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Hydraulic Boundary Conditions for Coastal Risk Management COMRISK Subproject 5

JOHANN DEKKER, AED WOLTERS, FRANK DEN HEIJER, SANDRA FRAIKIN

Summary

An inventory of the methods used to determine the hydraulic boundary conditions for the sea defences in the countries participating in the North Sea Coastal Managers Group was conducted. Based on the results of this inventory the various methods have been analysed and compared for a sea dike and a dune profile on the North Sea coast in The Netherlands. Though the general approach to determine the hydraulic boundary conditions is fairly similar, the differences in details of the methods can lead to crest heights that can vary several meters for the same return period. The approaches in the safety assessment of dune coasts are quite different, though a number of methods go back on the same research from the 1980-ies.

Due to these differences results of the various conducted risk-assessments are hardly comparable. The other way around, a common approach to risk assessment might thus lead to adaptations in safety-assessment methods in the various countries. On the other hand the knowledge questions, i.e. to reduce uncertainties in risk-analysis, are rather similar in the various countries. Joint research and further exchange of knowledge can and might lead to a convergence of the methods for risk assessment used in the various countries.

Zusammenfassung

Eine Bestandsaufnahme der Methoden zur Ermittlung der hydraulischen Randbedingungen für Küstenschutzbauwerke in den in der North Sea Coastal Managers Group vertretenen Ländern wurde durchgeführt. Basierend auf der Bestandsaufnahme wurden die verschiedenen Methoden vergleichend für einen Deich und eine Düne an der niederländischen Nordseeküste analysiert. Obwohl die Verfahren zur Bestimmung der Randbedingungen generell vergleichbar sind, können die Detailunterschiede in den angewandten Methoden zu Unterschieden von mehreren Metern in der resultierenden Deichhöhe für den gleichen Wiederkehrintervall führen. Die Verfahren zur Ermittlung der Sicherheitsstandards für Dünen variieren stark, obwohl mehrere Methoden auf die gleichen Forschungsergebnisse aus den frühen 1990ern beruhen.

Wegen dieser Unterschiede sind die Ergebnisse der durchgeführten Risikoanalysen kaum vergleichbar. Umgekehrt, ein gemeinsamer Ansatz zur Risikoanalyse kann zu Anpassungen bei den Ermittlungen der Sicherheitsstandards in den verschiedenen Ländern führen. Die Forschungsfragen hinsichtlich der Reduzierung der Unsicherheiten in Risikoanalysen sind in allen Ländern vergleichbar. Gemeinsame Forschung und weiterer Austausch von Erfahrungen können zu einer Harmonisierung der angewandten Ansätze zur Risikoanalyse in den Ländern führen.

Keywords

Coast, risk management, flood defence, hydraulic boundary conditions, dike design

Contents

1. Introduction

In 1996 national and regional coastal defence authorities in the United Kingdom, Belgium, The Netherlands, Germany and Denmark initiated a high level network of co-operation, the North Sea Coastal Managers Group (NSCMG). It was realised that, in order to achieve a transfer of knowledge and a balanced approach, a more comprehensive trans-national co-operation about risk management throughout the North Sea Region is indispensable. The NSCMG initiated a study to make an inventory of the risks, adopted safety levels and used techniques with regard to flooding of coastal areas in five countries to improve communication on this subject between the partners (DWW, 2001).

This previous study covered many aspects of flood risk in coastal areas, ranging from policy aspects and safety levels adopted in the various countries to technical aspects of dike design. One of the conclusions of this study was that the structural aspect is closely related to the way hydraulic boundary conditions are assessed. It was recommended to study the total process of hydraulic conditions together with the structural aspect to allow better comparison of the safety standards and methods applied in the various countries. In such a study the scope should include the structural aspects of dikes and dunes.

In Subproject 5 the focus is on the more technical aspects related to the design and safety assessment of the sea defences. In this subproject the way that hydraulic boundary conditions for the sea defences are derived and used is compared. The Road and Hydraulic Engineering Division (DWW) of the Directorate-General of Public Works and Water Management (in short Rijkswaterstaat) is coordinator of this subproject. DWW contracted WL | Delft Hydraulics to assist in the inventory and comparison of the methods.

2. Approach

2.1 Inventory

Subproject 5 started in 2002 with an inventory of the methodologies adopted by the various partners to assess the hydraulic boundary conditions (water level and wave conditions) and the way these are used in the design and/or safety assessment of the sea defences. This inventory was based on the response on a questionnaire that was sent to the partners together with a description of the methodology in The Netherlands. The information received from the partners has been summarized in WL | Delft Hydraulics (2005).

2.2 Analysis for selected locations

To get some more insight in possible reasons for the differences in the methodologies to determine the hydraulic boundary conditions, the results of the inventory have been brought a step further by comparing the results of the various methods. It would be interesting to see whether the heights of sea dikes in the six North Sea countries would be different if they were designed using the methodologies from other countries when adopting the same safety level.

Ideally all methods should be applied to a typical site in each of the partner countries. In this way differences due to different geography could be detected. This would mean 36 combinations of methods and sites, which was not feasible within the framework of the COMRISK project. The closest alternative was to apply all methods to a few selected sites. For practical reasons such as easy access to relevant data regarding water level, waves, wind and bathymetry, this was limited to sites in The Netherlands. Both a sea dike and a dune section have been considered. The following sites on the North Sea coast were selected:
- Petten sea defence, the sea dike near Petten,
- Dune coast at Callantsoog.

Both sites are in the province of North-Holland, north-northwest of Amsterdam. The location of these sites is shown in Fig. 1.

The Petten sea defence (Fig. 1, top right) is a sea dike with a crest at about 12.75 m above NAP (MSL). The lower part of the seaward side has a slope of 1:4.5, the upper part a slope of 1:3. Between those slopes is a 14 m wide berm at about 5.35 m above NAP. The inner slope is 1:3.

The dunes near Callantsoog consist of a single row with a width of about 100 m and a maximum height of about 20 m above NAP (Fig. 1, lower right).

Based on the information gathered through the questionnaires, the descriptions in earlier study (DWW, 2001) and other information (e.g. found on the website of the partners),

