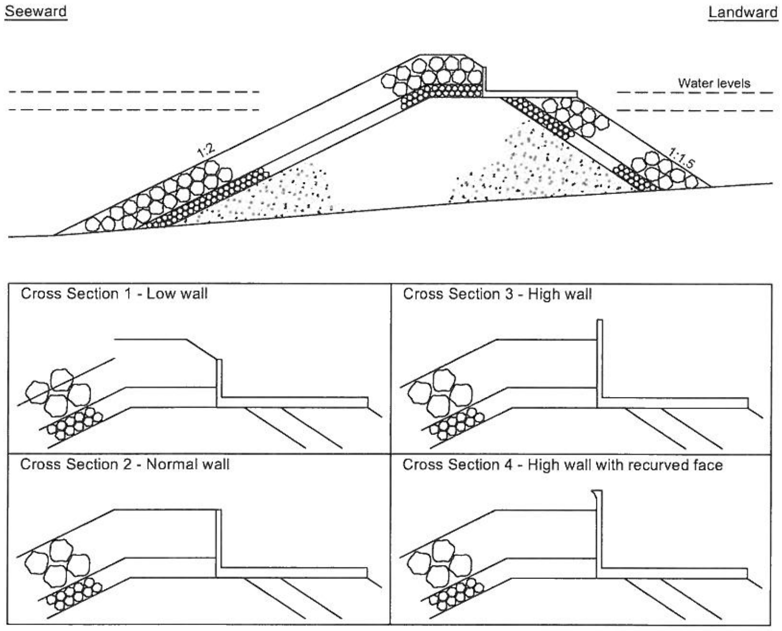

Figure 2. Tested cross sections by Lykke Andersen (2006).

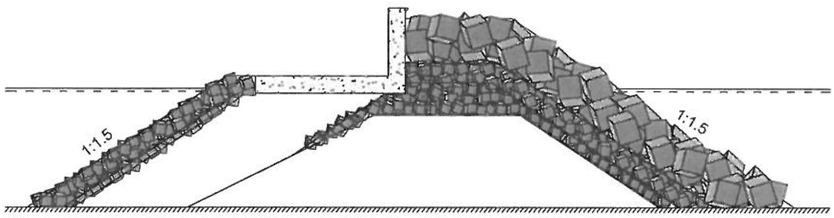

Figure 3. Tested cross sections by Burcharth et al. (2007).

DATA ANALYSIS

The data has in both small scale test programmes been acquired using the WaveLab2 software package by Aalborg University (2007). In this software package the pumps in the individual trays have been configured to start and stop automatically during data acquisition when the water level in the trays reaches certain predefined levels. The water levels in the chambers and the state of the pumps are stored in the data file. An example of the processing of the overtopping signals is shown for one chamber in Fig. 4. It can be seen that the cumulative overtopping volume time series calculated in this way can be used to identify single wave overtopping volumes and their landward distribution. WaveLab2 has also been used to separate incident and reflected waves using the method of Mansard & Funke (1980). As the overtopping is mainly dependent on the incident waves only incident wave parameters determined in this way are considered in the present paper.

