CHAPTER 8

Wave run-up and overtopping

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1 INTRODUCTION

Dikes protect the hinterland from flooding by storm surges from the sea or by high river discharges. Both situations are well known in low-laying countries and the safety in The Netherlands relies heavily on good and strong dikes. Design of dikes and other structures differs from examination of existing structures, but in both situations the same knowledge on hydraulic loads and resistance of the structure can be used. Geometrical parameters such as heights and slopes can be chosen and optimised in a design procedure, where they have to be determined for existing structures and examined against given criteria. The result of an examination may be a qualification ranging from good to unsafe; the latter result has to lead to design of a modified and safe structure.

The new law in The Netherlands on water defences (Staatsblad, 1996) states that existing water defences have to be examined every five years and a first round of examination is now underway. A code on examination of safety has been developed in the recent years. The examination can be divided into two main criteria: the height and the geotechnical stability of the water defence structure. This Chapter will deal only with the determination of the required height of dikes, i.e. wave run-up and wave overtopping.

Dikes usually have a rather mild slope, mostly of the order of 1:2 or milder. A dike consists of a toe construction, an outer slope, often with a berm, a crest of a certain height and an inner slope, see Figure 1. The outer slope may consist of various materials such as asphalt, a revetment of concrete blocks, or grass on a clay cover layer. Combinations of these are also possible.

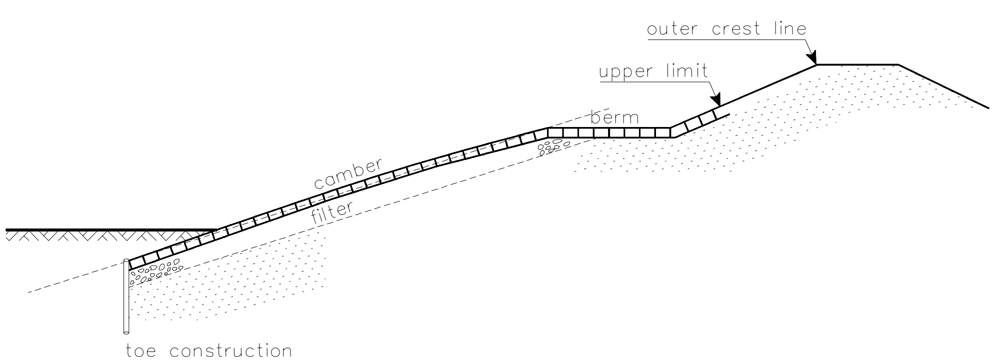

Figure 1 Cross-section of a dike: outer slope

Slopes are not always straight; the upper and lower parts do not always have a similar gradient if a berm has been applied. The crest height does not wholly depend on run-up or overtopping when designing or examining a dike. Design guidelines may account for a design water level, for an increase in this water level caused by sea level rise, both local wind set-up and squalls/ oscillations (resulting in the adjusted water level) and settlement. In the following the adjusted water level at the toe of the structure will be used.

2 CRITERIA FOR DIKE HEIGHTS

It is obvious that too low dikes will lead to flooding: either by direct overflow (rivers), or by breaching of the dike by too much wave overtopping. A safe approach is if no significant overtopping is allowed. In general this means that the crest height should not be lower than the 2%-wave run-up level. Another criterion for dike height design and examination is the admissible wave overtopping rate. This admissible overtopping rate depends on various conditions:

- how passable, practicable or trafficable the dike crest and inner berms must be in view of emergency measures under extreme conditions.

  Large scale tests in Delft Hydraulics' Deltaflume showed that, with a significant wave height of 1.5 m and an overtopping discharge of 25 l/s per m, a man on the crest of the dike (attached to a life line) could not resist the overtopping water and was swept away. From these experiments it was concluded that if people should be present on the crest of the dike the overtopping discharge should be less than 10 l/s per m.
- the admissible total volume of overtopping water with regard to storage or drain off.

  Storage or drain off problems behind the dike may have an effect on safety. If so, the overtopping should be limited.
- the resistance against erosion and local sliding of crest and inner slope due to overtopping water.

  The Dutch Guideline on river dykes (TAW, 1986) quotes "Which criterion applies depends of course also on the design of the dike and the possible presence of buildings. In certain cases, such as a covered crest and inner slopes, sometimes 10 l/s per m can be tolerated". In Dutch Guidelines it is assumed that the following average overtopping rates are allowable for the inner slope:
  - 0.1 l/s per m for sandy soil with a poor turf,
  - 1 l/s per m for clayey soil with relatively good grass.
  - 10 l/s per m with a clay protective layer and grass according to the standards for an outer slope or with a revetment construction

For examination of the dike height also the governing height has to be defined and determined. The dike has an outer slope, a certain crest width and an inner slope. Heights have to be measured at the *outer crest line*, see Figure 1, at least every 20 m of a dike section, and the lowest value is then taken as the governing dike height. A dike section is determined by similar characteristics within its length.

3 WAVE RUN-UP AND OVERTOPPING

Wave run-up and overtopping on sloping and vertical structures has been a major research topic in the recent years in Europe, mainly due to European collaboration in MAST-programmes which are financed by the European Union, national funds and own research funds of companies. Related papers are: Owen (1980), Franco et al. (1994), De Waal and Van der Meer (1992) and Van der Meer and Janssen (1995). All research related to wave run-up and overtopping on sloping dikes has been summarised in a second edition of a practical Dutch guideline which can be used for design and examination of dike heights. An English version of this early guideline is given by the work of Van der Meer and Janssen (1995). In the new edition this has been extended with respect to:

- a better description of the influence of a berm
- application (multiple berms, very long and high berms, very shallow water)
- varying roughness on slopes and berms, based on extensive Polish research (Szmytkiewicz et al. (1994))
- (almost) vertical walls on dikes
- overtopping formulae in accordance with run-up formulae

The new formulae and the main modifications and improvements have been described in this paper.