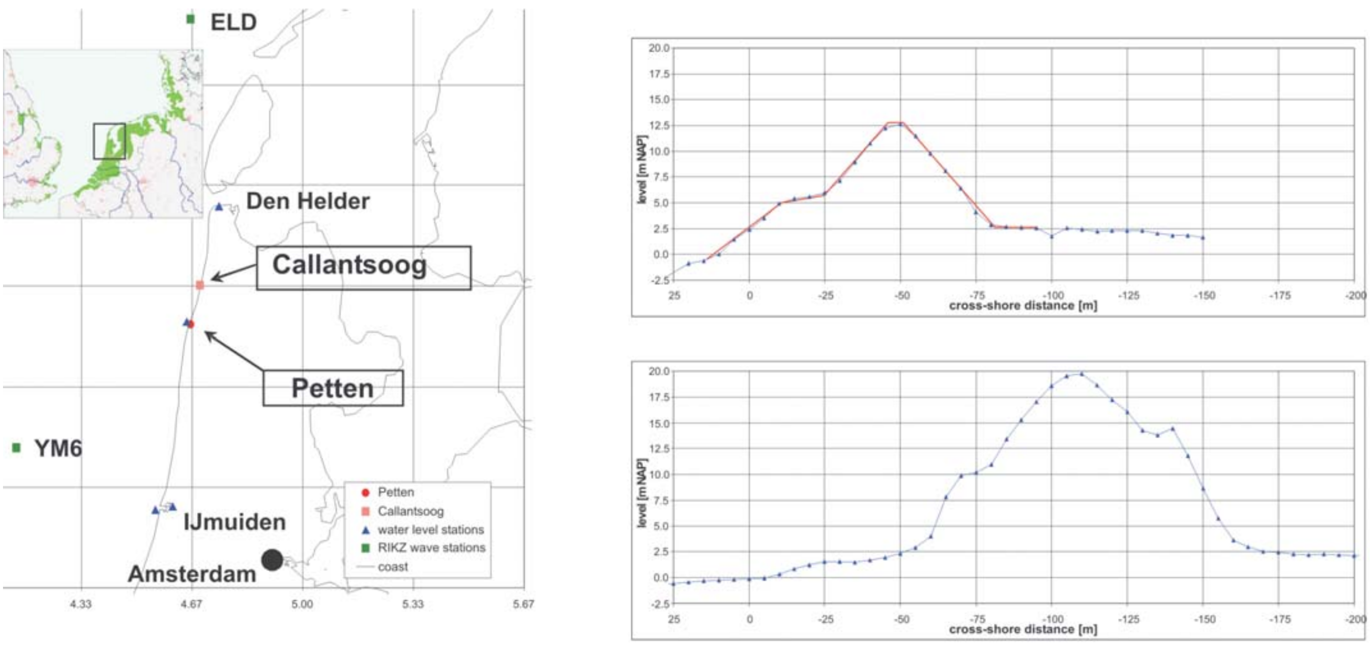

Fig. 1: Location of the selected sites (left) and typical sections of the dike at Petten (top right) and the dune at Callantsoog (lower right)

the procedures used in the six countries are described and compared based on data for the selected sites of Petten and Callantsoog. The procedure to design or evaluate a sea dike generally consists of two steps: determination of the hydraulic boundary conditions at the toe of the dike and calculation of the required crest height. The safety assessment of sandy coasts involves similar steps, the main difference being that the wave conditions are usually required in deeper water. This study therefore compares first the way the hydraulic boundary conditions are derived. Water level and wave conditions are treated separately. Then the procedures to determine the required crest height are compared. This includes a comparison of formulae for wave run-up and overtopping. These are used to assess the required crest height for the Petten sea defence according to the various methods.

The comparison presented in this study is based on a deterministic approach. All countries are developing probabilistic techniques to support assessing the risk of flooding of coastal areas. Comparing these by application to a selected case was not feasible within the present study. However, aspects such as wave run-up and overtopping formulae and criteria for these factors are also key elements in probabilistic methods. Thus, the values given in this report can not be used for actual assessment of water levels, crest levels and so on, they are indicative values to study differences between approaches in the various countries. Risk assessment using probabilistic techniques has been conducted in some of the case studies treated in the COMRISK subprojects 6 to 9.

3. Hydraulic boundary conditions

3.1 Water levels

All countries have fairly extensive networks of water level stations. These are used as basis to determine extreme water levels required as input for design and safety assessment of the sea defences. The number of stations in the countries ranges from 3 in Belgium, which has a fairly short stretch of coast along the North Sea to about 40 in the United Kingdom and even more in Germany, which has a long coast line with various estuaries. National authorities gather the data and the available information goes for some stations back for more than 100 years.

Recent data are generally stored as 10-minute averages after a quality check. Before data are used to determine design conditions by extreme value analysis the historic data are corrected for trends in sea level and/or the tidal amplitude over period of observations. In this way each record can be considered to be representative for the present situation. In most of the countries the required water levels for design and safety assessment are determined using probabilistic methods. This can be based on extrapolation of observed water levels (e.g. Denmark and The Netherlands) or on extrapolation of measured surges that are combined with the tidal component (e.g. Belgium). In the United Kingdom each of these methods may be applied as the contractor carrying out the study can use his own methods. In Niedersachsen (Germany) a deterministic method is used, which combines the tide with the highest observed surge. Schleswig-Holstein (Germany) combines this deterministic method with the probabilistic approach by using the maximum of the two. Most countries increase the design level to account for factor such as local wind set-up and relative sea level rise.

The results of the various methods to assess the design water levels are summarised in Fig. 2. The most striking in this figure is of course the single value independent from the probability of occurrence following the method Niedersachsen. This is inherent to the design

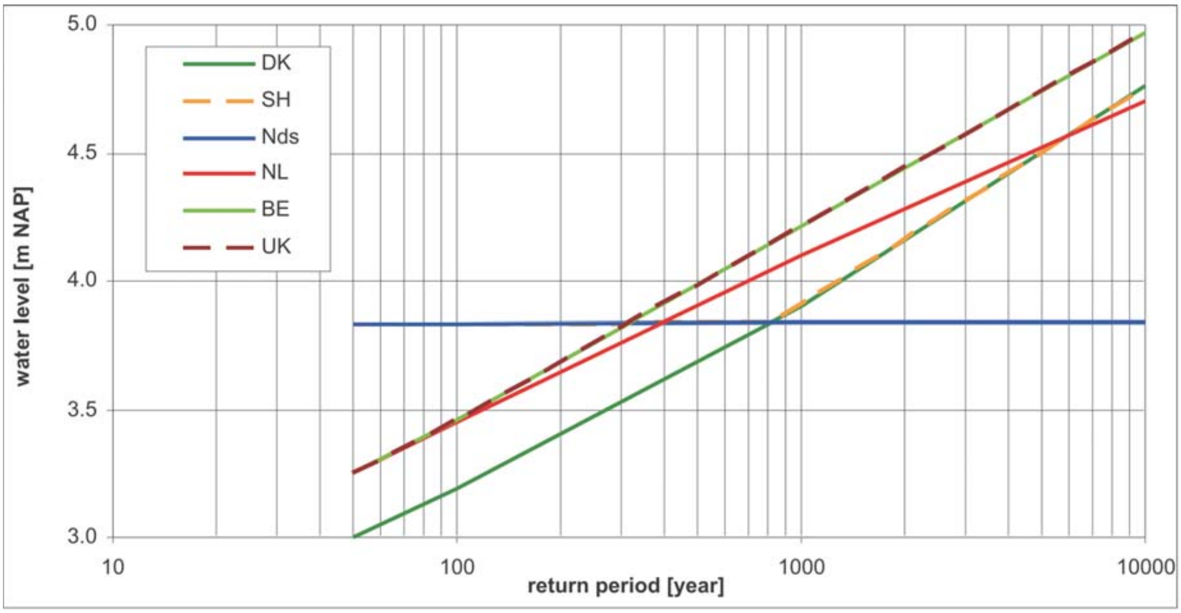

Fig. 2: Comparison of design water levels following different methods for Petten, The Netherlands

method *Einzelwert-Verfahren*, which "aims to avoid any exceedance" (quote from response to questionnaire; WL | Delft Hydraulics, 2005). It can be seen that the value for Petten following this method has a probability of occurrence between 1/400 (Dutch & Belgian method) and 1/800 (Danish method). In the comparison of methods for the Petten sea defence the results from the Belgian method have also been adopted for the United Kingdom.

It is further interesting to note that the water levels using the method of The Netherlands are for short return periods equal to those following the Belgian method, but for longer return periods closer to those from the Danish method. The differences between the results of these methods are in the order of 0.25 m, depending on the return period and method. For the longer return periods this is actually fairly small considering that the 95 %-confidence interval for the 1/10,000 year surge is in the order of 1 m. For the shorter return periods of 50 and 100 years, however, a better agreement was expected between the various methods. The difference for these return periods may be due to the use of not completely consistent data for the comparison (WL | Delft Hydraulics, 2005).

