Die Küste, 73 EurOtop (2007), 1-17

1. Introduction

1.1 Background

This manual describes methods to predict wave overtopping of sea defence and related coastal or shoreline structures. It recommends approaches for calculating mean overtopping discharges, maximum overtopping volumes and the proportion of waves overtopping a seawall. The manual will help engineers to establish limiting tolerable discharges for design wave conditions, and then use the prediction methods to confirm that these discharges are not exceeded.

1.1.1 Previous and related manuals

This manual is developed from, at least in part, three manuals: the (UK) Environment Agency Manual on Overtopping edited by BESLEY (1999); the (Netherlands) TAW Technical Report on Wave run-up and wave overtopping at dikes, edited by VAN DER MEER (2002); and the German Die Küste EAK (2002) edited by ERCHINGER. The new combined manual is intended to revise, extend and develop the parts of those manuals discussing wave run-up and overtopping.

In so doing, this manual will also extend and/or revise advice on wave overtopping predictions given in the CIRIA / CUR Rock Manual, the Revetment Manual by MCCONNELL (1998), British Standard BS6349, the US Coastal Engineering Manual, and ISO TC98.

1.1.2 Sources of material and contributing projects

Beyond the earlier manuals discussed in section 1.3, new methods and data have been derived from a number of European and national research programmes. The main new contributions to this manual have been derived from OPTICREST; PROVERBS; CLASH & SHADOW, VOWS and Big-VOWS and partly ComCoast. Everything given in this manual is supported by research papers and manuals described in the bibliography.

1.2 Use of this manual

The manual has been intended to assist an engineer analyse the overtopping performance of any type of sea defence or related shoreline structure found around Europe. The manual uses the results of research studies around Europe and further overseas to predict wave overtopping discharges, number of overtopping waves, and the distributions of overtopping volumes. It is envisaged that methods described here may be used for current performance assessments, and for longer-term design calculations. Users may be concerned with existing defences, or considering possible rehabilitation or new-build.

The analysis methods described in this manual are primarily based upon a deterministic approach in which overtopping discharges (or other responses) are calculated for wave and water level conditions representing a given return period. All of the design equations require data on water levels and wave conditions at the toe of the defence structure. The input water level should include a tidal and, if appropriate, a surge component. Surges are usually comprised of components including wind set-up and barometric pressure. Input wave conditions should take account of nearshore wave transformations, including breaking. Methods of calculating depth-limited wave conditions are outlined in Chapter 2.

All of the prediction methods given in this report have intrinsic limitations to their accuracy. For empirical equations derived from physical model data, account should be taken of the inherent scatter. This scatter, or reliability of the equations, has been described where possible or available and often equations for deterministic use are given where some safety has been taken into account. Still it can be concluded that overtopping rates calculated by empirically derived equations, should only be regarded as being within, at best, a factor of 1–3 of the actual overtopping rate. The largest deviations will be found for small overtopping discharges.

As however many practical structures depart (at least in part) from the idealised versions tested in hydraulics laboratories, and it is known that overtopping rates may be very sensitive to small variations in structure geometry, local bathymetry and wave climate, empirical methods based upon model tests conducted on generic structural types, such as vertical walls, armoured slopes etc may lead to large differences in overtopping performance. Methods presented here will not predict overtopping performance with the same degree of accuracy as structure-specific model tests.

This manual is not however intended to cover many other aspects of the analysis, design, construction or management of sea defences for which other manuals and methods already exist, see for example CIRIA/CUR (1991), BSI (1991), SIMM et al. (1996), BRAMPTON et al. (2002) and TAW guidelines in the Netherlands on design of sea, river and lake dikes. The manual has been kept deliberately concise in order to maintain clarity and brevity. For the interested reader a full set of references is given so that the reasoning behind the development of the recommended methods can be followed.

1.3 Principal types of structures

Wave overtopping is of principal concern for structures constructed primarily to defend against flooding: often termed sea defence. Somewhat similar structures may also be used to provide protection against coastal erosion: sometimes termed coast protection. Other structures may be built to protect areas of water for ship navigation or mooring: ports, harbours or marinas; these are often formed as breakwaters or moles. Whilst some of these types of structures may be detached from the shoreline, sometimes termed offshore, nearshore or detached, most of the structures used for sea defence form a part of the shoreline.

This manual is primarily concerned with the three principal types of sea defence structures: sloping sea dikes and embankment seawalls; armoured rubble slopes and mounds; and vertical, battered or steep walls.

Historically, sloping dikes have been the most widely used option for sea defences along the coasts of the Netherlands, Denmark, Germany and many parts of the UK. Dikes or embankment seawalls have been built along many Dutch, Danish or German coastlines protecting the land behind from flooding, and sometimes providing additional amenity value. Similar such structures in UK may alternatively be formed by clay materials or from a vegetated shingle ridge, in both instances allowing the side slopes to be steeper. All such embankments will need some degree of protection against direct wave erosion, generally using a revetment facing on the seaward side. Revetment facing may take many forms, but may commonly include closely-fitted concrete blockwork, cast in-situ concrete slabs, or asphaltic materials. Embankment or dike structures are generally most common along rural frontages.

A second type of coastal structure consists of a mound or layers of quarried rock fill, protected by rock or concrete armour units. The outer armour layer is designed to resist wave action without significant displacement of armour units. Under-layers of quarry or crushed rock support the armour and separate it from finer material in the embankment or mound. These porous and sloping layers dissipate a proportion of the incident wave energy in breaking and friction. Simplified forms of rubble mounds may be used for rubble seawalls or protection to vertical walls or revetments. Rubble mound revetments may also be used to protect embankments formed from relict sand dunes or shingle ridges. Rubble mound structures tend to be more common in areas where harder rock is available.