3.1 General formula on wave run-up

Wave run-up is often indicated by $R_{u2\%}$. This is the run-up level, vertically measured with respect to the (adjusted) still water level (SWL), which is exceeded by two per cent of the incoming waves. Note that the number of exceedance is here related to the number of incoming waves and not to the number of run-up levels.

The relative run-up is given by $R_{u2\%}/H_s$, with H_s the significant wave height, being the average value of the highest 1/3 part of the wave heights, or the wave height based on energy: $4\sqrt{m_0}$, with m_0 the zeroth moment of the energy density spectrum. This H_s is the significant wave height at the toe of the structure. The relative run-up is usually given as a function of the surf similarity parameter or breaker parameter which is defined as

$$\xi_{op} = \tan\alpha / \sqrt{s_{op}} \qquad (1)$$

where: ξ_{op} = the breaker parameter, α = the average slope angle and s_{op} = the wave steepness.

The wave steepness is a fictitious or computation quantity, especially meant to describe the influence of a wave period. This quantity is fictitious as the wave height at the location of the toe is related to the wave length in deep water ($gT_p^2/2\pi$).

With ξ_{op} < 2 - 2.5 the waves will tend to break on the dike or seawall slope. This is mostly the case with slopes of 1:3 or milder. For larger values of ξ_{op} the waves do not break on the slope any longer. In that case the slopes are often steeper than 1:3 and/or the waves are characterised by a smaller wave steepness (for example swell).

The general design formula that can be applied for wave run-up on dikes is given by:

$$R_{u2\%}/H_s = 1.6\, \gamma_b\, \gamma_f\, \gamma_\beta\, \xi_{op} \qquad \text{with a maximum of } 3.2\, \gamma_f\, \gamma_\beta \qquad (2)$$

with : γ_b = reduction factor for a berm, γ_f = reduction factor for slope roughness and
γ_β = reduction factor for oblique wave attack.

The formula is valid for the range $0.5 < \gamma_b\xi_{op} < 4$ or 5. The relative wave run-up $R_{u2\%}/H_s$ depends on the breaker parameter ξ_{op} and on three reduction factors, namely: for berms, roughness on the slope and for oblique wave attack. The question of how the reduction factors should be computed is given further on in the paper.

Equation 2 is shown in Figure 2 where the relative run-up $R_{u2\%}/(\gamma_f\gamma_\beta H_s)$ is set out against the breaker parameter $\gamma_b\xi_{op}$. The relative run-up increases till $\gamma_b\xi_{op}$ = 2 and remains constant for larger values. The latter is the case for relatively steep slopes and/or low wave steepnesses. The theoretical limit for a vertical structure ($\xi_{op} = \infty$) is $R_{u2\%}/H_s$ = 1.4 in Equation 2 or Figure 2, but this is far outside the application range considered here.

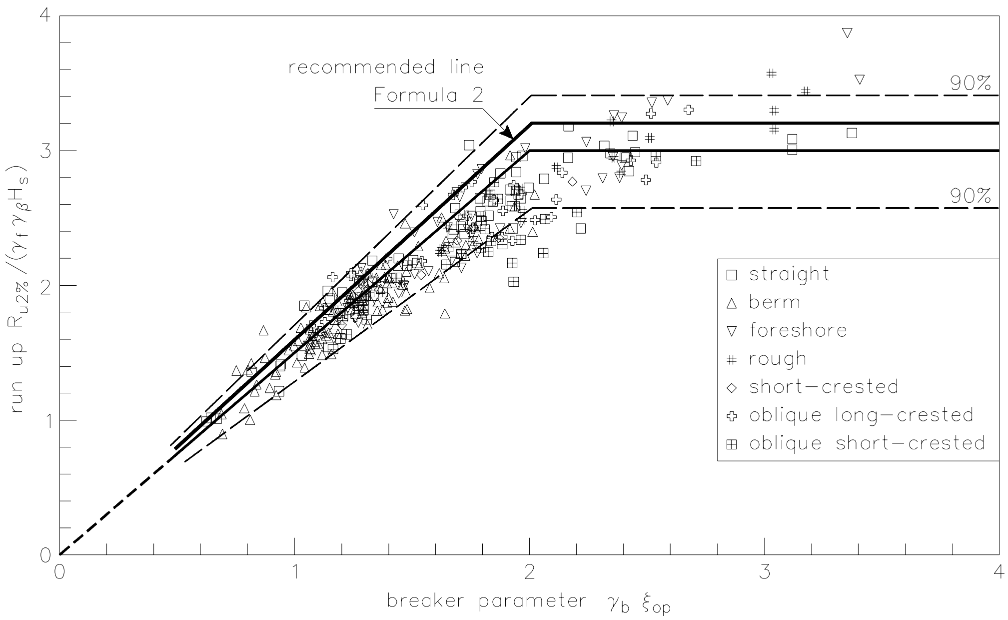

Figure 2 Wave run-up as a function of the breaker parameter, including data with possible influences