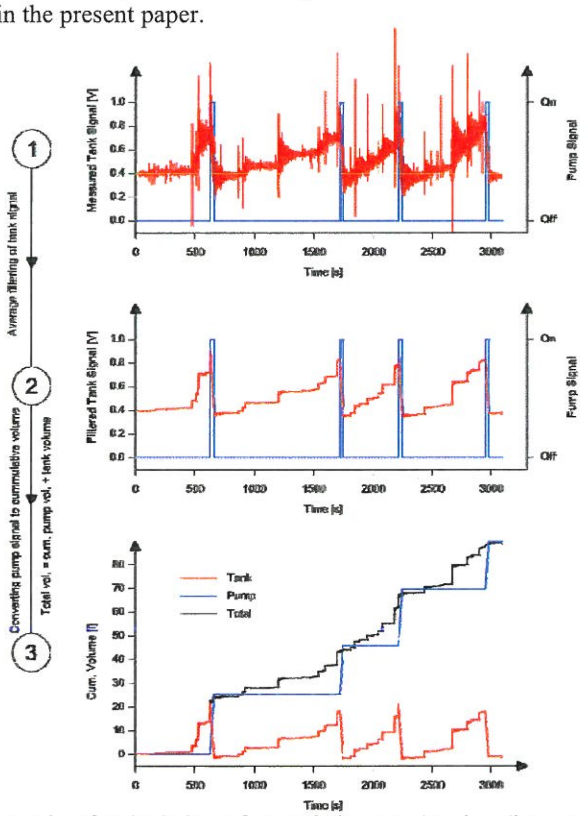

Figure 4. Example of calculation of cumulative overtopping time series for one overtopping chamber.

DISTRIBUTION OF SINGLE WAVE OVERTOPPING VOLUMES

Franco et al. (1994) found for vertical wall breakwaters that overtopping volumes from individual waves follow a two-parameter Weibull distribution. Van der Meer and Janssen (1995) found the same distribution valid also for dikes. In Eq. 1 such a distribution is given where the individual volumes are made dimensionless with the mean overtopping volume per overtopping wave $\overline{V} = V_{total} / N_{ow}$, where V_{total} is the total volume of overtopping water over a period where N_{ow} waves are overtopping.

$$F(V) = 1 - \exp\left[-\left(\frac{V/\overline{V}}{A}\right)^B\right] \qquad (1)$$

A is the scale parameter, B is the shape parameter and $F(V)$ is the non-exceedance probability which can be calculated from Weibulls plotting position formula:

$$F(V_i) = 1 - \frac{i}{N_{ow} + 1} \qquad (2)$$

where i is the rank. Combining Eq. 1 and 2 leads to:

$$\frac{V_i}{\overline{V}} = A \cdot \left[-Ln\left(\frac{i}{N_{ow} + 1}\right)\right]^{1/B} = A \cdot \left[Ln(N_{ow} + 1) - Ln(i)\right]^{1/B} \qquad (3)$$

The maximum overtopping of a single wave can be calculated by setting $i = 1$ which leads to:

$$\frac{V_{\max}}{\overline{V}} = A \cdot \left[Ln(N_{ow} + 1)\right]^{1/B} \qquad (4)$$

This equation is similar to that used by Franco et al. (1994) and van der Meer and Janssen (1995) except that they used N_{ow} instead of N_{ow} +1 in the logarithm. The consequence of this is that their equation predict V_{max} / $\overline{V}$ = 0 for N_{ow} = 1. Therefore, their equation is only valid for N_{ow} larger than five to ten. In quite many cases structures are designed so very few waves overtop the structure, especially when considering events with return periods of some few years. Therefore, the modification might be important in many cases.

Franco et al. (1994) and van der Meer and Janssen (1995) give the distribution parameters B = 0.75 and A = 0.84 for caisson breakwaters and dikes.

Eq. 4 could be rewritten to give the ratio of maximum single wave overtopping volume and total overtopping volume, i.e.:

$$\frac{V_{\max}}{V_{total}} = \frac{A}{N_{ow}} \cdot \left(Ln(N_{ow} + 1)\right)^{1/B} \qquad (5)$$

This equation is plotted in Fig. 5 for B = 0.75 and various values of A. In the figure is also included the model test data from Lykke Andersen (2006) and Burcharth et al. (2007). It can be seen that there is not much difference between the different types of top geometries tested. However, there is a tendency that the low steepness waves give slightly higher percentage of the overtopping volume in the maximum single wave than waves with higher steepness.

However, this influence is quite small and it could not be ruled out that it is simply due to scatter. Moreover, there seems not to be much difference between the data sets. In all of the figures in this paper data points with an o-shape is large scale data and all other data points are small scale data.