Along urban frontages, especially close to ports, erosion or flooding defence structures may include vertical (or battered/steep) walls. Such walls may be composed of stone or concrete blocks, mass concrete, or sheet steel piles. Typical vertical seawall structures may also act as retaining walls to material behind. Shaped and recurved wave return walls may be formed as walls in their own right, or smaller versions may be included in sloping structures. Some coastal structures are relatively impermeable to wave action. These include seawalls formed from blockwork or mass concrete, with vertical, near vertical, or steeply sloping faces. Such structures may be liable to intense local wave impact pressures, may overtop suddenly and severely, and will reflect much of the incident wave energy. Reflected waves cause additional wave disturbance and/or may initiate or accelerate local bed scour.

1.4 Definitions of key parameters and principal responses

Overtopping discharge occurs because of waves running up the face of a seawall. If wave run-up levels are high enough water will reach and pass over the crest of the wall. This defines the 'green water' overtopping case where a continuous sheet of water passes over the crest. In cases where the structure is vertical, the wave may impact against the wall and send a vertical plume of water over the crest.

A second form of overtopping occurs when waves break on the seaward face of the structure and produce significant volumes of splash. These droplets may then be carried over the wall either under their own momentum or as a consequence of an onshore wind.

Another less important method by which water may be carried over the crest is in the form of spray generated by the action of wind on the wave crests immediately offshore of the wall. Even with strong wind the volume is not large and this spray will not contribute to any significant overtopping volume.

Overtopping rates predicted by the various empirical formulae described within this report will include green water discharges and splash, since both these parameters were recorded during the model tests on which the prediction methods are based. The effect of wind on this type of discharge will not have been modelled. Model tests suggest that onshore winds have little effect on large green water events, however they may increase discharges under 1 l/s/m. Under these conditions, the water overtopping the structure is mainly spray and therefore the wind is strong enough to blow water droplets inshore.

In the list of symbols, short definitions of the parameters used have been included. Some definitions are so important that they are explained separately in this section as key parameters. The definitions and validity limits are specifically concerned with application of the given formulae. In this way, a structure section with a slope of 1:12 is not considered as a real slope (too gentle) and it is not a real berm too (too steep). In such a situation, wave run-up and overtopping can only be calculated by interpolation. For example, for a section with a slope of 1:12, interpolation can be made between a slope of 1:8 (mildest slope) and a 1:15 berm (steepest berm).

1.4.1 Wave height

The wave height used in the wave run-up and overtopping formulae is the incident significant wave height H_{m0} at the toe of the structure, called the spectral wave height, $H_{m0} = 4\,(m_0)^{½}$. Another definition of significant wave height is the average of the highest third of the waves, $H_{1/3}$. This wave height is, in principle, not used in this manual, unless formulae were derived on basis of it. In deep water, both definitions produce almost the same value, but situations in shallow water can lead to differences of 10–15 %.

In many cases, a foreshore is present on which waves can break and by which the significant wave height is reduced. There are models that in a relatively simple way can predict the reduction in energy from breaking of waves and thereby the accompanying wave height at the toe of the structure. The wave height must be calculated over the total spectrum including any long-wave energy present.

Based on the spectral significant wave height, it is reasonably simple to calculate a wave height distribution and accompanying significant wave height $H_{1/3}$ using the method of BATTJES and GROENENDIJK (2000).

1.4.2 Wave period

Various wave periods can be defined for a wave spectrum or wave record. Conventional wave periods are the peak period T_p (the period that gives the peak of the spectrum), the average period T_m (calculated from the spectrum or from the wave record) and the significant period $T_{1/3}$ (the average of the highest 1/3 of the waves). The relationship T_p/T_m usually lies between 1.1 and 1.25, and T_p and $T_{1/3}$ are almost identical.

The wave period used for some wave run-up and overtopping formulae is the spectral period $T_{m-1.0}$ (= m_{-1}/m_0). This period gives more weight to the longer periods in the spectrum than an average period and, independent of the type of spectrum, gives similar wave run-up or overtopping for the same values of $T_{m-1,0}$ and the same wave heights. In this way, wave run-up and overtopping can be easily determined for double-peaked and ‚flattened' spectra, without the need for other difficult procedures. Vertical and steep seawalls often use the $T_{m0,1}$ or T_m wave period.

In the case of a uniform (single peaked) spectrum there is a fixed relationship between the spectral period $T_{m-1.0}$ and the peak period. In this report a conversion factor ($T_p = 1.1\ T_{m-1.0}$) is given for the case where the peak period is known or has been determined, but not the spectral period.

1.4.3 Wave steepness and Breaker parameter

Wave steepness is defined as the ratio of wave height to wavelength (e.g. $s_0 = H_{m0}/L_0$). This will tell us something about the wave's history and characteristics. Generally a steepness of $s_0 = 0.01$ indicates a typical swell sea and a steepness of $s_0 = 0.04$ to 0.06 a typical wind sea. Swell seas will often be associated with long period waves, where it is the period that becomes the main parameter that affects overtopping.

But also wind seas may became seas with low wave steepness if the waves break on a gentle foreshore. By wave breaking the wave period does not change much, but the wave height decreases. This leads to a lower wave steepness. A low wave steepness on relatively deep water means swell waves, but for depth limited locations it often means broken waves on a (gentle) foreshore.

The breaker parameter, surf similarity or Iribarren number is defined as $\xi_{m-1,0} = \tan\alpha/(H_{m0}/L_{m-1,0})^{1/2}$, where α is the slope of the front face of the structure and $L_{m-1,0}$ being the deep water wave length $gT^2_{m-1,0}/2\pi$. The combination of structure slope and wave steepness gives a certain type of wave breaking, see Fig. 1.1. 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, see Fig. 1.2. Plunging waves break with steep and overhanging fronts and the wave tongue will hit the structure or back washing water; an example is shown in Fig. 1.3. The transition between plunging waves and surging waves is known as collapsing. The wave front 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.