3.2 Wave conditions

Most countries use fairly similar methods to assess the wave conditions in the vicinity of the sea defences. Only Schleswig-Holstein has a quite different approach. Though deep water waves and wind are measured on a location off Sylt since 1984 (21 years) as a basis for sand nourishment, the nearshore design wave conditions are direct assessed by correlating with the still water level. The approach in the other countries is based on a statistical evaluation of deep water wave data (either from measurements or hindcast) combined with wave propagation modelling to determine the corresponding conditions near the coast. In The Netherlands and Belgium relatively long datasets of wave measurements in deep water are available (20-25 years), which allows extreme value analysis directly on the measured wave heights. In Denmark time series of 8 years are available for most of the wave gauges, while in the United Kingdom the available timeseries of wave measurements cover periods of 1 to 4 years. In these countries the wave measurements are combined with wind data that cover longer periods using hindcast techniques.

For this study the deep-water station Eierlandse Gat (ELD, see Fig. 1) is the reference relevant for the coast of Petten and Callantsoog. The official Dutch values extreme wave conditions for this station (RIKZ, 1995) have been compared with an independent analysis of the data. The resulting wave heights for the selected return periods are included Table 1. It can be seen that the difference in wave height is 0.3–0.5 m. As it appeared that this difference has no significant effect on the nearshore conditions near the Petten sea defence (the remaining difference is only 1–2 cm) other methods to assess the deep water wave conditions have not been tested. In the comparison the official values were used for the method of The Netherlands, the results of the independent analysis for all other methods.

Table 1: Extreme wave conditions at Eierlandse Gat adopted in the comparison

method	waves 1/50		waves 1/100		waves 1/1,000		waves 1/10,000	
	H_s [m]	T_m [s]	H_s [m]	T_m [s]	H_s [m]	T_m [s]	H_s [m]	T_m [s]
NL	8.05	9.5	8.37	9.7	9.24	10.2	10.00	10.6
other	7.52		7.82		8.80		9.72	

As mentioned above, all countries except Schleswig-Holstein use numerical models to determine the design wave conditions at the toe of the sea defences. Within the scope of the present comparison of methods it was not feasible to carry out wave propagation simulations with the various models used in the different countries. Instead, the wave conditions at the toe of the Petten sea defence were determined by interpolation in the results of a large number of wave runs with the model SWAN, that have been stored in a database. This database contains for a large number of locations the characteristic wave parameters (H_{m0}, T_p, T_{m02}, T_{m-2-1}, direction) of SWAN runs for 3 water levels, 14 wind directions and 5–7 combinations of wind speed and offshore wave conditions. Based on the derived water level and deep water wave height, the significant wave height H_{m0}, mean wave period T_{m02} and the peak wave period T_p near the Petten sea defence were determined by bilinear interpolation from the results for the wind and wave direction 285 °N, which is the most unfavourable direction in this location. The results are shown in Table 2. It appeared that the results were fairly insensitive to the deep water wave height and that the water level is in fact governing the nearshore wave conditions. This is illustrated by the results for Niedersachsen, for which the water level is the same for all return periods.

In Schleswig-Holstein the nearshore wave conditions are determined by correlation with the water level using the following relations:

$$H_{1/3} = (SWL - DZ) * Gr$$
$$T_z = a + b * H_{1/3}$$

where $H_{1/3}$ is the significant wave height and T_z is the mean zero-crossing wave period. The coefficients DZ, Gr, a and b are parameters determined based on measurements. For the present comparison these parameters have been derived based on measurements from the Petten site for the season 2003–2004 (RIKZ, 2004). The results are included in Table 2. It is remarkable that the wave heights are significantly higher than those based on the SWAN simulations.

Table 2: Wave conditions at the toe of the Petten sea defence.

method	waves 1/50		waves 1/100		waves 1/1000		waves 1/10000	
	H_s [m]	T_{m02} [s]	H_s [m]	T_{m02} [s]	H_s [m]	T_{m02} [s]	H_s [m]	T_{m02} [s]
DK	2.61	6.56	2.74	6.66	3.19	6.93	3.69	7.15
SH	3.53	6.68	3.53	6.68	3.58	6.74	4.18	7.39
Nds	3.12	6.84	3.13	6.88	3.15	6.91	3.16	6.84
NL	2.78	6.71	2.91	6.80	3.31	6.97	3.66	7.11
B	2.76	6.64	2.90	6.75	3.37	7.03	3.80	7.22
UK	2.76	6.64	2.90	6.75	3.37	7.03	3.80	7.22

3.3 Wave periods

The database provides the spectral wave periods T_{m02}, T_{m-2-1} and T_p whereas the datafiles from the field measurements at Petten that were available provide the wave periods T_{m02} and $T_{H1/3}$. The formulae for wave run-up and overtopping that are used in the various countries contain characteristic wave periods that are not directly available. These wave periods have therefore been determined by assuming a certain constant ratio between different wave periods.

The formula for wave run-up used in Denmark contains the wave period T_m (DWW, 2001), without providing the definition. ANDERSEN (1998) gives the same formula with $\hat{T}$, also without defining this parameter. Here the expression from DWW has been adopted, as this reference provides also values for the coefficients in the equation. The wave period T_m was approximated by $T_p/1.15$, similar to the relation adopted in DWW (2001).

In the overtopping formulae used in Schleswig-Holstein, Niedersachsen and the United Kingdom the mean wave period T_m is used. This is the mean period of the waves in time domain also known as the zero-crossing period T_z. This characteristic wave period is not available in the database. Energy balance models such as SWAN can only provide wave periods in the frequency domain such as T_p, T_{m-10}, T_{m01} and T_{m02}. For the present study relations between time-domain period T_m and the frequency domain periods T_p and T_{m01} have been derived from flume test on the Petten profile that were performed for a large range of conditions (WL | Delft Hydraulics, 1999). The ratio between T_m and T_p shows a fairly large range, but seems to depend on the ratio of H_{m0} over the water depth. Based on the results of tests for conditions similar to the extreme hydraulic conditions used in the comparison a ratio of $T_p/T_m = 1.45$ has been adopted in this study. As the ratio T_p/T_{m02} for the most relevant conditions in the database was around 2, the ratio T_m/T_{m02} was taken as 1.4.

The formulae for wave overtopping applied in The Netherlands and Belgium use the spectral mean period T_{m-10}. This period is not available within the database. The ratio between T_{m-10} and T_{m02} depends largely on the spectral shape. For the present comparison the ratio has been estimated based on the nearshore measurements at the Petten site. From graphs presenting T_{m02} and T_{m-10} for two storms in the season 2003–2004 (RIKZ, 2004) it can be seen that the ratio between the two is quite different before, at and after the peak of the storm. At the peak of the storm the ratio T_{m-10}/T_{m02} is about 1.6–1.7. Before the peak the ratio is smaller, after the peak, the ratio is larger. For the comparison of the various methods a ratio of 1.65 has been adopted. This value is rather large, but this is because the wave spectra at the toe are non-standard spectra.

4. Approaches to dike evaluation/design

4.1 Comparison of methods

All partners determine the crest height from the design water level and the wave conditions near the sea defence. In the safety assessment of existing coastal defences additional margins are in some countries included for factors such as long waves, harbour resonance and trends in the sea level. In the design of new dike sections factors such as sea-level rise and the expected subsidence of the crest is taken into account.

The required height to account for waves is determined using a criterion either for wave run-up (DK, SH, Nds) or for wave overtopping (SH, NL, B, UK). For the wave run-up criteria, the run-up height follows directly from the formulas for wave run-up, which read in general form

$$Z_{n\%} = C\,\gamma_i\,\sqrt{H_s}\,T\tan\alpha \tag{1}$$

where $Z_{n\%}$ is the run-up level exceeded by n% of the waves, C is a coefficient, γ_i reduction factors for effects such as slope roughness, berms and angle of wave attack.