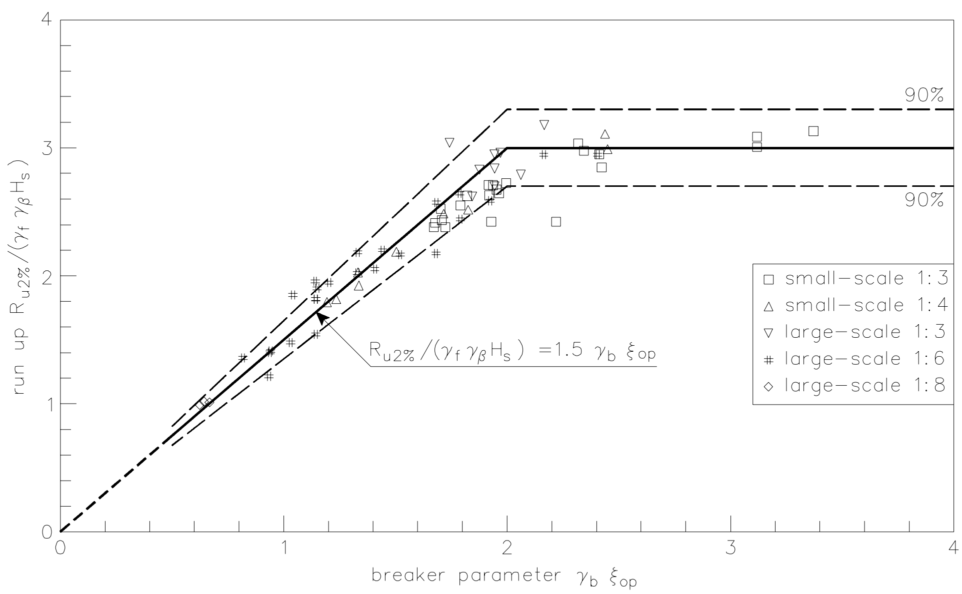

Figure 3 Wave run-up for a smooth slope with measured data

In manuals or guidelines it is advisable that one should not deal with a general trend in design formulae. Instead, it is recommended that a more conservative approach should be used. In many international standards a safety margin of one standard deviation is applied. This value is also incorporated in Equation 2. Figure 2 gives all the available data points pertaining to wave run-up. Equation 2 is only based on smooth and straight slopes under perpendicular wave attack. Those data points are only from the small-scale tests of De Waal and Van der Meer (1992), and from results of available large-scale tests which can be considered reliable. Amongst these large scale tests are the tests of Führböter et al. (1989). These data are shown in Figure 3.

The *average* value of the wave run-up can be described by:

$$R_{u2\%}/H_s = 1.5\ \gamma_b\gamma_f\gamma_\beta\ \xi_{op} \text{ with a maximum of } 3.0\ \gamma_f\gamma_\beta \qquad (3)$$

The scatter around equation 3 can be described by interpreting the coefficient 1.5 as a normally distributed stochastic variable with a mean value of 1.5 and a variation coefficient (standard deviation divided by the mean value) of $V = \sigma/\mu = 0.085$. Figure 2 shows all available data points pertaining to slopes with berms or roughness and obliquely incoming, short-crested waves. When all the influences are incorporated into one figure the scatter becomes larger than for only smooth, straight slopes, see Figure 3.

However, equation 3 should not be used for the wave run-up when deterministically designing dikes: for that purpose one should use equation 2. For probabilistic designs equation 3 should be taken with the above variation coefficient.

In TAW (1974) a formula is given for mild (milder than 1:2.5), smooth, straight slopes. After rearrangement, this formula has the form:

$$R_{u2\%}/H_s = 1.61\ \xi_{op} \qquad (4)$$

The formula is virtually identical to equation 2 except for the reduction factors and the limit for steeper slopes, which has here the value of 3.2. In other words, the run-up formula used for twenty years will be maintained and complemented on specific points.

3.2 General formula on wave overtopping

With wave overtopping the crest height is lower than the run-up levels of the highest waves. The parameter to be considered here is the crest freeboard R_c. This is the difference between SWL and the governing dike height. Wave overtopping is mostly given as an average discharge q per unit width, for example in m³/s per m or in l/s per m. The Dutch Guideline on river dykes (TAW, 1989) indicates that for relatively heavy seas and with wave heights of up to a few meters the 2%-wave run-up criterion yields an overtopping discharge of the order of 1 l/s per m. It becomes 0.1 l/s per m with lower waves such as those occurring in rivers. An acceptable overtopping of 1 l/s per m in the river area, instead of the 2%-wave run-up, can then lead to a reduction of the freeboard of the dike.

The former formulae on wave overtopping, see Van der Meer and Janssen (1995), made a distinction between breaking (plunging) and non-breaking (surging) waves on the slope. The new set of formulae relates to breaking waves and is valid up to a maximum which is in fact the non-breaking region. This procedure is in accordance with wave run-up on a slope. The new (rewritten) formula for wave overtopping on dikes is as follows:

$$\frac{q}{\sqrt{gH_s^3}} = \frac{0.06}{\sqrt{\tan\alpha}}\gamma_b\,\xi_{op}\exp\left(-4.7\frac{R_c}{H_s}\frac{1}{\xi_{op}\gamma_b\gamma_f\gamma_\beta\gamma_v}\right) \qquad (5)$$

with as maximum:

$$\frac{q}{\sqrt{gH_s^3}} = 0.2\exp(-2.3\frac{R_c}{H_s}\frac{1}{\gamma_f\gamma_\beta}) \tag{6}$$

where: q = average overtopping rate (m^3/s per m width); R_c = crest freeboard (m); and γ_v = reduction factor due to a vertical wall on a slope (-).

