Chapter 15

Wave Run-Up and Wave Overtopping at Armored Rubble Slopes and Mounds

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Wave overtopping and to a lesser extent wave run-up for armored rubble slopes and mounds have been subject to a number of investigations in the past. The objective of the present chapter is to summarize existing information to be present as a closed guidance on the use of wave run-up and wave overtopping formulae for a wide range of possible applications in practice. Therefore, guidance is given first on the use of wave run-up and wave overtopping formulae for simple slopes, excluding the effects of composite slopes, direction of wave attack, roughness, wave walls, etc. Then, formulae are presented to include these parameters in the calculation procedure. Guidance is also given on wave overtopping volumes, overtopping velocities, and the spatial distribution as well as for wave overtopping for shingle beaches. Finally, the effect of model and scale effects on the calculation of average overtopping rates are discussed. This chapter has mainly been composed from Chap. 6 of the *EurOtop* Overtopping Manual (2007), with some additions from Chap. 5. The present chapter is related to the previous Chap. 14 and the next Chap. 16 of this manual.

15.1. Introduction

Armored rubble slopes and mounds are characterized by a mound with some porosity or permeability, covered by a sloping porous armor layer consisting of large rock or concrete units. In contrast to dikes and embankment seawalls, the porosity of the structure and armor layer plays a role in wave run-up and overtopping. The cross section of a rubble mound slope, however, may have great similarities with an embankment seawall and may consist of various slopes.

As an example for armored slopes and mounds, a rock-armored embankment is given in Fig. 15.1.

Fig. 15.1. 1:4 rock-armored embankment.

15.2. Wave Run-Up and Run-Down Levels, Number of Overtopping Waves

15.2.1. *Introduction to wave run-up*

The wave run-up height is defined as the vertical difference between the highest point of wave run-up and the still water level (SWL) (Fig. 15.2). Due to the stochastic nature of the incoming waves, each wave will give a different run-up level. In many design formulae, the wave run-up height $R_{u2\%}$ is used as a characteristic parameter to describe wave run-up. This is the wave run-up height, which is exceeded by 2% of the number of incoming waves at the toe of the structure. The idea behind this was that if only 2% of the waves reach the crest of a coastal structure, the crest and inner slope do not need specific protection measures. It is for this reason that much research in the past has been focused on the 2%-wave run-up height. In the past decade the design or safety assessment has been changed to allowable overtopping instead of wave run-up.

Wave run-up has always been less important for rock-armored slopes and rubble mound structures, and the crest height of these types of structures has mostly been based on allowable overtopping, or even on allowable transmission (low-crested structures). Still an estimation or prediction of wave run-up is valuable as it gives a prediction of the number or percentage of waves which will reach the crest of the structure and eventually give wave overtopping. And this number is needed for a good prediction of individual overtopping volumes per wave, overtopping velocities, and flow depths.

The general formula that can be applied for the 2% mean wave run-up height is given by Eq. (15.1): The relative wave run-up height $R_{u,2\%}/H_{m0}$ in Eq. (15.1) is related to the breaker parameter $\xi_{m-1,0}$.

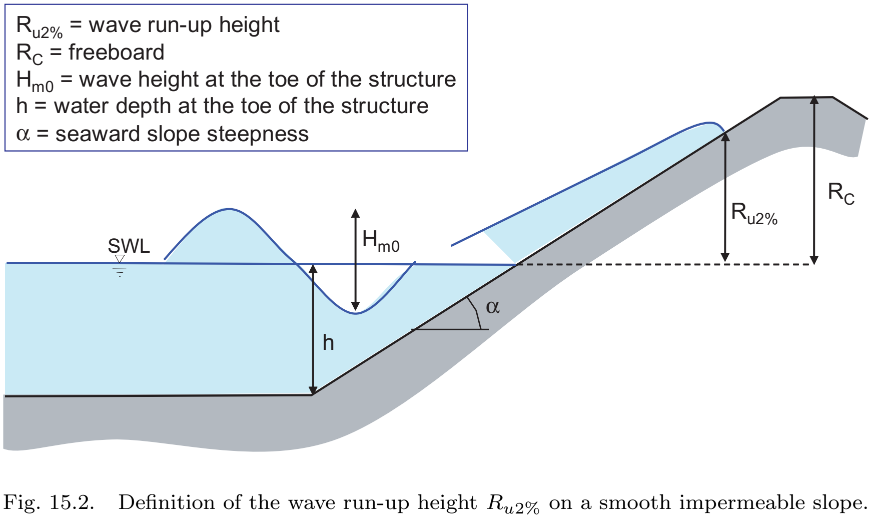

Fig. 15.2. Definition of the wave run-up height $R_{u2\%}$ on a smooth impermeable slope.

$$\frac{R_{u2\%}}{H_{m0}} = 1.65 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_{m-1,0} \tag{15.1}$$

with a maximum of $\frac{R_{u2\%}}{H_{m0}} = 1.00 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \left(4.0 - \frac{1.5}{\sqrt{\xi_{m-1,0}}}\right)$,

where

$R_{u2\%}$ = wave run-up height exceeded by 2% of the incoming waves [m];
H_{m0} = significant wave at the toe of the structure [m];
γ_b = influence factor for a berm [see 15.3.4(b)] [–];
γ_f = influence factor for roughness on the slope [–];
γ_β = influence factor for oblique wave attack (see 15.3.3) [–];
$\xi_{m-1,0}$ = breaker parameter = $\tan\alpha/\sqrt{s_{m-1,0}} = \tan\alpha\Big/\sqrt{2\pi H_{m0}/(gT_{m-1,0}^2)}$ [–];
$\tan\alpha$ = average slope angle (see Fig. 15.2) [–];
$T_{m-1,0}$ = spectral moment at the toe of the structure, based on m_{-1} and m_0 [s].

The breaker or surf similarity parameter $\xi_{m-1,0}$ relates the slope steepness $\tan\alpha$ (or $1/n$) to the fictitious wave steepness $s_{m-1,0} = 2\pi H_{m0}/(gT_{m-1,0}^2)$ and is often used to distinguish different breaker types. The combination of structure slope and wave steepness gives a certain type of wave breaking (Fig. 15.3).

For $\xi_{m-1,0} > 2$–3 waves are considered not to be breaking (surging waves), although there may still be some breaking, and for $\xi_{m-1,0} < 2$–3 waves are breaking. Waves on a gentle foreshore break as spilling waves and more than one breaker line can be found on such a foreshore. Plunging waves break with steep and overhanging fronts and the wave tongue will hit the structure or back washing water. The transition between plunging waves and surging waves is known as collapsing. The wavefront becomes almost vertical, and the water excursion on the slope (wave run-up + run-down) is often largest for this kind of breaking. Values are given for the majority of the larger waves in a sea state. Individual waves may still surge for generally plunging conditions or plunge for generally surging conditions.

The relative wave run-up height increases linearly with increasing $\xi_{m-1,0}$ in the range of breaking waves and small breaker parameters less than about 2. For nonbreaking waves and higher breaker parameters, the increase is less and

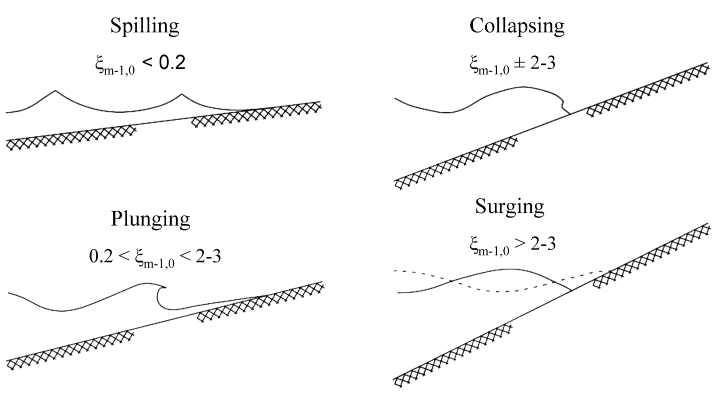

Fig. 15.3. Type of breaking on a slope.

becomes more or less horizontal. The relative wave run-up height $R_{u,2\%}/H_{m0}$ is also influenced by the geometry of the structure, the effect of wind, and the properties of the incoming waves.

15.2.2. *Wave run-up on rock-armored slopes*

Figure 15.4 gives 2% wave run-up heights for various rocks slopes with cot $\alpha = 1.5$, 2, 3, and 4, and for an impermeable and permeable core of the rubble mound. These run-up measurements were performed during the stability tests on rock slopes of van der Meer.[11] First of all, the graph gives values for a large range of the breaker parameter $\xi_{m-1,0}$, due to the fact that various slope angles were tested, but also with long wave periods (giving large $\xi_{m-1,0}$-values). Most breakwaters have steep slopes 1:1.5 or 1:2 only and then the range of breaker parameters is often limited to $\xi_{m-1,0} = 2$–4. The graph gives rock slope information outside this range, which may be useful also for slopes with concrete armor units.

