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Water levels and wave conditions

Die Küste

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Verfügbar unter/Available at: https://hdl.handle.net/20.500.11970/101582

Vorgeschlagene Zitierweise/Suggested citation:
Schüttrumpf, Holger; Kortenhaus, Andreas; Pullen, Tim; Allsop, William; Bruce, Tom; Van der Meer, Jentsje W. (2007): Water levels and wave conditions. In: Die Küste 73. Heide, Holstein: Boyens. S. 18-28.

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Die Küste, 73 EurOtop (2007), 18-28

2. Water levels and wave conditions

2.1 Introduction

This Overtopping Manual has a focus on the aspects of wave run-up and wave overtopping only. It is not a design manual, giving the whole design process of a structure. This chapter, therefore, will not provide a guide to the derivation of input conditions other than to identify the key activities in deriving water level and wave conditions, and particularly depth-limited wave conditions. It identifies the key parameters and provides a check-list of key processes and transformations. Comprehensive references are given to appropriate sources of information. Brief descriptions of methods are sometimes given, summary details of appropriate tools and models, and cross references to other manuals.

The main manuals and guidelines, which describe the whole design process of coastal and inland structures, including water levels and wave conditions are: The Rock Manual (1991), recently replaced by the updated Rock Manual (2007); The Coastal Engineering Manual; The British Standards; The German "Die Küste" (2002) and the DELOS Design Guidelines (2007).

2.2 Water levels, tides, surges and sea level changes

Prediction of water levels is extremely important for prediction of wave run-up levels or wave overtopping, which are often used to design the required crest level of a flood defence structure or breakwater. Moreover, in shallow areas the extreme water level often determines the water depth and thereby the upper limit for wave heights.

Extreme water levels in design or assessment of structures may have the following components: the mean sea level; the astronomical tide; surges related to (extreme) weather conditions; and high river discharges

2.2.1 Mean sea level

For coastal waters in open communication with the sea, the mean water level can often effectively be taken as a site-specific constant, being related to the mean sea level of the oceans. For safety assessments, not looking further ahead than about 5 years, the actual mean water level can be taken as a constant. Due to expected global warming, however, predictions in sea level rise for the next hundred years range roughly from 0.2 m to more than 1.0 m.

For design of structures, which last a long time after their design and construction phase, a certain sea level rise has to be included. Sometimes countries prescribe a certain sea level rise, which has to be taken into account when designing flood defence structures. Also the return period to include sea level rise may differ, due to the possibility of modification in future. An earthen dike is easy to increase in height and a predicted sea level rise for the next 50 years would be sufficient. A dedicated flood defence structure through a city is not easy to modify or replace. In such a situation a predicted sea level rise for the next 100 years or more could be considered.

2.2.2 Astronomical tide

The basic driving forces of tidal movements are astronomical and therefore entirely predictable, which enables accurate prediction of tidal levels (and currents). Around the UK and North Sea coast, and indeed around much of the world, the largest fluctuations in water level are caused by astronomical tides. These are caused by the relative rotation of both the sun and the moon around the earth each day. The differential gravitational effects over the surface of the oceans cause tides with well defined periods, principally semi-diurnal and diurnal. Around the British Isles and along coasts around the North Sea the semi-diurnal tides are much larger than the diurnal components.

In addition to the tides that result from the earth‘s rotation, other periodicities are apparent in the fluctuation of tidal levels. The most obvious is the fortnightly spring-neap cycle, corresponding to the half period of the lunar cycle.

Further details on the generation of astronomic tides, and their dynamics, can be found in the Admiralty Manual of Tides in most countries. These give daily predictions of times of high and low waters at selected locations, such as ports. Also details of calculating the differences in level between different locations are provided. Unfortunately, in practice, the prediction of an extreme water level is made much more complicated by the effects of weather, as discussed below.

2.2.3 Surges related to extreme weather conditions

Generally speaking the difference between the level of highest astronomical tide and, say, the largest predicted tide in any year is rather small (i.e. a few centimetres). In practice, this difference is often unimportant, when compared with the differences between predicted and observed tidal levels due to weather effects.

Extreme high water levels are caused by a combination of high tidal elevations plus a positive surge, which usually comprise three main components. A barometric effect caused by a variation in atmospheric pressure from its mean value. A wind set-up; in shallow seas, such as a the English Channel or the North Sea, a strong wind can cause a noticeable rise in sea level within a few hours. A dynamic effect due to the amplification of surge-induced motions caused by the shape of the land (e.g. seiching and funnelling).