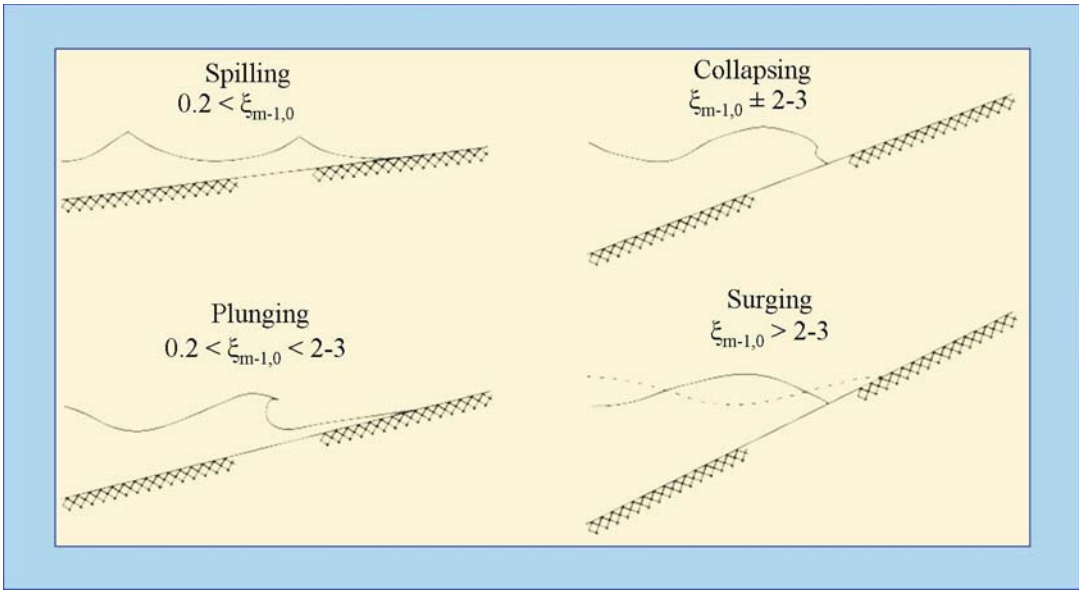

Fig. 1.1: Type of breaking on a slope

Fig. 1.2: Spilling waves on a beach; $\xi_{m-1,0} < 0.2$

Fig. 1.3: Plunging waves; $\xi_{m-1,0} < 2.0$

1.4.4 Parameter h*

In order to distinguish between non-impulsive (previously referred to as pulsating) waves on a vertical structure and impulsive (previously referred to as impacting) waves, the parameter h* has been defined.

$$h_* = \frac{h_s}{H_s}\frac{h_s}{L_o} \qquad 1.1$$

The parameter describes two ratios together, the wave height and wave length, both made relative to the local water depth h_s. Non-impulsive waves predominate when $h_* > 0.3$; impulsive waves when $h_* \leq 0.3$. Formulae for impulsive overtopping on vertical structures, originally used this h* parameter to some power, both for the dimensionless wave overtopping and dimensionless crest freeboard.

1.4.5 Toe of structure

In most cases, it is clear where the toe of the structure lies, and that is where the foreshore meets the front slope of the structure or the toe structure in front of it. For vertical walls, it will be at the base of the principal wall, or if present, at the rubble mound toe in front of it. It is possible that a sandy foreshore varies with season and even under severe wave attack. Toe levels may therefore vary during a storm, with maximum levels of erosion occurring during the peak of the tidal/surge cycle. It may therefore be necessary to consider the effects of increased wave heights due to the increase in the toe depth. The wave height that is always used in wave overtopping calculations is the incident wave height at the toe.

1.4.6 Foreshore

The foreshore is the section in front of the dike and can be horizontal or up to a maximum slope of 1:10. The foreshore can be deep, shallow or very shallow. If the water is shallow or very shallow then shoaling and depth limiting effects will need to be considered so that the wave height at the toe; or end of the foreshore; can be considered. A foreshore is defined as having a minimum length of one wavelength L_o. In cases where a foreshore lies in very shallow depths and is relatively short, then the methods outlined in Section 5.3.4 should be used.

A precise transition from a shallow to a very shallow foreshore is hard to give. At a shallow foreshore waves break and the wave height decreases, but the wave spectrum will retain more or less the shape of the incident wave spectrum. At very shallow foreshores the spectral shape changes drastically and hardly any peak can be detected (flat spectrum). As the waves become very small due to breaking many different wave periods arise.

Generally speaking, the transition between shallow and very shallow foreshores can be indicated as the situation where the original incident wave height, due to breaking, has been decreased by 50 % or more. The wave height at a structure on a very shallow foreshore is much smaller than in deep water situations. This means that the wave steepness (Section 1.4.3) becomes much smaller, too. Consequently, the breaker parameter, which is used in the formulae for wave run-up and wave overtopping, becomes much larger. Values of ξ_0 = 4 to 10 for the breaker parameter are then possible, where maximum values for a dike of 1:3 or 1:4 are normally smaller than say $\xi_0 = 2$ or 3.

Another possible way to look at the transition from shallow to very shallow foreshores, is to consider the breaker parameter. If the value of this parameter exceeds 5–7, or if they are swell waves, then a very shallow foreshore is present. In this way, no knowledge about wave heights at deeper water is required to distinguish between shallow and very shallow foreshores.

1.4.7 Slope

Part of a structure profile is defined as a slope if the slope of that part lies between 1:1 and 1:8. These limits are also valid for an average slope, which is the slope that occurs when a line is drawn between $-1.5\ H_{m0}$ and $+Ru_{2\%}$ in relation to the still water line and berms are not included. A continuous slope with a slope between 1:8 and 1:10 can be calculated in the first instance using the formulae for simple slopes, but the reliability is less than for steeper slopes. In this case interpolation between a slope 1:8 and a berm 1:15 is not possible.

A structure slope steeper than 1:1, but not vertical, can be considered as a battered wall. These are treated in Chapter 7 as a complete structure. If it is only a wave wall on top of gentle sloping dike, it is treated in Chapter 5.