The highest curve in Fig. 15.4 gives the prediction for smooth straight slopes ($\gamma_f = 1$). A rubble mound slope dissipates significantly more wave energy than an equivalent smooth and impermeable slope. Not only both the roughness and porosity of the armor layer cause this effect, but also the permeability of the under-layer and core contribute to it. Figure 15.4 shows the data for an impermeable core (geotextile on sand or clay underneath a thin under-layer) and for a permeable core (such as most breakwaters). The difference is most significant for large breaker parameters.

Equation (15.2) includes the influence factor for roughness γ_f. For two layers of rock on an impermeable core, $\gamma_f = 0.55$. This reduces to $\gamma_f = 0.40$ for two layers of rock on a permeable core. This influence factor is used in the linear part of the run-up formula, say, for $\xi_{m-1,0} \leq 1.8$. From $\xi_{m-1,0} = 1.8$, the roughness factor increases linearly up to 1 for $\xi_{m-1,0} = 10$, and it remains 1 for larger values.

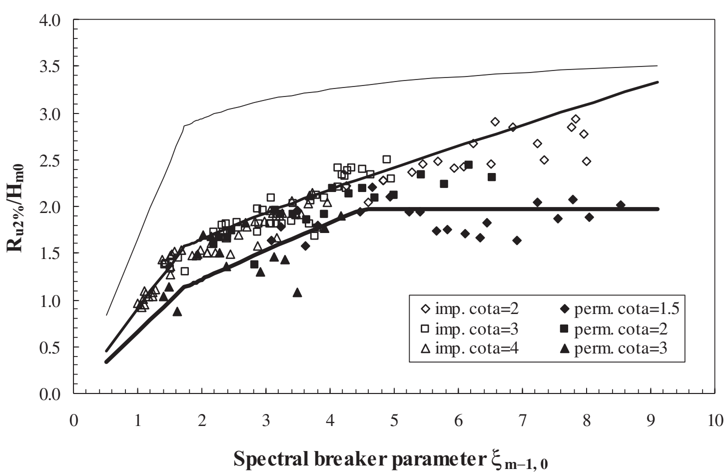

Fig. 15.4. Relative run-up on straight rock slopes with permeable and impermeable core, compared to smooth impermeable slopes.

The prediction for the 2% mean wave run-up value for rock or rough slopes can be described by

$$\frac{R_{u2\%}}{H_{m0}} = 1.65 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_{m-1,0}\,, \tag{15.2}$$

with a maximum of $\frac{R_{u2\%}}{H_{m0}} = 1.00 \cdot \gamma_b \cdot \gamma_{f\,\mathrm{surging}} \cdot \gamma_\beta \left(4.0 - \frac{1.5}{\sqrt{\xi_{m-1,0}}}\right)$.

From $\xi_{m-1,0} = 1.8$, the roughness factor $\gamma_{f\mathrm{surging}}$ increases linearly up to 1 for $\xi_{m-1,0} = 10$,
which can be described by
$\gamma_{f\mathrm{surging}} = \gamma_f + (\xi_{m-1,0} - 1.8) \cdot (1 - \gamma_f)/8.2$
$\gamma_{f\mathrm{surging}} = 1.0$ for $\xi_{m-1,0} > 10$.

For a permeable core, however, a maximum is reached for $R_{u2\%}/H_{m0} = 1.97$. The physical explanation for this is that if the slope becomes very steep (large $\xi_{m-1,0}$-value) and the core is impermeable, the surging waves slowly run up and down the slope, and all the water stays in the armor layer, leading to a fairly high run-up. The surging wave actually does not "feel" the roughness anymore and acts as a wave on a very steep smooth slope. For a permeable core, however, the water can penetrate into the core which decreases the actual run-up to a constant maximum (the horizontal line in Fig. 15.4).

Equation (15.2) may also give a good prediction for run-up on slopes armored with concrete armor units, if the right roughness factor is applied.

Deterministic design or safety assessment. For design or a safety assessment of the crest height, it is advised not to follow the average trend, but to include the uncertainty of the prediction. As the basic equation is similar for a smooth and a rough slope, the method to include uncertainty is also the same. This means that for a deterministic design or safety assessment, Eq. (15.3) should be used:

$$\frac{R_{u2\%}}{H_{m0}} = 1.75 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_{m-1,0}\,, \tag{15.3}$$

with a maximum of $\frac{R_{u2\%}}{H_{m0}} = 1.00 \cdot \gamma_b \cdot \gamma_{f\,\mathrm{surging}} \cdot \gamma_\beta \left(4.3 - \frac{1.6}{\sqrt{\xi_{m-1,0}}}\right)$.

From $\xi_{m-1,0} = 1.8$, the roughness factor $\gamma_{f\mathrm{surging}}$ increases linearly up to 1 for $\xi_{m-1,0} = 10$, which can be described by
$\gamma_{f\mathrm{surging}} = \gamma_f + (\xi_{m-1,0} - 1.8) \cdot (1 - \gamma_f)/8.2$
$\gamma_{f\mathrm{surging}} = 1.0$ for $\xi_{m-1,0} > 10$.

For a permeable core a maximum is reached for $R_{u2\%}/H_{m0} = 2.11$.

Probabilistic design. For probabilistic calculations, Eq. (15.2) should be used together with a normal distribution and variation coefficient of CoV = 0.07. For prediction or comparison of measurements, the same Eq. (15.2) is used, but now for instance with the 5% lower and upper exceedance lines.

15.2.3. *Number of overtopping waves or overtopping percentage*

Till now only the 2% run-up value has been described. It might be that one is interested in another percentage, for example, on the design of breakwaters where the crest height may be determined by an allowable percentage of overtopping waves, say, 10–15%. A few ways exist to calculate run-up heights for other percentages, or to calculate the number of overtopping waves for a given crest height. van der Meer and Stam[10] give two methods. One is an equation like Eq. (15.2) with a table of coefficients for the 0.1%, 1%, 2%, 5%, 10%, and 50% (median). Interpolation is needed for other percentages.

The second method gives a formula for the run-up distribution as a function of wave conditions, slope angle, and permeability of the structure. The distribution is a two-parameter Weibull distribution. With this method, the run-up can be calculated for every percentage required. Both methods apply to straight rock slopes only and will not be described here. The given references, however, give all the details.

The easiest way to calculate run-up (or overtopping percentage) different from 2% is to take the 2%-value and assume a Rayleigh distribution. The probability of overtopping $P_{ov} = N_{ow}/N_w$ (the percentage is simply 100 times larger) can be calculated by

$$P_{ov} = N_{ow}/N_w = \exp\left[-\left(\sqrt{-\ln 0.02}\frac{R_c}{R_{u,2\%}}\right)^2\right], \qquad (15.4)$$

where

P_{ov} = probability of overtopping [–];
N_{ow} = number of overtopping waves in a sea state [–];
N_w = number of waves in a sea state [–];
R_c = crest freeboard [m].

Equation (15.4) can be used to calculate the probability of overtopping, given a crest freeboard R_c or to calculate the required crest freeboard, given an allowable probability or percentage of overtopping waves.

One warning should be given in applying Eqs. (15.2)–(15.4). The equations give the run-up level in percentage or height on a straight (rock-armored) slope. This is not the same as the number of overtopping waves or overtopping percentage. Figure 15.5 gives the difference. The run-up is always a point on a straight slope, where for a rock-armored slope or mound the overtopping is measured some distance away from the seaward slope and on the crest, often behind a crown wall. This means that Eqs. (15.2)–(15.4) always give an overestimation of the number of overtopping waves.

Figure 15.6 shows measured data for rubble mound breakwaters armored with Tetrapods, Accropode$^{\text{TM}}$, or a single layer of cubes. All tests were performed at Delft Hydraulics.

The test setup was more or less similar to Fig. 15.4 with a crown wall height R_c a little lower than the armor freeboard A_c. CLASH-data on specific overtopping tests for various rock and concrete armored slopes were added to Fig. 15.6. This figure gives only the percentage of overtopping waves passing the crown wall.