A fourth component, wave set-up causes an increase in water levels within the surf zone at a particular site due to waves breaking as they travel shoreward. Unlike the other three positive surge components, wave set-up has only an extremely localised effect on water levels. Wave set-up is implicitly reproduced in the physical model tests on which the overtopping equations are based. There is, therefore, no requirement to add on an additional water level increase for wave set-up when calculating overtopping discharges using the methods reported in this document.

Negative surges are made up of two principal components: a barometric effect caused by high atmospheric pressures and wind set-down caused by winds blowing offshore. Large positive surges are more frequent than large negative ones. This is because a depression causing a positive surge will tend to be more intense and associated with a more severe wind condition than anticyclones.

Surges in relatively large and shallow areas, like the southern part of the North Sea, play an important role in estimating extreme water levels. The surges may become several meters for large return periods. The easiest means of predicting extreme water levels is to analyse long term water level data from the site in question. However, where no such data exists, it may be necessary to predict surge levels using theoretical equations and combine these levels with tidal elevations in order to obtain an estimation of extreme water levels.

More than 100 years' of high water level measurements in the Netherlands is shown in Fig. 2.1 along with the extrapolation of the measurements to extreme low exceedance probabilities, such as 10^{-4} or only once in 10,000 years.

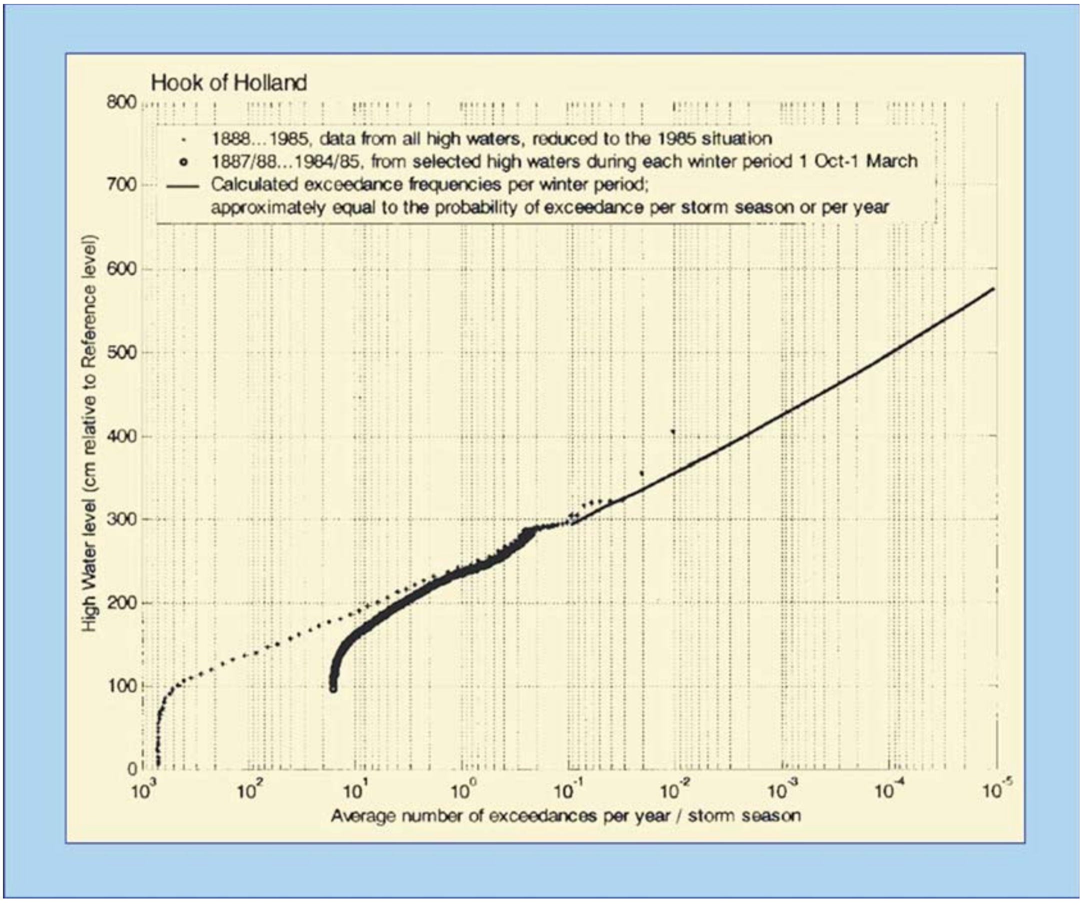

Fig. 2.1: Measurements of maximum water levels for more than 100 years and extrapolation to extreme return periods

2.2.4 High river discharges

Coastal flood defences face the sea or a (large) lake, but flood defences are also present along tidal rivers. Extreme river discharges determine the extreme water levels along river flood defences. During such an extreme water level, which may take a week or longer, a storm may generate waves on the river and cause overtopping of the flood defence. In many cases the required height of a river dike does not only depend on the extreme water level, but also on the possibility of wave overtopping. It should be noted that the occurrence of the extreme river discharge, and extreme water level, are independent of the occurrence of the storm. During high river discharges, only "normal" storms; occurring every decade; are considered, not the extreme storms.