To allow comparison of the formulas for run-up and overtopping, the wave overtopping formulas have been rewritten to obtain a direct expression for the required crest level above the still water line. Where the general shape of the formulas for the overtopping rate is

$$q = c_1 \frac{\gamma_i H_s T}{\sqrt{\cot\alpha}} \exp\left(-c_2 \frac{R_c \cot\alpha}{\gamma_i \sqrt{H_s} T} \right) \tag{2}$$

with a maximum of

$$q = c_3 \sqrt{H_s^3} \exp\left(-c_4 \frac{R_c}{\gamma_i H_s} \right) \tag{3}$$

where q is the overtopping rate, R_c the crest height above the still water level and c_1, c_2, c_3 and c_4 are coefficients. Rewriting these equations, it follows that the crest level above the still water line as function of the criterion for the overtopping rate and the wave conditions is given by

$$R_c = -c_2 \gamma_i \frac{\sqrt{H_s} T}{\cot\alpha} \ln\left(c_1 \frac{q\sqrt{\cot\alpha}}{\gamma_i H_s T} \right) \tag{4}$$

with a maximum of

$$R_c = -c_4 \gamma_i H_s \ln\left(c_3 \frac{q}{\sqrt{H_s^3}} \right) \tag{5}$$

The expressions for the wave run-up and wave overtopping that are used in the different countries have been compared by calculating the required crest level above the still water line as function of the wave height. This comparison is carried out for a straight smooth 1:4 slope.

This is similar to the representative slope of the Petten sea defence under design conditions. The effects of berms, surface roughness, shallow foreshores or wave attack under an angle with the dike have not been considered.

The wave periods corresponding to the significant wave height have been calculated assuming JONSWAP type spectrum with $\bullet_0 = 3.3$. (It should be noted that the spectral shape in shallow water close to the dike is usually significantly different due to breaking.) The peak wave period has been calculated using the relation $T_p = C\sqrt{H_s}$ in which C is 4.5 corresponding to a wave steepness of $s_p = 0.03$. Other wave period parameters (e.g. T_m and T_{m-10}) have been calculated from the peak period using the relations $T_m = T_{m02} = T_p/1.28634$ and $T_{m-10} = T_p/1.10706$.

For this combination of slope and wave conditions (steepness) the overtopping formulae for breaking waves are governing. The left panel of Fig. 3 shows that the required relative crest levels show a considerable scatter. The ratio between the highest and lowest value is in the order of 1.5, which means a difference in crest height of several meters for a significant wave height of 2–3 m. To compare the formula for non-breaking waves, the required crest levels were computed for a slope of 1:2.5. The right panel of Fig. 3 shows again the large spread in the crest level required to have the same amount of overtopping. It is interesting to note that the formulae used in Niedersachsen and The Netherlands give nearly the same results in this case. Note that in Belgium the formulae from The Netherlands are used.

One of the obvious observations from the figures above is that the shape of the curves for the relation $H_s - R_c$ for methods based on wave run-up is different to those based on wave overtopping. The curves based on wave run-up show a linear relation, whereas the curves for overtopping criteria show a non-linear relation: the required crest level is progressively increasing with the significant wave height.

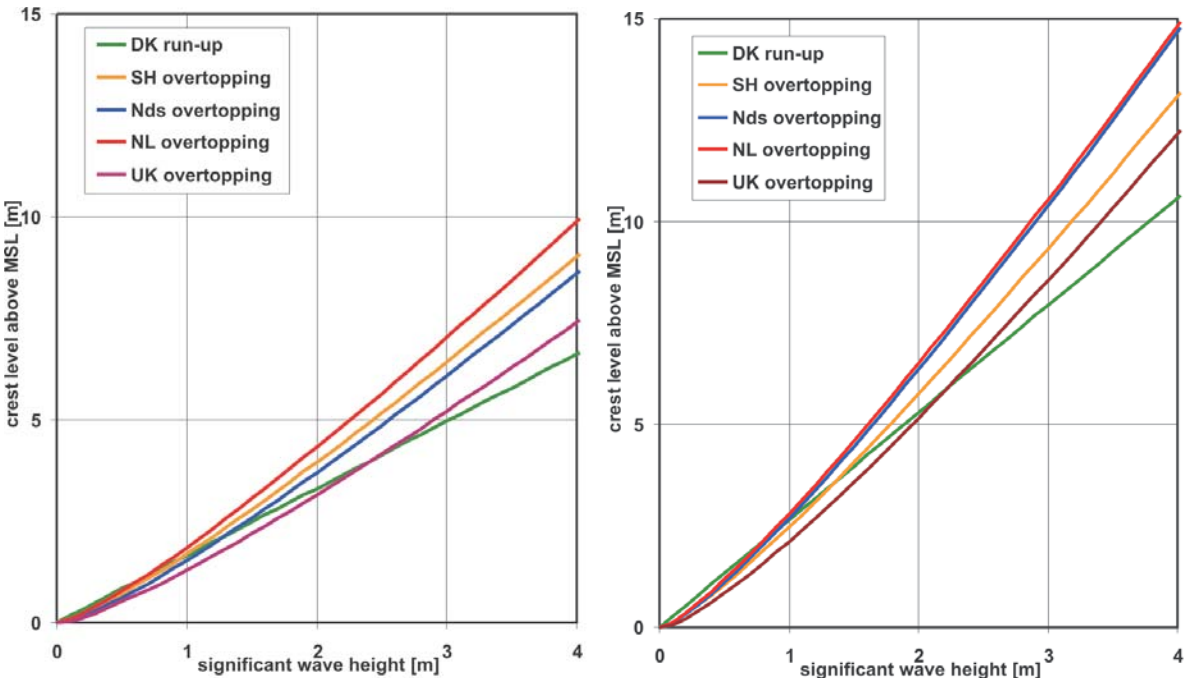

Fig. 3: Comparison of the crest level above MSL for breaking (left) and non-breaking (right) waves following different methods

It is further interesting to note that the different countries make different distinctions in their criteria for wave overtopping. Denmark is the only country where the angle of the inner slope is an explicit factor in the allowable percentage of overtopping. Both Denmark and The Netherlands have further different criteria depending on the quality of the top layer of the inner slope for grass dikes, whereas other countries use a single criterion.

The required hydraulic boundary conditions for the various applied methods to assess the design height of the sea defences are the water level and the wave height and period at the toe of the sea dike. The wave height parameter can be H_s or H_{m0}; the difference between these two is usually not very large. For the wave period different characteristic parameters are being used of which the mean period T_m is mostly used (DK, SH, UK). Other characteristic periods that are used are the peak period T_p (Nds) and the spectral mean wave period T_{m-10} (NL, B and recently in Nds).

4.2 The height of the Petten sea defence

The formulae for wave run-up and overtopping were applied to the data for the Petten sea defence to assess the required height of this sea defence. Where the comparison of the various formulae in Section 4.1 was carried out without the effects of reduction factors for roughness, berm etc., it was ensured that the right corrections were included when applying the formulae to the Petten sea defence. It was found that these reduction factors are calculated differently in the various methods, especially the factors for the effect of a berm and for the effect of the angle of wave attack. The latter has no influence on this comparison as the waves are assumed to approach the coast perpendicularly.

The required crest levels according to the various methods are shown in left panel of Fig. 4. This comparison is based on the water levels, wave conditions, run-up or overtopping formula and associated criteria for run-up or the overtopping discharges as applied in the six countries. Note that for Niedersachsen accidentally overtopping has been considered instead of run-up.