Figure 4 gives prediction curves as a function of the breaker parameter, similar to Figures 2 and 3, but now with the dimensionless wave overtopping on the vertical axis instead of the wave run-up and with various curves in relation to the crest height. Figure 4 yields for a straight, smooth slope of 1:3 and different curves are given for relative freeboards of R_c/H_s = 1, 2 and 3. Around ξ_{op} = 2 the wave overtopping reaches its maximum just as for wave run-up.

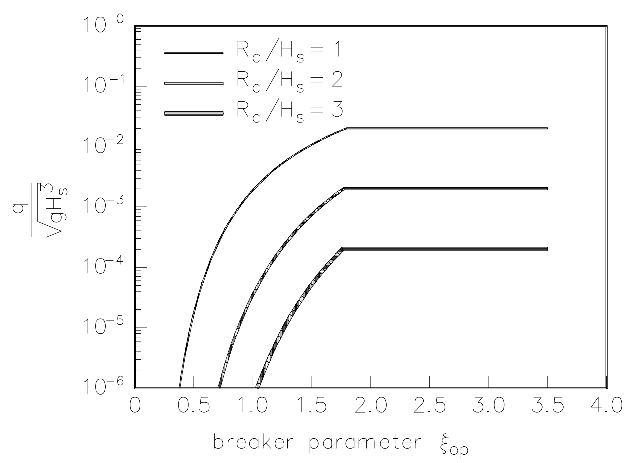

Figure 4 Dimensionless wave overtopping on a straight, smooth 1:3 slope as a function of the breaker parameter and for various relative crest heights

Figure 5 gives an overall view of available information on breaking waves. It shows the results of many tests and gives a good idea of the scatter. In this figure the important parameters are given along the axes, all existing data points are given with a mean and 95% confidence bands and typical applications are indicated along the vertical axis. Besides data of Delft Hydraulics also data for smooth slopes from Owen (1980) and Führböter et al. (1989) are included. The dimensionless overtopping discharge is given on the ordinate and the dimensionless crest height on the abscissa. The overtopping discharge is given on a logarithmic scale, assuming a kind of exponential function. Owen (1980) was the first who gave explicitly the exponential relationship between dimensionless overtopping discharge and relative crest height. Most of other researchers have used this kind of relationship to describe their overtopping data. The differences then are mainly the treatment of the influence of the wave period or steepness and the slope angle. In fact equation 5 includes the original equation of Owen (1980) and also his data.

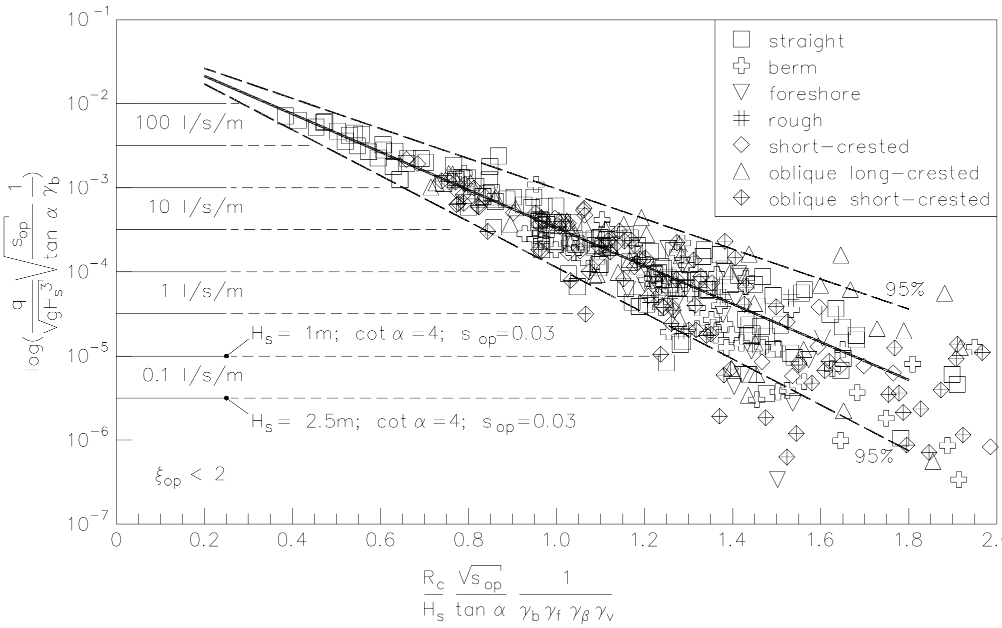

Figure 5 Wave overtopping data for breaking waves with mean, confidence bands and with an indication of typical applications

The average of all the observations in Figure 5 is not described by equation 5, but by a similar equation where the coefficient 4.7 has been changed to 5.2. The reliability of the formula is given by taking the coefficient 5.2 as a normally distributed stochastic variable with an average of 5.2 and a standard deviation $\sigma = 0.55$. By means of this standard deviation also confidence bands ($\mu \pm x\sigma$) can be drawn with for x times the standard deviation (1.64 for the 90% and 1.96 for the 95% confidence limit). The average for all data at the maximum, described by equation 6 is given by a coefficient 2.6 instead of 2.3. The reliability of this formula can be given by taking the coefficient 2.6 as a normally distributed stochastic variable having a standard deviation of $\sigma = 0.35$. As is the case with wave run-up, a somewhat more conservative formula should be applied for design and examination purposes than the average value. An extra safety of about one standard deviation is taken into account in equations 5 and 6. Figure 6 shows the recommended equation 5, the mean and the 95% confidence limits. Also in this Figure 6 the formula from TAW (1974) is drawn and is practically the same as the recommended line.