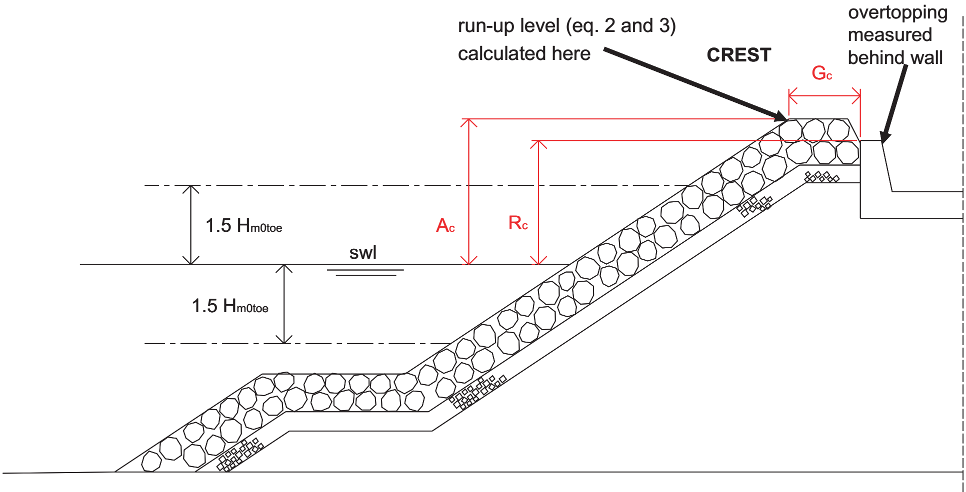

Fig. 15.5. Run-up level and location for overtopping differ.

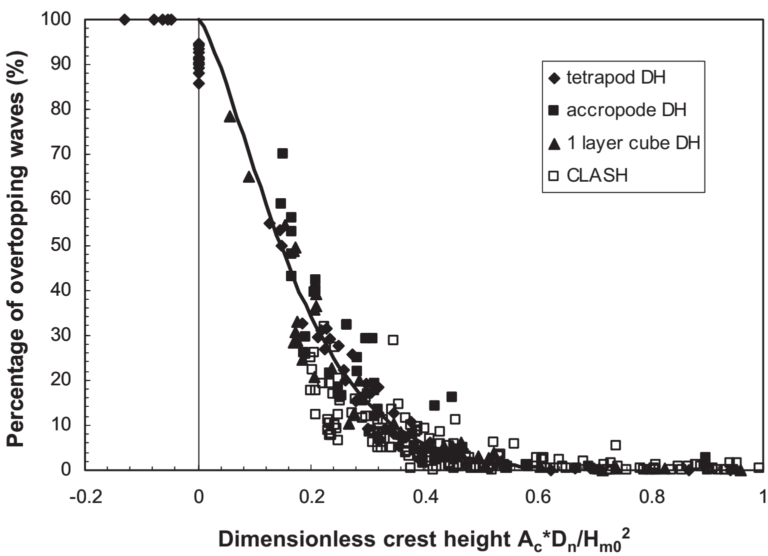

Fig. 15.6. Percentage of overtopping waves for rubble mound breakwaters as a function of relative (armor) crest height and armor size ($R_c \leq A_c$).

Analysis showed that the size of the armor unit relative to the wave height had influence, which gave a combined parameter $A_c \cdot D_n/H_{m0}^2$, where D_n is the nominal diameter of the armor unit.

The figure covers the whole range of overtopping percentages, from complete overtopping with the crest at or lower than SWL to no overtopping at all. The CLASH-data give maximum overtopping percentages of about 30%. Larger percentages mean that overtopping is so large that it can hardly be measured and that wave transmission starts to play a role.

Taking 100% overtopping for zero freeboard (the actual data are only a little lower), a Weibull curve can be fitted through the data. Equation (15.5) can be used to predict the number or percentage of overtopping waves or to establish the armor crest level for an allowable percentage of overtopping waves.

$$P_{ov} = N_{ow}/N_w = \exp\left[-\left(\frac{A_c D_n}{0.19 H_{m0}^2}\right)^{1.4}\right]. \tag{15.5}$$

It is clear that Eqs. (15.2)–(15.4) will come to more overtopping waves than Eq. (15.5). But both estimations together give a designer enough information to establish the required crest height of a structure given an allowable overtopping percentage.

15.2.4. *Wave run-down on rock-armored slopes*

When a wave on a structure has reached its highest point, it will run down on the slope till the next wave meets this water and run-up starts again. The lowest point to where the water retreats, measured vertically to SWL, is called the run-down level. Run-down often is less or not important compared to wave run-up, but both together they may give an idea of the total water excursion on the slope. Therefore, only a first estimate of run-down on straight rock slopes is given here, based on the same tests of van der Meer,[11] but re-analyzed with respect to the use of the spectral wave period $T_{m-1,0}$. Figure 15.7 gives an overall view.

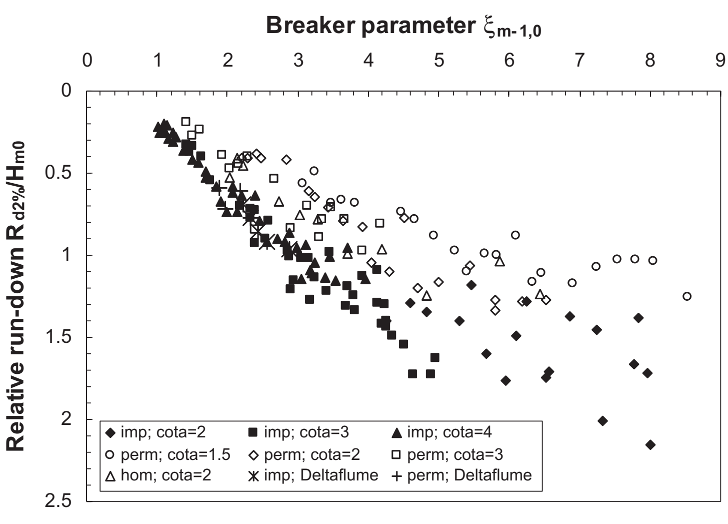

Fig. 15.7. Relative 2% run-down on straight rock slopes with impermeable core (imp), permeable core (perm), and homogeneous structure (hom).

The graph shows clearly the influence of the permeability of the structure as the solid data points (impermeable core) generally show larger run-down than the open data symbols of the permeable core. Furthermore, the breaker parameter $\xi_{m-1,0}$ gives a fairly clear trend of run-down for various slope angles and wave periods. Figure 15.7 can be used directly for design purposes, as it also gives a good idea of the scatter.

15.3. Overtopping Discharges

15.3.1. *Simple armored slopes*

The mean overtopping discharge is often used to judge allowable overtopping. It is easy to measure, and an extensive database on mean overtopping discharge has been gathered in CLASH. This mean discharge does not of course describe the real behavior of wave overtopping, where only large waves will reach the top of the structure and give overtopping. Random individual wave overtopping is random in time, and each wave gives a different overtopping volume. But the description of individual overtopping is based on the mean overtopping, as the duration of overtopping multiplied with this mean overtopping discharge gives the total volume of water overtopped by a certain number of overtopping waves. The mean overtopping discharge has been described in this section. The individual overtopping volumes is the subject in Sec. 15.4.1.

Wave overtopping occurs if the crest level of the coastal structures is lower than the highest wave run-up level. In that case, the freeboard R_c defined as the vertical difference between the SWL and the crest height becomes important (Fig. 15.2). Wave overtopping depends on the freeboard R_c and increases for decreasing freeboard height R_c. Usually, wave overtopping for rubble slopes and mounds is described by an average wave overtopping discharge q, which is given in m^3/s per m width, or in l/s per m width.

An average overtopping discharge q can only be calculated for quasi-stationary wave and water level conditions, a so-called sea state. If the amount of water overtopping for a structure during a storm is required, the average overtopping discharge has to be calculated for each more or less constant storm water level and constant wave conditions.

Many model studies were performed to investigate the average overtopping discharge for specific dike geometries or wave conditions. For practical purposes, empirical formulae were fitted through experimental model data which obey often one of the following expressions:

$$Q^* = Q_0 \left(1 - R^*\right)^b \quad \text{or} \quad Q^* = Q_0 \exp\left(-b \cdot R^*\right) . \tag{15.6}$$

Q^* is a dimensionless overtopping discharge, R_* is a dimensionless freeboard height, Q_0 describes wave overtopping for zero freeboard, and b is a coefficient which describes the specific behavior of wave overtopping for a certain structure. Schüttrumpf[8] summarized expressions for the dimensionless overtopping discharge Q^* and the dimensionless freeboard height R^*.

The dimensionless overtopping discharge $Q^* = q/\sqrt{gH_{m0}^3}$ is a function of the wave height, originally derived from the Weir formula.