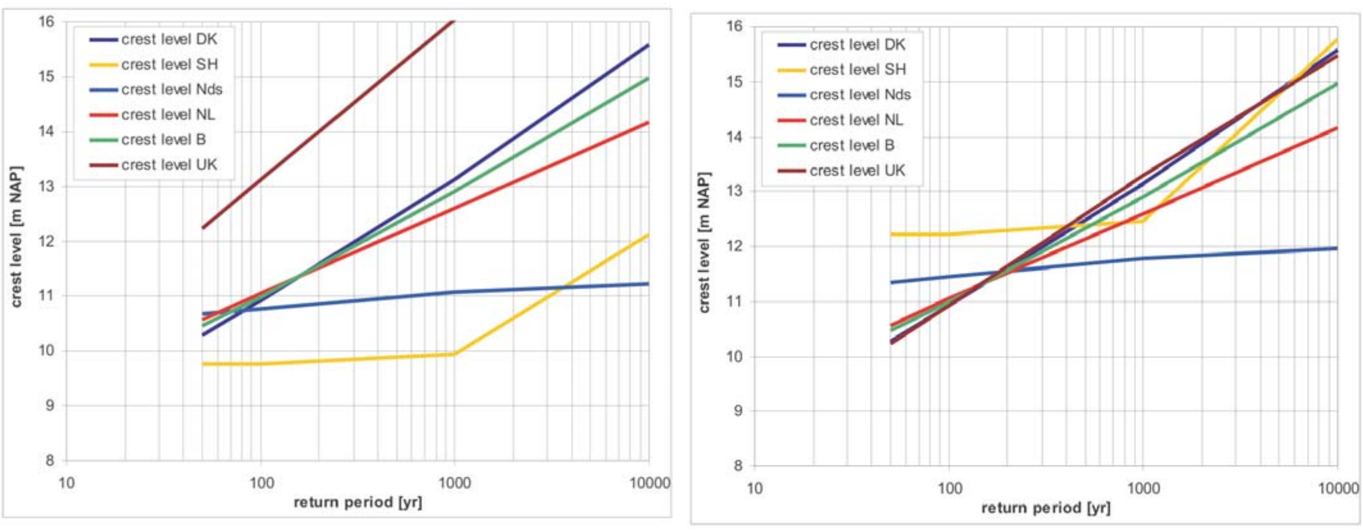

Fig. 4: Required crest level above NAP (·MSL) for the Petten sea defence following different methods; left panel: basic comparison, right panel: comparison of modified methods for SH, Nds and UK (see text)

It can be seen that three methods give results that differ up to about 1m, three other methods show much larger differences. The fairly flat line for Niedersachsen is caused by the design water level, which is independent of the return period. Investigation of the large differences for Schleswig-Holstein, Niedersachsen and the UK showed that these could be traced back to a few specific factors that are discussed below.

For Schleswig-Holstein three factors cause a large part of the differences. The first factor is the reduction factor for a shallow foreshore that is applied in agreement with the original expression by Van der Meer (TAW, 1999). This factor (approx. 0.83), which causes the difference between the results for 50 and 100 year for Schleswig-Holstein and Niedersachsen, has not been included in the most recent formulae used in the Netherlands (TAW, 2002), as the use of another representative wave period (T_{m-10}) left insufficient evidence for retaining this factor. The Schleswig-Holstein expression for non-breaking waves includes further a reduction factor for a berm, a factor that is not included in the formulae for non-breaking waves from other countries. Finally, the formula for breaking waves contains a factor $1.25T_m$ to approximate the peak period T_a in the original expression of Van der Meer (TAW, 1999) on which the formula of Schleswig-Holstein is based. As mentioned above, this factor is more in the order of 1.45 for the considered conditions at Petten. The right panel of Fig. 4 shows the required crest level with these three modifications to the expressions of Schleswig-Holstein. It can be seen that the results are more in line with those of the other countries.

The expression for overtopping given in the EAK (2002) appears to contain the factor $\cot\alpha$ where several similar equations for the other countries have the square root of this factor on the same position (see Eq. 4). Using the square root leads to a higher crest level as shown in the right panel of Fig. 4.

For the United Kingdom the main cause of the differences seems to be the way the representative slope is calculated in the presence of a berm. Following the expression used in the United Kingdom this slope is around 3.2, whereas other methods lead to values around 4.0. This has a significant effect on the required crest level as can be seen in the right panel of Fig. 4, where the expressions from the United Kingdom have been combined with the Dutch equation for the effect of a berm. The line for the United Kingdom nearly coincides with the curve for Denmark.

Fig. 4 shows further that the methods of Denmark, The Netherlands and Belgium give fairly similar results for shorter return periods, but for the longer return periods the differences are increasingly larger. For the return period of 10,000 years the difference is up to 1.5 m. As the overtopping formulae are the same, the different results for the methods of Belgium and The Netherlands are entirely caused by the difference in water level: the 0.27 m higher water level leads to a 0.8 m higher required crest height (10,000 year return period).

The results of the various methods were further analysed by varying certain input parameters and criteria so that these are gradually the same for all methods. Using the same hydraulic input conditions (water level and waves) in the (modified) methods appears to lead to fairly small differences between the results (Fig. 5, left panel). Note that for each method the appropriate characteristic wave period has been used. These were all based on the same deep water conditions, the SWAN results in the database and the ratios between these characteristic periods mentioned in section 3.3.

Using both the same hydraulic boundary conditions and the same criteria for wave overtopping (2 l/s/m) and a comparable criterion for run-up (2 % run-up level) clearly increases the differences in the required crest level (Fig 5, right panel). It can be seen that increasing the overtopping criterion from 1 l/s/m to 2 l/s/m (Nds, NL, B methods) leads to crest heights that are lower by about 1 m. Given the fairly explicit statement in e.g. the German guidelines

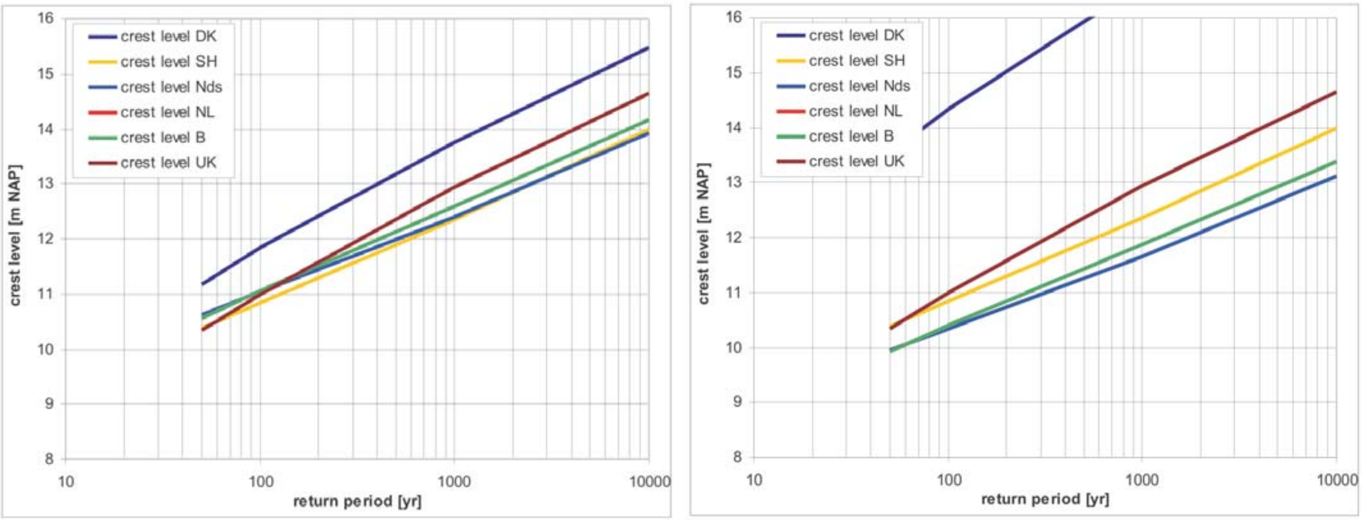

Fig. 5: Required crest levels above NAP with the same hydraulic boundary conditions (left panel) and with the same hydraulic boundary conditions and the same criteria (right panel)

(EAK, 2002) that "the criteria must be used with utmost care" and that "further research is required to complete" the information "and to specify the criteria with greater accuracy" this seems to be one of the larger gaps in present knowledge regarding sea dikes.

