Prepared for:

Rijkswaterstaat, DWW

COMRISK

Sub-project 5: Hydraulic boundary conditions

Report

June 2005

COMRISK

Interreg North Sea Region

WL | delft hydraulics

H4203

Prepared for:

Rijkswaterstaat, DWW

COMRISK

Sub-project 5: Hydraulic boundary conditions

Johan Dekker

Report

June 2005

wL | delft hydraulics

WL | delft hydraulics

CLIENT:	Rijkswaterstaat, DWW
TITLE:	COMRISK Subproject 5: Hydraulic boundary conditions

ABSTRACT:

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 Management 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.

REFERENCES: Offer by WL | Delft Hydraulics dated 6 September 2004
Order by DWW dated 30 Septtember 2004 (ref. AK/044292, verplichtingennummer 3101/1922; project 'Vergelijking randvoorwaarden i.k.v. COMRISK SP5'

VER	ORIGINATOR		DATE	REMARKS	REVIEW		APPROVED BY	
1	J. Dekker		9 February 2005	Draft	M.R.A. van Gent		W.M.K. Tilmans	
2	J. Dekker		29 June 2005	Final	A.R. van Dongeren		W.M.K. Tilmans	

PROJECTIDENTIFICATION:	H4203
KEYWORDS:	Coastal safety, hydraulic conditions, North Sea
NUMBER OF PAGES:	60 + appendices
CONFIDENTIAL:	☐ YES, until (date) ☒ NO
STATUS:	☐ PRELIMINARY ☐ DRAFT ☒ FINAL

Contents

List of Figures

List of Tables

List of Symbols and Abbreviations

Symbol	Unit	Description
B	[m]	berm width
H_{m0}	[m]	significant wave height derived from the wave spectrum $4\sqrt{m_0}$
H_s	[m]	significant wave height, average of highest 1/3 of waves
$H_{1/3}$	[m]	other notation used for H_s
L	[m]	wave length
q	[m^3/s/m]	average overtopping discharge per unit crest length
R_c	[m]	crest height relative to still water level (SWL)
T_m	[s]	mean wave period (time domain analysis)
T_{m02}	[s]	mean wave period (spectral analysis) $\sqrt{m_0/m_2}$
T_{m-10}	[s]	spectral mean wave period (spectral analysis) m_{-1}/m_0
T_p	[s]	spectral peak wave period
α	[°]	slope angle
β	[°]	angle of wave attack (0 is perpendicular)
γ_b	[-]	reduction factor for a berm (run-up, overtopping)
γ_f	[-]	reduction factor for the surface roughness (run-up, overtopping)
γ_h	[-]	reduction factor for a shallow foreshore (run-up, overtopping)
γ_β	[-]	reduction factor for the angle of wave attack (run-up, overtopping)

Abbreviation	Description
BE	Belgium
DK	Denmark
NAP	Normaal Amsterdams Peil; reference level in The Netherlands
NN	Normal Null; reference level in Germany
NL	Netherlands
Nds	Niedersachsen
RWS	Rijkswaterstaat; Directorate General of Transport and Public Works in The Netherlands
SH	Schleswig-Holstein
SWL	Still Water Level
UK	United Kingdom

1 Introduction

1.1 General background

In 1996 national and regional coastal defence authorities in the UK, 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 transnational 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 also be extended to include the structural aspects of dunes and other flood protection structures, including storm surge barriers.

On the basis of these considerations, the idea of COMRISK was born. COMRISK aims at improved coastal flood risk management through a transfer and evaluation of knowledge and methods as well as pilot studies. The project runs from July 2002 to June 2005 and consists of the "umbrella" project and nine subprojects. In COMRISK many of the aspects touched on in the earlier study are treated in more detail. 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 (part of the Ministry of Transport, Public Works and Water Management in The Netherlands) is coordinator of this subproject. DWW contracted WL | Delft Hydraulics to assist in the inventory and comparison of the methods. COMRISK is co-financed by the European Union.

1.2 Approach to sub-project 5

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 – some were very generous with background documentation in their response – has been summarized into 6 separate documents that describe the methodology used by each of the partner countries. In Germany coastal defence is a task of the *länder*, so both Niedersachsen and Schleswig-Holstein are partners in this project. These documents (attached as Appendix A to F to this report) treat:

- the basic data that are used,
- the way these are processed to determine design/safety assessment conditions
- how the conditions along the toe of the sea defence are obtained from the data at the measurement locations
- the design formulae that are used for sea dikes (only the parameters required height and armour layer are considered)
- the way the strength or safety of dune coasts is evaluated

This inventory shows interesting similarities and differences in the methodologies adopted around the North Sea. A first impression was presented in a workshop in Ribe, Denmark, in November 2003. One of the most striking items is the large difference in the required safety level for the sea defences, which ranges from once in 50 years to once in 10,000 years. Other differences are in the technical approach, e.g. direct extrapolation of extreme high water levels versus separation of tide and surge with extrapolation of the surge. Possible reasons for differences may be the different geography of the various areas and the difference in population density. Similarities might be caused by the fact that the methodologies of all partners have their basis in the same technical-scientific literature.

Comparison

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. A first step thereto could be to make a comparison of 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 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 were selected:

- Petten sea dike, a sea dike directly at the North Sea,
- Dune coast at Callantsoog.

The location of these sites is shown in Figure 1.1. A description is presented in Section 2.1.

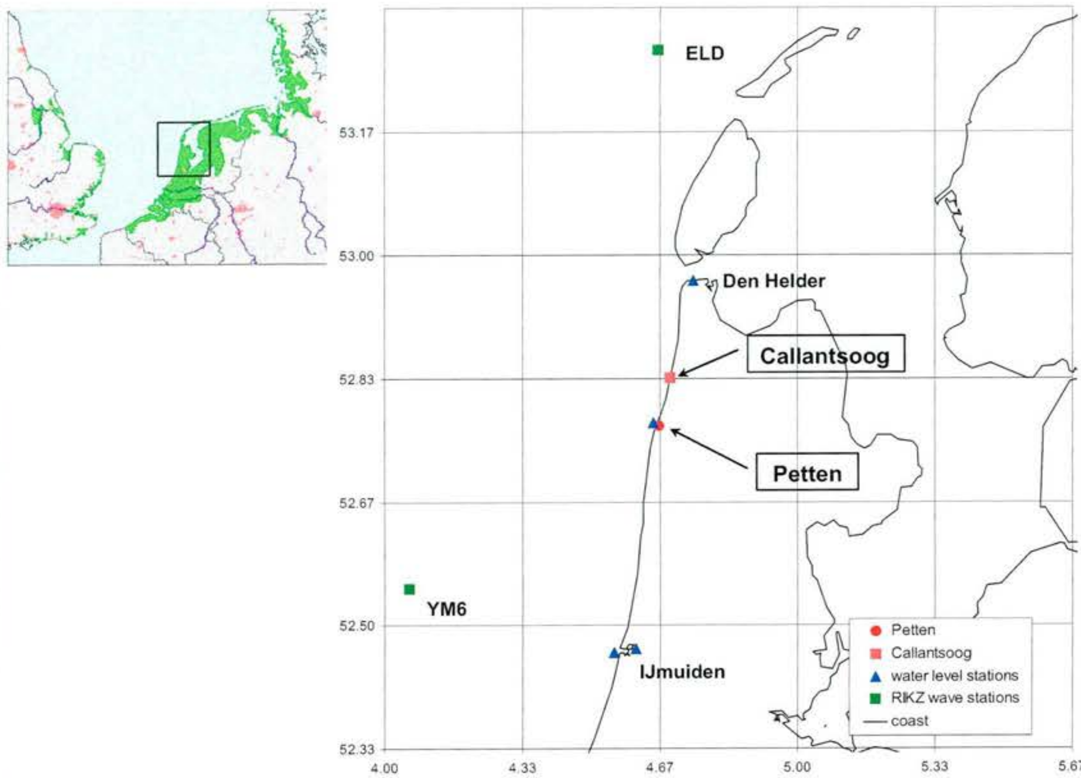


Figure 1.1 Location of the Petten Sea-defence and Callantsoog

1.3 Approach to comparison

In the study comparing approaches in the five countries along the North Sea coast carried out earlier (DWW, 2001) it was already found that the differences in crest height of a sea dike can be fairly large, due to the different safety level that is adopted and due to the differences in the design procedures. This difference can be in the order of a few meters. Based on two example cases the study showed that when the same design procedure is used, the difference in crest height due to the safety level can be up to about 1.5m. If the same safety level is adopted the crest height can vary 2 to 3m due to the difference in design procedure.

However, when crossing the border from one country to another, the difference in crest height may not be as dramatic as the above suggests, though the heights are certainly different on the Dutch-German border (Klein Breteler, personal communication). This means that the combination of safety level and design procedure may lead to similar dike heights. The purpose of the present comparison is therefore to study the various methods in more detail to see whether the differences in crest height found in 2001 remain, when all steps in the process to assess the crest height are carried out according to the same methodology. A reliable comparison can only be made when the hydraulic boundary conditions and the applied criteria are consistent with the formulae used to calculate the crest height. By comparing the each of the steps in the methods to determine the crest height, we also hope to find possible reasons for differences that may remain.

