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Armoured rubble slopesa and mounds

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6. Armoured rubble slopes and mounds

6.1 Introduction

This manual describes three types of flood defences or coastal structures:

- coastal dikes and embankment seawalls,
- armoured rubble slopes and structures,
- and vertical and steep seawalls.

Sometimes there will be combinations and it will be difficult to place them only in one category. For example, a vertical wall or sloping embankment with a large rock berm in front. Armoured rubble slopes and mounds () are characterized by a mound with some porosity or permeability, covered by a sloping porous armour layer consisting of large rock or concrete units. In contrast to dikes and embankment seawalls the porosity of the structure and armour 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 rubble mound structures are to some extent similar to dikes and embankment seawalls, the basic wave run-up and overtopping formulae are taken from Chapter 5. They will then be modified, if necessary, to fit for rubble mound structures. Also for most definitions the reader is referred to Chapter 5 (or Chapter 1.4). More in particular:

- the definition of wave run-up (Fig. 5.3)
- the general wave run-up formula (Equation 5.1)
- the general wave overtopping formula (Equation 5.8 or 5.9)
- the influence factors γ_b, γ_f and γ_β
- the spectral wave period $T_{m-1,0}$
- the difference in deterministic and probabilistic approach

The main calculation procedure for armoured rubble slopes and mounds is given in Table 6.1.

Table 6.1: Main calculation procedure for armoured rubble slopes and mounds

	Deterministic design	Probabilistic design
Wave run-up height (2%)	Eq. 6.2	Eq. 6.1
Wave runu-up height for shingle beaches		Eq. 6.20
Mean wave overtopping discharge	Eq. 6.5	Eq. 6.6
Mean overtopping discharge for berm breakwaters		Eq. 6.9 – 6.11
Percentage of overtopping waves		Eq. 6.4
Individual overtopping volumes	Eqs. 6.15-6.16	Eqs. 6.15-6.16
Effect of armour roughness	Table 6.2	Table 6.2
Effect of armour crest berm	Eq. 6.7	Eq. 6.7
Effect of oblique waves	Eq. 6.8 for overtopping	Eq. 6.8 for overtopping
Overtopping velocities		Eqs. 6.17 – 6.18
Scale and model uncertenties	Eqs. 6.12 – 6.14	Eqs. 6.12 – 6.14

Fig. 6.1: Armoured structures

6.2 Wave run-up and run-down levels, number of overtopping waves

Through civil engineering history the wave run-up and particularly the 2 % run-up height was important for the design of dikes and coastal embankments. Till quite recently the 2 % run-up height under design conditions was considered a good measure for the required dike height. With only 2 % of overtopping waves the load on crest and inner side were considered so small that no special measurements had to be taken with respect to strength of these parts of a dike. Recently, the requirements for dikes changed to allowable wave overtopping, making the 2 % run-up value less important in engineering practice.

Wave run-up has always been less important for rock slopes and rubble mound structures and the crest height of these type 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.

Fig. 6.2 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 (1988). 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 angels 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 armour units.

The highest curve in Fig. 6.2 gives the prediction for smooth straight slopes, see Fig. 5.1 and Equation 5.3. A rubble mound slope dissipates significantly more wave energy than an equivalent smooth and impermeable slope. Both the roughness and porosity of the armour layer cause this effect, but also the permeability of the under layer and core contribute to it. Fig. 6.2 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 5.1 includes the influence factor for roughness γ_f. For two layers of rock on an impermeable core γ_f = 0.55. This reduces to γ_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 $\xi_0 \leq 1.8$. 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. 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 ξ_0-value) and the core is impermeable, the surging waves slowly run up and down the slope and all the water stays in the armour layer, leading to 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 an 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. 6.2).

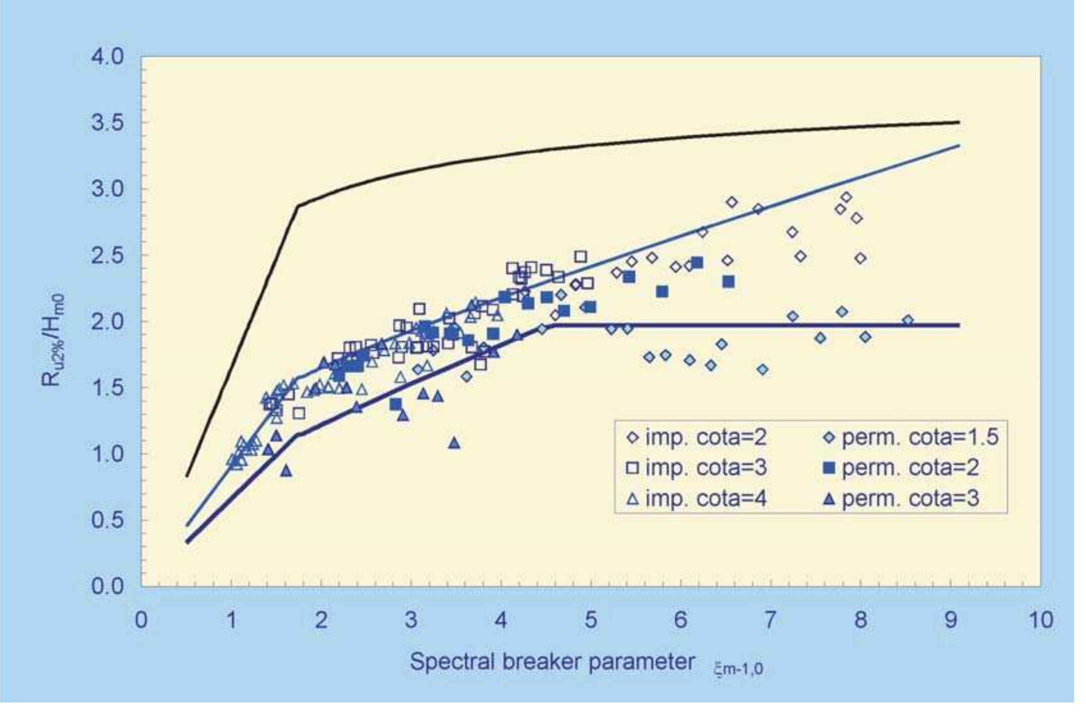

Fig. 6.2: Relative run-up on straight rock slopes with permeable and impermeable core, compared to smooth impermeable slopes