If the safety level adopted in the countries is also taken into consideration, the difference in crest level is even larger. The safety level adopted in Denmark is between 50 and 200 yr, in The Netherlands 2,000, 4,000 or 10,000 yr and in Belgium 1,000 yr. In the United Kingdom a cost-effective solution is determined without adopting a specific uniform safety level.

From the comparison and the analysis it appeared further that it is important that the formulae are used with the appropriate characteristic wave period. In the nearshore zone often adopted relations between the various characteristic wave periods based on a standard spectral shape such as a JONSWAP spectrum are not valid and their use may lead to erroneous results for the required crest height.

Testing the sensitivity of the crest height for the wave height and the wave period it appears that a 10 % different wave period has for the Petten sea defence a larger effect on the required crest height than a 10 % higher wave height. If wave propagation models are used to assess the conditions at the toe of the dike, it is therefore important that the model not only predicts the wave heights well; a correct prediction of the characteristic wave period is even more important. This means for generally applied wave models such as MIKE21 and SWAN that they must capable of accurately predicting the spectral shape in complex shallow water areas. It is known that e.g. SWAN has to be improved in this aspect.

5. Approaches to the safety assessment of dunes

5.1 Description of methods

Denmark uses a fairly simple criterion for the safety assessment of the dunes. These must have a minimum width of 40 m at a height of 5 m above MSL for unprotected dunes and 30 m for dunes protected by a revetment.

In Schleswig-Holstein the sandy coasts are not included in the regular assessment of safety against flooding. This is due to the different concept adopted here. According to the coastal defence concept no dune erosion is allowed at all. Where necessary sand depots are created high on the beach which should be sufficient to prevent erosion of the actual dunes under design conditions. Calculations of the cross-shore transport are carried out to assess the required reserve of sand on the beach.

In Niedersachsen numerical simulations are used to determine the dune erosion during a design storm. Up to 2003 the model NEWDUNE was used. This model is comparable with the model EDUNE by KRIEBEL (1989), which is briefly described in (EAK, 2002). The NEWDUNE model was developed by Newe at LWI University of Braunschweig. NEWDUNE is based on an equilibrium profile. The transition slope in deeper water of 1:12.5 and the 1:1 slope of the dune above the zone of wave attack are taken after VELLINGA (1983). Since 2003/2004 the numerical model UNIBEST-DE (English version of DUROSTA; STEETZEL, 1993) is used in addition to the NEWDUNE model. The experience in Niedersachsen shows that both models give comparable results for design conditions, but that NEWDUNE overestimates the erosion for more regular events. After the numerical simulations are carried out the remaining dune width at a level of NN+8 m (approx. 8 m above MSL) is taken as an indicator for the strength of the considered dune profile. A width of 15 m remaining after a simulated storm surge event is judged to be sufficient.

The method to calculate dune erosion in The Netherlands (and also in Belgium) is based on an equilibrium profile after VELLINGA (1983) consisting of a dune front with a slope of 1:1 above the water line, a parabolic beach profile of to a depth of about $0.75H_{0,s}$ and a the transition to the original seabed on the seaward side with a slope of 1:12.5 (Fig. 6, top). This

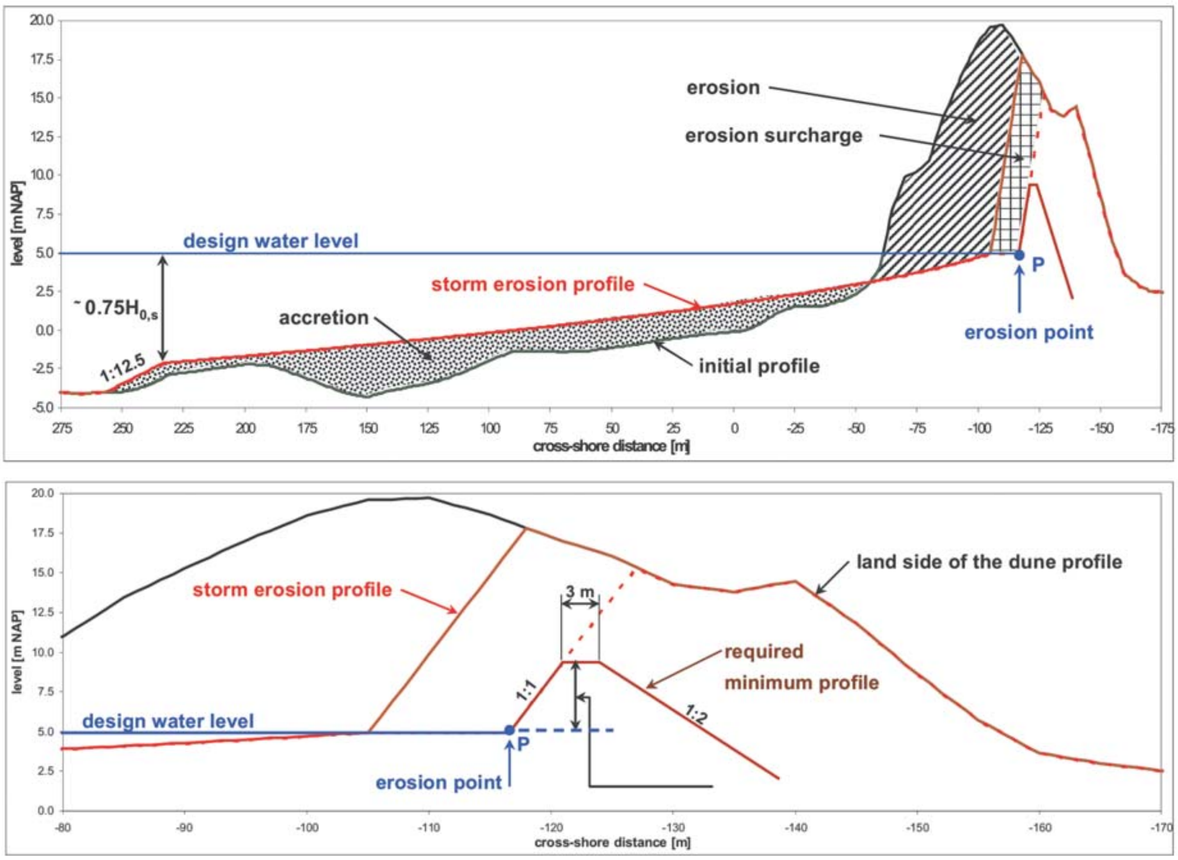

Fig. 6: Principles of the calculation of the profile after dune erosion in The Netherlands

equilibrium profile is fitted to the cross-section of the dune in such a way that the amount of erosion of the dune is equal to the amount of deposition below the water level. An additional amount of erosion equal to 25 % of the erosion above the still water level is added to account for the uncertainty in storm duration and the inaccuracy of the model. The remaining profile must have certain minimum dimensions (Fig. 6, bottom).