Based on the information gathered through the questionnaires (sometimes including various supporting documents provided by the partners), the descriptions in earlier study (DWW, 2001) and other information (e.g. found on the website of the partners), 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 assess the safety of 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 the four case studies treated in subprojects 6 to 9.

1.4 Set-up of report

After this introduction Chapter 2 gives first a brief description of the selected sites and the data that have been used. The following Chapters present the comparison of the way the hydraulic boundary conditions for the water level (Chapter 3) and the wave conditions (Chapter 4) are derived. The procedures to assess the required crest height for sea dikes are described and discussed in Chapter 5. The approaches to the safety assessment of sandy coasts are described in Chapter 6. Finally Chapter 7 presents some general conclusions and recommendations. Background information for the six countries is presented in the country descriptions in the appendices.

2 Sites and data

2.1 Description of the sites

2.1.1 Petten

The dike section near Petten considered in this comparison is a part of the Hondsbossche and Pettemer Zeewering. This is a sea dike of about 5.5 km long in the province of North-Holland on the North Sea coast of The Netherlands. The Pettemer Zeewering is the northerly 1.7 km of this sea dike. The majority of this part of the coast consists of dunes, but early in the 16th century started the construction of a dike protecting this low-lying part of the coast. Since then the adjacent dune coast has retreated so that the Pettemer Zeewering is now protruding 50 – 100 m relative to the dunes. Figure 2.1 shows an aerial photograph of this dike section.

Figure 2.1 The Petten Sea-defence (photo: RWS - RIKZ/ AGI)

The site was selected for the present comparison, because it is an important section in the coastal defence of the western part of The Netherlands. A further advantage of this dike section is that Rijkswaterstaat is carrying out field measurements at the site since 1994.

Each winter season (roughly from October to May) various hydraulic parameters are measured on several locations along an 8 km cross-shore section. This provides additional data on the nearshore hydraulic conditions that are useful for this study.

The Pettemer Zeewering has a berm and the lower part of the slope is protected with an asphalt top layer. The slope protection in the upper part consists of basalton blocks. The inner slope, the crest and a small part of the outer slope have a grass cover. The toe of the dike is at a level of about NAP-0.5 m. The lower part of the seaward side has a slope of 1:4.5 up to a level of NAP+5.0 m. The berm between NAP+5.0 m and NAP+5.7 m has a slope of 1:20 and is about 14m wide. The upper part has a slope of 1:3 and the crest is at NAP+12.75 m. Figure 2.2 shows this cross-section together with a recent survey of one section of the dike (section 21.00).

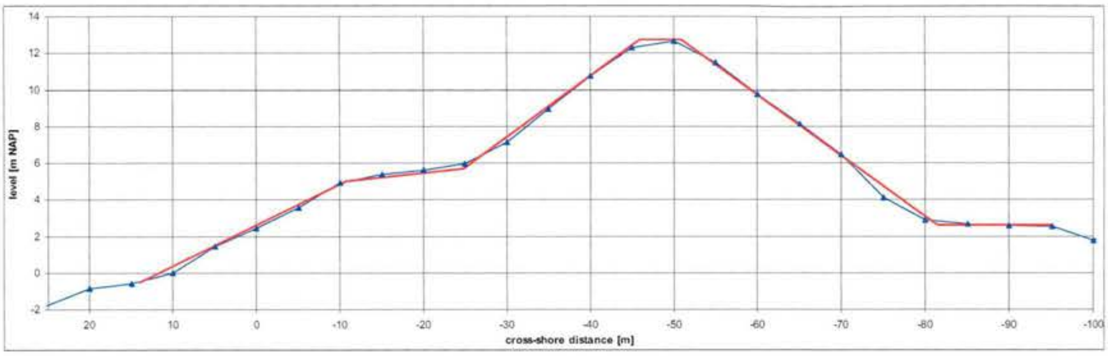


Figure 2.2 Typical cross-section of the Petten Sea-defence in March/April 2003 (blue line, section 21.00) and the approximation used in this study (red line)

A typical section of the foreshore at Petten is shown in Figure 2.3, but it must be noted that this is just a snapshot at one moment in time. In reality the bathymetry is highly variable and changes even during severe storms. Especially the position of the nearshore bar is changing continuously.

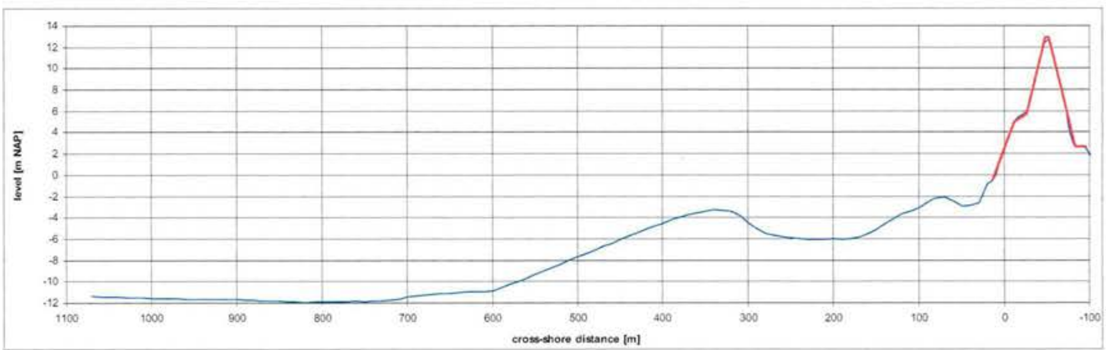


Figure 2.3 Typical cross-section of the foreshore at Petten Sea-defence in March/April 2003 (section 21.00)

2.1.2 Callantsoog

The dune coast considered in this study is located near Callantsoog, about 7.5 km north of Petten (Figure 1.1). Between Petten and Callantsoog the dunes protecting the land are rather wide (1-2 km), but near the village of Callantsoog the dunes consist of only a single row some 100m in width. An aerial photograph of the coast near Callantsoog is shown in Figure 2.4. The photo shows the village with the narrow strip of dunes on the left. The very right of the picture shows the transition to the coastal section were the dunes are about 2 km wide. The approximate location of the section considered in this study is shown with the yellow line.

Figure 2.4 The dunes near Callantsoog (photo: RWS - RIKZ/ AGI)

The row of dunes reaches a height of about 20 m as can be seen in Figure 2.5. The land behind the dunes has an elevation of about NAP+2.5 m. The foreshore in this cross-section of the dunes is shown in Figure 2.6. It can be seen that the seabed is at 4-5m below NAP for a distance of about 600 m. It then drops to about NAP-8 m, from where it becomes gradually deeper to about 10 m-NAP at about 1500 m from the shore.

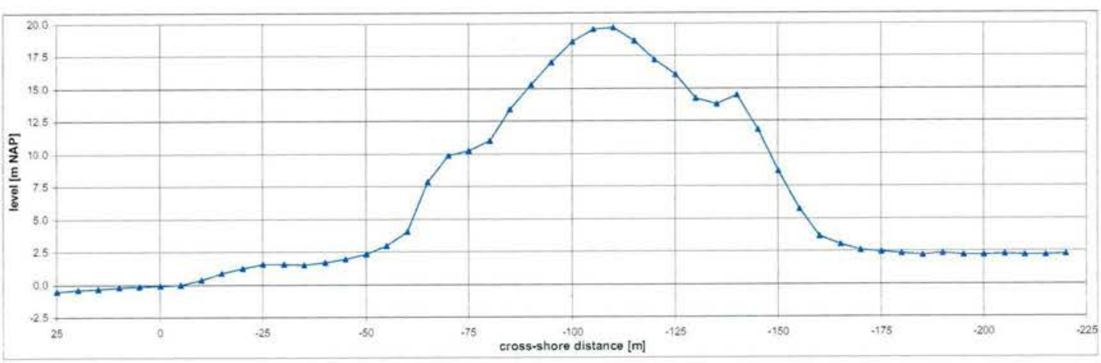


Figure 2.5 Typical cross-section of the dunes near Callantsoog in March/May 2003 (section 13.60)

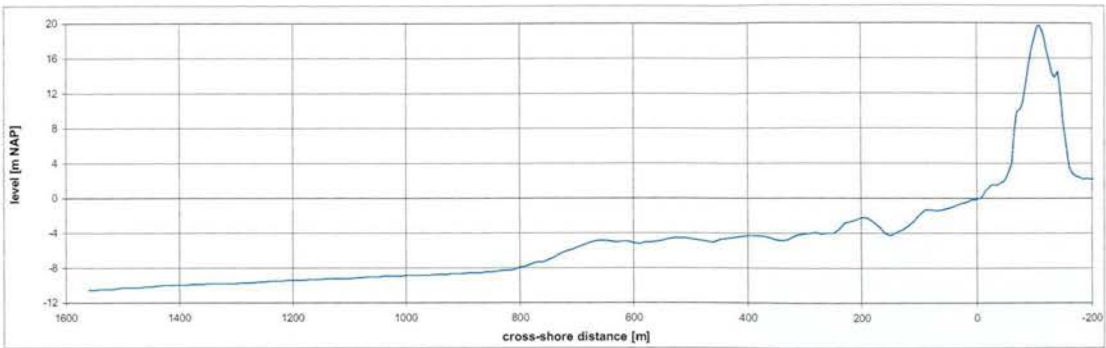


Figure 2.6 Typical cross-section of the foreshore near Callantsoog in March/May 2003 (section 13.60)

2.2 Description of the hydraulic data

2.2.1 Water level

The stations with water level measurements relevant for the present study are IJmuiden, Petten and Den Helder. Figure 1.1 shows the location of these sites. Historical data on the observed water levels in these stations were downloaded from the Rijkswaterstaat database (`www.waterbase.nl`). Table 2.1 gives a summary of these data. Summary sheets with information on tidal levels, high water levels for various exceedance frequencies and some observed extreme events were obtained from a second database of Rijkswaterstaat (`www.waternormalen.nl`). These summary sheets show that the high water levels for different exceedance frequencies at Petten can be approximated by interpolating between the values for Den Helder and IJmuiden based on the distance between the stations (up to 3 cm difference). Note that this does not apply for the tidal levels.