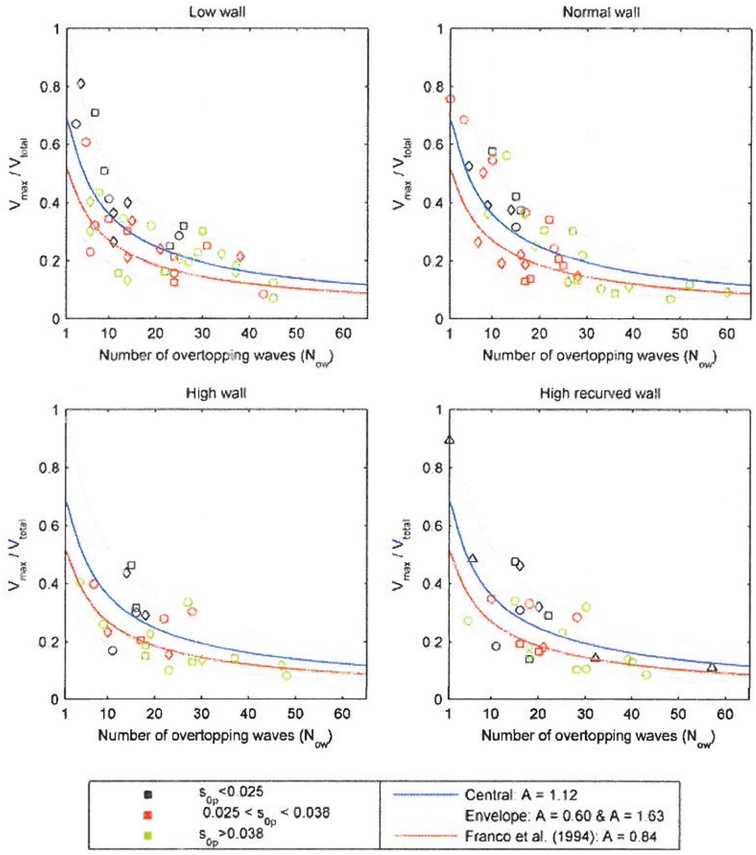

Figure 5. Comparison of measured maximum single wave overtopping volumes and those calculated from Eq. 5 using *B* = 0.75 and various values of *A*. Three different data sets are included in the figure identified by the different shapes of the points. The different shading of the points relate to wave steepness.

Fig. 5 shows that $A = 0.84$ as given by Franco et al. (1994) and van der Meer and Janssen (1995) is close to a lower bound when the number of overtopping waves is small. For a large number of overtopping waves their equation seems to be quite close to a central estimate. The lower and upper envelope curves are given by $A = 0.6$ and $A = 1.63$, respectively. $A = 1.63$ corresponds to $V_{max} / V_{total} = 1$ for $N_{ow} = 1$ which is the expected value for a single overtopping wave.

However, when only one wave is giving green water overtopping there will in most cases be other waves giving spray. That is the reason for many of the data points being significantly below the curve given by $A = 1.63$ also for small values of N_{ow}.

To apply Eq. 5 or Fig. 5 for design purposes a method to estimate the mean overtopping discharge and the number of overtopping waves is needed.

Several methods to estimate mean overtopping discharges exists. Lately a data base including approximately 10,000 overtopping test has been established during the CLASH project and a neural network model has been developed to estimate mean discharges, De Rouck (2005). The neural network model has been presented by van Gent et al. (2007) and seems quite accurate for many types of structures. Therefore, this item is not discussed any further in the present paper.

The existing knowledge on the number of overtopping waves is much less than knowledge about mean discharges. Some few empirical formulae exist, which validity is investigated in the next section for the two data sets.