Where rivers enter the sea both systems for extreme water levels may occur. Extreme storms may give extreme water levels, but also extreme river discharges. The effect of extreme storms and surges disappear farther upstream. Joint probabilistic calculations of both phenomena may give the right extreme water levels for design or safety assessment.

2.2.5 Effect on crest levels

During design or safety assessment of a dike, the crest height does not just depend on wave run-up or wave overtopping. Account must also be taken of a reference level, local sudden gusts and oscillations (leading to a corrected water level), settlement and an increase of the water level due to sea level rise.

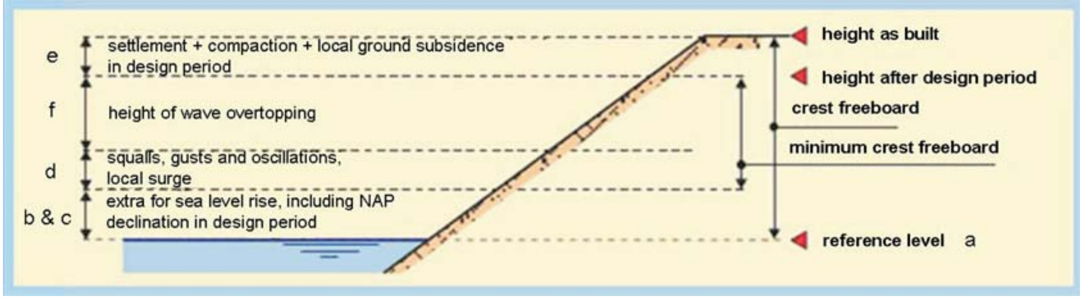

Fig. 2.2: Important aspects during calculation or assessment of dike height

The structure height of a dike in the Netherlands is composed of the following contributions; see also the Guidelines for Sea and Lake Dikes [TAW, 1999-2]:

a) the reference level with a probability of being exceeded corresponding to the legal standard (in the Netherlands this is a return period between 1,250 and 10,000 years;
b) the sea level rise or lake level increase during the design period;
c) the expected local ground subsidence during the design period;
d) an extra due to squalls, gusts, seiches and other local wind conditions;
e) the expected decrease in crest height due to settlement of the dike body and the foundation soils during the design period;
f) the wave run-up height and the wave overtopping height.

Contributions (a) to (d) cannot be influenced, whereas contribution (e) can be influenced. Contribution (f) also depends on the outer slope, which can consist of various materials, such as an asphalt layer, a cement-concrete dike covering (stone setting) or grass on a clay layer. A combination of these types is also possible. Slopes are not always straight, and the upper and lower sections may have different slopes and also a berm may be applied. The design of a covering layer is not dealt with in this report. However, the aspects related to berms, slopes and roughness elements are dealt with when they have an influence on wave run-up and wave overtopping.

2.3 Wave conditions

In defining the wave climate at the site, the ideal situation is to collect long term instrumentally measured data at the required location. There are very few instances in which this is even a remote possibility. The data of almost 30 years' of wave height measurements is shown in Fig. 2.3. These are the Dutch part of the North Sea with an extrapolation to very extreme events.

It is however more likely that data in deep water, offshore of a site will be available either through the use of a computational wave prediction model based on wind data, or on a wave model. In both of these cases the offshore data can be used in conjunction with a wave transformation model to provide information on wave climate at a coastal site. If instrumentally measured data is also available, covering a short period of time, this can be used for the calibration or verification of the wave transformation model, thus giving greater confidence in its use.