Station	Coordinates		Period	Details
Den Helder	111850	553230	1932-2003	1932-1960: 3-hourly data (at 2:40, 5:40, etc) 1961-1970: 3-hourly data (at 2:00, 5:00, etc) 1971-1986: hourly data 1987-2003: 10-min data
Petten Zuid	105230	531960	1977-2003	1977-1985: hourly data 1986-2003: 10-min data
IJmuiden Buitenhaven	98430	497500	1981-2003	1981-1987: hourly data 1988-2003: 10-min data
IJmuiden Noordersluis	101850	498010	1924-1983	1924-1960: 3-hourly data (at 2:40, 5:40, etc) 1961-1970: 3-hourly data (at 2:00, 5:00, etc) 1971-1983: hourly data

Table 2.1 Details of water level data for relevant stations

The above mentioned data are all water levels relative to NAP. For Den Helder and IJmuiden Rijkswaterstaat (RIKZ) provided further time series of the observed surge at high water. This is the surge defined as difference between the observed high water level and the corresponding tidal high water level irrespective of the moment when these occur (*scheve opzet*).

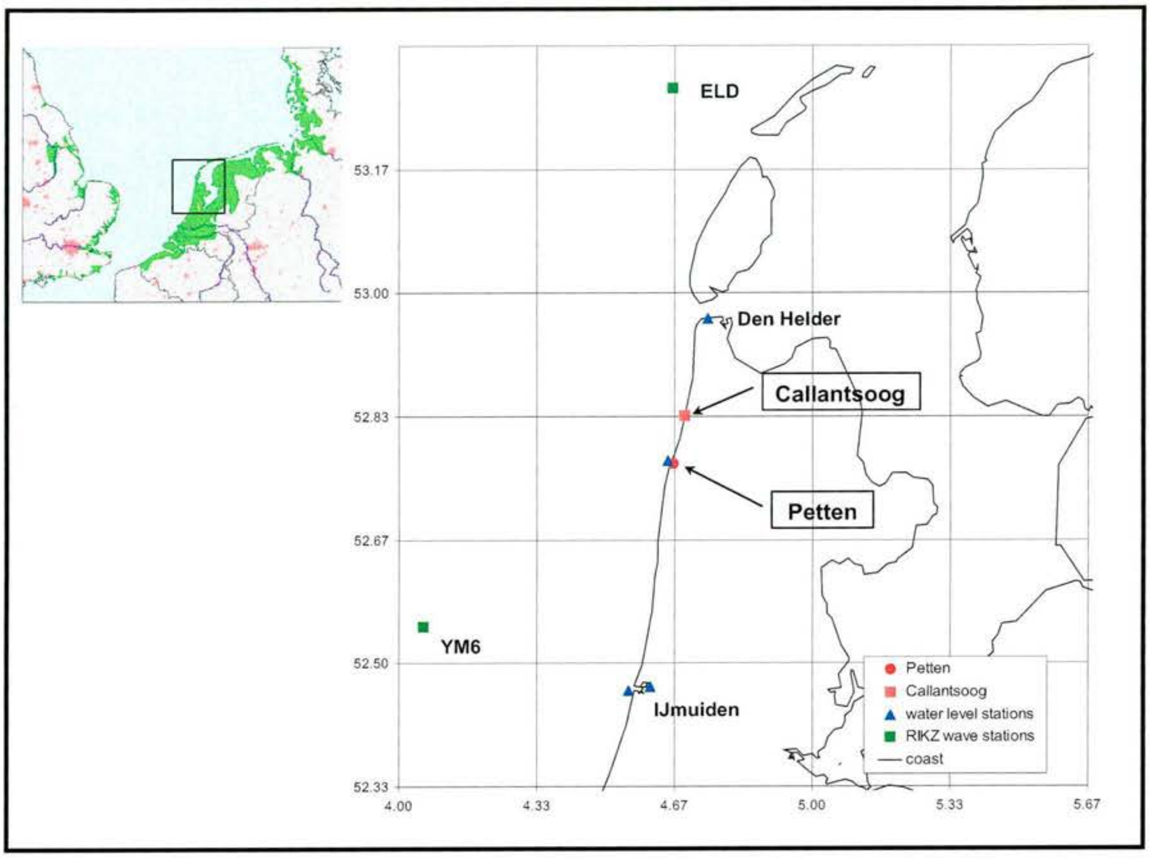


Figure 2.7 Location of water level stations and wave buoys near Petten and Callantsoog

2.2.2 Waves

Data on the deep water wave conditions for 9 measuring stations around The Netherlands are available in a database of RIKZ (`www.golfklimaat.nl`). Complete time series at 3-hourly intervals are available in all stations for the period 1979-2001. Gaps in the measurements are filled in by estimating the relevant parameters using a neural network technique based on data from surrounding locations, wind speeds and measurements in the preceding hours.

The files include apart from date and time the following parameters:

- wave height H_{m0}
- accuracy of the wave height H_{m0} (standard deviation)
- wave height $H_{1/3}$
- wave height H_{TE3} (=H_{m0} of the low-frequency part of the spectrum, 0.03 – 0.10Hz)
- wave period T_{m02}
- wave period $T_{H1/3}$
- wave direction Th_0
- wind direction
- wind speed

- water level
- surge

The wind and water level data are for some stations from a relevant nearby station. A code indicates the origin of the data (measured vs. interpolated, distance of wind/water level station).

From the 9 available stations Eierlandse Gat (ELD) and IJmuiden (YM6) are most relevant for the sites of Petten and Callantsoog. Their location is shown in Figure 1.1.

2.2.3 Nearshore wave conditions

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. This would require installation of and familiarizing with various programs only for the purpose of a few computations. Several of these are also commercial software for which a license is required. If the models are well validated and/or calibrated, either more general or for the specific sites that are considered, each model should provide nearshore conditions that are in the same order of magnitude. Instead the Rand2001 database has been used in the present comparison to assess the nearshore wave conditions.

The Rand2001 database consists of a user interface program that can be used to retrieve data from large MSAccess databases containing the results of many wave propagation simulations with the program SWAN. Databases are available for five coastal areas. One of these contains data for selected sections of the North Sea coast, mainly sections of the coastal defence protected by hard structures such as dikes or breakwaters. Other databases cover the dikes along the Western Scheldt estuary, the Eastern Scheldt estuary, the westerly part of the Wadden Sea and the easterly part of the Wadden Sea. The databases provide the results of the SWAN simulations in locations close to the toe of the dike at a typical spacing of about 200 m.

The database for the North Sea coast contains the results of 219 SWAN simulations for 3 water levels (2, 4, 6m above NAP), 14 directions and 5-7 combinations of wind speeds and corresponding offshore wave conditions (Alkyon, 1999). For the comparison of the design of the Pettemer Zeewering the results in the location X=105627 m, Y=531679 m were exported from the database. This provides a table containing for all combinations of water level, wind direction and wind speed the corresponding nearshore wave conditions including significant wave height, various characteristic wave periods (such as T_{m02}, T_{m-2-1}, T_p), the wave direction and the directional spreading. In the SWAN simulations the deep-water wave height was varied based on the data from the stations ELD and YM6. In the Rand2001 the deep-water wave conditions at ELD are stored as reference values. These are also included in the exported table with results.

3 Assessing the water level

3.1 General

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 more in Germany, which have a long coast line with various estuaries. National authorities gather the data and the available information goes for some stations back for 100 years or more.

3.2 Data processing methods

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 generally 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.

3.3 Extreme value analysis

3.3.1 Denmark

In Denmark extreme water levels are determined by extrapolation of observed water levels (Kystdirektoratet, 2002) using a Peak-over-Threshold method. From the various extreme value distributions that were applied, the Log-Normal distributions showed usually the best fit to the data in the most southerly stations along the North Sea coast (Wadden area) and in the tidal inlets. In the other locations the Weibull distribution performed better. These two distributions are fitted to the data above a certain threshold level to determine the extreme water levels for return periods of 20, 50 and 100 years. The parameters of the extreme value distributions are determined using the Maximum Likelihood method. The adopted threshold level leads to 1 observation for every 2-5 years of the observation period.

For the evaluation of extreme water levels at Petten and Callantsoog the Weibull and Log-Normal distributions were fitted to various selections from the available data (see Table 2.1) using WL | Delft Hydraulics program for extreme value analysis SCATTER/EVA. The results showed that both distributions gave usually similar results for the shorter return periods (say up to 100 years), but that the Log-Normal distribution has a tendency to produce extremely high values for the longer return periods.