Deterministic design or safety assessment. The equation, including a standard deviation of safety, should be used for deterministic design or safety assessment:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(-4.3\frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \tag{15.7}$$

with a maximum of $\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp\left(-2.3\frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right)$,
where γ_v = the influence of a small wall on top of the embankment.

Probabilistic design. The mean prediction should be used for probabilistic design, or prediction of or comparison with measurements. This equation is given by

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(-4.75\frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \tag{15.8}$$

with a maximum of $\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp\left(-2.6\frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right)$.

The reliability of Eq. (15.8) is described by taking the coefficients 4.75 and 2.6 as normally distributed stochastic parameters with means of 4.75 and 2.6 and standard deviations $\sigma = 0.5$ and 0.35, respectively. For probabilistic calculations, Eq. (15.8) should be taken together with these stochastic coefficients. For predictions of measurements or comparison with measurements also Eq. (15.8) should be taken with, for instance, 5% upper and lower exceedance curves.

It has to be mentioned that the first part of Eqs. (15.7) and (15.8) is valid for mostly breaking waves. Considering the steep slopes of armored rubble slope and mounds this part has less importance in practice than the second part of the equation, describing the maximum of overtopping. In that case, the relative freeboard does not depend on the breaker parameter $\xi_{m-1,0}$ for nonbreaking waves (Fig. 15.8), as the line is horizontal.

This means that a composite slope and even a, not too long, berm leads to the same overtopping discharge as for a simple straight rubble mound slope. Only when the average slope becomes so gentle that the maximum part in Eqs. (15.7) and (15.8) do not apply anymore, then a berm and a composite slope will have effect on the overtopping discharge. Generally, average slopes around 1:2 or steeper do not show influence of the slope angle, or only to a limited extent, and the maximum part in Eqs. (15.7) and (15.8) are valid.

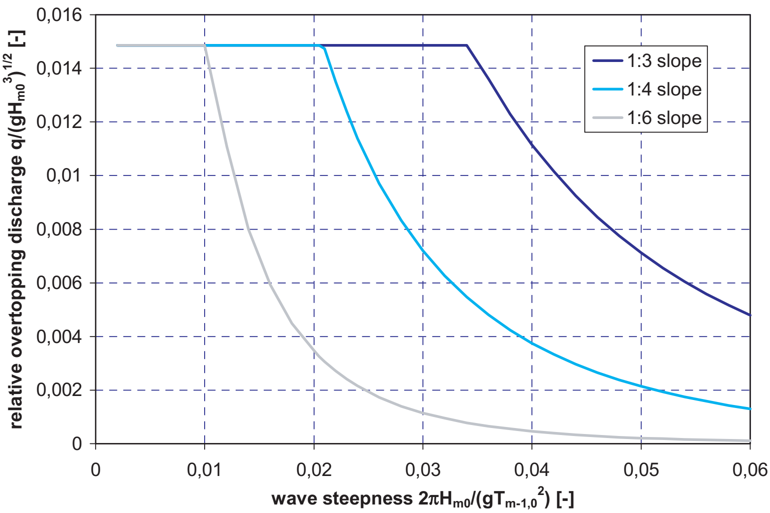

Fig. 15.8. Wave overtopping as a function of the fictitious wave steepness $s_{m-1,0} = 2\pi H_{m0}/(gT_{m-1,0}{}^2)$ and a smooth slope.

As part of the EU research program CLASH[2] tests were undertaken to derive roughness factors for rock-armored slopes and different armor units on sloping permeable structures. Overtopping was measured for a 1:1.5 sloping permeable structure at a reference point $3D_{n50}$ from the crest edge, where D_{n50} is the nominal diameter. The wave wall had the same height as the armor crest, so $R_c = A_c$. As discussed in Sec. 15.2 and Fig. 15.5, the point to where run-up can be measured and the location of overtopping may differ. Normally, a rubble mound structure has a crest width of at least $3D_{n50}$. Waves rushing up the slope reach the crest with an upward velocity. For this reason, it is assumed that overtopping waves reaching the crest will also reach the location $3D_{n50}$ further.

Results of the CLASH-work are shown in Fig. 15.9 and Table 15.1. Figure 15.9 gives all data together in one graph. Two lines are given, one for a smooth slope, Eq. (15.8) with $\gamma_f = 1.0$, and one for rubble mound 1:1.5 slopes, with the same equation, but with $\gamma_f = 0.45$. The lower line only gives a kind of average, but shows clearly the very large influence of roughness and permeability on wave overtopping. The required crest height for a steep rubble mound structure is at least half of that for a steep smooth structure, for similar overtopping discharge. It is also for this reason that smooth slopes are often more gentle in order to reduce the crest heights.

In Fig. 15.9, one-layer systems, like Accropode™, CORE-LOC®, Xbloc®, and one layer of cubes, have solid symbols. Two-layer systems have been given by open symbols. There is a slight tendency that one-layer systems give a little more overtopping than two-layer systems, which is also clear from Table 15.1. Equation (15.8) can be used with the roughness factors in Table 15.1 for the prediction of mean overtopping discharges for rubble mound breakwaters. Values in italics in Table 15.1 have been estimated/extrapolated, based on the CLASH results.

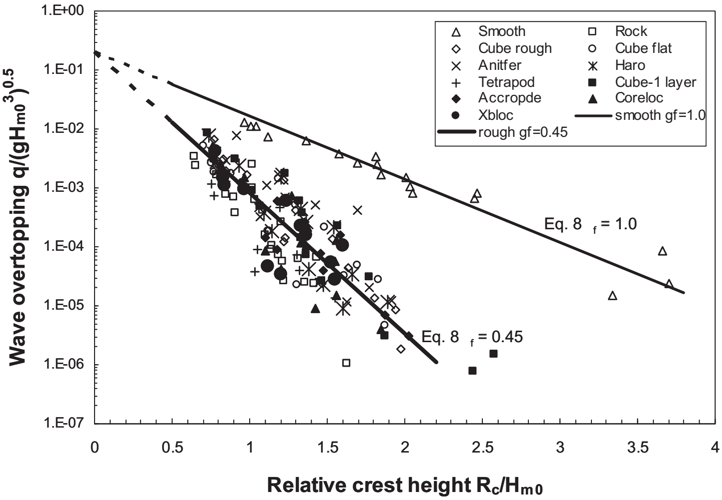

Fig. 15.9. Mean overtopping discharge for 1:1.5 smooth and rubble mound slopes.

Table 15.1. Values for roughness factors γ_f for permeable rubble mound structures with slope of 1:1.5. Values in italics are estimated/extrapolated.

Type of armor layer	γ_f
Smooth impermeable surface	1.00
Rocks (one layer, impermeable core)	0.60
Rocks (one layer, permeable core)	0.45
Rocks (two layers, impermeable core)	0.55
Rocks (two layers, permeable core)	0.40
Cubes (one layer, random positioning)	0.50
Cubes (two layers, random positioning)	0.47
Antifers	0.47
HARO's	0.47
Accropode™	0.46
Xbloc®	0.45
CORE-LOC®	0.44
Tetrapods	0.38
Dolosse	*0.43*

15.3.2. *Effect of armored crest berm*

Wave overtopping on simple straight slope include an armored crest berm up to about three nominal diameters. It is clear, however, that wide crests will certainly decrease the overtopping as much more energy will be dissipated in a wider crest. The crest width can be described by G_c (see Fig. 15.5). The *EA-Manual* (Besley[1]) describes in a simple and effective way the influence of a wide crest. First, the wave overtopping discharge should be calculated for a simple slope, with a crest width up to $3D_{n50}$. Then, the following reduction factor on the overtopping discharge can be applied:

$$C_r = 3.06 \exp(-1.5G_c/H_{m0}), \quad \text{with maximum } C_r = 1. \tag{15.9}$$

Equation (15.9) gives no reduction for a crest width smaller than about 0.75 H_{m0}. This is fairly close to about $3D_{n50}$ and is, therefore, consistent. A crest width of $1H_{m0}$ reduces the overtopping discharge to 68%, a crest width of $2H_{m0}$ gives a reduction to 15%, and for a wide crest of $3H_{m0}$, the overtopping reduces to only 3.4%. In all cases, the crest wall has the same height as the armor crest: $R_c = A_c$.

Equation (15.9) was determined for a rock-armored slope and can be considered as conservative, as for a slope with Accropode, more reduction was found.

15.3.3. *Effect of oblique waves*

In the CLASH-project, specific tests on a rubble mound breakwater were performed with a slope of 1:2 and armored with rock or cubes[7] to investigate the effect of oblique waves on wave overtopping. The structure was tested both with long-crested and short-crested waves, but only the results by short-crested waves are given. Results for the effect of oblique waves on smooth slopes, dikes, or embankments are given in the *EurOtop* Overtopping Manual,[4] and in the TAW-report.[9] Here, only the results for armored rubble mound slopes will be discussed.