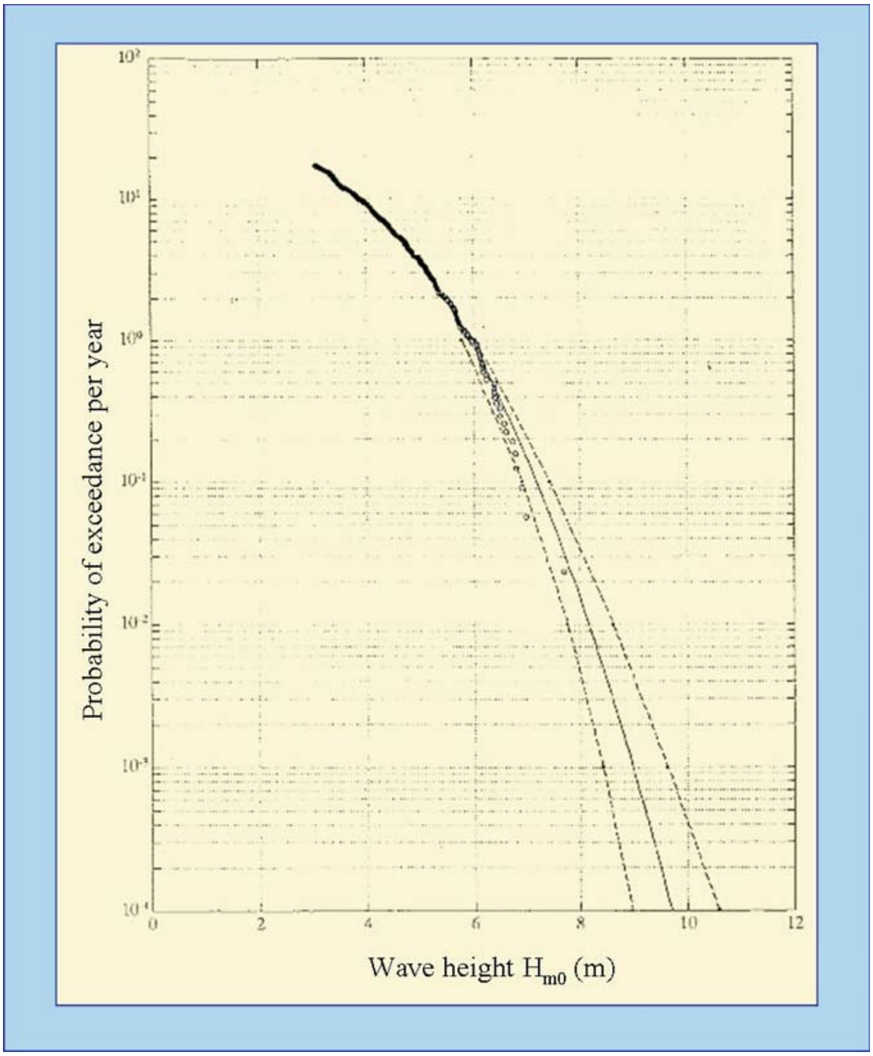

Fig. 2.3: Wave measurements and numerical simulations in the North Sea (1964–1993), leading to an extreme distribution

Wind generated waves offshore of most coasts have wave periods in the range 1s to 20s. The height, period and direction of the waves generated will depend on the wind speed, duration, direction and the 'fetch', i.e. the unobstructed distance of sea surface over which the wind has acted. In most situations, one of either the duration or fetch become relatively unimportant. For example, in an inland reservoir or lake, even a short storm will produce large wave heights. However, any increase in the duration of the wind will then cause no extra growth because of the small fetch lengths. Thus such waves are described as 'fetch limited'. In contrast, on an open coast where the fetch is very large but the wind blows for only a short period, the waves are limited by the duration of the storm. Beyond a certain limit, the exact fetch length becomes unimportant. These waves are described as 'duration limited'.

On oceanic shorelines the situation is usually more complicated. Both the fetch and duration may be extremely large, waves then become "fully developed" and their height depends solely on the wind speed. In such situations the wave period usually becomes quite large, and long period waves are able to travel great distances without suffering serious diminution. The arrival of 'swell', defined as waves not generated by local and/or recent wind conditions, presents a more challenging situation from the viewpoint of wave predictions.

2.4 Wave conditions at depth-limited situations

Wave breaking remains one phenomenon that is difficult to describe mathematically. One reason for this is that the physics of the process is not yet completely understood. However, as breaking has a significant effect on the behaviour of waves, the transport of sediments, the magnitude of forces on coastal structures and the overtopping response, it is represented in computational models. The most frequent method for doing this is to define an energy dissipation term which is used in the model when waves reach a limiting depth compared to their height.

There are also two relatively simple empirical methods for a first estimate of the incident wave conditions in the surf zone. The methods by GODA (1980) and OWEN (1980) are regularly used. GODA (1980) inshore wave conditions are influenced by shoaling and wave breaking. These processes are influenced by a number of parameters such as the sea steepness and the slope of the bathymetry. To take all the important parameters into account GODA (1980) provided a series of graphs to determine the largest and the significant wave heights (H_{max} and H_s) for 1:10, 1:20, 1:30 and 1:100 sloping bathymetries.