NUMBER OF OVERTOPPING WAVES

Van der Meer and Janssen (1995) gives the following formula to calculate the number of overtopping waves (N_{ow}):

$$\frac{N_{ow}}{N_w} = \exp\left[-\left(\frac{R_c / H_s}{c}\right)^2\right] \tag{6}$$

where N_w is the number of waves, R_c is the freeboard and H_s is the incident significant wave height at the toe. The value of the c coefficient follows from the assumption that the run-up distribution is similar to the distribution of the waves, i.e. run-up is assumed to be a linear phenomenon. By assuming the waves Rayleigh distributed, van der Meer and Janssen (1995) found for non-breaking waves ($\xi > 2$):

$$c = 1.62\gamma_h \gamma_f \gamma_\beta \tag{7}$$

where the γ-values take into account a shallow foreshore, roughness and angle of attack. In Fig. 6 the probability of overtopping is plotted against the relative freeboard. The curve of van der Meer and Janssen (1995), Eq. 6, is also included assuming $H_{m0} \approx H_s$, $\gamma_f = 0.40$ and $\gamma_h = 1.0$. As freeboard, R_c in Eq. 6, is used the armour crest freeboard (A_c) as this was found to give the best fit also for the high wall case where the wall freeboard is significantly larger than the armour crest freeboard. It can be seen that the formula of van der Meer and Janssen (1995) gives in most cases values in the correct order of magnitude, but the trend of the data seems different from that given by Eq. 6. Therefore, the number of overtopping waves is overestimated for small values of A_c/H_{m0}. This is not expected to be due to the discarded influence of γ_h as the wave heights at the toe are in most cases close to Rayleigh distributed and $h_{toe}/H_{m0,toe}$ is always larger than 3. A special note has to be given to the data of Burcharth et al. (2207) as there is a big difference between these test results and those from Lykke Andersen (2006). This might partly be due to the use of large cubes instead of rock as armour. The data of Burcharth et al. (2007) corresponds to a higher roughness factor ($\gamma_f \approx 0.55$), which has a large impact on the number of overtopping waves as shown in Fig. 6. This also shows that the reliability of Eq. 6 highly depends on a very good estimate on the roughness factor.

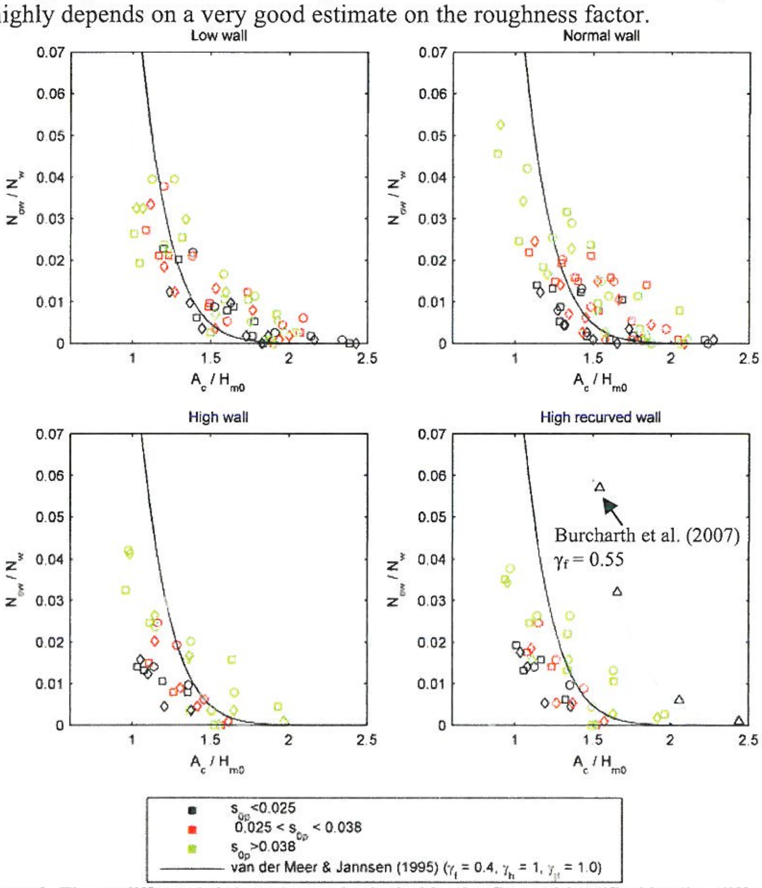

Figure 6. Three different data sets are included in the figure identified by the different shapes of the points. The different shadings relate to wave steepness.