For oblique waves, the angle of wave attack β (deg.) is defined as the angle between the direction of propagation of waves and the axis perpendicular to the structure (for perpendicular wave attack, $\beta = 0°$). And the direction of wave attack is the angle after any change of direction of the waves on the foreshore due to refraction. Just like for smooth slopes, the influence of the angle of wave attack is described by the influence factor γ_β. Just as for smooth slopes, there is a linear relationship between the influence factor and the angle of wave attack, but the reduction in overtopping for rock slopes is faster with increasing angle:

$$\gamma_\beta = 1 - 0.0063|\beta| \qquad \begin{array}{l} \text{for } 0° \leq |\beta| \leq 80° \\ \text{for } |\beta| > 80°, \text{ the result of } \beta = 80° \text{can be applied.} \end{array} \tag{15.10}$$

The wave height and period are linearly reduced to 0 for $80° \leq |\beta| \leq 110°$. For $|\beta| > 110°$, the wave overtopping is set to $q = 0$.

15.3.4. *Composite slopes and berms, including berm breakwaters*

In every formula where a cot α or breaker parameter $\xi_{m-1,0}$ is present, a procedure has to be described how a composite slope has to be taken into account. Hardly any specific research exists for rubble mound structures, and, therefore, the procedure for composite slopes at sloping impermeable structures like dikes and sloping seawalls is assumed to be applicable.

(a) Average slopes. A characteristic slope is required to be used in the breaker parameter $\xi_{m-1,0}$ for composite profiles or bermed profiles to calculate wave run-up or wave overtopping. Theoretically, the run-up process is influenced by a change of slope from the breaking point to the maximum wave run-up height. Therefore, often it has been recommended to calculate the characteristic slope from the point of wave breaking to the maximum wave run-up height. This approach needs some calculation effort, because of the iterative solution since the wave run-up height $R_{u2\%}$ is unknown. For the breaking limit, a point on the slope can be chosen which is $1.5H_{m0}$ below the still water line.

It is recommended to use also a point on the slope $1.5H_{m0}$ above water as a first estimate to calculate the characteristic slope and to exclude a berm (Fig. 15.10).

$$\text{First estimate: } \tan\alpha = \frac{3 \cdot H_{m0}}{L_{\text{Slope}} - B}\,. \tag{15.11}$$

As a second estimate, the wave run-up height from the first estimate is used to calculate the average slope [L_{Slope} has to be adapted (see Fig. 15.11)]:

$$\text{Second estimate: } \tan\alpha = \frac{\left(1.5 \cdot H_{m0} + R_{u2\%(\text{from 1st estimate})}\right)}{L_{\text{Slope}} - B}\,. \tag{15.12}$$

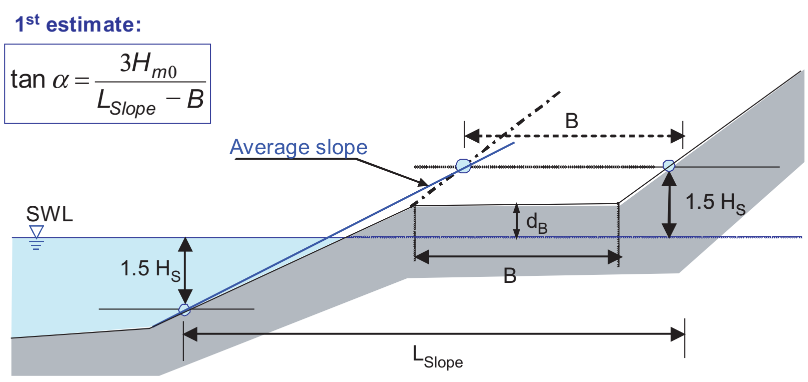

Fig. 15.10. Determination of the average slope (first estimate).

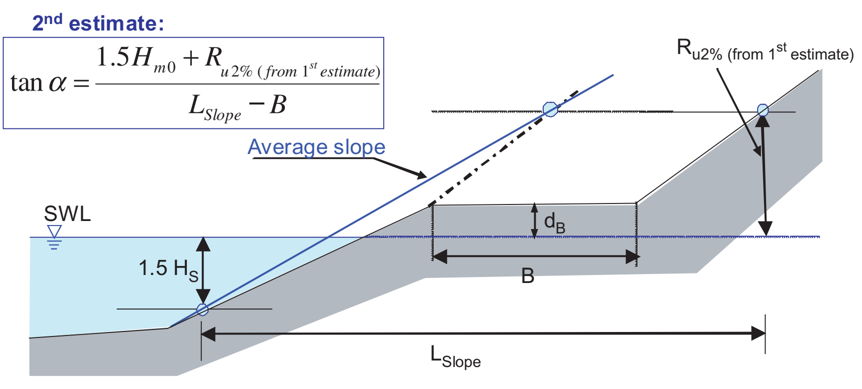

Fig. 15.11. Determination of the average slope (second estimate).

If the run-up height or $1.5H_{m0}$ comes above the crest level, then the crest level must be taken as the characteristic point above SWL.

Also, the influence of a berm in a sloping profile has been adapted from smooth sloping sea dikes for rubble mound structures. There is, however, often a difference in the effect of composite slopes or berms for rubble mound and smooth gentle slopes. On gentle slopes, the breaker parameter $\xi_{m-1,0}$ has large influence on wave overtopping (see Eqs. (15.2) and (15.3) as the breaker parameter will be quite small). Rubble mound structures often have a steep slope, leading to the formula for "nonbreaking" waves, the maximum in Eqs. (15.7) and (15.8). In these equations, no slope angle or breaker parameter is present, and the effect of a small berm will be very small and probably negligible.

(b) Influence of berms. A berm is a part of a dike or breakwater profile in which the slope varies between horizontal and 1:15. A berm is defined by the width of the berm, B, and by the vertical difference d_B between the middle of the berm and the SWL (Fig. 15.12). The width of the berm, B, may not be greater than $0.25 \cdot L_{m-1,0}$. If the berm is horizontal, the berm width B is calculated according to Fig. 15.12. The lower and the upper slope are extended to draw a horizontal berm without changing the berm height d_B. The horizontal berm width is therefore shorter than the angled berm width. d_B is 0 if the berm lies on the still water line. The characteristic parameters of a berm are defined in Fig. 15.12.

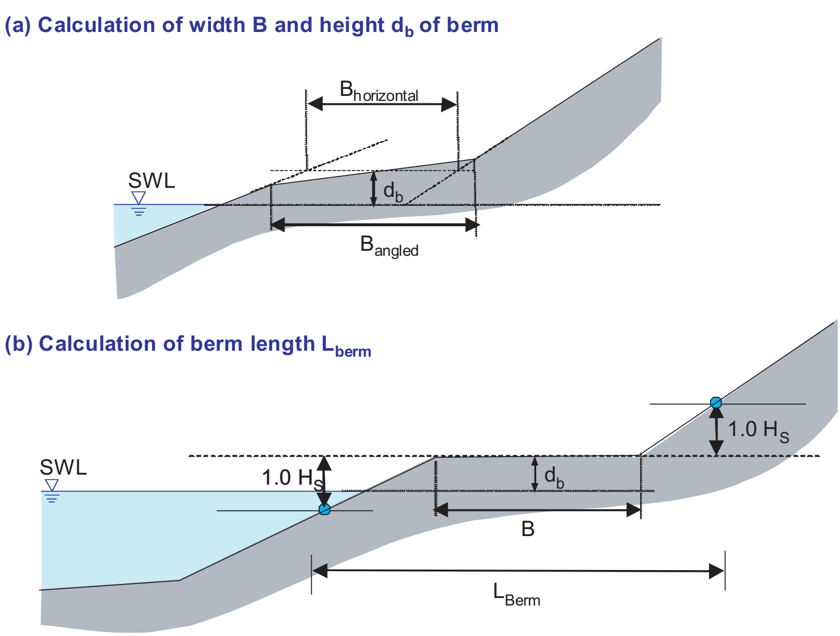

Fig. 15.12. Determination of the characteristic berm length L_{Berm}.