Also, in Figure 5 several overtopping discharges are illustrated, namely, 0.1, 1, 10 and 100 l/s per m. The discharges apply for a 1:4 slope and a wave steepness of $s_{op} = 0.03$. The upper line of the interval applies to a significant wave height of 1.0 m (for example river dikes) and the lower one for a wave height of 2.5 m (for example for sea dikes).

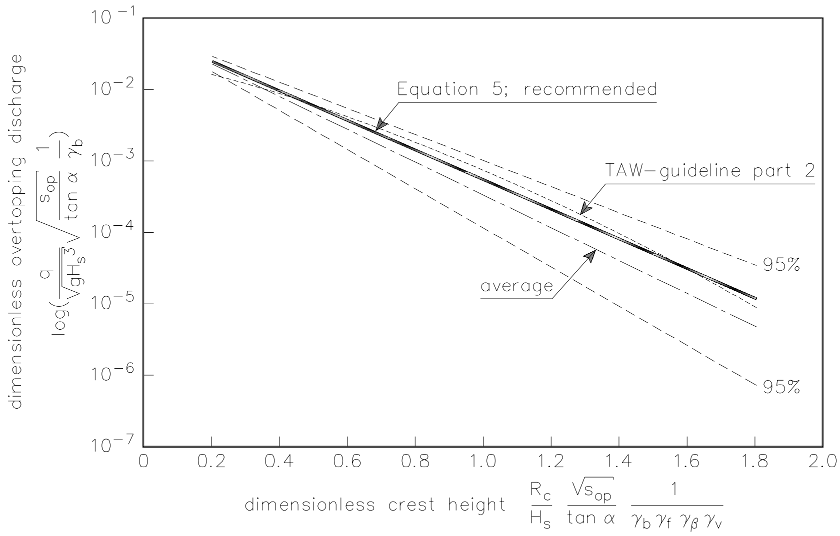

Figure 6 Wave overtopping with confidence limits

4 REDUCTION FACTORS ON WAVE RUN-UP AND WAVE OVERTOPPING

Above equations, described as general formulae on wave run-up and wave overtopping, include the effects of a berm, friction on the slope, oblique wave attack and a wall on the slope. These effects will be described now.

4.1 Definition of the average slope angle

Research is very often performed with nice straight slopes and the definition of tanα is then obvious. In practice, however, a dike slope may consist of various more or less straight parts and the definition of the slope angle needs to be more precisely defined. The slope angle becomes an average slope angle. Figure 7 gives the definition that is used in this paper.

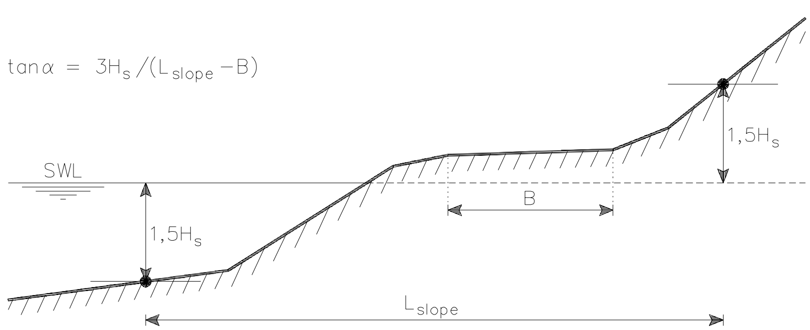

Figure 7 Determination of the average slope angle

The wave action is concentrated on a certain part of the slope around the water level. Examination of many tests showed that the part 1.5 H_s above and below the water line is the governing part. As berms are treated separately the berm width should be omitted from the definition of the average slope. The average slope is then defined as:

$$\tan\alpha = 3H_s/(L_{slope} - B) \quad (7)$$

Where L_{slope} = the horizontal length between the two points on the slope 1.5 H_s above and below the water line, and B = the berm width.

4.2 Reduction factor γ_b for a berm

A berm is defined as a flat part in a slope profile with a slope not steeper than 1:15. The berm itself is described by its berm width, B (see Figure 7), and by its location with respect to the still water level, d_h. This depth parameter is the vertical distance between the still water level and *the middle of the berm*. A berm at the still water level gives $d_h = 0$.

The influence of a berm on wave run-up and wave overtopping is graphically given in Figure 8. The horizontal axis gives the relative depth d_h/H_s of the berm below (positive) or above (negative) the still water level. The vertical axis shows the reduction factor γ_b to be used in equations 2, 3 and 5. The reduction is limited to a value of 0.6 and reaches its minimum value (maximum influence) if the berm is *at* the still water level: $d_h/H_s = 0$. No influence of a berm is found if the berm is higher than the 2%-runup level on the down slope. The influence vanishes if the berm is more than two wave heights below the still water level. Various curves show the influence of the relative berm width B/H_s. A larger berm width gives a larger influence.

The influence of the *berm width* can be described by taking into account the change in slope angle with and without the berm. Now the influence is described with respect to *the middle of the berm* (and not with respect to the water level), see Figure 9. The ratio B/L_{berm} describes this influence, where L_{berm} is the horizontal length of the total slope (including the berm) between the points 1.0 H_s above and below the *middle of the berm*.