The two distributions were fitted to all available water level data from Den Helder and IJmuiden. To obtain independent data a time-window of 24 hours was applied. The threshold level was chosen in such a way that the number of remaining events was about half of the period of observation. This is similar to the number of observations used in Danish procedure. Based on the goodness-of-fit parameters it appeared that in Den Helder the Weibull distribution showed the best fit and in IJmuiden the Log-Normal distribution. The extreme water levels predicted using the mentioned distributions are shown in Table 3.1. The value for Petten was determined from the values for Den Helder and IJmuiden by interpolation based on the distance to these stations.

station	**probability of occurrence**			
	1/50 yr	**1/100 yr**	**1/1,000 yr**	**1/10,000 yr**
Den Helder (Weibull)	2.95	3.14	3.77	4.42
Petten-Zuid	3.00	3.19	3.90	4.76
IJmuiden (Log-Normal)	3.08	3.27	4.11	5.29

Table 3.1 Design water level using the Danish method

3.3.2 Schleswig-Holstein and Niedersachsen

In the Germany three different criteria form the basis for the design water level (EAK, 2002):

- The single value criterion (*Einzelwertverfahren*) combines a number of single values that do not belong to the same high water event. The design water level consists of the sum of
  - the level of mean high water above mean sea level (NN)
  - the difference between the highest high water and mean high water,
  - the highest observed wind set-up,
  - a safety margin to account for trend in the mean high water or mean sea level
- The reference value criterion (*Vergleichsverfahren*) specifies that the water level should not be lower than the highest storm level recorded so far corrected for any trends between its occurrence and the present,
- The statistical criterion (*Statistisches Verfahren*) requires that water level should have a probability of occurrence of n = 0.01 (once in 100 years) considered for the year 2000.

In Schleswig-Holstein the highest of the three criteria is used as design level of the sea defences (SH, 2002). For the North Sea coast of Schleswig-Holstein the statistical criterion is the governing criterion. For the Baltic coast the design level is determined by the reference value criterion (extreme storm surge of 1872).

In Niedersachsen the highest of the first two criteria is adopted as the design water level.

These methods have been applied to the data for Den Helder and IJmuiden. Schleswig Holstein uses the Jenkinson-D distribution to assess the water level for the statistical criterion. As a tool for this distribution was not available, the water levels with a probability of occurrence of 1/100 from the Danish procedure were used for the statistical criterion. The highest observed water level was available in the summary sheets from RWS. These occurred all during the severe storm of 1 February 1953. For the highest observed surge the highest value in the data with high water surge values was taken. From the results for Den Helder and IJmuiden the corresponding value for Petten has been estimated by interpolation. The results are shown in Table 3.2. It should be noted that the highest water level found by interpolating between the values for Den Helder and IJmuiden (NAP+3.48m) is higher than the highest recorded water level at Petten Zuid given in the summary sheets of RWS (NAP+3.20m).

It can be seen in Table 3.2 that the single value method is determining in this case. The statistical method, which is only used in Schleswig-Holstein, leads to the lowest values. The design water level of NAP+3.84m at Petten is similar to the value with a probability of 1/800 according to the Danish method.

	statistical method	**reference value method**	**single value method**		
station	1/100 yr water level [m NAP]	highest water level [m NAP]	highest surge [m]	MHWS [m NAP]	MHWS + surge [m NAP]
Den Helder	3.14	3.25	2.85	0.66	3.51
Petten-Zuid	3.19	3.48	2.89	0.95	3.84
IJmuiden	3.27	3.85	2.94	1.15	4.09

Table 3.2 Design water level according to the three criteria used in the German method.

If the statistical criterion of 1/100 that is used in Schleswig-Holstein, is taken as a more general statistical criterion and modified according to the probabilities considered in this comparison, it appears that the statistical criterion is governing for the return periods longer than about 800 years (see Table 3.3). In Schleswig-Holstein the statistical criterion is also governing for the coastal areas along the North Sea.

method	**probability of occurrence**			
	1/50 yr	**1/100 yr**	**1/1,000 yr**	**1/10,000 yr**
Schleswig-Holstein	3.84	3.84	3.90	4.76
Niedersachsen	3.84			

Table 3.3 Design water level at Petten for different probabilities using the methods of Schleswig-Holstein and Niedersachsen.

3.3.3 The Netherlands

The design water levels in The Netherlands are based on a combined approach of statistical analysis and physical research based on modelling (RIKZ, 1993a,b,c,d). In the statistical evaluation various distributions were tested. The "distribution free method-0" (VVM-0, *Verdelings vrije Methode-0*) was adopted based on theoretical considerations and because of the results for the 5 stations considered in the evaluation. This method was then also applied to the data of other stations. The design water levels for the sea defences in between the water level stations was determined interpolation based on the results of a hindcast study of extreme water levels.

Table 3.4 shows the design water levels for various probabilities of occurrence. These are taken from the summary sheets of RWS. It can be seen that the design water level increases towards the south.

station	probability of occurrence			
	1/50 yr	1/100 yr	1/1,000 yr	1/10,000 yr
Den Helder	3.20	3.40	3.95	4.45
Petten-Zuid	3.25	3.45	4.10	4.70
IJmuiden	3.40	3.60	4.35	5.15

Table 3.4 Design water level using the Dutch method

3.3.4 Belgium

In Belgium the design water level is based on a statistical evaluation of high water surges. These are defined as the difference between the maximum observed water level and the maximum astronomical water level at high water. These do not necessarily have to occur at the same moment, since the storm surge will delay the time of maximum water level.

The design water level is based on the theoretical work of Beirlant (1996). For this method no theoretical distribution is assumed, but three typical distributions are examined each time, distinguished by the extreme value index γ. The Weibull, exponential and log-normal distributions have an index $\gamma=0$; the Pareto distributions an index $\gamma>0$ and the (rarely occurring) beta-distributions an index $\gamma<0$.

This procedure could not completely be reproduced in the present comparison, but the approach was approximated by carrying out an extreme value analysis on the HW-surges for Den Helder and IJmuiden. surges. Four distributions available within our EVA program were fitted to the POT values (thresholds of 1.4m for Den Helder and 1.2m for IJmuiden). The distribution with the best fit (based on the results of the goodness-of-fit tests Chi-square, Kolmogorov-Smirnov and regression) was adopted. For both sites this appeared to be the Weibull-distribution with Maximum-Likelihood estimators for the parameters of the distribution. The surge in Petten was determined by interpolation between the values for Den Helder and IJmuiden. The results are shown in the top of Table 3.5. To these extreme surges the mean high water in each of the locations was added to obtain the design water level for the selected return periods. These are also shown in Table 3.5.

	station	probability of occurrence			
		1/50 yr	1/100 yr	1/1,000 yr	1/10,000 yr
surge [m]	Den Helder	2.47	2.68	3.41	4.18
	Petten-Zuid	2.44	2.65	3.40	4.16
	IJmuiden	2.39	2.61	3.37	4.14
water level [m NAP]	Den Helder	3.05	3.26	3.99	4.76
	Petten-Zuid	3.25	3.46	4.21	4.97
	IJmuiden	3.36	3.58	4.34	5.11

Table 3.5 Design water level using the Belgian method.

It can be seen that the surges at the two sites are nearly the same, the values in IJmuiden being slightly smaller. This is opposite of what is usually found. It is interesting to note that the design water levels for IJmuiden are about the same as the official values according to the procedure adopted in The Netherlands (see Table 3.4). The values found here for Den Helder are lower than the official values for the shorter return periods and higher for the longer return periods.

3.3.5 United Kingdom

In the United Kingdom there is no strictly prescribed method to be adopted to determine the design water levels. Each consultant involved in a project can use his own methods. Lacking information on the methods that consultants in the UK use, the procedures applied in the UK can not be reproduced here. To determine the extreme water level in a project, the usually adopted approach at WL | Delft Hydraulics is to carry out an extreme value analysis on the surges and combine this with the tide. This combination can be done by simply adding a typical astronomical level (as done above in the approximation of the Belgian method), but can also be more sophisticated by considering the joint occurrence of the astronomical level and the surge. For the present comparison of the various methods, we have adopted for the UK method the water levels shown in Table 3.5 for the Belgian method.

3.3.6 Summary

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 factors such as local wind set-up and relative sea level rise.