A berm reduces wave run-up or wave overtopping. The influence factor γ_b for a berm consists of two parts.

$$\gamma_b = 1 - r_B\,(1 - r_{db}) \qquad \text{for } 0.6 \le \gamma_b \le 1.0. \tag{15.13}$$

The first part (r_B) stands for the width of the berm L_{Berm} and becomes 0 if no berm is present.

$$r_B = \frac{B}{L_{\text{Berm}}}\,. \tag{15.14}$$

The second part (r_{db}) stands for the vertical difference d_B between the SWL and the middle of the berm and becomes 0 if the berm lies on the still water line. The reduction of wave run-up or wave overtopping is maximum for a berm on the still water line and decreases with increasing d_B. Thus, a berm lying on the still water line is most effective. A berm lying below $2 \cdot H_{m0}$ or above $R_{u2\%}$ has no influence on wave run-up and wave overtopping.

Different expressions are used for r_{dB} in Europe. Here, an expression using a cosine-function for r_{db} (Fig. 15.13) is recommended which is also used in PC-OVERTOPPING (see Chap. 14).

$$\begin{aligned} r_{db} &= 0.5 - 0.5\cos\left(\pi\frac{d_b}{R_{u2\%}}\right) \quad \text{for a berm above SWL}\,,\\ r_{db} &= 0.5 - 0.5\cos\left(\pi\frac{d_b}{2 \cdot H_{m0}}\right) \quad \text{for a berm below SWL}\,,\\ r_{db} &= 1 \text{ for berms lying outside the area of influence}\,. \end{aligned} \tag{15.15}$$

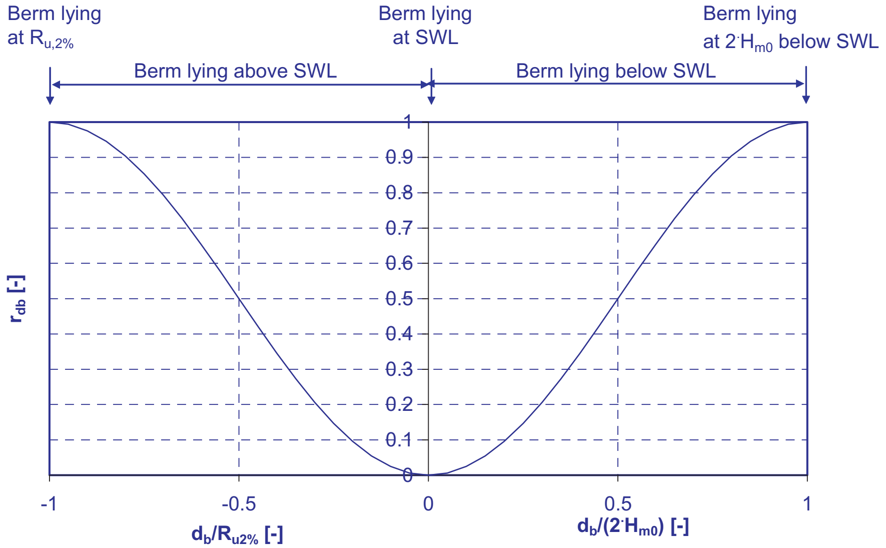

Fig. 15.13. Influence of the berm depth on factor r_{dh}.

The maximum influence of a berm is actually always limited to $\gamma_B = 0.6$. This corresponds to an optimal berm width B on the SWL of $B = 0.4 \cdot L_{\mathrm{Berm}}$.

The definition of a berm is made for a slope smoother than 1:15, while the definition of a slope is made for slopes steeper than 1:8. If a slope or a part of the slope lies in between 1:8 and 1:15, it is required to interpolate between a bermed profile and a straight profile. For wave run-up, this interpolation is written by

$$R_{u2\%} = R_{u2\%(1:8\mathrm{slope})} + \left(R_{u2\%(\mathrm{Berm})} - R_{u2\%(1:8\mathrm{slope})}\right) \cdot \frac{(1/8 - \tan\alpha)}{(1/8 - 1/15)}. \quad (15.16)$$

A similar interpolation procedure should be followed for wave overtopping.

15.3.5. *Wave overtopping on a berm breakwater*

A specific type of rubble mound structure is the berm breakwater. The original idea behind the berm breakwater is that a large berm, consisting of fairly large rock, is constructed into the sea with a steep seaward face. The berm height is higher than the minimum required for construction with land-based equipment. Due to the steep seaward face the first storms will reshape the berm and finally a structure will be present with a fully reshaped S-profile. Such a profile has then a gentle 1:4 or 1:5 slope just below the water level and steep upper and lower slopes (see Fig. 15.14).

The idea of the reshaping berm breakwater has evolved in Iceland to a more or less nonreshaping berm breakwater (Fig. 15.15). The main difference is that during rock production from the quarry, care is taken to gather a few percent of really big rocks. Only a few percent is required to strengthen the corner of the berm and part of the down slope and upper layer of the berm in such a way that reshaping will hardly occur. An example with various rock classes (class I being the largest) is given in Fig. 15.16. Therefore, distinction has been made between conventional reshaping berm breakwaters and the nonreshaping Icelandic type berm breakwater.

In order to calculate wave overtopping on reshaped berm breakwaters, the reshaped profile should be known. The basic method of profile reshaping is given in van der Meer,[11] and the program BREAKWAT (WL | Delft Hydraulics) is able to calculate the profile. The first method described here to calculate wave overtopping at reshaping berm breakwaters is by using Eqs. (15.7) or (15.8) which have been developed for smooth slopes. Equations (15.7) and (15.8) include the effect of an average slope with the roughness factors given in Table 15.1 of $\gamma_f = 0.40$ for

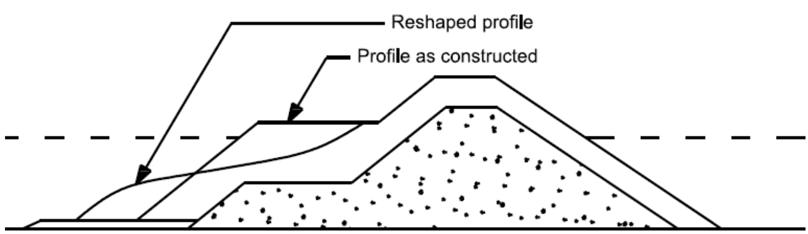

Fig. 15.14. Conventional reshaping berm breakwater.

Fig. 15.15. Icelandic berm breakwater.

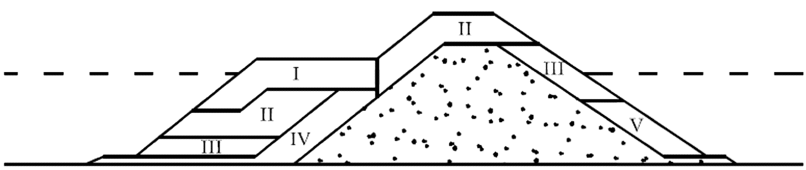

Fig. 15.16. Nonreshaping Icelandic berm breakwater with various classes of big rocks.

reshaping berm breakwaters and $\gamma_f = 0.35$ for nonreshaping Icelandic berm breakwaters. The method of composite slopes and berms should be applied as described above.

The second method is to use the CLASH neural network. As overtopping research at that time on berm breakwaters was limited, this method also gives quite some scatter, but a little less than the first method described above.

Recent information on berm breakwaters has been described by Andersen.[5] Only part of his research was included in the CLASH database and consequently in the Neural Network prediction method. He performed about 600 tests on reshaping berm breakwaters and some 60 on nonreshaping berm breakwaters (fixing the steep slopes by a steel net). The true nonreshaping Icelandic type of berm breakwaters with large rock classes, has not been tested and, therefore, his results might lead to an overestimation.

One comment should be made on the application of the results of Andersen.[5] The maximum overtopping discharge measured was only $q/(gH_{m0}^3)^{0.5} = 10^{-3}$. In practical situations with wave heights around 5 m, the overtopping discharge will then be limited to only a few l/s per m width. For berm breakwaters and also for conventional rubble slopes and mounds, allowable overtopping may be much higher than this value.

The final result of the work of Andersen[5] is a quite complicated formula, based on multi-parameter fitting. The advantage of such a fitting is that by using a large number of parameters, the data set used will be quite well described by the formula. The disadvantage is that physical understanding of the working of the formula, certainly outside the ranges tested, is limited. But, due to the fact that so many structures were tested, this effect may be neglected.