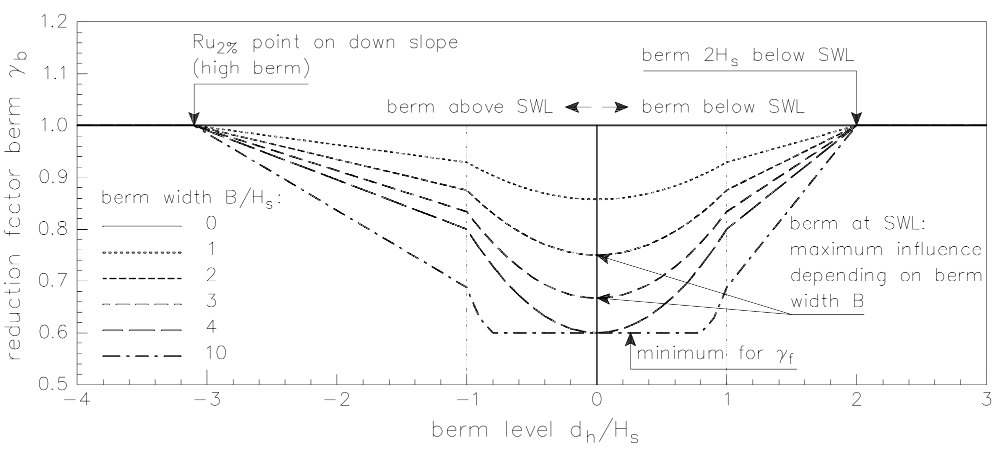

Figure 8 The reduction factor γ_b for a bermed slope

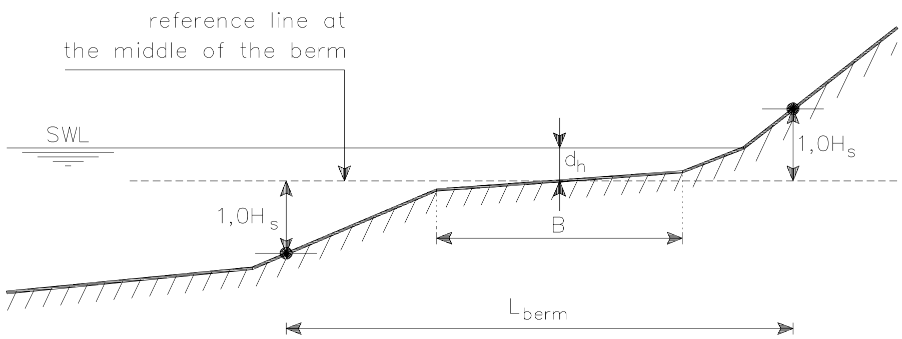

Figure 9 Definition of L_{berm}

Figure 8 can be described by the following equation:

$$\gamma_b = 1 - \frac{B}{L_{berm}}\left(1 - 0.5\left(\frac{d_h}{H_s}\right)^2\right) \qquad (8)$$

with $0.6 \leq \gamma_b \leq 1.0$ and $-1.0 \leq d_h/H_s \leq 1.0$

Between $d_h = 1\ H_s$ and $d_h = 2\ H_s$ the reduction factor γ_b increases linearly to $\gamma_b = 1$ (the influence of the berm reduces linearly to zero). See Figure 8. With a high berm the influence also decreases linearly from $\gamma_b = 1 - 0.5B/L_{berm}$ at $d_h/H_s = -1\ H_s$ to $\gamma_b = 1$ if $R_{u2\%}$ is reached on the down slope.

The berm is most effective when lying at the still water level, or around it if upper and lower slopes are different. An optimum berm width will be obtained if the reduction factor reaches the value of 0.6. In principle, this optimum berm width can be determined with equation 8 for every berm geometry (with *one* berm). For a berm at the still water level the optimum berm width becomes:

$$B = 0.4\ L_{berm} \qquad (9)$$

It might be possible that a dike profile includes more than one berm. Then the reduction factors have to be determined separately for each berm. The total reduction factor is the product of the separate reduction factors, of course with a minimum of 0.6. The ratio B/L_{berm} in equation 8 should then be modified a little:

$$B/L_{berm} = B_i / (L_{berm,i} - \sum L_{other\ berms}), \qquad (10)$$

with "i" for the berm considered.

4.3 Reduction factor γ_f for roughness on the slope

The influence of a kind of roughness on the slope is given by the reduction factor γ_f. Reduction factors for various types of revetments have been published earlier. The origin of these factors dates back to Russian investigations performed in the 1950's with regular waves. A table on these factors was further developed in TAW (1974) and published in several international manuals. New studies, often large-scale, and conducted with random waves have led to a new table (Table 1) of reduction factors for rough slopes.

Type of slope	Reduction factor γ_f	Old reduction factors
Smooth, concrete, asphalt	1.0	1.0
Closed, smooth, block revetment	1.0	0.9
Grass (3 cm)	0.95	0.85-0.90
Block revetment (basalt, basalton)	0.90	0.85-0.90
1 rubble layer (H_s/D = 1.5-3)	0.60	0.80
¼ of placed block revetment (0.5*0.5 m^2) 9 cm above slope	0.75	

Table 1 Reduction factors γ_f for a rough slope

The reduction factors in Table 1 apply for $\gamma_b\xi_{op} < 3$. Above $\gamma_b\xi_{op} = 3$, the reduction factor increases linearly to 1 at $\gamma_b\xi_{op} = 5$. The reduction factors in Table 1 apply if the part between 0.25 $R_{u2\%,\ smooth}$ below and 0.5 $R_{u2\%,\ smooth}$ above the still water level is covered with roughness. The extension "smooth" means the wave run-up on a smooth slope. If the coverage is less, the reduction factor has to be reduced, see further.