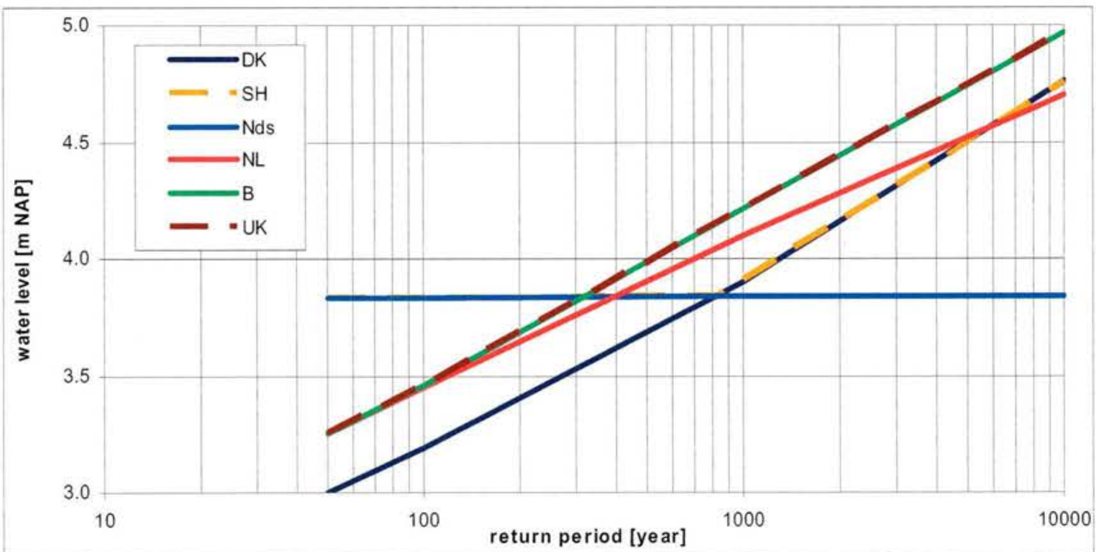
The results of the various methods to assess the design water levels are summarised in Table 3.6 and Figure 3.1. The most striking in this table is of course the single value independent from the probability of occurrence following the method Niedersachsen. This is inherent to the design method *Einzelwert-Verfahren*, which "aims to avoid any exceedance"(quote from response to questionnaire). It can be seen that the value for Petten following this method has a probability of occurrence between 1/400 (NL & BE method) and 1/800 (DK method).

method	**probability of occurrence**			
	1/50 yr	**1/100 yr**	**1/1,000 yr**	**1/10,000 yr**
DK	3.00	3.19	3.90	4.76
SH	3.84	3.84	3.90	4.76
Nds *	3.84			
NL	3.25	3.45	4.10	4.70
BE & UK	3.25	3.46	4.21	4.97

*) Niedersachsen uses a deterministic approach

Table 3.6 Design water level using the various methods (in m above NAP).

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 and probably within the accuracy of various methods. The 95%-confidence interval for the 1/10,000 yr surge is in the order of 1 m.



Note: the curve for SH coincides with the curve for Nds for return periods between 50 and 700 yr

Figure 3.1 Comparison of design water levels following different methods.

For the shorter return periods of 50 and 100 years, however, we would expect to have better agreement between the various methods. This may be caused by the fact that the Danish method was applied to different data. The Dutch method and our approximation of the Belgium method were carried out based on the highest observed water level at high water and on the surges at high water respectively. For the Danish method measured water level data at regular intervals were used. In the period before 1971 these are 3-hourly data, which is too coarse to represent the tide accurately. This means that high water may be missed in the data used for the Danish method.

4 Assessing the wave conditions

4.1 General

Most countries use fairly similar methods to assess the wave conditions in the vicinity of the sea defences. Their approach is mostly based on a combination of deep water wave data (either from measurements or hindcast) combined with wave modelling to determine the corresponding conditions near the coast. Only Schleswig-Holstein has a quite different approach which is based on direct assessment of the nearshore conditions by correlating the wave conditions to the still water level. Schleswig-Holstein measures deep water waves and wind on a location off Sylt since 1984 (21 years) as a basis for sand nourishment. 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.

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. In the present comparison, the Rand2001 database (see Section 2.2.3) has been used to assess the nearshore wave conditions. The method adopted in Schleswig-Holtein to determine the nearshore wave conditions has been applied using wave measurements near this dike (see Section 4.3).

In the present comparison, the Rand2001 database (see Section 2.2.3) has been used to assess the nearshore wave conditions. The significant wave height H_{m0}, mean wave period T_{m02} and the peak wave period T_p were interpolated from the results based on the derived water level and deep water wave height. 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.

4.2 Deep water wave conditions

The first line of Table 4.1 shows the official Dutch values for the extreme wave conditions for the station Eierlandse Gat, the reference station used in the RAND2001 database for the section of the coast of Petten and Callantsoog. These have been determined using a Weibull-distribution with a fixed shape parameter of 2.62 and the threshold significant wave height of 4.0m.

For this comparison we have carried out an extreme value analysis on these wave height data using in-house software. From various distributions that were tested a 3-parameter Weibull-distribution showed the best fit to the data above the threshold of 4.8m. The resulting wave heights for the selected return periods are included in Table 4.1. It can be seen that the difference in wave height is 0.3-0.5m. As it appeared that this difference has no significant effect on the nearshore conditions when using the RAND2001 database (the remaining difference is only 1-2 cm, see Section 4.3) no other methods to assess the deep water wave conditions have not been tested.

method	waves 1/50		waves 1/100		waves 1/1000		waves 1/10000	
	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	

Table 4.1 Extreme wave conditions at Eierlandse Gat adopted in the comparison

4.3 Wave conditions at the toe

4.3.1 Wave conditions based on numerical modelling

All countries except Schleswig-Holstein use numerical models to determine the design wave conditions at the toe of the sea defences. As mentioned above, the nearshore wave heights for these methods have been assessed based on the RAND2001 database. From this database the results in the location X=105627m, Y=531679m, which is closest to the considered section of the dike, were exported from the database. The results from the SWAN simulations for the wind and wave direction 285°N, which is the most unfavourable direction in this location, where used to determine the wave conditions at the toe of the Pettemer Zeewering. The significant wave height H_{m0}, mean wave period T_{m02} and the peak wave period T_p were determined by bilinear interpolation in the Rand2001 results based on the design water level (see Table 3.6) and the deep water significant wave height (Table 4.1). The results are shown in Table 4.2.

method	waves 1/50		waves 1/100		waves 1/1000		waves 1/10000	
	H_s [m]	T_m [s]	H_s [m]	T_m [s]	H_s [m]	T_m [s]	H_s [m]	T_m [s]
DK	2.61	6.56	2.74	6.66	3.19	6.93	3.69	7.15
SH	3.54	6.69	3.54	6.69	3.54	6.69	3.54	6.69
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
BE	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

Table 4.2 Wave conditions at the toe of the Pettemer Zeewering.

4.3.2 Wave conditions using the method of Schleswig-Holstein

In Schleswig-Holstein the design wave conditions in front of the sea defence are determined by correlation with the water level using the following relations:

$$H_{1/3} = (SWL - DZ) * Gr \tag{4.1}$$

$$T_z = a + b * H_{1/3} \tag{4.2}$$

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). It has been assumed that T_{m02} is representative for T_z.

The correlation between SWL and the significant wave height in station 6, which is close to the shore, is shown in the left panel of Figure 4.1 for all wave heights larger than H_s = 0.5m. The right panel of the figure shows the correlation between the wave height and the mean wave period for all wave heights over H_s = 0.75m in this location. As the automatic fitting procedures gave results that seemed to err on the low side for higher water levels and higher wave heights (lines shown in blue), the correlation was determined using a visual fit to the data (shown in red).

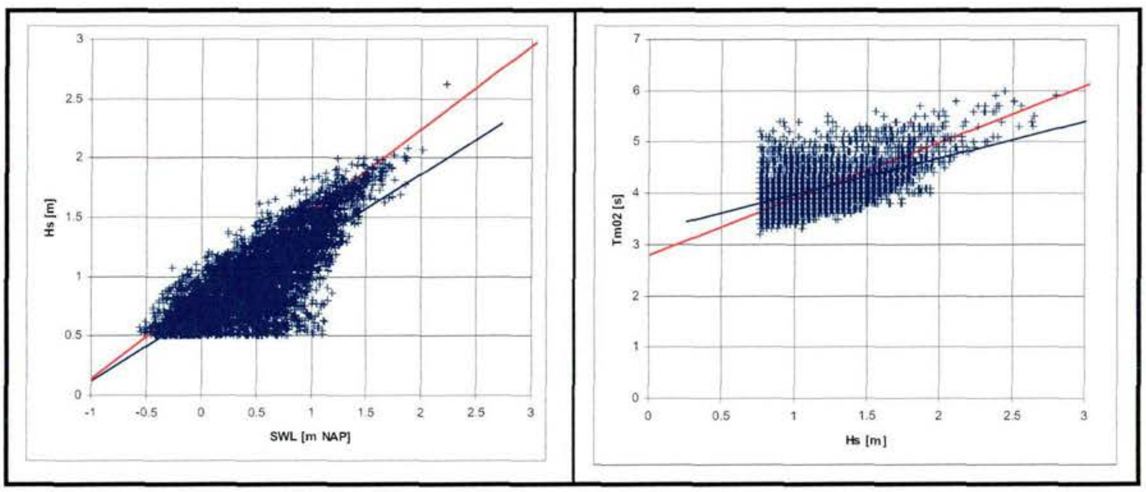


Figure 4.1 Correlation of water level and significant wave height (left) and significant wave height and mean wave period (right) near the Petten sea defence (measuring location 6)

From this correlations the following parameters were derived:

- DZ = -1.21m
- Gr = 0.7
- a = 2.8
- b = 1.1

The wave heights and periods for the design water level that follow from this correlation are included in Table 4.2. It can be seen that the wave heights are considerably higher than those derived for the same water level using the Rand2001 database (as given in Table 4.2 for Niedersachsen, Nds). Possible reasons for the difference are:

- a difference in depth in the output location of the numerical model,
- the fact that the measured wave heights are a combination of incoming and reflected waves, whereas the waves in the Rand2001 database are only the incoming wave component,
- the capability of the model to reproduce reality.