The formula is valid for berm breakwaters with no superstructure and gives the overtopping discharge at the back of the crest ($A_c = R_c$). In order to overcome the problem that one encounters when calculating the reshaped profile before any overtopping calculation can be done, the formula is based on the "as built" profile, before reshaping. Instead of calculating the profile, a part of the formula predicts the influence of waves on recession of the berm. The parameter used is called f_{Ho}, which is an indicative measure of the reshaping, and can be defined as a "factor accounting for the influence of stability numbers." Note that f_{Ho} is a dimensionless factor and not the direct measure of recession and that H_o and T_o are also dimensionless parameters (see below).

$$\begin{aligned} f_{Ho} &= 19.8 s_{om}^{-0.5} \exp(-7.08/H_o) \quad \text{for } T_o \geq T_o^* \,, \\ f_{Ho} &= 0.05 H_o T_o + 10.5 \quad \text{for } T_o < T_o^* \,, \end{aligned} \tag{15.17}$$

where

$$H_o = H_{m0}/\Delta D_{n50}, T_o = (g/D_{n50})^{0.5} T_{m0,1},$$

and

$$T_o^* = \{19.8 s_{om}^{-0.5} \ \exp(-7.08/H_o) - 10.5\}/(0.05 H_o).$$

The berm level d_h is also taken into account as an influence factor, d_h^*. Note that the berm depth is positive if the berm level is below SWL, and therefore, for berm breakwaters often negative. Note also that this influence factor is different from that for a bermed slope. This influence factor is described by

$$\begin{aligned} d_h^* &= (3H_{m0} - d_h)/(3H_{m0} + R_c) \quad \text{for } d_h < 3H_{m0} \,, \\ d_h^* &= 0 \quad \text{for } d_h \geq 3H_{m0} \,. \end{aligned} \tag{15.18}$$

The final overtopping formula then takes into account the influence factor on recession, f_{Ho}, the influence factor of the berm level, d_h^*, the geometrical parameters R_c, B, and G_c, and the wave conditions H_{m0} and the mean period $T_{m0,1}$. It means that the wave overtopping is described by a spectral mean period, and not by $T_{m-1,0}$.

$$\begin{aligned} & q/(gH_{m0}^3)^{0.5} = 1.79 \cdot 10^{-5} \ (f_{Ho}^{1.34} + 9.22) s_{op}^{-2.52}{}_*, \\ & \exp[-5.63(R_c/H_{m0})^{0.92} - 0.61(G_c/H_{m0})^{1.39} - 0.55 h_{b*}^{1.48} (B/H_{m0})^{1.39} \,. \end{aligned} \tag{15.19}$$

Equation (15.19) is only valid for a down slope of 1:1.25 and an upper slope of 1:1.25. For other slopes, one has to reshape the slope to a slope of 1:1.25, keeping the volume of material the same and adjusting the berm width B and for the upper slope also the crest width G_c. Note also that in Eq. (15.19), the peak wave period

T_p has to be used to calculate s_{op}, where the mean period $\mathrm{T}_{m0,1}$ has to be used in Eq. (15.17).

Although no tests were performed on the nonreshaping Icelandic berm breakwaters (see Fig. 15.16), a number of tests were performed on nonreshaping structures by keeping the material in place with a steel net. The difference may be that Icelandic berm breakwaters show a little less overtopping, due to the presence of larger rocks and, therefore, more permeability. The tests showed that Eq. (15.19) is also valid for nonreshaping berm breakwaters, if the reshaping factor $f_{Ho} = 0$.

15.3.6. *Effect of wave walls*

Most breakwaters have a wave wall, capping wall, or crest unit on the crest, simply to end the armor layer in a good way and to create access to the breakwater. For design, it is advised not to design a wave wall much higher than the armor crest, for the simple reason that wave forces on the wall will increase drastically if directly attacked by waves and not hidden behind the armor crest. For rubble mound slopes as a seawall, design waves might be a little lower than for breakwaters and a wave wall might be one of the solutions to reduce wave overtopping. Nevertheless, one should realize the increase in wave forces in designing a wave wall significantly above the armor crest.

Equations (15.7) and (15.8) for a simple rubble mound slope include a berm of $3D_{n50}$ wide and a wave wall at the same level as the armor crest: $A_c = R_c$. A little lower wave wall will hardly give larger overtopping, but no wave wall at all would certainly increase overtopping. Part of the overtopping waves will then penetrate through the crest armor. No formulae are present to cope with such a situation, unless the use of the Neural Network prediction method.

Various researchers have investigated wave walls higher than the armor crest. None of them compared their results with a graph like Fig. 15.9 for simple rubble mound slopes. During the writing of the *EurOtop* Overtopping Manual, 2007, some of the published equations were plotted in Fig. 15.7 and most curves fell within the scatter of the data. Data with a wider crest gave significantly lower overtopping, but that was due to the wider crest, not the higher wave wall. In essence, the message is: use the height of the wave wall R_c and not the height of the armored crest A_c in Eqs. (15.7) and (15.8) if the wall is higher than the crest. For a wave wall lower than the crest armor the height of this crest armor should be used. The Neural Network prediction might be able to give more precise predictions.

15.3.7. *Scale and model effect corrections*

Results of the recent CLASH-project suggested significant differences between field and model results on wave overtopping. This has been verified for different sloping rubble structures. Results of the comparisons in this project have led to a scaling procedure which is mainly dependent on the roughness of the structure γ_f [–]; the seaward slope m of the structure [–]; the mean overtopping discharge, upscaled to prototype, q_{ss} [m^3/s/m]; and whether wind is considered or not.

Data from the field are naturally scarce, and hence the method can only be regarded as tentative. Furthermore, it is only relevant if mean overtopping rates are

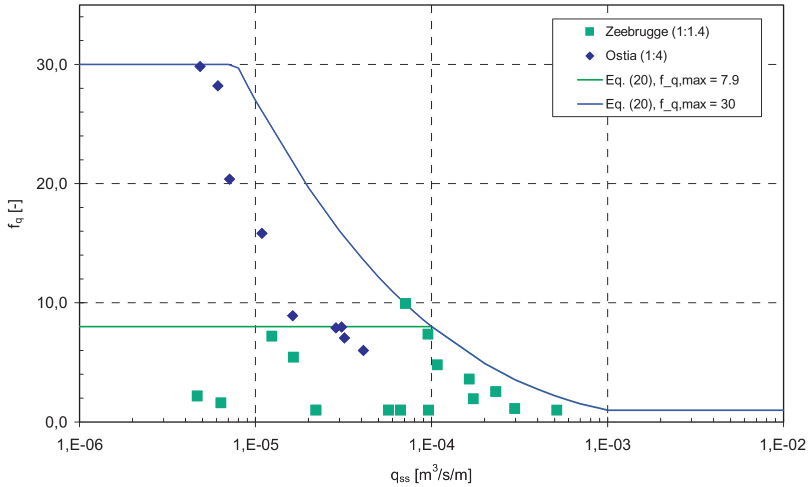

Fig. 15.17. Proposed adjustment factor applied to data from two field sites (Zeebrugge 1:1.4 rubble mound breakwater, and Ostia 1:4 rubble slope).

lower than 1.0 l/s/m but may include significant adjustment factors below these rates. Due to the inherent uncertainties, the proposed approach tries to be conservative. It has, however, been applied to pilot cases in CLASH and has proved good corrections with these model data.

The adjustment factor f_q for the model and scale effects can be determined as follows (Fig. 15.17):

$$fq = \begin{cases} 1.0 & \text{for } q_{ss} \geq 10^{-3}\text{m}^3/\text{s/m}\,, \\ \min\{(-\log\ q_{ss} - 2)^3; f_{q,\max}\} & \text{for } q_{ss} \geq 10^{-3}\text{m}^3/\text{s/m}\,, \end{cases} \tag{15.20}$$

where $f_{q,\max}$ is an upper bound to the adjustment factor f_q and can be calculated as follows:

$$f_{q,\max} = \begin{cases} f_{q,r} & \text{for } \gamma_f \leq 0.7\,, \\ 5 \cdot \gamma_f \cdot (1 - f_{q,r}) + 4.5 \cdot (f_{q,r} - 1) + 1 & \text{for } 0.7 < \gamma_f \leq 0.9\,, \\ 1.0 & \text{for } \gamma_f > 0.9\,. \end{cases} \tag{15.21}$$

In Eq. (15.21), $f_{q,r}$ is the adjustment factor for rough slopes which is mainly dependent on the slope of the structure and whether wind needs to be included or not:

$$f_{q,r} = \begin{cases} 1.0 & \text{for } m \leq 0.6\,, \\ f_w \cdot (8.5 \cdot m - 4.0) & \text{for } 0.6 < m \leq 4.0\,, \\ f_w \cdot 30 & \text{for } m > 4.0\,, \end{cases} \tag{15.22}$$

in which $m = \cot\alpha$ (slope of structure); f_w accounts for the presence of wind and is set to $f_w = 1.0$ if there is wind and $f_w = 0.67$ if there is no wind.