Research on roughness units on a slope, such as blocks and ribs, has also been performed by Szmytkiewitcz et al. (1994). Their results have been re-analysed by the authors. The width of a block or rib is defined by f_b, the height by f_h and the rib distance by f_L. The optimal rib distance is $f_L/f_b = 7$, with an area of application of f_L/f_b = 5-8. If the total area is covered with blocks or ribs and if the height is at least $f_h/H_s = 0.15$, the following (minimal) reduction factors are found:

Block, covered area 1/25th of total $\gamma_f = 0.80$
Block, covered area 1/9th of total $\gamma_f = 0.70$
Ribs, rib distance f_L/f_b = 5-8 $\gamma_f = 0.65$

A larger block or rib height dan $f_h/H_s = 0.15$ has no further reducing effect. If the height is smaller one can interpolate linearly to $\gamma_f = 1$ for $f_h/H_s = 0$. Just like for the reduction factors in Table 1, the application of the reduction factors is limited by $\gamma_b\xi_{op} < 3$ and increase linearly to 1 for $\gamma_b\xi_{op} = 5$.

Run-up formulae for rock slopes with a double layer of rock have been given by Van der Meer and Stam (1992). After some modification from mean period to peak period the equations become:

$$R_{u2\%}/H_s = 0.88\ \xi_{op} \quad \text{for } \xi_{op} < 1.5 \text{ and}$$
$$R_{u2\%}/H_s = 1.1\ \xi_{op}^{0.46} \quad \text{for } \xi_{op} > 1.5 \qquad (11)$$

One can use these formulae to calculate wave-runup for rock slopes. For wave overtopping one still has to use the reduction factor γ_f. This reduction factor is found by calculating the $R_{u2\%}$ both for the rock slope and for a smooth slope for the same ξ_{op} and by determining the ratio of these figures.

It is possible that roughness is only present on a small part of the slope. First of all, tests showed that roughness solely below the still water level (and a smooth slope above) does not have any influence. If also roughness above the still water level is present an average weighing can be done over the area 0.25 $R_{u2\%, smooth}$ below and 0.5 $R_{u2\%, smooth}$ above the water level. The part to be taken into account below SWL may never exceed the part above SWL. Suppose that within the given area three different slope sections exist with lengths of respectively l_1, l_2 and l_3 and reduction factors of $\gamma_{f,1}$, $\gamma_{f,2}$ and $\gamma_{f,3}$. The average reduction factor for roughness becomes then:

$$\gamma_f = \frac{\gamma_{f,1} l_1 + \gamma_{f,2} l_2 + \gamma_{f,3} l_3}{l_1 + l_2 + l_3} \tag{12}$$

As artificial roughness works in a fairly small area, the entire reduction can be reached by just placing the roughness in this area. Costs for this artificial roughness may be much smaller than if a total slope is covered by artificial roughness.

4.4 Reduction factor γ_β for the angle of wave attack

The angle of the wave attack ß is defined as the angle of the propagation direction with respect to the normal of the alignment axis of the dike. Perpendicular wave attack is therefore given by ß = 0°. The reduction factor for the angle of wave attack is given by γ_β. Until recently few investigations were carried out with obliquely incoming waves but these investigations had been performed with long-crested waves. "Long-crested" means that the length of the wave crest is in principle assumed to be infinite. In investigations with long-crested waves the wave crest is as long as the wave board and the wave crests propagate parallel to one another.

In nature, waves are short-crested. This implies that the wave crests have a certain length and the waves a certain main direction. The individual waves have a direction around this main direction. The extent to which they vary around the main direction (directional spreading) can be described by a spreading value. Only long swell, for example coming from the ocean, has such long crests that it may virtually be called "long-crested". A wave field with strong wind is short-crested.

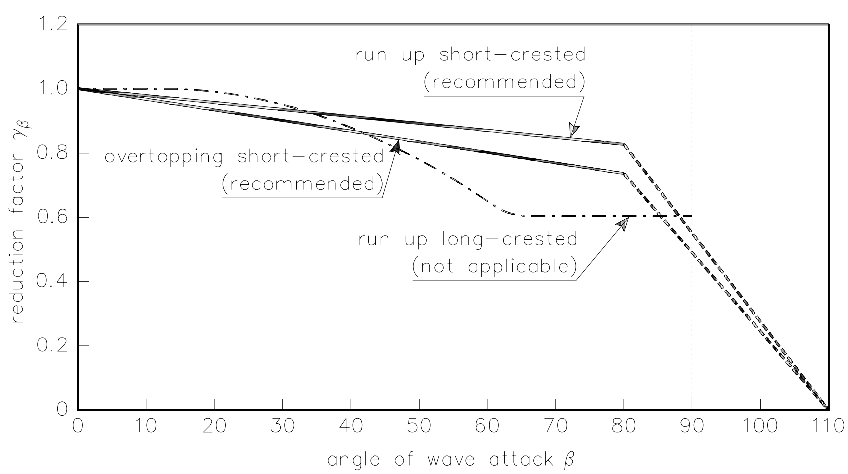

Figure 10 The reduction factor for oblique wave attack γ_β

In Van der Meer and Janssen (1995) results of an investigation were described into wave run-up and overtopping where the influence of obliquely incoming waves and directional spreading was studied. Figure 10 summarises these results. The reduction factor γ_β has been set out against the angle of wave attack ß.

Long-crested waves with 0° < ß < 30° cause virtually the same wave run-up as with perpendicular attack. Outside of this range, the reduction factor decreases fairly quickly to about 0.6 at ß = 60°. With short-crested waves the angle of wave attack has apparently less influence. This is mainly caused by the fact that within the wave field the individual waves deviate from the main direction ß. For both run-up and overtopping with short-crested waves the reduction factor decreases linearly to a certain value at ß = 80°. This is around γ_β = 0.8 for the 2% run-up and around 0.7 for overtopping. So, for wind waves the reduction factor has a minimum of 0.7 - 0.8 and not 0.6, as was found for long-crested waves. Since a wave field under storm conditions can be considered to be short-crested, it is recommended that the lines in Figure 10 be used for short-crested waves.