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

$$\frac{R_{u2\%}}{H_{m0}} = 1.65 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_{m-1,0} \qquad \text{with a maximum of}$$

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

From $\xi_{m-1,0} = 1.8$ the roughness factor $\gamma_{f\,surging}$ increases linearly up to 1 for $\xi_{m-1,0} = 10$, which can be described by: 6.1

$\gamma_{f\,surging} = \gamma_f + (\xi_{m-1,0} - 1.8) * (1 - \gamma_f)/8.2$

$\gamma_{f\,surging} = 1.0$ for $\xi_{m-1,0} > 10$.

For a permeable core a maximum is reached for $R_{u2\%}/H_{m0} = 1.97$

Equation 6.1 may also give a good prediction for run-up on slopes armoured with concrete armour units, if the right roughness factor is applied (see Section 6.3).

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, see Section 5.2. 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 Equation 5.4 should be used and adapted accordingly as in Equation 6.1:

$$\frac{R_{u2\%}}{H_{m0}} = 1.75 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_{m-1,0} \qquad \text{with a maximum of}$$

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

From $\xi_{m-1,0} = 1.8$ the roughness factor $\gamma_{f\,surging}$ increases linearly up to 1 for $\xi_{m-1,0} = 10$, which can be described by: 6.2

$\gamma_{f\,surging} = \gamma_f + (\xi_{m-1,0} - 1.8) * (1 - \gamma_f)/8.2$

$\gamma_{f\,surging} = 1.0$ 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 Equation 6.1 is used together with a normal distribution and variation coefficient of σ' = 0.07. For prediction or comparison of measurements the same Equation 6.1 is used, but now for instance with the 5 % lower and upper exceedance lines.

Till now only the 2 % run-up value has been described. It might be that one is interested in an other percentage, for example for 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 (1992) give two methods. One is an equation like 6.1 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 wanted. Both methods apply to straight rock slopes only and will not be described here. The given references, however, give all 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. This is similar to the method in Chapter 5 for dikes and embankment seawalls. The probability of overtopping $P_{ov} = N_{ow}/N_w$ (the percentage is simply 100 times larger) can be calculated by:

$$P_{ov} = N_{ow} / N_w = \exp\left[-\left(\sqrt{-\ln 0.02}\,\frac{R_C}{R_{u,2\%}}\right)^2\right] \qquad 6.3$$

Equation 6.3 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 Equations 6.1, 6.2 and 6.3. The equations give the run-up level in percentage or height on a straight (rock) slope. This is not the same as the number of overtopping waves or overtopping percentage. Fig. 6.3 gives the difference. The run-up is always a point on a straight slope, where for a rock slope or armoured mound the overtopping is measured some distance away from the seaward slope and on the crest, often behind a crown wall. This means that Equations 6.1, 6.2 and 6.3 always give an over estimation of the number of overtopping waves.