Results obtained from a simple 1D energy decay numerical model (VAN DER MEER, 1990) in which the influence of wave breaking is included, are presented in Fig. 2.4. This method has also been described in the Rock Manual (1991) and the updated version of this Rock Manual (2007). Tests have shown that wave height predictions using the design graphs from this model are accurate for slopes ranging from 1:10 to 1:100. For slopes flatter than 1:100, the predictions for the 1:100 slopes should be used.

The method for using these graphs is:

1. Determine the deep-water wave steepness, $s_{op} = H_{so}/L_{op}$ (where $L_{op} = gT_p^2/(2\pi)$). This value determines which graphs should be used. Suppose here for convenience that $s_{op} = 0.043$, then the graphs of Fig. 2.4 for $s_{op} = 0.04$ and 0.05 have to be used, interpolating between the results from each.
2. Determine the local relative water depth, h/L_{op}. The range of the curves in the graphs covers a decrease in wave height by 10 per cent to about 70 per cent. Limited breaking

occurs at the right hand side of the graphs and severe breaking on the left-hand side. If h/L_{op} is larger than the maximum value in the graph this means that there is no or only limited wave breaking and one can then assume no wave breaking (deep-water wave height = shallow-water wave height).

3. Determine the slope of the foreshore (m = tan α). Curves are given for range m = 0.075 to 0.01 (1:13 to 1:100). For gentler slopes the 1:100 slope should be used.
4. Enter the two selected graphs with calculated h/L_{op} and read the breaker index H_{m0}/h from the curve of the calculated foreshore slope.
5. Interpolate linearly between the two values of H_{m0}/h to find H_{m0}/h for the correct wave steepness.