4.4 Wave period ratios

The RAND2001 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 the German *länder* Schleswig-Holstein and Niedersachsen 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 RAND2001 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, 1999a). 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 test 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 RAND2001 was around 2, the ratio T_m/T_{m02} was taken as 1.4.

The formulae for wave overtopping used in The Netherlands and Belgium use the spectral mean period T_{m-10} instead of the mean wave period T_m (assumed to be equal to T_{m02}). The ratio between these two periods 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 carried out in Section 5.5 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. For standard single-peaked spectra (Pierson-Moskowitz or JONSWAP) much lower values need to be adopted.

5 Approaches to dike evaluation / design

5.1 General

The approach to the cross-section of the sea dikes of the North Sea countries is quite different. In Denmark and Schleswig-Holstein were the design of new dike sections is carried out by a central authority the cross-section of new or improved dikes is more or less the same. In Denmark the dikes are usually grass-dikes with an outer slope of 1:7 to 1:10 and an inner slope of 1:3. Both outer and inner slope have a clay cover for protection against wave attack and wave overtopping (Andersen, 1998).

In The Netherlands the cross-section of the sea dikes differs from location to location and depends on the local circumstances and history. Regional water boards are in charge of design and maintenance of the sea dikes based on national guidelines. Most dikes have an outer slope with a protection of asphalt or stone revetment. The inner slope consists usually of grass; depending on the circumstances the crest and the upper part of the outer slope occasionally too.

In Belgium the hard coastal defences on the North Sea coast are mostly located in the towns and the dikes are completely covered with stone or asphalt.

5.2 Crest height

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. Table 5.1 shows a diagram with the various components taken into account in the required crest level.

	DK	SH	Nds	NL	BE	UK
land subsidence	X	X	X	X	X	
sea level rise		X	X	X	X	
squalls, oscillations and local wind set-up			X	X	X	
settlement /compaction of structure		X	X	X	X	
safety margin			X	X		

Table 5.1 Additional factors taken into account in determination of required crest level.

From the above factors taken into account to determine the crest level, the largest differences seem to be in the required height to account for the effect of waves. As this is the largest factor on more exposed locations, the methods to determine this factor are compared in some more detail in the next section.

5.3 Comparison of run-up / overtopping formulae

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). The formulas that are used in the various countries are given in the country descriptions in the Appendices. For the wave run-up criteria, the run-up height computed with the given formulas can directly be used to determine the crest level. 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 above formulas for the overtopping rate is

$$q = a \cdot \exp\left(-bR_c\right) \tag{5.1}$$

it follows that the crest level above the still water line as function of the overtopping rate is given by

$$R_c = -\frac{1}{b}\ln\left(\frac{q}{a}\right) \tag{5.2}$$

The crest levels as function of the criterion for the overtopping rate and the wave conditions are thus given by the formulas given below. For completeness the formula for run-up used in Denmark is also repeated here.

Denmark

In Denmark wave run-up is used to determine the contribution of the waves in the required crest level of the dike. The wave run-up is computed using:

$$Z_n = C_n \cdot T_m \cdot \sqrt{g \cdot H_s} \cdot \tan\alpha \tag{5.3}$$

in which Z_n is the run-up level (n is the percentage which depends on the inner slope, see Appendix A), C_n is a coefficient that depends on the percentage n and T_m is the characteristic period. Which characteristic wave period is meant is not exactly clear; the earlier study (DWW, 2001) used $T_p/1.15$. This is probably an approximation of another characteristic wave period, but lacking the detailed information $T_p/1.15$ has also been used in this study. It may be noted here that Eq. (5.3) was not used in the most recent dike design for the Rejsby Dike in 2001. The cross-section of this dike was directly based on scale model tests at DHI.

Schleswig-Holstein

Rewriting the equations for wave overtopping given in Appendix B leads to the following expressions for the required relative crest level:

$$R_c = -\frac{\sqrt{g}}{5.2\sqrt{2\pi}} \cdot \frac{\gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_h}{\cot\alpha} \cdot \sqrt{H_s} \cdot 1.25T_m \cdot \ln\left(\frac{\sqrt{2\pi}}{0.06g} \cdot \frac{q\sqrt{\cot\alpha}}{H_s \cdot 1.25T_m}\right) \tag{5.4}$$

with a maximum of

$$R_c = -\frac{1}{2.6}\cdot\gamma_b\cdot\gamma_f\cdot\gamma_\beta\cdot\gamma_h\cdot H_s\cdot\ln\left(\frac{q}{0.2\sqrt{g\,H_s^3}}\right) \tag{5.5}$$

in which T_m is the mean wave period in time domain (often indicated as T_z, the zero-crossing wave period). The available documentation (SH, 2002) mentions that these equations are based on the run-up formulae by van der Meer without detailed reference. This probably refers to a study carried out by WL | Delft Hydraulics (1993; see also TAW, 1999). The factor $1.25T_m$ is apparently an approximation of the peak period T_p that is used by the Dutch TAW (see also TAW, 1999).

Niedersachsen

The equations for wave run-up given in Appendix C used in Niedersachsen are:

$$z_{2\%} = 1.6\cdot\gamma_b\cdot\gamma_f\cdot\gamma_\beta\cdot\sqrt{\frac{g}{2\pi}}\cdot\sqrt{H_s}\cdot T_p\cdot\tan\alpha \tag{5.6}$$

with a maximum of

$$z_{2\%} = 3.2\cdot\gamma_f\cdot\gamma_\beta\cdot H_s \tag{5.7}$$

in which $z_{2\%}$ is the run-up level exceeded by 2% of the waves, H_s the significant wave height, T_p the peak wave period. and $\tan\alpha$ the angle of the slope.

The Netherlands

Rewriting the equations for wave overtopping given in Appendix D leads to the following expressions for the required relative crest level:

$$R_c = -\frac{\sqrt{g}}{4.3\sqrt{2\pi}}\cdot\frac{\gamma_b\cdot\gamma_f\cdot\gamma_\beta\cdot\gamma_v}{\cot\alpha}\cdot\sqrt{H_{m0}}\,T_{m-10}\cdot\ln\left(\frac{\sqrt{2\pi}}{0.067g}\cdot\frac{q\sqrt{\cot\alpha}}{\gamma_b H_{m0}T_{m-10}}\right) \tag{5.8}$$

with a maximum of

$$R_c = -\frac{1}{2.3}\cdot\gamma_f\cdot\gamma_\beta\cdot H_{m0}\cdot\ln\left(\frac{q}{0.2\sqrt{gH_{m0}^3}}\right) \tag{5.9}$$

These formulae are also used in Belgium.

United Kingdom

Rewriting the expressions for wave overtopping from the Rock Manual (CUR/CIRIA, 1991) and the Overtopping Manual (HR Wallingford, 1999) give the expressions in Eq. (5.10) and Eq. (5.11) respectively.

$$R_c = -\frac{\sqrt{g}}{b_{RM}} \cdot r \cdot \sqrt{H_s}\, T_m \cdot \ln\left(\frac{q}{a_{RM} H_s T_m}\right) \tag{5.10}$$

$$R_c = -\frac{\sqrt{g}}{b_{OM}} \cdot r \cdot \sqrt{H_s}\, T_m \cdot \ln\left(\frac{q}{a_{OM} \sqrt{g}\, H_s T_m}\right) \tag{5.11}$$

For a 1:3 smooth slope Eq. (5.11) (from the Overtopping Manual) gives crest heights that are 25-35% higher than those according to Eq. (5.10). As the Overtopping Manual is a more recent publication than the Rock Manual, Eq. (5.11) has been used in the comparison with the expressions from other countries. (Note that a new edition of the Rock Manual is expected in the 2005).

It should be noted that though the same notation is used for several parameters such as the representative slope angle (for structures with compound slope) and the reduction factors γ, the way these factors are computed differs from country to country. This is not presented here as the different formulae are compared using a plane 1:3 slope with perpendicular wave attack. In the application to the Pettemer Zeewering (Section 5.5) these parameters are computed in the appropriate ways.

5.3.1 Comparison of formulae

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 the same order as the representative slope of the Pettemer Zeewering under design conditions. The effects of berms, surface roughness, shallow foreshores or wave attach 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 $\gamma_0 = 3.3$. (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

$$T_p = C\sqrt{H_s} \tag{5.12}$$

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 following relations

$$T_m = T_{m02} = T_p / 1.28634 \tag{5.13}$$

$$T_{m-10} = T_p / 1.10706 \tag{5.14}$$

For this combination of slope and wave conditions the overtopping formulae for breaking waves (Eq. (5.4) and (5.8)) are governing. The required crest height is shown in Figure 5.1.