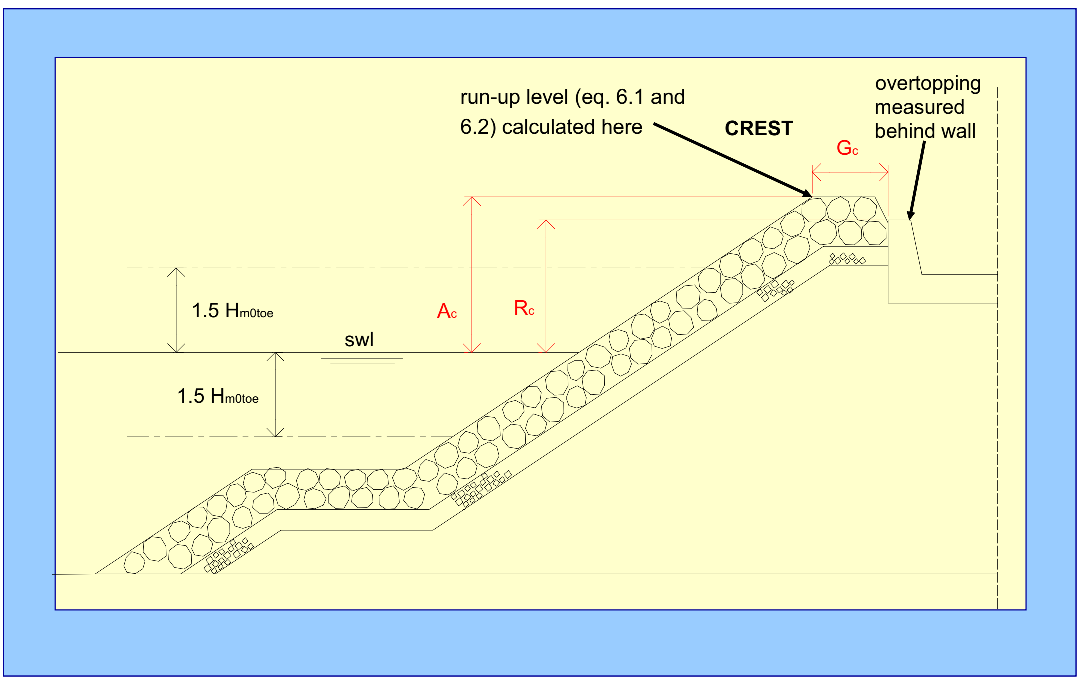

Fig. 6.3: Run-up level and location for overtopping differ

Fig. 6.4 shows measured data for rubble mound breakwaters armoured with Tetrapods (DE JONG 1996), Accropode™ or a single layer of cubes (VAN GENT et al. 1999). All tests were performed at Delft Hydraulics. The test set-up was more or less similar to Fig. 6.2 with a crown wall height R_c a little lower than the armour freeboard A_c. CLASH-data on specific overtopping tests (see Section 6.3) for various rock and concrete armoured slopes were added to Fig. 6.4. This Fig. gives only the percentage of overtopping waves passing the crown wall. Analysis showed that the size of the armour unit relative to the wave height had influence, which gave a combined parameter A_c*D_n/H_{m0}^2, where D_n is the nominal diameter of the armour unit.

The Fig. 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 6.4 can be used to predict the number or percentage of overtopping waves or to establish the armour crest level for an allowable percentage of overtopping waves.

$$P_{ov} = N_{ow} / N_w = \exp\left[-\left(\frac{A_c D_n}{0.19 H_{m0}^2}\right)^{1.4}\right] \qquad 6.4$$

It is clear that equations 6.1–6.3 will come to more overtopping waves than equation 6.4. But both estimations together give a designer enough information to establish the required crest height of a structure given an allowable overtopping percentage.

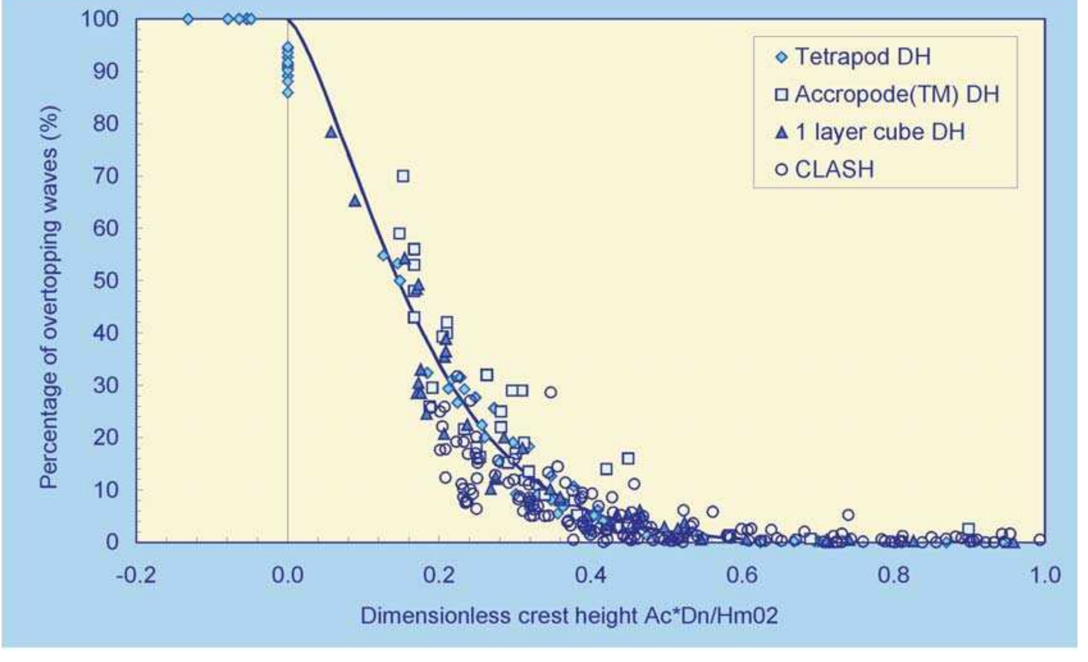

Fig. 6.4: Percentage of overtopping waves for rubble mound breakwaters as a function of relative (armour) crest height and armour size ($R_c \leq A_c$)

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 (1988), but re-analysed with respect to the use of the spectral wave period $T_{m-1,0}$. Fig. 6.5 gives an overall view.

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. Fig. 6.5 can be used directly for design purposes, as it also gives a good idea of the scatter.