Besley (1999) has presented two different sets of equations for the probability of overtopping on sloping structures. In the first one, Eq. 8, the probability of overtopping is related to the dimensionless overtopping discharge

$Q_* = q/(T_m \cdot g \cdot H_s)$. The formulae were fitted to data from tests with a variety of sloping structures. Eq. 8 is evaluated against the present data in Fig. 7.

$$\frac{N_{ow}}{N_w} = 55.4 \cdot Q_*^{0.634} \quad \text{for } Q_* < 8 \cdot 10^{-4}$$
$$\frac{N_{ow}}{N_w} = 2.50 \cdot Q_*^{0.199} \quad \text{for } 8 \cdot 10^{-4} \leq Q_* < 1 \cdot 10^{-2} \qquad (8)$$
$$\frac{N_{ow}}{N_w} = 1 \quad \text{for } Q_* > 1 \cdot 10^{-2}$$

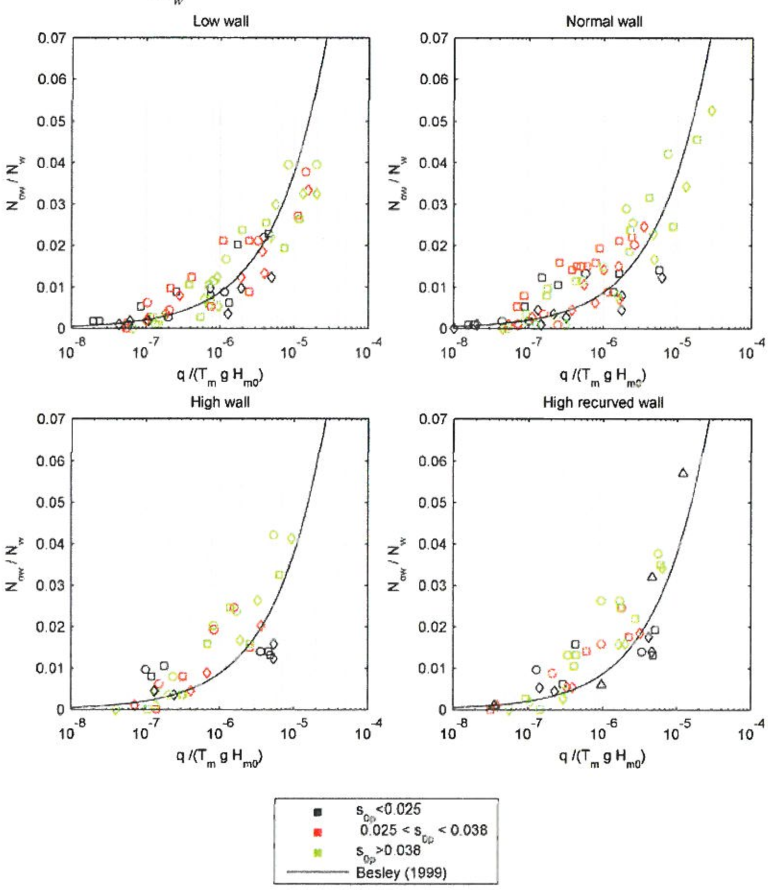

Figure 7. Evaluation of Besley (1999) formula (Eq. 8). Three different data sets are included in the figure identified by the different shapes of the points. The different shadings of the points relate to wave steepness.

It can be seen that this method fits very well to both data sets, and the observed differences between the two data sets seen in Fig. 6 are not present. This means that Eq. 8 is independent of the front geometry and the roughness factor. Therefore, Eq. 8 is highly recommended for the estimation of the number of overtopping waves on rubble mound breakwaters.