For oblique waves, different reduction factors apply to run-up levels or to overtopping discharges. The cause for this is that here the incoming wave energy per unit length of structure is less than that for perpendicular wave attack. The wave overtopping is defined as a volume per unit length, while the run-up does not depend on the structure length. The use of the lines given in Figure 10 for short-crested waves is recommended and can be described by the following formulae:

For the 2%-wave run-up with short-crested waves:

$$\gamma_\beta = 1 - 0.0022\beta \quad (\text{ß in degrees}) \tag{13}$$

For wave overtopping with short-crested waves:

$$\gamma_\beta = 1 - 0.0033\beta \quad (\text{ß in degrees}) \tag{14}$$

For wave angles with β > 80° the reduction factor will of course rapidly diminish. As at β = 90° still some wave run-up can be expected, certainly for short-crested waves, it is fairly arbitrary stated that between β = 80° and 110° the reduction factor linearly decreases to zero.

4.5 Reduction factor γ_v for a (vertical) wall on a slope

In some situations it may occur that a vertical wall or a very steep slope at the top of a slope has been designed to reduce wave overtopping. These walls are relatively small and not comparable with vertical structures like caissons or quay walls. Due to limited research the application of a reduction factor γ_v is restricted by the following area:

- slopes from 1:2.5 to 1:3.5, possibly with a berm with dimensions B/H_s = 2-3 or B/L_{op} = 0.05-0.08
- the toe of the wall should lie between 1.2 H_s above and below the still water level
- the minimum height of the wall (with a high toe level) is about 0.5 H_s and the maximum height (with a low toe level) is about 3 H_s.

An average slope has been defined in Figure 7. With a vertical wall this procedure will very soon lead to a large value of the breaker parameter ξ_{op}, which means that waves do not break in such situations. In fact the wall is situated at the top of the slope and waves will possibly break on the slope before they reach the wall. In order to keep the relationship between the type of breaking and the breaker parameter, *the vertical wall has to be schematised as a 1:1 slope,* starting at the toe of the wall. Then the procedure of Figure 7 and equation 7 can be applied.

The reduction factor for a vertical wall on top of a slope is $\gamma_v = 0.65$. Data with a vertical wall and with the application of the reduction factor are shown in Figure 11. The lines in the graph are similar to Figure 6. Wave overtopping will increase again if the wall is not completely vertical, but a little sloping. With a steep slope of 1:1 the reduction factor becomes $\gamma_v = 1$. For a steeper slope, between 1:1 and vertical, one may interpolate linearly:

$$\gamma_v = 1.35 - 0.0078\alpha_{slope} \tag{16}$$

with α_{slope} = the angle of the steep wall (between 45° at 1:1 and 90° for a vertical wall).

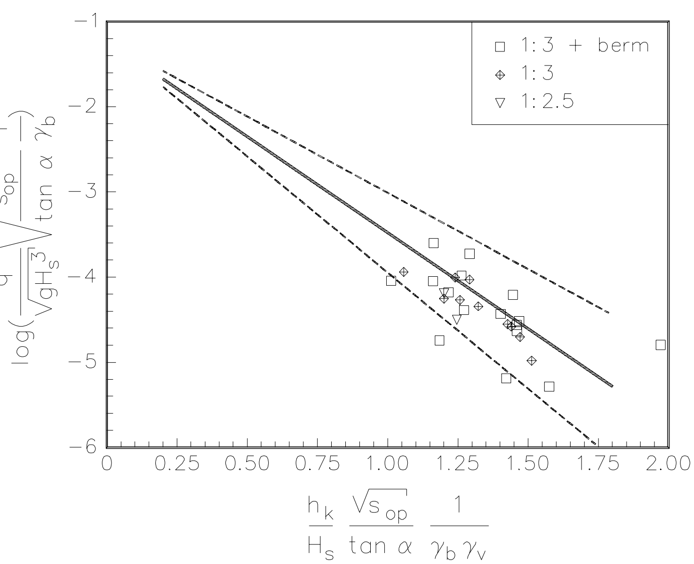

Figure 11 Data with a vertical wall on top of a (bermed) slope with $\gamma_v = 0.65$

5 CONCLUSIONS

Formulae have been given for conceptual design on wave run-up and wave overtopping of dikes and sloping structures. Influences of structure geometry and wave boundary conditions have been included in a practical way by the use of reduction factors. Formulae were given for deterministic design, including some safety, but also for probabilistic design, giving the average trend and the reliability by a variation coefficient.

A final remark on applicability of the formulae should be given for a specific situation that may happen quite often in practice. This is the situation where waves from deep water break substantially on the foreshore and decrease also substantially in height. Most of the formulae given in this chapter, but also for other references, apply to fairly deep water or moderate shallow water conditions. These conditions can be characterized by conditions where the wave height decreases less than 30-40% from the deep water wave height, due to breaking.

In some cases, however, the decrease in wave height is much more and can amount up to 70-80% (only 20-30% of the original wave height remains). The heavy breaking gives a very wide spectrum with energy present over a wide range of frequencies and a peak period can hardly be detected. In those situations, the extreme shallow water situations, the formulae in this chapter may not be valid. Research is going on in this area, but needs time to be performed and evaluated.

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