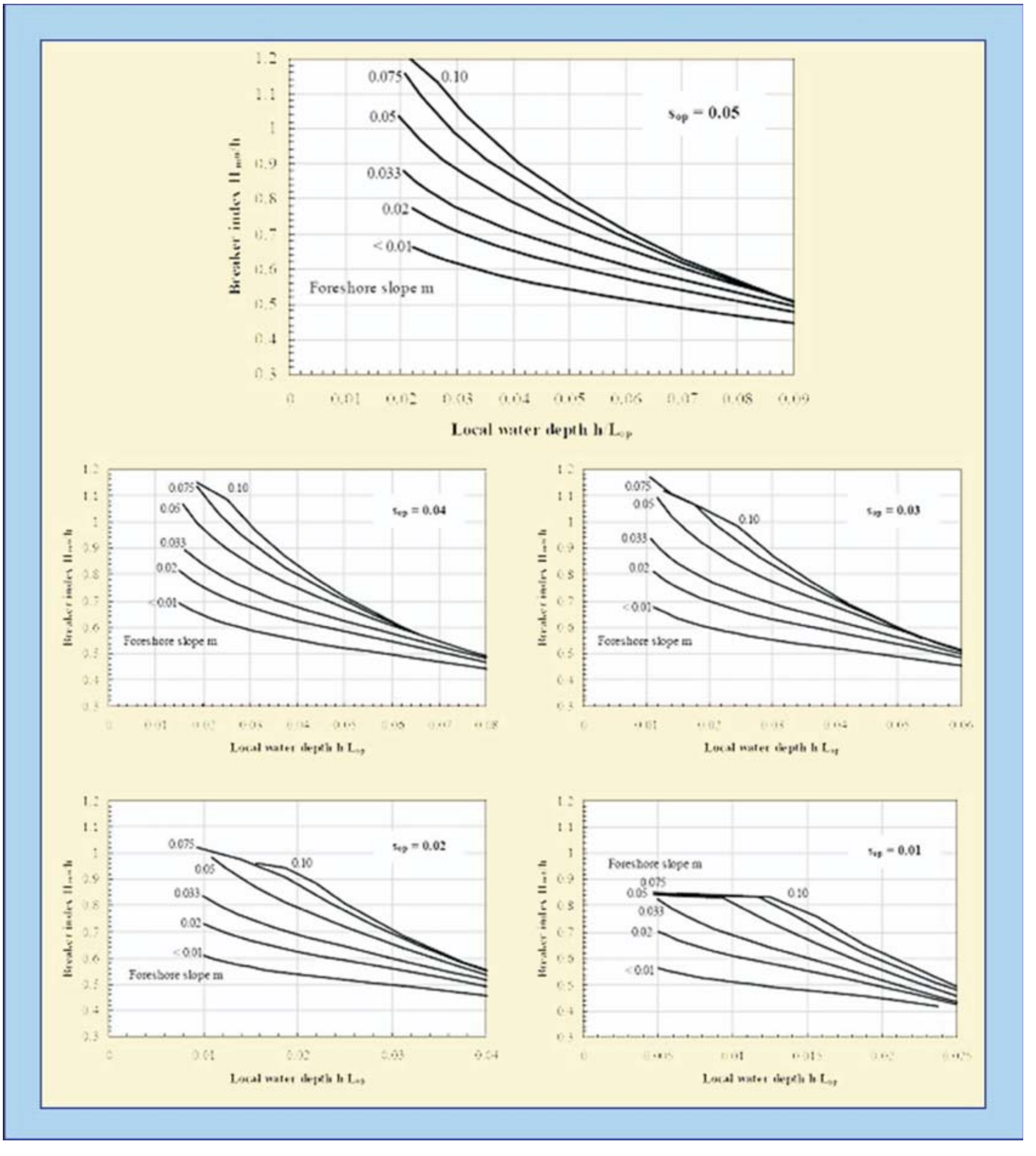

Fig. 2.4: Depth-limited significant wave heights for uniform foreshore slopes

Example. Suppose H_{so} = 6 m, T_p = 9.4 s, foreshore slope is 1:40 (m = 0.025). Calculate the maximum significant wave height H_{m0} at a water depth of h = 7 m.

1. The wave conditions on deep water give s_{op} = 0.043. Graphs with s_{op} = 0.04 and 0.05 have to be used.
2. The local relative water depth h/L_{op} = 0.051.
3. The slope of the foreshore (m = 0.025) is in between the curves for m = 0.02 and 0.033.
4. From the graphs, H_{m0}/h = 0.64 is found for s_{op} = 0.04 and 0.68 is found for s_{op} = 0.05.
5. Interpolation for s_{op} = 0.043 gives H_{m0}/h = 0.65 and finally a depth-limited spectral significant wave height of H_{m0} = 3.9 m.

Wave breaking in shallow water does not only affect the significant wave height H_{m0}. Also the distribution of wave heights will change. In deep water wave heights have a Rayleigh distribution and the spectral wave height H_{m0} will be close to the statistical wave height $H_{1/3}$. In shallow water these wave heights become different values due to the breaking process. Moreover, the highest waves break first when they feel the bottom, where the small waves stay unchanged. Actually, this gives a non-homogeneous set of wave heights: broken waves and non-broken waves. For this reason Battjes and Groenendijk (2000) developed the composite Weibull distribution for wave heights in shallow water.

Although prediction methods in this manual are mainly based on the spectral significant wave height, it might be useful in some cases to consider also other definitions, like the 2%-wave height $H_{2\%}$ or $H_{1/10}$, the average of the highest 1/10-the of the waves. For this reason a summary of the method of Battjes and Groenendijk (2000) is given here. The example given above with a calculated H_{m0} = 3.9 m at a depth of 7 m on a 1:40 slope foreshore has been explored further in Fig. 2.5.

$$H_{m0} = 4\sqrt{m_0} \; H_{m0} = 4\sqrt{m_0}$$
$$H_{rms} = \left(2.69 + 3.24\sqrt{m_0}/h\right)\sqrt{m_0} \qquad 2.1$$

where H_{rms} = root mean square wave height. The transition wave height, H_{tr}, between the lower Rayleigh distribution and the higher Weibull distribution (see Fig. 2.5) is then given by:

$$H_r = (0.35 + 5.8 \tan\alpha)h \qquad 2.2$$

One has then to compute the non-dimensional wave height H_{tr}/H_{rms}, which is used as input to Table 2 of Battjes and Groenendijk (2000) to find the (non-dimensional) characteristic heights: $H_{1/3}/H_{rms}$, $H_{1/10}/H_{rms}$, $H_{2\%}/H_{rms}$, $H_{1\%}/H_{rms}$ and $H_{0.1\%}/H_{rms}$. Some particular values have been extracted from this table and are included in Table 2.1, only for the ratios $H_{1/3}/H_{rms}$, $H_{1/10}/H_{rms}$, and $H_{2\%}/H_{rms}$.