1.4.8 Berm

A berm is part of a structure profile in which the slope varies between horizontal and 1:15. The position of the berm in relation to the still water line is determined by the depth, dh, the vertical distance between the middle of the berm and the still water line. The width of a berm, B, may not be greater than one-quarter of a wavelength, i.e., $B < 0.25\ L_o$. If the width is greater, then the structure part is considered between that of a berm and a foreshore, and wave run-up and overtopping can be calculated by interpolation. Section 5.3.4 gives a more detailed description.

1.4.9 Crest freeboard and armour freeboard and width

The crest height of a structure is defined as the crest freeboard, R_c, and has to be used for wave overtopping calculations. It is actually the point on the structure where overtopping water can no longer flow back to the seaside. The height (freeboard) is related to SWL. For rubble mound structures, it is often the top of a crest element and not the height of the rubble mound armour.

The armour freeboard, A_c, is the height of a horizontal part of the crest, measured relative to SWL. The horizontal part of the crest is called G_c. For rubble mound slopes the armour freeboard, A_c, may be higher, equal or sometimes lower than the crest freeboard, R_c, Fig. 1.4.

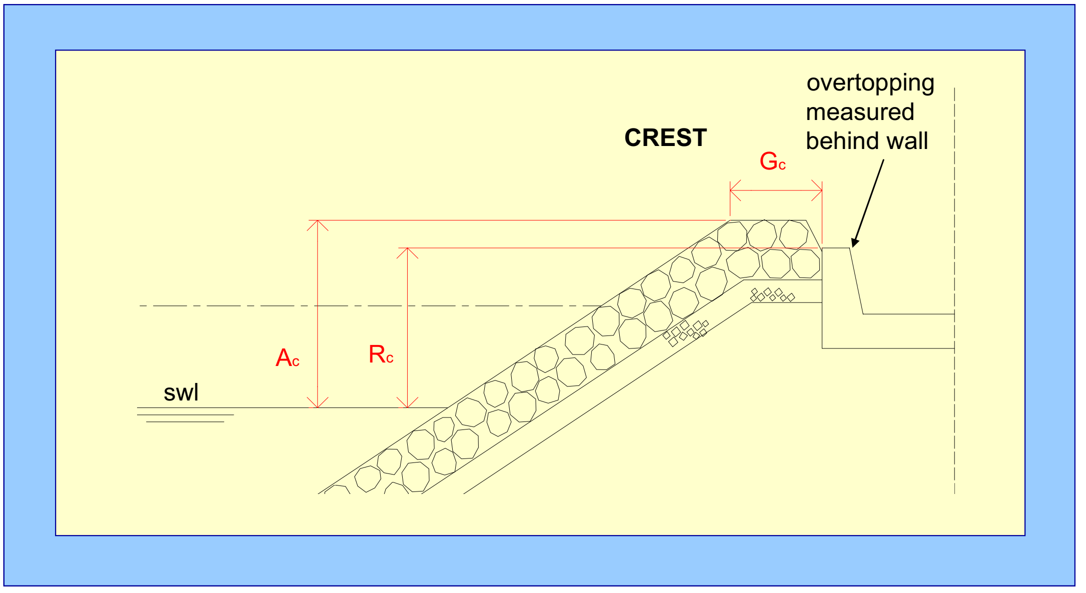

Fig. 1.4: Crest freeboard different from armour freeboard

The crest height that must be taken into account during calculations for wave overtopping for an upper slope with quarry stone, but without a wave wall, is not the upper side of this quarry stone, A_c. The quarry stone armour layer is itself completely water permeable, so that the under side must be used instead, see Fig. 1.5. In fact, the height of a non or only slightly water-permeable layer determines the crest freeboard, R_c, in this case for calculations of wave overtopping.

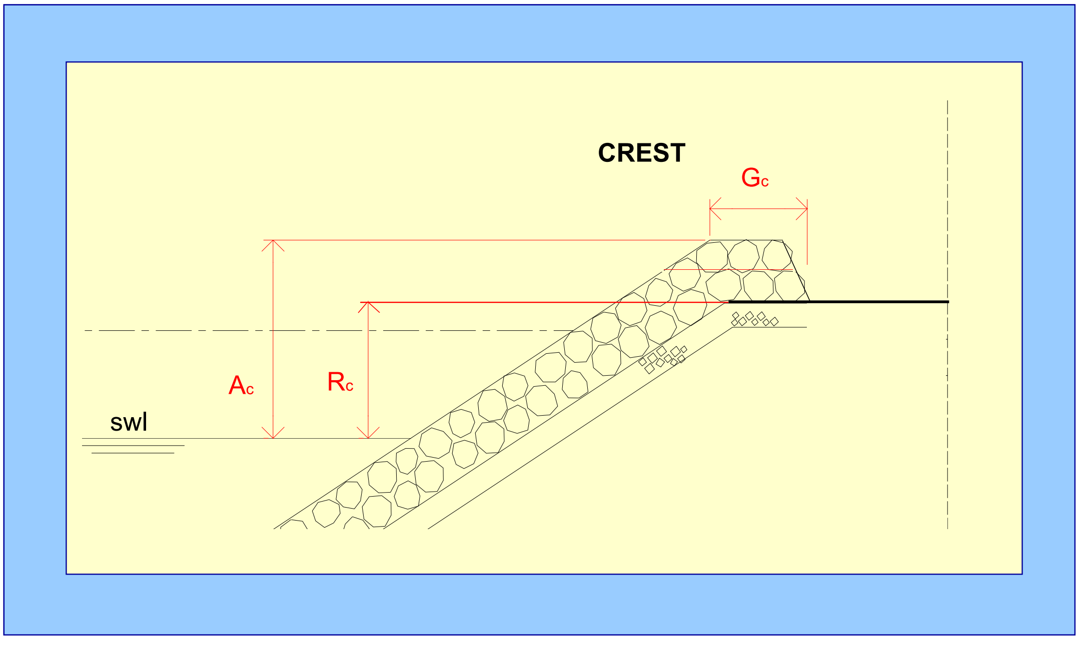

Fig. 1.5: Crest freeboard ignores a permeable layer if no crest element is present

The crest of a dike, especially if a road runs along it, is in many cases not completely horizontal, but slightly rounded and of a certain width. The crest height at a dike or embankment, R_c, is defined as the height of the outer crest line (transition from outer slope to crest). This definition therefore is used for wave run-up and overtopping. In principle the width of the crest and the height of the middle of the crest have no influence on calculations for wave overtopping, which also means that $R_c = A_c$ is assumed and that $G_c = 0$. Of course, the width of the crest, if it is very wide, can have an influence on the actual wave overtopping.