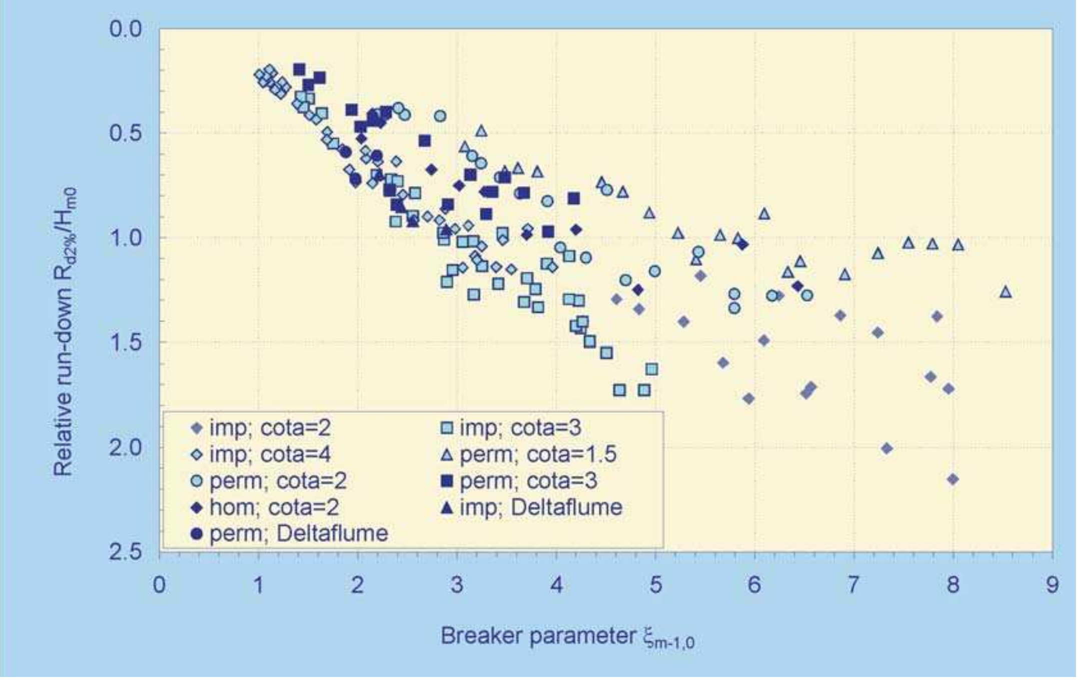

Fig. 6.5: Relative 2 % run-down on straight rock slopes with impermeable core (imp), permeable core (perm) and homogeneous structure (hom)

6.3 Overtopping discharges

6.3.1 Simple armoured 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 of course not describe the real behaviour of wave overtopping, where only large waves will reach the top of the structure and give overtopping. Random individual wave overtopping means 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 Section 6.4

Just like for run-up, the basic formula for mean wave overtopping discharge has been described in Chapter 5 for smooth slopes (Equation 5.8 or 5.9). The influence factor for roughness should take into account rough structures. Rubble mound structures often have steep slopes of about 1:1.5, leading to the second part in the overtopping equations.

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

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp\left(-2.3 \frac{R_C}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right) \qquad 6.5$$

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

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp\left(-2.6 \frac{R_C}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right) \qquad 6.6$$

The coefficient 2.6 in Equation 6.6 gives the mean prediction and its reliability can be described by a standard deviation of $\sigma = 0.35$.

As part of the EU research programme CLASH (BRUCE et al. 2007) tests were undertaken to derive roughness factors for rock slopes and different armour units on sloping permeable structures. Overtopping was measured for a 1:1.5 sloping permeable structure at a reference point $3D_n$ from the crest edge, where D_n is the nominal diameter. The wave wall had the same height as the armour crest, so $R_c = A_c$. As discussed in Section 6.2 and Fig. 6.3, 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 $3D_n$. 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 $3D_n$ further.

Results of the CLASH-work is shown in Fig. 6.6 and Table 6.2. Fig. 6.6 gives all data together in one graph. Two lines are given, one for a smooth slope, Equation 6.4 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. 6.6 one-layer systems, like Accropode™, CORE-LOC®, Xbloc® and 1 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 6.2. Equation 6.4 can be used with the roughness factors in Table 6.2 for prediction of mean overtopping discharges for rubble mound breakwaters. Values in italics in Table 6.2 have been estimated/extrapolated, based on the CLASH results.

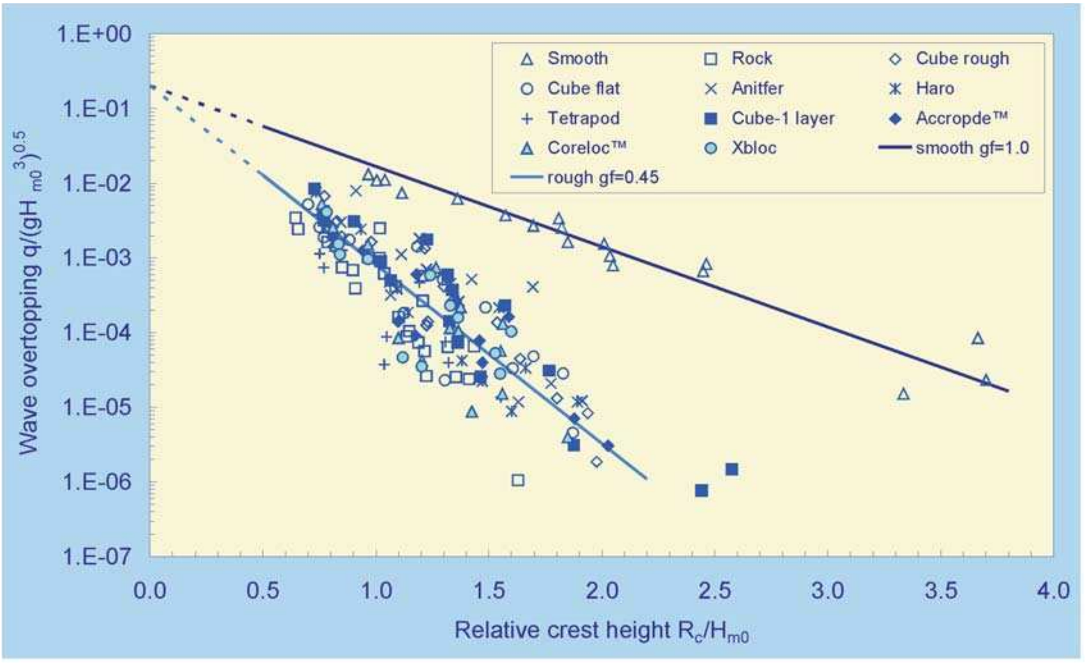

Fig. 6.6: Mean overtopping discharge for 1:1.5 smooth and rubble mound slopes

Table 6.2: Values for roughness factor γ_f for permeable rubble mound structures with slope of 1:1.5. Values in italics are estimated/extrapolated

Type of armour layer	γ_f
Smooth impermeable surface	1.00
Rocks (1 layer, impermeable core)	0.60
Rocks (1 layer, permeable core)	0.45
Rocks (2 layers, impermeable core)	0.55
Rocks (2 layers, permeable core)	0.40
Cubes (1 layer, random positioning)	0.50
Cubes (2 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*