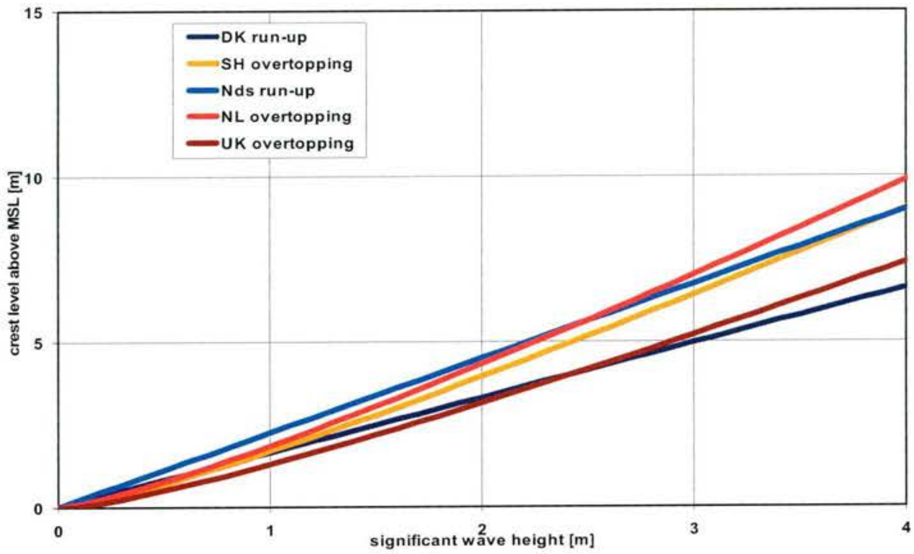


Figure 5.1 Comparison of formulae for run-up and overtopping for breaking waves

It can be seen 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 (Eq. (5.5) and (5.9)), the required crest levels were computed again slope of 1:2.5. This comparison is shown in Figure 5.2. The figure 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.

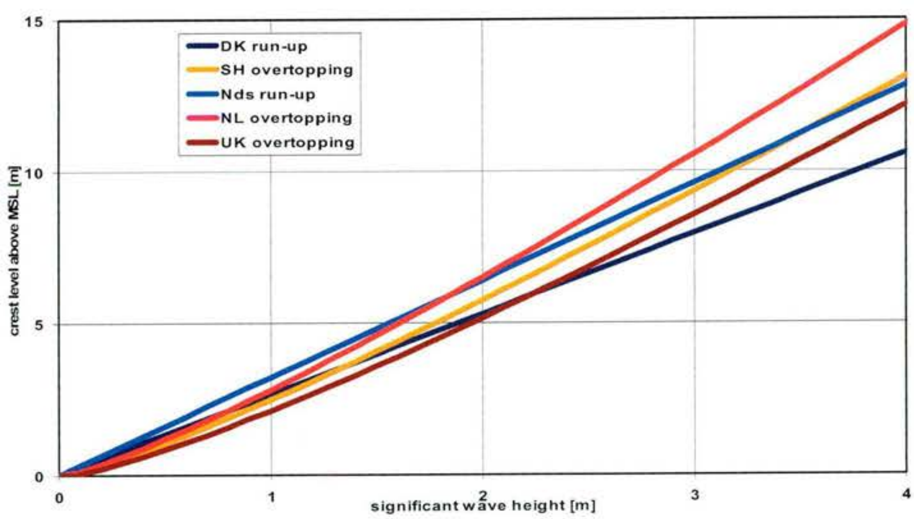


Figure 5.2 Comparison of formulae for run-up and overtopping for non-breaking waves

5.3.2 Discussion

One of the obvious observations from Figure 5.1 and Figure 5.2 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.

It is further interesting to note that the different countries make different distinctions in their criteria for wave overtopping. Denmark is the only country were the angle of the inner slope is an explicit factor in the allowable percentage of overtopping (see Table 2.1). 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).

5.4 Run-up and overtopping criteria

Analysing the various methods to determine and use the hydraulic boundary conditions, the large variation in criteria for wave overtopping is remarkable. 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. The Dutch guideline specifies e.g. the following criteria for the mean overtopping discharge:

- 0.1 l/s/m for a sandy soil with an unsatisfactory grass cover;
- 1 l/s/m for a clayey soil with a reasonably good grass cover;
- 10 l/s/m for a clay layer and grass cover according to the standards of the outer slope or in case of a revetment cover.

The German guidelines (EAK, 2002) provides also indicative values for the allowable mean overtopping discharge (see Table 5.2), but is very explicit in stating that these must be used with utmost care and that further research is required to complete the table and to specify the criteria with greater accuracy. The Dutch criterion for a good grass cover matches with the lower criterion for "grass dike, damage if crest not protected" of the German guideline.

	mean overtopping discharge					damages
buildings			q	<	0.001	Mostly no damage
	0.001	<	q	<	0.03	Small damage to building parts
			q	>	0.03	Massive damage
stone/concrete			q	<	50	Mostly no damage
cover	50	<	q	<	200	Damage for unprotected crest
			q	>	200	Damage possible
grass dike			q	<	1	No damage
	1	<	q	<	10	Damage if crest not protected
			q	>	10	Damage

Table 5.2 Some criteria for the mean overtopping discharge for structural safety (in l/s/m; after EAK2002).

The values in the German guideline differ, however, from the criteria for overtopping given in the Overtopping Manual (HR Wallingford, 1999) as can be seen in Table 5.3, which gives the tolerable mean overtopping discharges for an embankment seawall (with a back slope, thus a dike). The difference with the German guidelines for a grass dike is a factor 2. For the Pettemer Zeewering this means difference in crest height of about 1 m.

mean overtopping discharge					damages
		q	<	2	No damage
2	<	q	<	20	Damage if crest not protected
20	<	q	<	50	Damage if back slope not protected
		q	>	50	Damage even if fully protected

Table 5.3 Criteria for mean overtopping discharge for an embankment seawall (in l/s/m; after HR Wallingford, 1999).

5.5 The height of the Pettemer Zeewering

5.5.1 Introduction

The formulae for wave run-up and overtopping were applied to the data for the Pettemer Zeewering to assess the required height of this sea defence. Where the comparison of the various formulae in Section 5.3 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 Pettemer Zeewering. 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 incidence. The latter has no influence on this comparison as the waves are assumed to approach the coast perpendicularly.

5.5.2 Results

The required crest levels according to the various methods are shown in Figure 5.3. The values and some relevant intermediate parameters of the calculation are shown in Table 5.4. In the comparison the criteria for run-up or the overtopping discharges as applied in the different countries have been used. These are shown in the first line of Table 5.4.

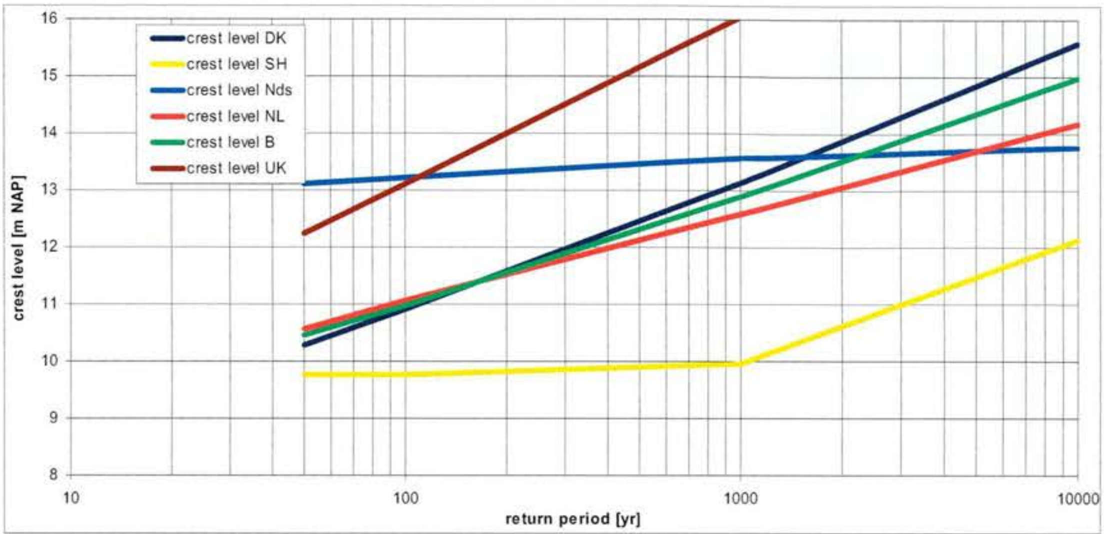


Figure 5.3 Required crest level above NAP (≈MSL) for the Pettemer Zeewering following different methods for selected return periods

It can be seen that the difference between the methods for the same return period is sometimes several meters. Three methods (DK, NL, B) give results that differ up to about 1.5m, 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. Considering that the return period of their deterministic approach is in the order of 400-800 years (see Section 3.3.6), the required crest level according to this method is not very far from the results of the methods for Denmark, The Netherlands and Belgium.

The methods of Schleswig-Holstein and the United Kingdom differ most from the other methods. These differences could be traced back to a few specific factors that are discussed in Section 5.5.3.1.