If an impermeable slope or a vertical wall have a horizontal crest with at the rear a wave wall, then the height of the wave wall determines R_c and the height of the horizontal part determines A_c, see Fig. 1.6.

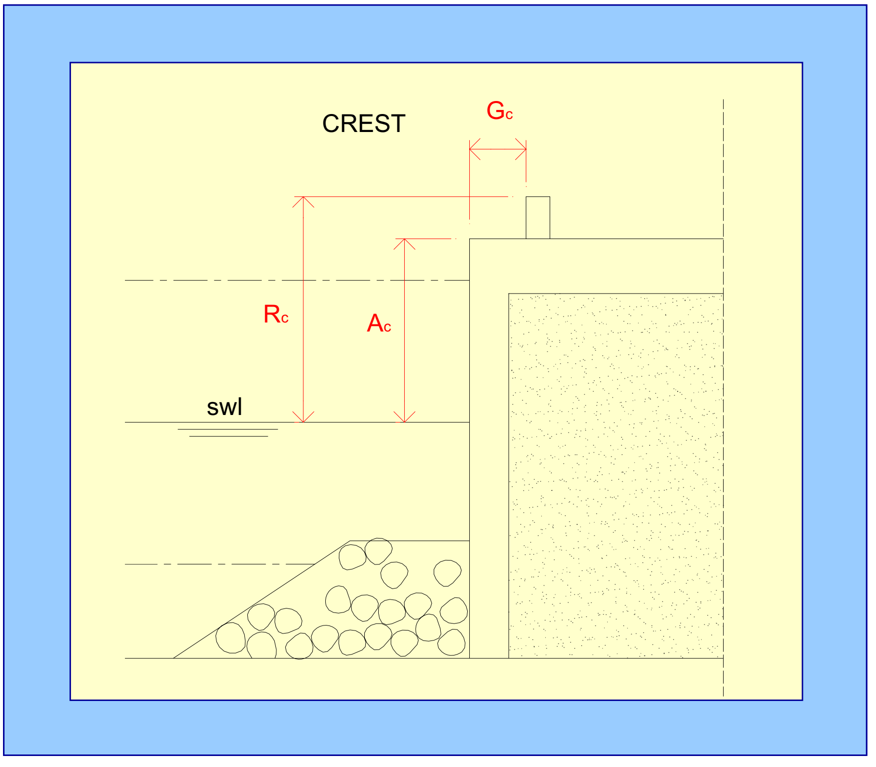

Fig. 1.6: Crest configuration for a vertical wall

1.4.10 Permeability, porosity and roughness

A smooth structure like a dike or embankment is mostly impermeable for water or waves and the slope has no, or almost no roughness. Examples are embankments covered with a placed block revetment, an asphalt or concrete slope and a grass cover on clay. Roughness on the slope will dissipate wave energy during wave run-up and will therefore reduce wave overtopping. Roughness is created by irregularly shaped block revetments or artificial ribs or blocks on a smooth slope.

A rubble mound slope with rock or concrete armour is also rough and in general more rough than roughness on impermeable dikes or embankments. But there is another difference, as the permeability and porosity is much larger for a rubble mound structure. Porosity is defined as the percentage of voids between the units or particles. Actually, loose materials always have some porosity. For rock and concrete armour the porosity may range roughly between 30 %–55 %. But also sand has a comparable porosity. Still the behaviour of waves on a sand beach or a rubble mound slope is different.

This difference is caused by the difference in permeability. The armour of rubble mound slopes is very permeable and waves will easily penetrate between the armour units and dissipate energy. But this becomes more difficult for the under layer and certainly for the core of the structure. Difference is made between "impermeable under layers or core" and a "permeable core". In both cases the same armour layer is present, but the structure and under layers differ.

A rubble mound breakwater often has an under layer of large rock (about one tenth of the weight of the armour), sometimes a second under layer of smaller rock and then the core of still smaller rock. Up-rushing waves can penetrate into the armour layer and will then sink into the under layers and core. This is called a structure with a "permeable core".

An embankment can also be covered by an armour layer of rock. The under layer is often small and thin and placed on a geotextile. Underneath the geotextile sand or clay may be present, which is impermeable for up-rushing waves. Such an embankment covered with rock has an "impermeable core". Run-up and wave overtopping are dependent on the permeability of the core.

In summary the following types of structures can be described:

Smooth dikes and embankments:	smooth and impermeable
Dikes and embankments with rough slopes:	some roughness and impermeable
Rock cover on an embankment:	rough with impermeable core
Rubble mound breakwater:	rough with permeable core

1.4.11 Wave run-up height

The wave run-up height is given by $R_{u2\%}$. This is the wave run-up level, measured vertically from the still water line, which is exceeded by 2 % of the number of incident waves. The number of waves exceeding this level is hereby related to the number of incoming waves and not to the number that run-up.

A very thin water layer in a run-up tongue cannot be measured accurately. In model studies on smooth slopes the limit is often reached at a water layer thickness of 2 mm. For prototype waves this means a layer depth of about 2 cm, depending on the scale in relation to the model study. Very thin layers on a smooth slope can be blown a long way up the slope by a strong wind, a condition that can also not be simulated in a small scale model. Running-up water tongues less than 2 cm thickness actually contain very little water. Therefore it is suggested that the wave run-up level on smooth slopes is determined by the level at which the water tongue becomes less than 2 cm thick. Thin layers blown onto the slope are not seen as wave run-up.