6.3.2 Effect of armoured crest berm

Simple straight slopes including an armoured crest berm of less than about 3 nominal diameters ($G_c \approx 3D_n$) will reduce overtopping. It is, however, possible to reduce overtopping with a wide crest as much more energy can be dissipated in a wider crest. BESLEY (1999) 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 $3D_n$. Then the following reduction factor on the overtopping discharge can be applied:

$$C_r = 3.06\exp(-1.5G_c/H_{m0}) \qquad G_c/H_{m0} \quad \text{with maximum } C_r = 1 \qquad 6.7$$

Equation 6.7 gives no reduction for a crest width smaller than about 0.75 H_{m0}. This is fairly close to about $3D_n$ and is, therefore, consistent. A crest width of 1 H_{m0} reduces the overtopping discharge to 68 %, a crest width of 2 H_{m0} gives a reduction to 15 % and for a wide crest of 3 H_{m0} the overtopping reduces to only 3.4 %. In all cases the crest wall has the same height as the armour crest: $R_c = A_c$.

Equation 6.7 was determined for a rock slope and can be considered as conservative, as for a slope with Accropode more reduction was found.

6.3.3 Effect of oblique waves

Section 5.5.3 describes the effect of oblique waves on run-up and overtopping on smooth slopes (including some roughness). But specific tests on rubble mound slopes were not performed at that time. In CLASH, however, this omission was discovered and specific tests on a rubble mound breakwater were performed with a slope of 1:2 and armoured with rock or cubes (ANDERSEN and BURCHARTH, 2004). The structure was tested both with long-crested and short-crested waves, but only the results by short-crested waves will be given.

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: β = 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 is much faster with increasing angle:

$$\gamma_\beta = 1 - 0.0063\,|\beta| \qquad \text{for } 0^\circ \le |\beta| \le 80^\circ$$
$$\text{for } |\beta| > 80^\circ \text{ the result } \beta = 80^\circ \text{ can be applied} \qquad 6.8$$

The wave height and period are linearly reduced to zero for $80^\circ \le |\beta| \le 110^\circ$, just like for smooth slopes, see Section 5.3.3. For $|\beta| > 110^\circ$ the wave overtopping is assumed to be $q = 0$ m³/s/m.

6.3.4 Composite slopes and berms, including berm breakwaters

In every formula where a cotα 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. The procedure has been described in Section 5.3.4.

Also the influence of a berm in a sloping profile has been described in Section 5.3.4 and can be used for rubble mound structures. There is, however, often a difference in 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 Equations 5.8 and 5.9 as the breaker parameter will be quite small. Rubble mound structures often have a steep slope, leading to the formula for "non-breaking" waves, Equations 6.5 and 6.6. In these equations there is no form factor present.

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 in Equations 5.8 or 5.9 does 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 extend.

A specific type of rubble mound structure is the berm breakwater (see Fig. 6.7). 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

Fig. 6.7: Icelandic Berm breakwater

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. 6.8.

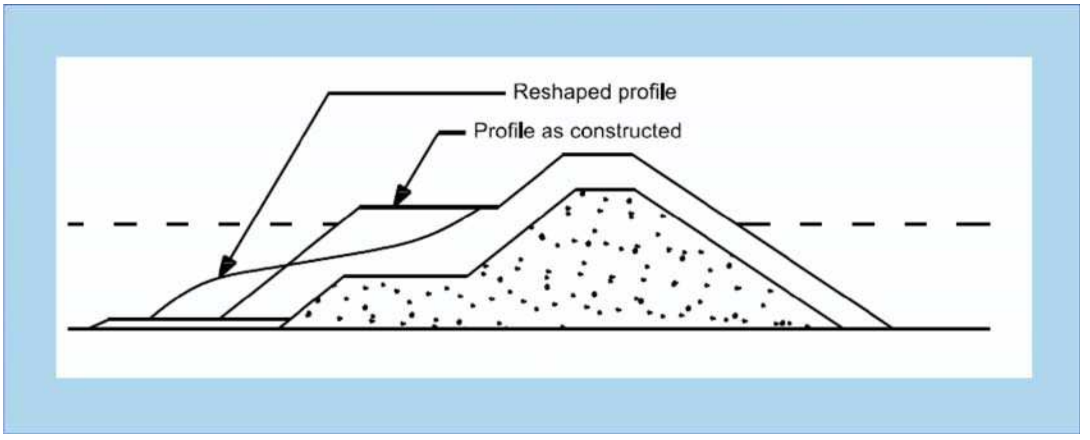

Fig. 6.8: Conventional reshaping berm breakwater

The idea of the reshaping berm breakwater has evolved in Iceland to a more or less non-reshaping berm breakwater. The main difference is that during rock production from the quarry care is taken to gather a few percent of really big rock. 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. 6.9. Therefore distinction has been made between conventional reshaping berm breakwaters and the non-reshaping Icelandic type berm breakwater.