This set of equations include the case of smooth dikes which will, due to $\gamma_f = 0.9$ in this case, always lead to an adjustment factor of $f_q = 1.0$. In case of a very rough

1:4 slope with wind $f_{q,\max} = f_{qr} = 30.0$, which is the maximum, the factor can get to (but only if the mean overtopping rates get below $q_{ss} = 10^{-5}$ m^3/s per m). The latter case and a steep rough slope is illustrated in Fig. 15.14.

15.4. Individual Overtopping Waves

15.4.1. *Overtopping volumes per wave*

The following section is a summary of Sec. 14.2.2 in Chap. 14 of this handbook. Parts of that section are repeated in the following with a focus on rubble mound structures.

Wave overtopping is a dynamic and irregular process and the mean overtopping discharge, q, does not cover this aspect. But by knowing the storm duration, t, and the number of overtopping waves in that period, N_{ow}, it is easy to describe this irregular and dynamic overtopping, if the overtopping discharge, q, is known. Each overtopping wave gives a certain overtopping volume of water, V, with dimension m^3 per m width or l per m width.

As many equations in this chapter, the two-parameter Weibull distribution describes the behavior quite well. This equation has a shape parameter, b, and a scale parameter, a. The shape parameter gives a lot of information on the type of distribution.

The exceedance probability, P_V, of an overtopping volume per wave is similar to Eqs. (15.23) and (15.24).

$$P_V = P\left(\underline{V} \leq V\right) = 1 - \exp\left[-\left(\frac{V}{a}\right)^{0.75}\right], \qquad (15.23)$$

with

$$a = 0.84 \cdot T_m \cdot \frac{q}{P_{ov}} = 0.84 \cdot T_m \cdot q \cdot N_w/N_{ow} = 0.84 \cdot q \cdot t/N_{ow}\,. \qquad (15.24)$$

Equation (15.24) shows that the scale parameter depends on the overtopping discharge, but also on the mean period (not the spectral period $T_{m-1,0}$!) and probability of overtopping, or which is similar, on the storm duration and the actual number of overtopping waves.

The probability of wave overtopping for rubble mound structures has been described in Sec. 15.2 and Eq. (15.4).

Equations for calculating the overtopping volume per wave for a given probability of exceedance is given by Eq. (15.25):

$$V = a \cdot \left[-\ln\left(1 - P_V\right)\right]^{4/3}\,. \qquad (15.25)$$

The maximum overtopping during a certain event is fairly uncertain, as most maxima, but depends on the duration of the event. In a 6-h period, one may expect a larger maximum than only during 15 min. The maximum during an event can be calculated by Eq. (15.26):

$$V_{\max} = a\ \left[\ln\left(N_{ow}\right)\right]^{4/3}\,. \qquad (15.26)$$

15.4.2. *Overtopping velocities and spatial distribution*

The hydraulic behavior of waves on rubble mound slopes and on smooth slopes like dikes, is generally based on similar formulae, as clearly shown in this chapter. This is different, however, for overtopping velocities and spatial distribution of the overtopping water. A dike or sloping impermeable seawall generally has an impermeable and more or less horizontal crest. Up-rushing and overtopping waves flow over the crest, and each overtopping wave can be described by a maximum velocity and flow depth. These velocities and flow depths form the description of the hydraulic loads on crest and inner slope and are part of the failure mechanism "failure or erosion of inner slopes by wave overtopping."

This is different for rubble mound slopes or breakwaters where wave energy is dissipated in the rough and permeable crest and where often overtopping water falls over a crest wall onto a crest road or even on the rear slope of a breakwater. A lot of overtopping water travels over the crest and through the air before it hits something else.

Only recently in CLASH and a few other projects at Aalborg University, attention has been paid to the spatial distribution of overtopping water at breakwaters with a crest wall.[6] The spatial distribution was measured by various trays behind the crest wall. Figure 15.18 gives different cross sections with a setup of three arrays.

Up to six arrays have been used. The spatial distribution depends on the level with respect to the rear side of the crest wall and the distance from this rear wall (Fig. 15.19). The coordinate system (x, y) starts at the rear side and at the top of the crest wall, with the positive y-axis downward.

The exceedance probability F of the travel distance is defined as the volume of overtopping water passing a given x- and y-coordinate, divided by the total overtopping volume. The probability, therefore, lies between 0 and 1, with 1 at the crest wall. The spatial distribution can be described with the following equations, which have slightly been rewritten and modified with respect to the original formulae by Andersen and Burcharth.[6] The probability F at a certain location can be

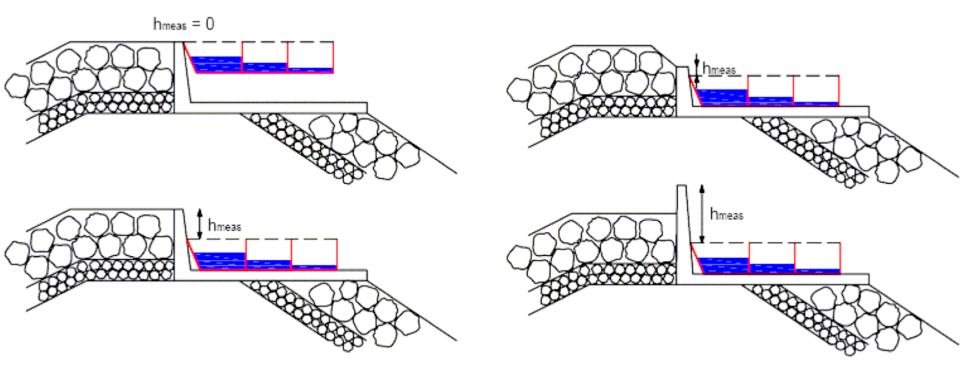

Fig. 15.18. Definition of y for various cross sections.

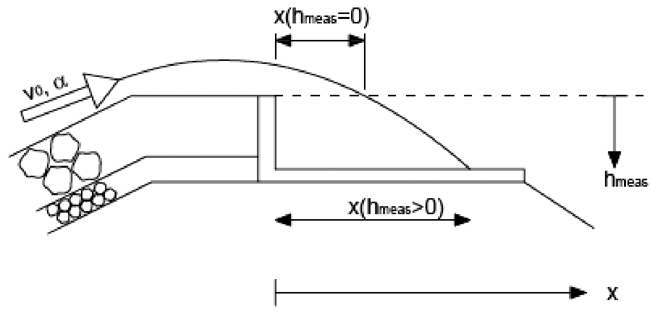

Fig. 15.19. Definition of x- and y-coordinate for spatial distribution.

described by

$$F(x,y) = \exp[-1.3/H_{m0} \cdot \{\max(x/\cos\beta - 2.7ys_{op}^{0.15}; 0)\}]\,. \tag{15.27}$$

Equation (15.27) can be rewritten to calculate directly the travel distance x (at a certain level y) by rewriting the above equation:

$$x/\cos\beta = -0.77H_{m0}\ln(F) + 2.7ys_{op}^{0.15}. \tag{15.28}$$

Suppose $\cos\beta = 0$, then we get $F = 1$ with $x = 0$; $F = 0.1$ with $x = 1.77H_s$; and $F = 0.01$ with $x = 3.55H_s$.

It means that 10% of the volume of water travels almost two wave heights through the air and 1% of the volume travels more than 3.5 times the wave height. These percentages will be higher if $y \neq 0$, which is often the case with a crest unit.

The validity of Eqs. (15.27) and (15.28) is for rubble mound slopes of approximately 1:2 and for angles of wave attack between $0° \leq |\beta| < 45°$. It should be noted that the equation is valid for the spatial distribution of the water through the air behind the crest wall. All water falling on the basement of the crest unit will, of course, travel on and will fall into the water behind and/or on the slope behind.

15.5. Overtopping of Shingle Beaches

Shingle beaches differ from the armored slopes principally in the size of the beach material, and hence its mobility. The typical stone size is sufficiently small to permit significant changes of beach profile, even under relatively low levels of wave attack. A shingle beach may be expected to adjust its profile to the incident wave conditions, provided that sufficient beach material is available. Run-up or overtopping levels on a shingle beach are therefore calculated without reference to any initial slope.

The equilibrium profile of shingle beaches under (temporary constant) wave conditions is described by van der Meer.[11] The most important profile parameter for run-up and overtopping is the crest height above SWL, h_c. For shingle with $D_{n50} < 0.1\,\text{m}$, this crest height is only a function of the wave height and wave steepness. Note that the mean wave period is used, not the spectral wave period $T_{m-1,0}$.

$$h_c/H_{m0} = 0.3 s_{om}^{-0.5} . \tag{15.29}$$

Only the highest waves will overtop the beach crest and most of this water will percolate through the material behind the beach crest. Equation (15.29) gives a run-up or overtopping level which is more or less close to $Ru_{2\%}$.

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