Run-up is relevant for smooth slopes and embankments and sometimes for rough slopes armoured with rock or concrete armour. Wave run-up is not an issue for vertical structures. The percentage or number of overtopping waves, however, is relevant for each type of structure.

1.4.12 Wave overtopping discharge

Wave overtopping is the mean discharge per linear meter of width, q, for example in $m^3/s/m$ or in l/s/m. The methods described in this manual calculate all overtopping discharges in $m^3/s/m$ unless otherwise stated; it is, however, often more convenient to multiply by 1000 and quote the discharge in l/s/m.

In reality, there is no constant discharge over the crest of a structure during overtopping. The process of wave overtopping is very random in time and volume. The highest waves will push a large amount of water over the crest in a short period of time, less than a wave period. Lower waves will not produce any overtopping. An example of wave overtopping measurements is shown in Fig. 1.7. The graphs shows 200 s of measurements. The lowest graph (flow depths) clearly shows the irregularity of wave overtopping. The upper graph gives the cumulative overtopping as it was measured in the overtopping tank. Individual overtopping volumes can be distinguished, unless a few overtopping waves come in one wave group.

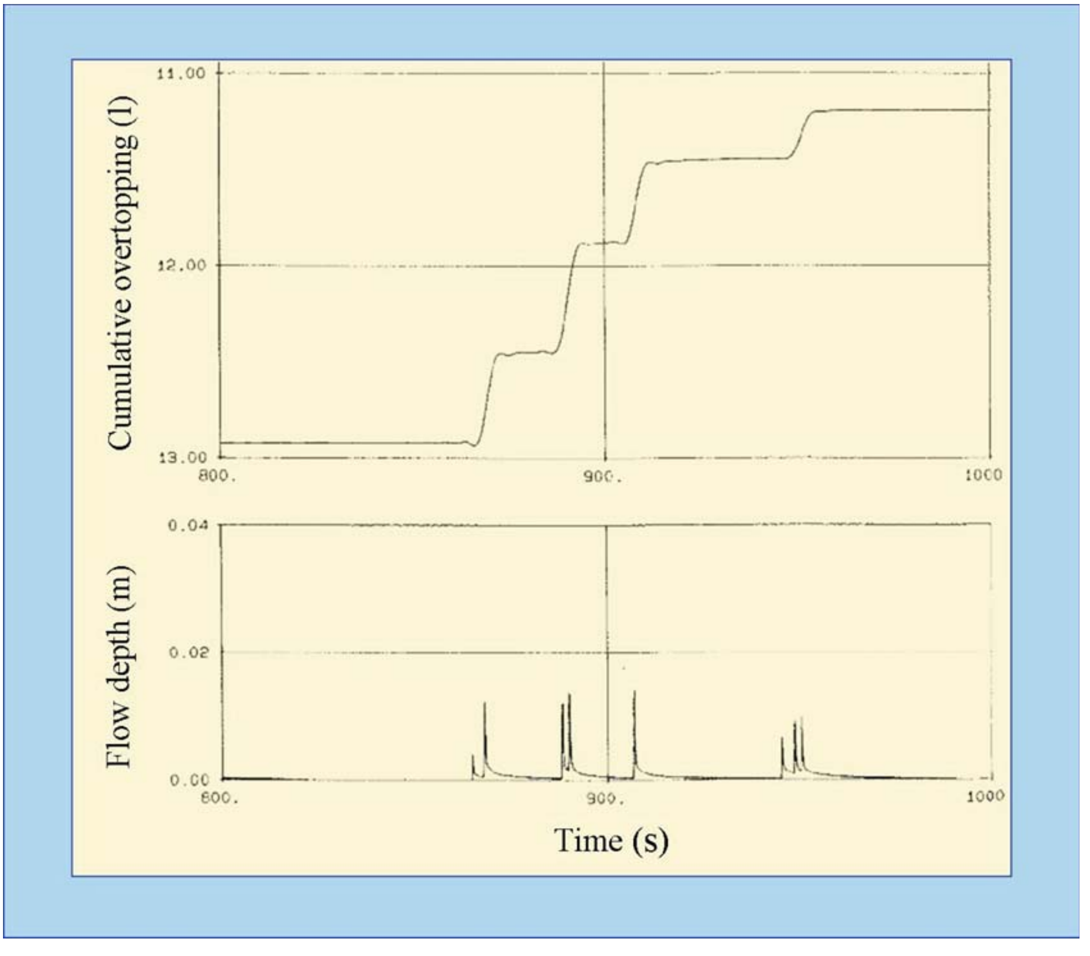

Fig. 1.7: Example of wave overtopping measurements, showing the random behaviour

Still a mean overtopping discharge is widely used as it can easily be measured and also classified:

$q < 0.1$ l/s per m:	Insignificant with respect to strength of crest and rear of structure.
$q = 1$ l/s per m:	On crest and inner slopes grass and/or clay may start to erode.
$q = 10$ l/s per m:	Significant overtopping for dikes and embankments. Some overtopping for rubble mound breakwaters.
$q = 100$ l/s per m:	Crest and inner slopes of dikes have to be protected by asphalt or concrete; for rubble mound breakwaters transmitted waves may be generated.

1.4.13 Wave overtopping volumes

A mean overtopping discharge does not yet describe how many waves will overtop and how much water will be overtopped in each wave. The volume of water, V, that comes over the crest of a structure is given in m³ per wave per m width. Generally, most of the overtopping waves are fairly small, but a small number gives significantly larger overtopping volumes.

The maximum volume overtopped in a sea state depends on the mean discharge q, on the storm duration and the percentage of overtopping waves. In this report, a method is given by which the distribution of overtopping volumes can be calculated for each wave. A longer storm duration gives more overtopping waves, but statistically, also a larger maximum volume. Many small overtopping waves (like for river dikes or embankments) may create the same mean overtopping discharge as a few large waves for rough sea conditions. The maximum volume will, however, be much larger for rough sea conditions with large waves.