The second method of Besley, Eq. 9, is valid for simple slopes only and is quite similar to the method used by van der Meer and Janssen (1995), except that a different dimensionless freeboard is used.

$$\frac{N_{ow}}{N_w} = \exp\left[-C \cdot \left(\frac{R_c}{T_m\sqrt{g \cdot H_s}} \cdot \frac{1}{\gamma_f}\right)^2\right] \tag{9}$$

C is a parameter that depends on the slope of the structure. The value of C is 37.8 for a 1:2 slope and 63.8 for a 1:1.0 slope. In the present analysis it was found that this equation predicts too strongly the influence of the wave period when the waves are non-breaking. Therefore, this formula is not considered any further in the present paper.

TRAVEL DISTANCE OF SINGLE WAVE OVERTOPPING VOLUMES

If a good estimate of the average discharge is available, it is possible with Eqs. 5 and 8 to estimate the maximum single wave overtopping volume. The last objective of the present paper is to give a formula to estimate the travel distance of these large single wave overtopping volumes. Previously Lykke Andersen & Burcharth (2006) has presented a formula for the landward distribution of the average discharge. Eq. 10 is a rewritten version of the original equation in which the travel distance is made dimensionless using H_{m0} instead of L_0 but at the same time the powers on s_{0p} is adjusted so the two formulae are identical.

It is clear that the spatial distribution of the overtopping water must be dependent on the elevation at which the distribution is wanted or measured. To describe this effect the parameter h_{Level} was introduced being the vertical distance from the crown wall crest level to the level of interest, cf. Fig. 8.

$$\frac{q_{\text{passing x}}}{q_{\text{total}}} = \exp\left(-1.1 \cdot s_{0p}^{-0.05} \cdot \frac{\max\left(x / \cos(\beta) - 2.7 \cdot h_{\text{level}} \cdot s_{0p}^{0.15}, 0\right)}{H_{m0}}\right) \tag{10}$$

where s_{0p} is the fictitious deep water peak wave steepness, β is the wave obliquity and x is the travel distance at a distance h_{level} below the crown wall, cf. Fig. 8.

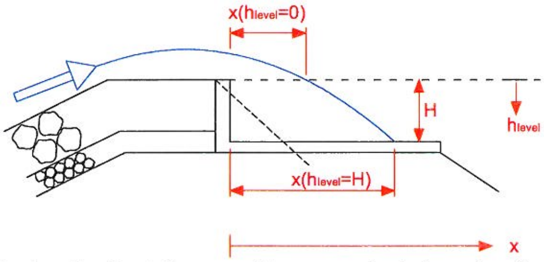

Figure 8. Sketch showing the influence of h_{level} on splash down location.

In Fig. 9 the evaluation of this formula on the total overtopping volume and on the maximum single wave overtopping volume is given. It can be seen that the travel distance of the maximum and average overtopping volume is not so different. However, more scatter is observed on the maximum overtopping volume.

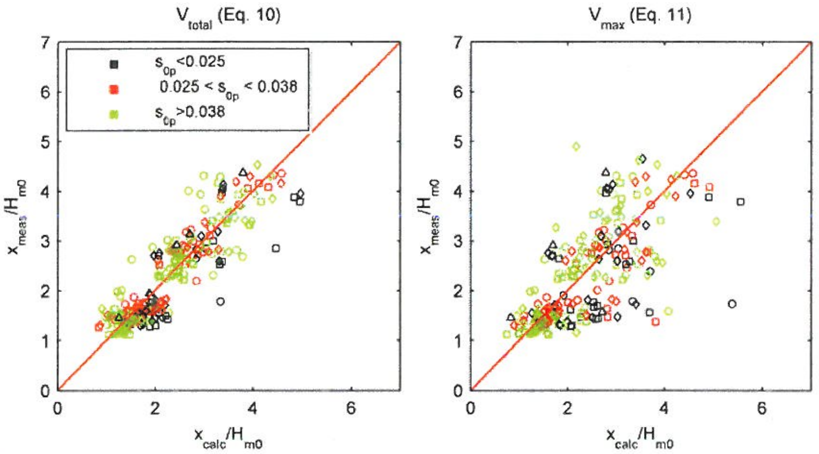

Figure 9. Evaluation of Eq. 10 and 11.