Table 2.1: Values of dimensionless wave heights for some values of H_{tr}/H_{rms}

Characteristic height	Non-dimensional transitional wave H_{tr}/H_{rms}									
	0.05	0.50	1.00	1.20	1.35	1.50	1.75	2.00	2.50	3.00
$H_{1/3}/H_{rms}$	1.279	1.280	1.324	1.371	1.395	1.406	1.413	1.415	1.416	1.416
$H_{1/10}/H_{rms}$	1.466	1.467	1.518	1.573	1.626	1.683	1.759	1.786	1.799	1.800
$H_{2\%}/H_{rms}$	1.548	1.549	1.603	1.662	1.717	1.778	1.884	1.985	1.978	1.978

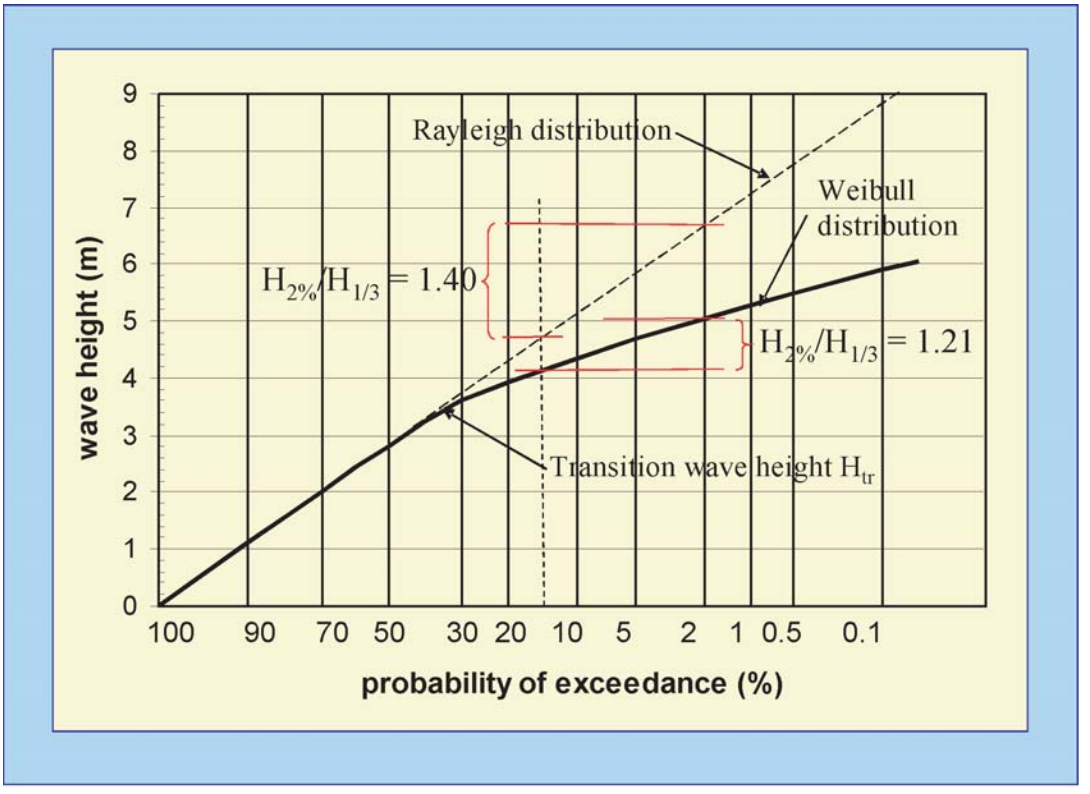

Fig. 2.5: Computed composite Weibull distribution. H_{m0} = 3.9 m; foreshore slope 1:40 and water depth h = 7 m

The final step is the computation of the dimensional wave heights from the ratios read in the table and the value of H_{rms}. For the given example one finds: $H_{1/3}$ = 4.16 m; $H_{1/10}$ = 4.77 m and $H_{2\%}$ = 5.4 m. Note that the value $H_{2\%}/H_{1/3}$ changed from 1.4 for a Rayleigh distribution (see Fig. 2.5) to a value of 1.21.

2.5 Currents

Where waves are propagating towards an oncoming current, for example at the mouth of a river, the current will tend to increase the steepness of the waves by increasing their height and decreasing their wave length. Refraction of the waves by the current will tend to focus the energy of the waves towards the river mouth. In reality both current and depth refraction are likely to take place producing a complex wave current field. It is clearly more complicated to include current and depth refraction effects, but at sites where currents are large they will have a significant influence on wave propagation. Computational models are available to allow both these effects to be represented.

2.6 Application of design conditions

The selection of a given return period for a particular site will depend on several factors. These will include the expected lifetime of the structure, expected maximum wave and water level conditions and the intended use of the structure. If for instance the public are to have access to the site then a higher standard of defence will be required than that to protect farm land. Further examples are given in Chapter 3.