1.5 Probability levels and uncertainties

This section will briefly introduce the concept of uncertainties and how it will be dealt with in this manual. It will start with a basic definition of uncertainty and return period. After that the various types of uncertainties are explained and more detailed descriptions of parameters and model uncertainties used in this manual will be described.

1.5.1 Definitions

Uncertainty may be defined as the relative variation in parameters or error in the model description so that there is no single value describing this parameter but a range of possible values. Due to the random nature of many of those variables used in coastal engineering most of the parameters should not be treated deterministically but stochastically. The latter assumes that a parameter x shows different realisations out of a range of possible values. Hence, uncertainty may be defined as a statistical distribution of the parameter. If a normal distribution is assumed here uncertainty may also be given as relative error, mathematically expressed as the coefficient of variation of a certain parameter x:

$$\sigma'_x = \frac{\sigma_x}{\mu_x} \tag{1.2}$$

where σ_x is the standard deviation of the parameter and μ_x is the mean value of that parameter. Although this definition may be regarded as imperfect it has some practical value and is easily applied.

The return period of a parameter is defined as the period of time in which the parameter occurs again on average. Therefore, it is the inverse of the probability of occurrence of this parameter. If the return period T_R of a certain wave height is given, it means that this specific wave height will only occur once in T_R years on average.

It should be remembered that there will not be exactly T_R years between events with a given return period of T_R years. If the events are statistically independent then the probability that a condition with a return period of T_R years will occur within a period of L years is given by $p = 1-(1-1/nT_R)^{nL}$, where n is the number of events per year, e.g., 2920 storms of three hours duration. Hence, for an event with a return period of 100 years there is a 1 % chance of recurrence in any one year. For a time interval equal to the return period, $p = 1-(1-1/nTr)^{nTr}$ or $p \approx 1 - 1/e = 0.63$. Therefore, there is a 63 % chance of occurrence within the return period. Further information on design events and return periods can be found in the British Standard Code of practice for Maritime Structures (BS6349 Part 1 1984 and Part 7 1991) or the PIANC working group 12 report (PIANC, 1992). Also refer to Section 2.6.

1.5.2 Background

Many parameters used in engineering models are uncertain, and so are the models themselves. The uncertainties of input parameters and models generally fall into certain categories; as summarised in Fig. 1.8.

- Fundamental or statistical uncertainties: elemental, inherent uncertainties, which are conditioned by random processes of nature and which can not be diminished (always comprised in measured data)
- Data uncertainty: measurement errors, inhomogeneity of data, errors during data handling, non-representative reproduction of measurement due to inadequate temporal and spatial resolution
- Model uncertainty: coverage of inadequate reproduction of physical processes in nature
- Human errors: all of the errors during production, abrasion, maintenance as well as other human mistakes which are not covered by the model. These errors are not considered in the following, due to the fact that in general they are specific to the problems and no universal approaches are available.

If Normal or Gaussian Distributions for x are used 68.3 % of all values of x are within the range of $\mu_x(1 \pm \sigma_x)$, 95.5 % of all values within the range of $\mu_x \pm 2\sigma_x$ and almost all values (97,7 %) within the range of $\mu_x \pm 3\sigma_x$, see Fig. 1.9. Considering uncertainties in a design, therefore, means that all input parameters are no longer regarded as fixed deterministic parameters but can be any realisation of the specific parameter. This has two consequences: Firstly, the parameters have to be checked whether all realisations of this parameter are really physically sound: E.g., a realisation of a normally distributed wave height can mathematically become negative which is physically impossible. Secondly, parameters have to be checked against realisations of other parameters: E.g., a wave of a certain height can only exist in certain water depths and not all combinations of wave heights and wave periods can physically exist.