$$\frac{V_{max,passing\ x}}{V_{max,total}} = \exp\left(-1.1 \cdot s_{0p}^{-0.05} \cdot \frac{\max\left(x/\cos(\beta) - 2.7 \cdot h_{level} \cdot s_{0p}^{0.15}, 0\right)}{H_{m0}}\right) \qquad (11)$$

As the present data is from large and small scale model tests, the influence of wind has not been taken into account. The presented formula for landward distribution of overtopping is limited to green water overtopping and cannot predict travel distance of wind carried spray. The influence of scale effects on the landward distribution is expected to be quite small as indicated by the test results in large and small scale.

In the EU-CLASH project differences between prototype and small scale overtopping results have been observed. These differences might be due to model effects (e.g. wind) or scale effects. Wind and scale effects are expected to be most pronounced for relatively small overtopping discharges ($q < 1$ l/sm).

CONCLUSIONS

- Experimental results on overtopping of individual waves have been analysed.
- A calculation procedure for calculating single wave overtopping volumes and their travel distance has been presented.
- One of the two formulae presented by Besley (1999) predict the number of waves overtopping with great accuracy in all of the tested cases.
- The existing formula for single wave overtopping volume has been slightly modified to account for cases with few overtopping waves. Upper and lower envelope curves has been given for single wave overtopping volume.
- The spatial distribution of the travel distance of the water in the maximum single wave overtopping volumes is quite similar to that found for the mean discharge.

REFERENCES

Aalborg University, 2007. WaveLab2 homepage, *http://hydrosoft.civil.aau.dk/wavelab.*

Besley, P., 1999. Wave overtopping of seawalls – design and assessment manual. R & D
Technical Report W 178, Environment *Agency, Bristol, ISBN 1 85705 069 X.*

Burcharth, H. F., Meinert, P., Lykke Andersen, T., Skals, K., 2007. Report on 2D model testing of the breakwater for a new port at Puerto de la Cruz, Tenerife. *Department of Civil Engineering, Aalborg University.*

De Rouck, J., 2005. CLASH – D46: Final Report. *Ghent University, Belgium.*

Franco, L., de Gerloni, M., van der Meer, J. W., 1994. Wave Overtopping on Vertical and Composite Breakwaters. *Proceedings of the24th International Coastal Engineering Conference.*

Lykke Andersen, T., 2006. Hydraulic Response of Rubble Mound Breakwaters. Scale Effecgs – Berm Breakwaters. *PhD thesis from Department of Civil Engineering, Aalborg University. ISSN 0909-4296 Series Paper No. 27.*

Lykke Andersen, T.,Burcharth, H. F., 2006. Landward Distribution of Wave Overtopping for Rubble Mound Breakwaters. *Proceedings of the First International Conference on the Application of Physical Modelling to Port and Coastal Protection,* Porto, Portugal (in Press).

Mansard, E. P. D. and Funke, E. R., 1980. The measurement of incident and reflected spectra using a least squares method. *Proceedings of the 17ᵗ International Conference on Coastal Engineering,* Sydney, Australia.

van der Meer, J. W., Janssen, W., 1995. Wave Run-Up and Wave Overtopping at Dikes. *Wave Forces on Inclined and Vertical Wall Structures.*

van Gent, M. R. A., van den Boogaard, H. F. P., Pozueta, B., Medina, J. R., 2007. Neural network modelling of wave overtopping at coastal structures. *Coastal Engineering, Vol. 54, pp. 586-593.*

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Overtopping
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