A way of considering an event with a given return period, T_R, is to consider that (for $T_R \geq 5$ years) the probability of its occurrence in any one year is approximately equal to $1/T_R$. For example, a 10,000 year return period event is equivalent to one with a probability of occurrence of 10^{-4} in any one year.

Over an envisaged lifetime of N years for a structure (not necessarily the same as the design return period) the probability of encountering the wave condition with return period T_R, at least once, is given by:

$$P\,(\underline{T_R} \geq T_R) = 1 - (1 - 1/T_R)^N \qquad 2.3$$

Fig. 2.6 presents curves for this encounter probability with values between 1 per cent and 80 per cent shown as a function of T_R and N. It follows that there will not be exactly T_R years between events with a given return period of T_R years. It can be seen that for a time interval equal to the return period, 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 1974 and Part 7 1991), the PIANC working group 12 report (PIANC 1992) and in the new Rock Manual (2007).

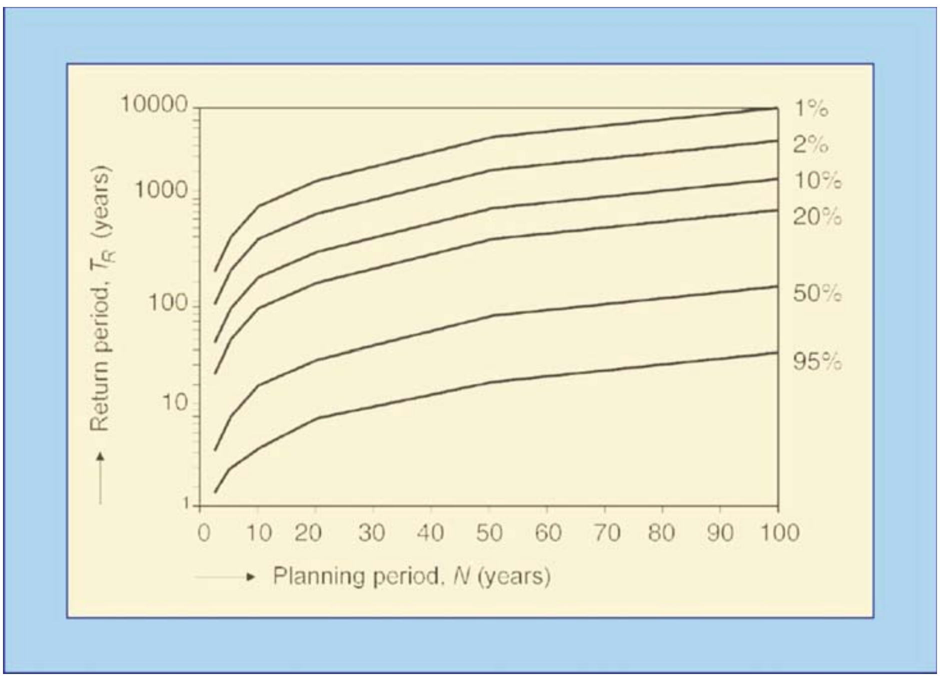

Fig. 2.6: Encounter probability

2.7 Uncertainties in inputs

Principal input parameters discussed in this section comprised water levels, including tides, surges, and sea level changes. Sea state parameters at the toe of the structure have been discussed and river discharges and currents have been considered.

It is assumed here that all input parameters are made available at the toe of the structure. Depending on different foreshore conditions and physical processes such as refraction, shoaling and wave breaking the statistical distributions of those parameters will have changed over the foreshore. Methods to account for this change are given in BATTJES & GROENENDIJK (2000) and elsewhere.

If no information on statistical distributions or error levels is available for water levels or sea state parameters the following assumptions should be taken: all parameters are normally distributed; significant wave height H_s or mean wave height H_{m0} have a coefficient of variation σ_x' = 5.0 %; peak wave period T_p or mean wave period $T_{m-1.0}$ have a coefficient of variation σ_x' = 5.0 %; and design water level at the toe σ_x' = 3.0 %, see SCHÜTTRUMPF et al. (2006).

The aforementioned values were derived from expert opinions on these uncertainties. About 100 international experts and professionals working in coastal engineering have been interviewed for this purpose. Although these parameters may be regarded rather small in relation to what GODA (1985) has suggested results have been tested against real cases and found to give a reasonable range of variations. It should be noted that these uncertainties are applied to significant values rather than mean sea state parameters. This will both change the type of the statistical distribution and the magnitude of the standard deviation or the coefficient of variation.

Guidance on hydraulic boundary conditions for the safety assessment of Dutch water defences can be found in Hydraulische Randvoorwaarden, RWS 2001 (Due to be updated in 2007).