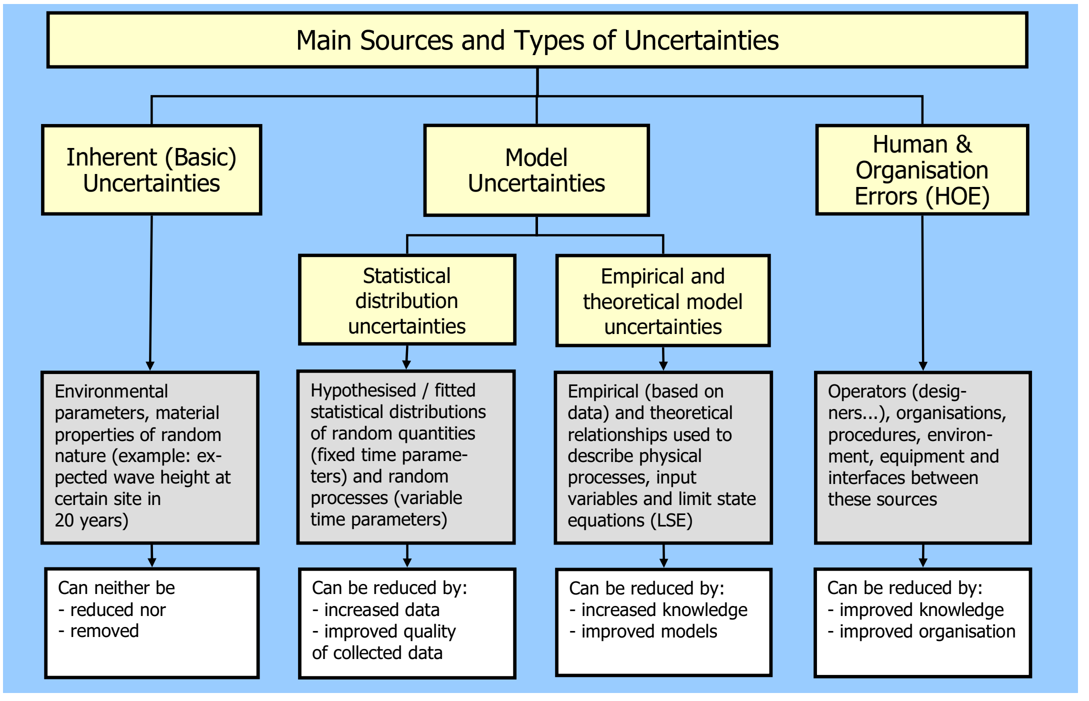

Fig. 1.8: Sources of uncertainties

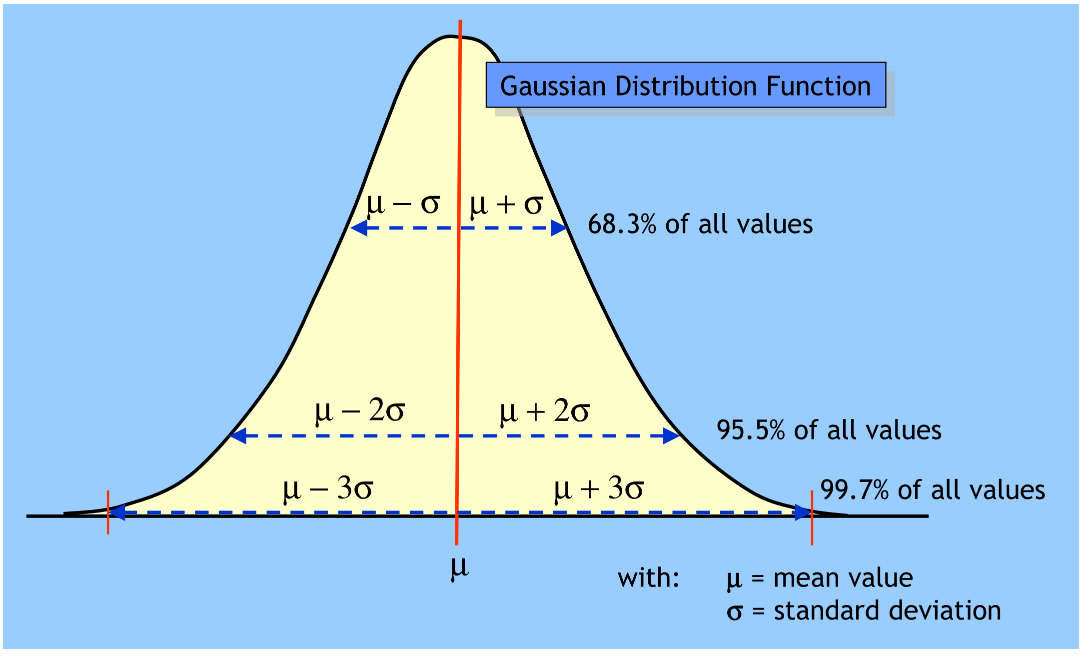

Fig. 1.9: Gaussian distribution function and variation of parameters

In designing with uncertainties this means that statistical distributions for most of the parameters have to be selected extremely carefully. Furthermore, physical relations between parameters have to be respected. This will be discussed in the subsequent sections as well.

1.5.3 Parameter uncertainty

The uncertainty of input parameters describes the inaccuracy of these parameters, either from measurements of those or from their inherent uncertainties. As previously discussed, this uncertainty will be described using statistical distributions or relative variation of these parameters. Relative variation for most of the parameters will be taken from various sources such as: measurement errors observed; expert opinions derived from questionnaires; errors reported in literature.

Uncertainties of parameters will be discussed in the subsections of each of the following chapters discussing various methods to predict wave overtopping of coastal structures. Any physical relations between parameters will be discussed and restrictions for assessing the uncertainties will be proposed.

1.5.4 Model uncertainty

The model uncertainty is considered as the accuracy, with which a model or method can describe a physical process or a limit state function. Therefore, the model uncertainty describes the deviation of the prediction from the measured data due to this method. Difficulties of this definition arise from the combination of parameter uncertainty and model uncertainty. Differences between predictions and data observations may result from either uncertainties of the input parameters or model uncertainty.

Model uncertainties may be described using the same approach than for parameter uncertainties using a multiplicative approach. This means that

$$q = m \cdot f(x_i) \qquad 1.3$$

where m is the model factor [–]; q is the overtopping ratio and f(x) is the model used for prediction of overtopping. The model factor m is assumed to be normally distributed with a mean value of 1.0 and a coefficient of variation specifically derived for the model.

These model factors may easily reach coefficients of variations up to 30 %. It should be noted that a mean value of m = 1.0 always means that there is no bias in the models used. Any systematic error needs to be adjusted by the model itself. For example, if there is an over-prediction of a specific model by 20 % the model has to be adjusted to predict 20 % lower results. This concept is followed in all further chapters of this manual so that from here onwards, the term 'model uncertainties' is used to describe the coefficient of variation σ', assuming that the mean value is always 1.0. The procedure to account for the model uncertainties is given in section 4.9.1.

Model uncertainties will be more widely discussed in the subsections of each chapter describing the models. The subsections will also give details on how the uncertain results of the specific model may be interpreted.

1.5.5 Methodology and output

All parameter and model uncertainties as discussed in the previous sections are used to run the models proposed in this manual. Results of all models will again follow statistical distributions rather than being single deterministic values. Hence, interpretation of these results is required and recommendations will be given on how to use outputs of the models.

Key models for overtopping will be calculated using all uncertainties and applying a Monte-Carlo-simulation (MCS). Statistical distributions of outputs will be classified with regard to exceedance probabilities such as: very safe, where output is only exceeded by 2 % of all results, corresponding to a return period of 50 years which means that the structure is expected to be overtopped only once during its lifetime of 50 years; safe, where output is exceeded by 10 % of all results, corresponding to a return period of 10 years; medium safe, where output corresponds to mean values plus one standard deviation; and probabilistic, where output is exceeded by 50 % of all results and may be used for probabilistic calculations.