In the United Kingdom the dune erosion is considered to be part of the beach response to storms. Both numerical and physical models are used to predict the beach response to extreme conditions. A certain length of retreat implies failure.

Several countries use methods or criteria for dune erosion that are based on the method developed by VELLINGA (1983). This method has been derived for peak wave periods up to 12 s. A few years ago it was recognized in The Netherlands, based on much longer time series, that longer wave periods have to be taken into account for the design conditions. Recent small-scale flume tests have shown that the erosion is significantly more for longer wave periods. The method to determine dune erosion after VELLINGA (1983) is not valid for these longer periods. Further research and development of new procedures to calculate dune erosion more accurately for these conditions is therefore relevant.

5.2 The strength of the Callantsoog dune profile

The methods to assess the strength of the dune profile from Denmark and The Netherlands have been compared for the selected profile at Callantsoog. The numerical method used in Niedersachsen was not available. The erosion according to the method of The Netherlands was computed using UCIT (Universal Coastal Intelligence Toolkit), a program developed at WL | Delft Hydraulics for coastal management applications. In UCIT the erosion can be calculated for a few selected return periods between 500 and 10,000 years. The result is shown in Fig. 7.

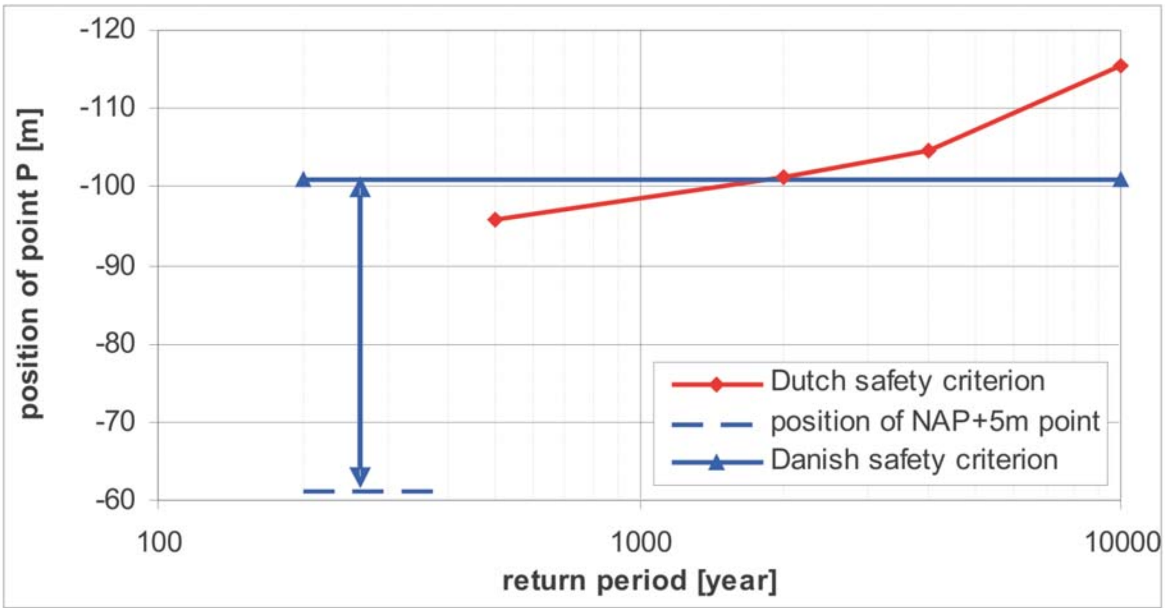

Fig. 7: Comparison of the Dutch and Danish safety criteria for dune erosion for the profile at Callantsoog

As Denmark uses a safety levels in the order of 100 years and the probability of the deterministic design water level in Niedersachsen is in the order of 400–800 years, the criteria for dune erosion in these countries have been compared with the calculated erosion according to the original method in The Netherlands for a return period of 500 years (Fig. 8). It appears that the dune width at NAP+5 m is about 95 m, which is well above the Danish criterion of 40 m at a level of 5 m above MSL. The Danish criterion of 40 m is just equal to the calculated retreat of the dunefront following the Dutch method for a 2,000 year return period. This indicates that the criterion seems to be adequate for the safety levels usually adopted in Denmark, but that depends also on the height of the dune: for a dune with the same width but a lower height the retreat according to the Dutch method would be larger. For the safety level adopted in The Netherlands (10,000 years) the Danish criterion would be insufficient.

The indicator used in Niedersachsen, the remaining width at 8 m above MSL, is for the 500 year return period close to 50 m. This is well above the criterion of 15 m used that is applied in Niedersachsen. For the 10,000 year return period the remaining width at NAP + 8 m is about 30 m, still sufficient according to the criterion applied in Niedersachsen.

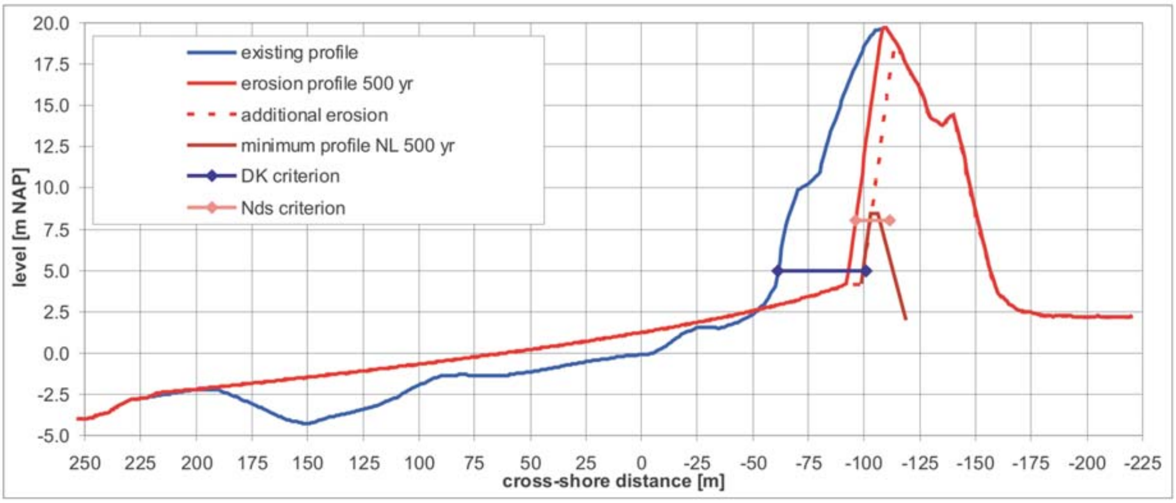

Fig. 8: Comparison of the calculated dune erosion for the Callantsoog dune profile according to the Dutch method with criteria for Denmark, Niedersachsen and The Netherlands (500 yr return period)

6. Discussion, conclusions and recommendations

6.1 Discussion

From the comparison of the various methods to assess the hydraulic boundary conditions and their use for safety assessment of the sea defences it can be concluded that the general approach in the North Sea countries is fairly similar. The boundary conditions are usually obtained by statistical evaluation and extrapolation of water levels and deep-water wave conditions followed by numerical modelling to obtain the wave conditions nearshore. The results are used in fairly similar expressions for run-up and overtopping. The differences are mostly in the details such as coefficients and some specific aspects of the applications.

Water levels at different return periods are usually based on extrapolation of long time-series of measured data. Remarkable here is the method used in Niedersachsen which leads to a single design level irrespective of the probability. This is not suitable in a risk-based approach. In the method used in Denmark to predict water levels for extreme events, different statistical distributions are used for different locations (Weibull and Log-Normal) depending on the quality of the fit.