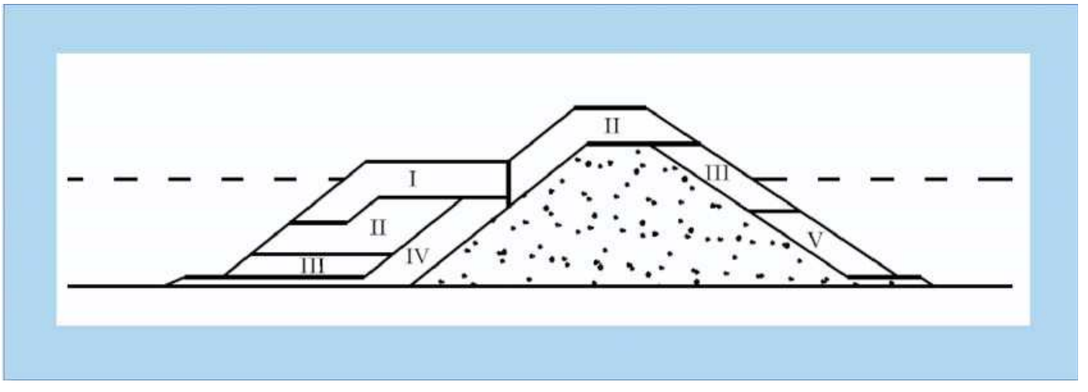

Fig. 6.9: Non-reshaping Icelandic berm breakwater with various classes of big rock

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 (1988) and the programme BREAKWAT (WL | Delft Hydraulics) is able to calculate the profile. The first method described here to calculate wave overtopping at reshaping berm breakwaters is the method described in Chapter 5 (equations 5.8 or 5.9) with the roughness factors given in Table 6.1 of $\gamma_f = 0.40$ for reshaping berm breakwaters and $\gamma_f = 0.35$ for non-reshaping 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 (Section 4.4). As overtopping research at that time on berm breakwaters was limited, also this method gives quite some scatter, but a little less than the first method described above.

Recent information on berm breakwaters has been described by LYKKE ANDERSEN (2006). 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 non-reshaping berm breakwaters (fixing the steep slopes by a steel net). The true non-reshaping 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 LYKKE ANDERSEN (2006). 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 LYKKE ANDERSEN (2006) 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 negligible.

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 has to calculate 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_{H0}, 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_{H0} is a dimensionless factor and not the direct measure of recession and that H_0 and T_0 are also dimensionless parameters.

$$f_{H0} = 19.8 s_{0m}^{-0.5} \exp(-7.08/H_0) \qquad \text{for } T_0 \geq T_0^*$$
$$f_{H0} = 0.05\, H_0 T_0 + 10.5) \qquad \text{for } T_0 < T_0^* \qquad 6.9$$

where $H_0 = H_{m0}/\Delta D_{n50}$, $T_0 = (g/D_{n50})^{0.5}\, T_{m0,1}$,
and $T_0^* = \{19.8\, s_{0m}^{-0.5} \exp(-7.08/H_0) - 10.5\}/(0.05\, H_0)$.

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 than for a bermed slope, see Section 5.3.4. This influence factor is described by:

$$d_h^* = (3H_{m0} - d_h)/(3H_{m0} + R_c) \qquad \text{for } d_h < 3H_{m0}$$
$$d_h^* = 0 \qquad \text{for } d_h \geq 3H_{m0} \qquad 6.10$$

The final overtopping formula then takes into account the influence factor on recession, f_{H0}, the influence factor of the berm level, d_h^*, the geometrical parameters R_c, B and G_c, 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, not by $T_{m-1,0}$.

$$\frac{q}{\left(gH_{m0}^3\right)^{0.5}} = 1.79\times10^{-5}\left(f_{H0}^{1.34}+9.22\right)s_{op}^{-2.52}e^{\left(-5.63\left(\frac{R_c}{H_{m0}}\right)^{0.92}-0.61\left(\frac{G_c}{H_{m0}}\right)^{1.39}-0.55h_{b*}^{1.48}\left(\frac{B}{H_{m0}}\right)^{1.39}\right)} \quad 6.11$$

Equation 6.11 is only valid for a lower 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 Equation 6.11 the peak wave period T_p has to be used to calculate s_{op}, where the mean period $T_{m0,1}$ has to be used in Equation 6.9.

Although no tests were performed on the non-reshaping Icelandic berm breakwaters (see Fig. 6.7), a number of tests were performed on non-reshaping 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 rock and, therefore, more permeability. The tests showed that Equation 6.11 is also valid for non-reshaping berm breakwaters, if the reshaping factor $f_{H0} = 0$.

6.3.5 Effect of wave walls

Most breakwaters have a wave wall, capping wall or crest unit on the crest, simply to end the armour 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 armour crest, for the simple reason that wave forces on the wall will increase drastically if directly attacked by waves and not hidden behind the armour crest. For rubble mound slopes as a shore protection, 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 realise the increase in wave forces if designing a wave wall significantly above the armour crest.

Equations 6.5 and 6.6 for a simple rubble mound slope includes a berm of $3D_n$ wide and a wave wall at the same level as the armour 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 armour. No formula are present to cope with such a situation, unless the use of the Neural Network prediction method (Section 4.4).

Various researchers have investigated wave walls higher than the armour crest. None of them compared their results with a graph like Fig. 6.6 for simple rubble mound slopes. During the writing of this manual some of the published equations were plotted in Fig. 6.6 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 armoured crest A_c in Equations 6.5 and 6.6 if the wall is higher than the crest. For a wave wall lower than the crest armour the height of this crest armour should be used. The Neural Network prediction might be able to give more precise predictions.

6.3.6 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 cot α of the structure [–]; the mean overtopping discharge, up-scaled 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. It is furthermore only relevant if mean overtopping rates are 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 fq for model and scale effects can be determined as follows:

$$f_q = \begin{cases} 1.0 & \text{for } q_{ss} \geq 10^{-3}\,\text{m}^3/\text{s/m} \\ \min\left\{ \left(-\log q_{ss} - 2\right)^3; f_{q,\max} \right\} & \text{for } q_{ss} \geq 10^{-3}\,\text{m}^3/\text{s/m} \end{cases} \qquad 6.12$$

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

$$f_{q,\max} = \begin{cases} f_{q,r} & \text{for } \gamma_f \leq 0.7 \\ 5 \cdot \gamma_f \cdot (1 - f_{q,r}) + 4.5 \cdot (f_{q,r} - 1) + 1 & \text{for } 0.7 < \gamma_f \leq 0.9 \\ 1.0 & \text{for } \gamma_f > 0.9 \end{cases} \qquad 6.13$$

and $f_{q,r}$ 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.

$$f_{q,r} = \begin{cases} 1.0 & \text{for } \cot\alpha \leq 0.6 \\ f_w \cdot (8.5 \cdot \cot\alpha - 4.0) & \text{for } 0.6 < \cot\alpha \leq 4.0 \\ f_w \cdot 30 & \text{for } \cot\alpha > 4.0 \end{cases} \qquad 6.14$$

in which 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 γ_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 gets below $q_{ss} = 10^{-5}$ $m^3/s/m$). The latter case and a steep rough slope is illustrated in Fig. 6.8.