The different methods to assess the design water levels lead to differences in the water level of 20–30 cm. For the shorter return periods, that are similar to the period of observations, this is more than might be expected. This may be caused by differences in the used data. For the longer return periods of 1,000 and 10,000 years the difference of 20–30 cm is not very large considering that the confidence interval which is in the order of 1m for these return periods. The fairly small difference in water level may have larger effects on the required crest level due to the effect that this difference has on the wave height near the toe of sea dikes and subsequently on the wave run-up and overtopping. For the Petten sea defence a difference in water level thus leads to a difference in crest height of 0.6–1.0 m.

The formulae to determine the required crest height of the dikes use the wave conditions at the toe of the structure as input parameter. These wave conditions are often limited by the local water depth. In the commonly adopted method to determine the wave conditions at the toe of the sea dikes using numerical models, the design water level and the applied model are factors determining for the nearshore wave conditions. In these depth-limited conditions the deep-water wave conditions has a negligible influence on the nearshore conditions. The quality of the input for the crest level calculation depends therefore largely on the ability of the model to predict the correct wave height and the required characteristic wave period.

From the comparison of the various methods to determine the required crest height of the Petten sea defence it is concluded that the different methods lead to crest heights that vary in the order of magnitude of meters for the same return period. This confirms the results of the earlier study (DWW, 2001). The largest differences are caused by the formulae used to calculate wave run-up and overtopping. If some specific factors in the formulae are modified and the same hydraulic boundary conditions are used, the remaining difference is still in the order of 1 m. but a difference of a few decimetres in the water level or a different overtopping criterion lead also to differences in crest height in the order of 1 m.

The countries use different overtopping criteria for their sea dikes. The guidelines used in Germany and the United Kingdom give rather vague ranges for the conditions were damage may occur. Denmark and The Netherlands give clear criteria, which depend on the condition of the toplayer of the dike. These criteria require a somewhat subjective judgement of the quality of the grass cover and sand/clay layer in which the grass grows. The choice of the overtopping criteria is therefore to some extent a subjective decision and can be a reason for differences, especially because the required crest level is fairly sensitive to the applied criterion. Increasing the allowable overtopping rate from 1 l/s/m to 2 l/s/m leads for Petten to a crest height that is 0.6–1.0 m lower; a more strict run-up criterion of 2 % instead of 10 % in the Danish method leads to a crest that is about 3 m higher.

Even if the dike fails to meet the given criterion for overtopping or run-up, the dike does not necessarily fail immediately. The structure still has a certain remaining strength. For the development of risk-based approaches it is necessary to obtain more insight into the criteria for overtopping and run-up and the remaining strength once these are exceeded.

The approaches to the safety assessment of sandy coasts are quite different and range from time-dependent simulation of dune erosion (Niedersachsen) via an equilibrium profile method (The Netherlands, Belgium) to a simple criterion for the required width of the dunes (Denmark), though the latter is also based on erosion estimates using an equilibrium profile. The fixed criterion for the dune width used in Denmark is not related to a probability of exceedance. This is not suitable in a risk-based approach of flooding.

Several countries use methods or criteria for dune erosion that are based on the method developed by VELLINGA for wave periods up to 12 s (1983). Recent studies have shown that the erosion is significantly more for longer wave periods. Further research and development of new procedures to calculate dune erosion more accurately for these conditions is therefore relevant.

6.2 Conclusions and recommendations

Based on the comparison and analysis of the applied methods to determine the hydraulic boundary conditions and their use in the safety assessment of the sea defences the following general conclusions can be drawn:

- The methods used in the various countries to determine the required crest level lead to dike heights that can vary several meters for the same return period.
- Major factors for these differences in the crest height of sea dikes are:
  - The statistical methods to assess the design water level;
  - The quality of the prediction of the wave parameters at the toe of the dike;
  - The run-up and overtopping formulae including specific reduction factors and the way the representative slope is calculated for compound slopes and berms;
  - The strength criteria for overtopping and run-up.
- For the safety assessment of sandy coasts several countries use methods based on the work of VELLINGA (1983). The way in which this has been implemented in tools and criteria is quite different.
- Due to differences in methods adopted to determine the hydraulic boundary conditions and the strength criteria the results of risk assessments are hardly comparable. The other way around, a common approach to risk assessment might thus lead to adaptations in dike design in the various countries.

Based on the above conclusions the following general recommendations can be made:

- To further improve insight in the differences in the various methods to determine the hydraulic boundary conditions and in the strength formulations combined research, either in joint projects or by exchange of results, is recommended. On the longer term, this might lead to a convergence of the methods for risk assessment used in the various countries. Relevant aspects may include:
  - Statistical methods for determination of design water level for very long return periods;
  - Improving the quality of wave modelling tools by extensive validation for typical applications such as open coasts, estuaries and Wadden sea areas, including exchange of data for this purpose;
  - Development for better defined criteria for wave run-up / overtopping that leave less opportunity for subjective choices and that have a clear relation with the actual risk of failure and flooding.
- To obtain more insight into differences due to the geographical situation in each country, it is recommended to carry out a comparative risk analysis using a single method to derive hydraulic boundary conditions for a number of selected sites in the countries of the North Sea Coastal Managers Group.

7. Literature

ANDERSEN, J. O.: Flood protection in the Danish Wadden Sea Area. Coastal Engineering ,98 Proceedings of the Conference, American Society of Civil engineering, Page 3542–3552, 1998.

DWW: Flooding risk in coastal areas; An inventory of risks, safety levels and probabilistic techniques in five countries along the North Sea coast. Road and Hydraulic Engineering Division (DWW), 27 pp + appendices, 2001.

EAK: Empfehlungen für Küstenschutzwerke (Recommendations for coastal protection works). Die Küste, Heft 65, Kuratorium für Forschung im Küsteningenieurwesen, 2002.

HR Wallingford Ltd: Wave Overtopping of Seawalls. Design and Assessment Manual. R&D Technical Report W178, February 1999.

KRIEBEL, D. L.: User Manual for Dune Erosion Model EDUNE. U.S. Naval Academy, Annapolis, 1989.

RIKZ: Golfrandvoorwaarden langs de Nederlandse kust op relatief diep water (Wave boundary conditions along the coast of The Netherlands on relatively deep water). Report RIKZ-95.024 (2 vol.: text + appendices), December 1995.

RIKZ: Rapportage veldmetingen Pettermer Zeewering; Stormseizoen 2003–2004 (Report field measurements Petten sea defence, Storm season 2003–2004). Report RIKZ/2004.032, September 2004.

STEETZEL, H. J.: Cross-shore Transport during Storm Surges. PhD Thesis, Delft University of Technology. Also published as TAW Report No. C1-93.05, 1993.

TAW: Leidraad Toetsen op Veiligheid (Guidelines Assessment on Safety). Technische Adviescommissie voor de Waterkeringen, August 1999.

TAW: Technisch Rapport Golfoploop en Golfoverslag bij Dijken (Technical report wave run-up and wave overtopping at dikes). Technische Adviescommissie voor de Waterkeringen, May 2002.

VELLINGA, P.: Predictive Computational Model for Beach and Dune Erosion during Storm Surges. Proc. of Coastal Structures '83, Arlington / Virginia, ASCE, pp. 806–819, 1983.

WL | Delft Hydraulics: Physical model investigations on coastal structures with shallow foreshores; 2D model tests on the Petten Sea-defence. Report of Task 3.3 of the OPTICREST project (ref. MAS3-CT97-0116), author: M. R. A. van Gent, Prepared for: Commission of the European Communities, 1999.

WL | Delft Hydraulics: COMRISK; Sub-project 5: Hydraulic boundary conditions. Report H4203, prepared for: Rijkswaterstaat, DWW, 2005.