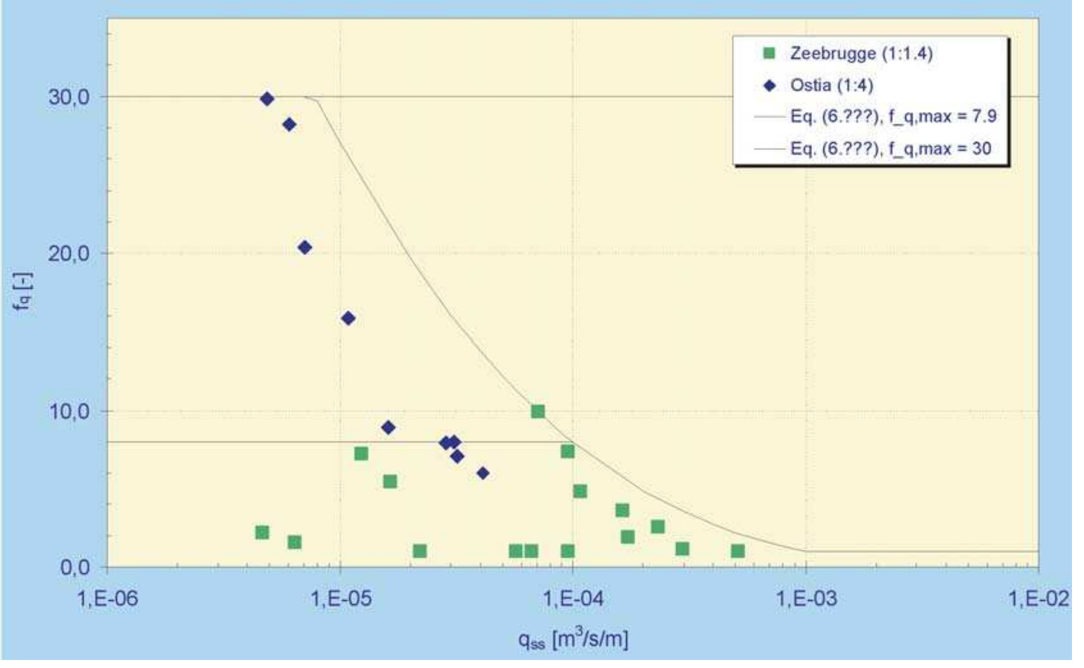

Fig. 6.10: Proposed adjustment factor applied to data from two field sites (Zeebrugge 1:1.4 rubble mound breakwater, and Ostia 1:4 rubble slope)

6.4 Overtopping volumes per wave

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. The general distribution of overtopping volumes for coastal structures has been described in Section 4.2.2.

As with many equations in this manual, the two-parameter Weibull distribution describes the behaviour quite well. This equation has a shape parameter, b, and a scale parameter, a. For smooth sloping structures an average value of b = 0.75 was found to indicate the distribution of overtopping volumes (see Section 5.4). The same average value will be used for rubble mound structures, which makes smooth and rubble mound structures easily comparable. The exceedance probability, P_V, of an overtopping volume per wave is then similar to Equations 4.2 and 4.3.

$$P_V = P(\underline{V} \leq V) = 1 - \exp\left[-\left(\frac{V}{a}\right)^{0.75}\right] \qquad 6.15$$

with:

$$a = 0.84 \cdot T_m \cdot \frac{q}{P_{ov}} = 0.84 \cdot T_m \cdot q \cdot N_w / N_{ow} = 0.84 \cdot q \cdot t / N_{ow} \qquad 6.16$$

Equation 6.16 shows that the scale parameter depends on the overtopping discharge, but also on the mean period 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 Section 6.2, Fig. 6.4 and Equation 6.4.

Equations for calculating the overtopping volume per wave for a given probability of exceedance, is given by Equation 5.34. The maximum overtopping during a certain event is fairly uncertain, as most maxima, but depends on the duration of the event. In a 6 hours period one may expect a larger maximum than only during 15 minutes. The maximum during an event can be calculated by Equation 5.35.

6.5 Overtopping velocities and spatial distribution

The hydraulic behaviour 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, see Section 5.5. 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 (LYKKE ANDERSEN and BURCHARTH, 2006). The spatial distribution was measured by various trays behind the crest wall. Fig. 6.11 gives different cross-sections with a set-up of three arrays. Up to six arrays have been used. The spatial distribution depends on the level with

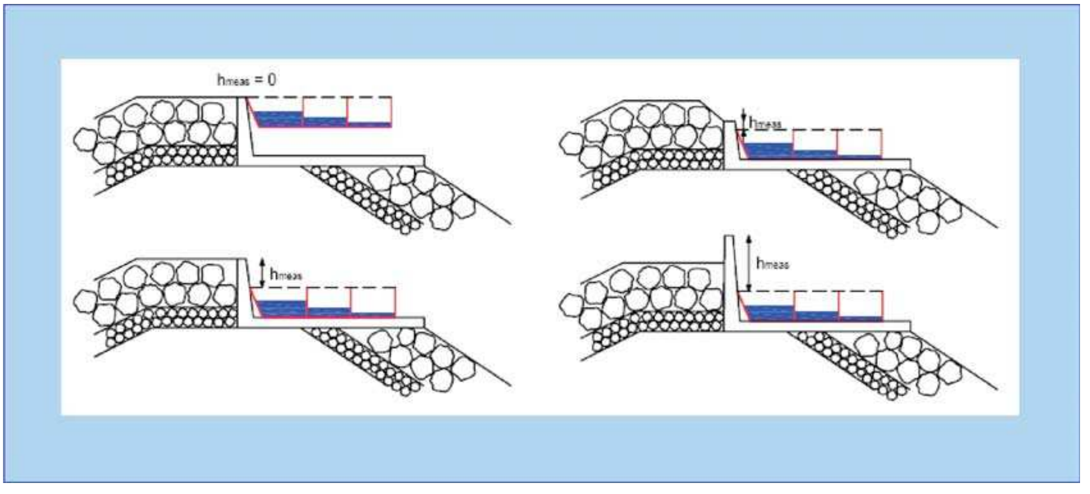

Fig. 6.11: Definition of y for various cross-sections

respect to the rear side of the crest wall and the distance from this rear wall, see Fig. 6.12. The coordinate system (x, y) starts at the rear side and at the top of the crest wall, with the positive y-axis downward.

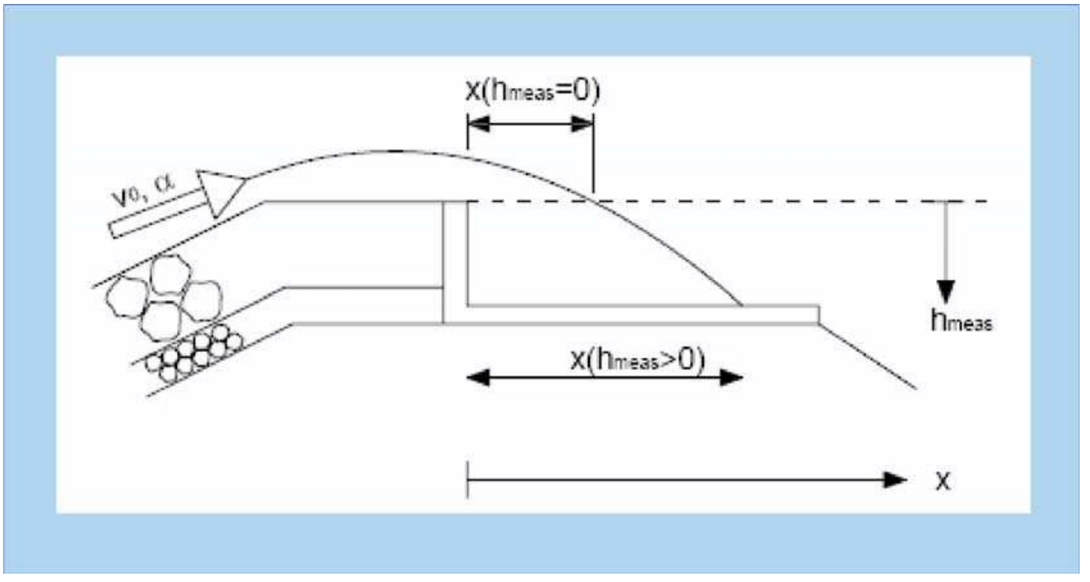

Fig. 6.12: Definition of x- and y-coordinate for spatial distribution

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 LYKKE ANDERSEN and BURCHARTH (2006). The probability F at a certain location can be described by:

$$F(x, y) = \exp\left(\frac{-1.3}{H_{m0}}.\left\{\max\left(\frac{x}{\cos\beta} - 2.7ys_{op}^{0.15}, 0\right)\right\}\right) \qquad 6.17$$

Equation 6.17 can be rewritten to calculate the travel distance x directly (at a certain level y) by rewriting the above equation:

$$\frac{x}{\cos\beta} = -0.77H_{m0}\ln(F) + 2.7ys_{op}^{0.15} \qquad 6.18$$

Suppose cosβ = 0, then we get:

F = 1	x = 0
F = 0.1	x = 1.77 H_s
F = 0.01	x = 3.55 H_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 ≠ 0, which is often the case with a crest unit.

The validity of Equations 6.17 and 6.18 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.

6.6 Overtopping of shingle beaches

Shingle beaches differ from the armoured 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 (1988). The most important profile parameter for run-up and overtopping is the crest height above SWL, h_c. For shingle with $D_{n50} < 0.1$ 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 $T_{m-1,0}$.

$$h_c/H_{m0} = 0.3\, s_{om}^{-0.5} \qquad 6.19$$

Only the highest waves will overtop the beach crest and most of this water will percolate through the material behind the beach crest. Equation 6.19 gives a run-up or overtopping level which is more or less close to $Ru_{2\%}$.

6.7 Uncertainties

Since wave overtopping formulae are principally identical to the ones for sea dikes, uncertainties of the models proposed in this chapter should be dealt with in the same way as those proposed in section 5.8 already.

It should however be noted that some of the uncertainties of the relevant parameters might change. For rubble mound structures the crest height is about 30 % more uncertain than for smooth dikes and will result in about 0.08 m. Furthermore, the slope uncertainty increases by about 40 % to 2.8 %. All uncertainties related to waves and water levels will remain as discussed within section 5.8.

The minor changes in these uncertainties will not affect the lines as shown in Fig. 5.43. Hence, the same proposal accounting for uncertainties as already given in Section 5.8 is applied here.

Again, it should be noted that only uncertainties for mean wave overtopping rates are considered here. Other methods as discussed in this chapter were disregarded but can be dealt with using the principal procedure as discussed in Section 1.5.4.