For the longer return periods the differences between the methods of Denmark, The Netherlands and Belgium are increasingly larger. For the return period of 10,000 years the difference is about 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.

return period	parameter	unit	DK	SH	Nds	NL	B	UK
criterion	run-up overtopping	[%] [l/s/m]	10%	2.0	2%	1.0	1.0	2.0
50	water level	[m NAP]	3.00	3.84	3.84	3.25	3.25	3.25
	H_s	[m]	2.61	3.53	3.12	2.78	2.76	2.76
	T^{*})	[s]	11.79	9.36	9.52	11.07	10.96	9.30
	R_c	[m]	7.28	5.92	7.65	7.32	7.21	8.99
	crest level	[m NAP]	10.28	9.76	13.11	10.57	10.46	12.24
100	water level	[m NAP]	3.19	3.84	3.84	3.45	3.46	3.46
	H_s	[m]	2.74	3.53	3.13	2.91	2.90	2.90
	T^{*})	[s]	12.00	9.36	9.61	11.22	11.14	9.45
	R_c	[m]	7.73	5.92	7.75	7.61	7.52	9.66
	crest level	[m NAP]	10.92	9.76	13.22	11.06	10.98	13.12
1,000	water level	[m NAP]	3.90	3.90	3.84	4.10	4.21	4.21
	H_s	[m]	3.19	3.58	3.15	3.31	3.37	3.37
	T^{*})	[s]	12.50	9.43	9.91	11.50	11.60	9.84
	R_c	[m]	9.23	6.04	8.08	8.50	8.69	11.83
	crest level	[m NAP]	13.13	9.95	13.56	12.60	12.90	16.04
10,000	water level	[m NAP]	4.76	4.76	3.84	4.70	4.97	4.97
	H_s	[m]	3.69	4.18	3.16	3.66	3.80	3.80
	T^{*})	[s]	12.89	10.35	10.09	11.74	11.91	10.10
	R_c	[m]	10.82	7.37	8.26	9.47	10.00	13.76
	crest level	[m NAP]	15.58	12.12	13.75	14.17	14.97	18.73

*) DK: $T_p/1.15$; SH: $T_{m02}*1.4$ (for T_m); Nds: T_p, NL: T_{m-10}, B: T_{m-10}, UK: T_{m02}.

Table 5.4 Required crest level above NAP (≈MSL) for the Petten sea defence following different methods; basic comparison.

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. whereas the deterministic approach in Niedersachsen has for the site of Petten to a return period in the order of 400-800 years. . The safety level in The Netherlands is depending on the location 2,000, 4,000 or 10,000 year and in Belgium 1,000 year is adopted. In the United Kingdom a cost-effective solution is determined without adopting a specific uniform safety level. The crest levels for these safety levels are marked in Table 5.4.

5.5.3 Analysis and discussion

Investigation of the large differences for Schleswig-Holstein and the UK showed that these could be traced back to a few specific factors. These are discussed in the next section. To obtain more insight in possible causes for the differences between the methods, the following parameters in the methods were one by one taken the same:

- Design water level
- Wave conditions
- Run-up and overtopping criterion

The results are described in the following sections.

5.5.3.1 Differences by specific factors

The results of the comparison of the different methods (Figure 5.3) show that especially the results for Schleswig-Holstein and the UK differ considerably with several of the other methods. By comparing intermediate results of the calculation this could be traced back to a few specific factors.

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) 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. This can be seen from comparing Eq. (5.5) and (5.9): apart from slightly different constants the factors γ_b and γ_h are the main difference. Due to this factor the expression for non-breaking waves is governing for the Schleswig-Holstein method, whereas the expressions for breaking waves are governing for the other countries.
Finally, the formula for breaking waves contains a factor $1.25T_m$ to approximate the peak period T_p 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. Figure 5.4 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.

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 Figure 5.4, where the expressions from the United Kingdom have been combined with the Dutch equation for the effect of a berm. The crest levels are 2 m (50 yr) to 3.25 m (10,00yr) lower and line for the United Kingdom nearly coincides with the curve for Denmark.

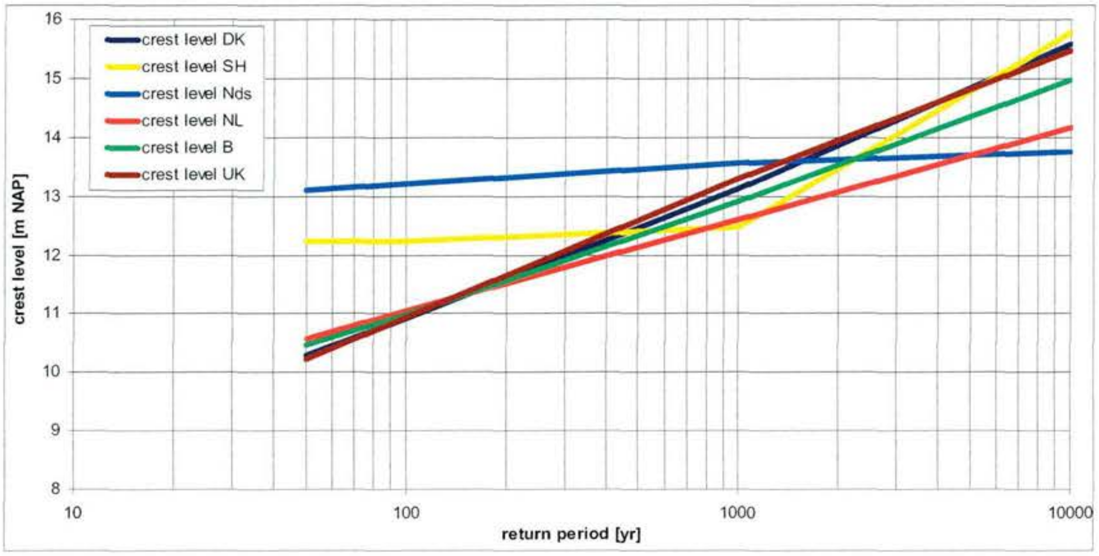


Figure 5.4 Required crest level above NAP (≈MSL) for the Pettemer Zeewering following different methods; modified methods for Schleswig-Holstein and the United Kingdom.

5.5.3.2 Uniform water levels

To obtain some more insight in the effect of the water level on the results, especially for the German countries, the water level values for The Netherlands were also adopted for the other countries. The results are shown in Figure 5.5. This leads of course to quite different curves for Schleswig-Holstein and Niedersachsen. For the method of Schleswig-Holstein the crest level is lower for the shorter return periods 50 and 100 years, higher for 1,000 year and about the same for 10,000 yr. The method of Niedersachsen gives lower crest levels up to a return period of about 400 yr. For longer return periods the crest levels are higher (about 2 m for 10,000 yr). The method of Denmark gives values that are 0.4-0.5 m higher for return periods up to 1,000 yr. For 10,000 yr the crest level decreases a little. Following the methods of Belgium and the United Kingdom the decrease in water level of 0.27 m for 10,000 yr leads to a decrease in crest height of about 0.8 m. The decreasing water depth leads to a lower wave height and period, which in turn requires a smaller relative crest height R_c.

The very small difference between the curves for The Netherlands and Belgium is entirely caused by the difference in deep water wave height. This leads to nearshore wave heights that are up to 2 cm different. This shows that the deep water wave height is nearly irrelevant for the nearshore wave height in this case. This implies that the accuracy of the wave model used to transfer the deep-water conditions to the shore has a larger effect than the accuracy of the design wave height in deep water. In other words, the wave model and the water depth determine the nearshore wave conditions.

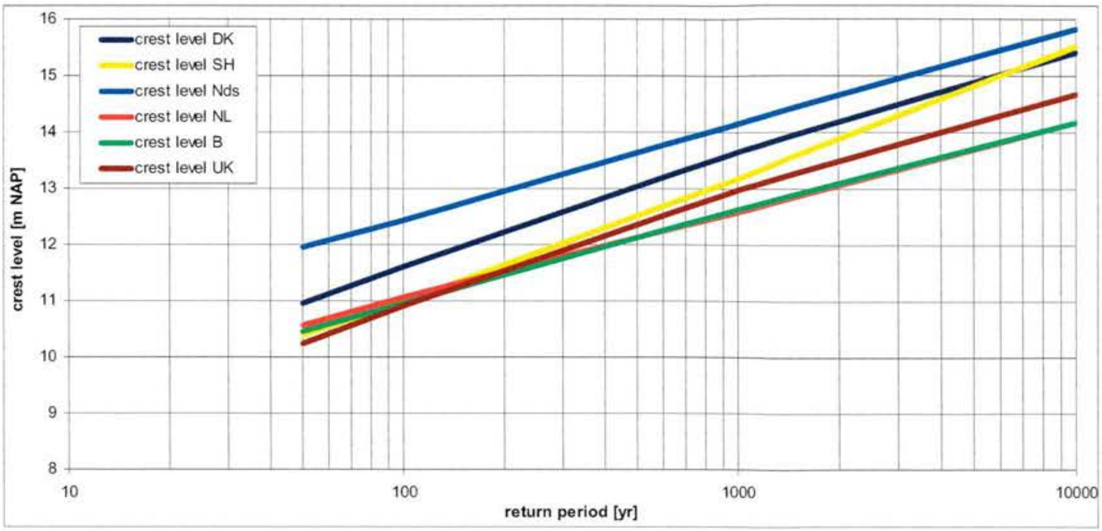


Figure 5.5 Required crest level above NAP (≈MSL) for the Pettemer Zeewering following different methods using the same water levels.

It is further interesting to note that the Danish method leads to crest levels that are significantly higher than those for the Dutch method, although the Dutch overtopping criterion is stricter than the Danish criterion. Following the Dutch method an overtopping of 2 l/s/m and a run-up level of 2% lead to similar crest levels (only a few cm difference).

5.5.3.3 Uniform water levels and wave conditions

The combined effect of using the same water level and wave conditions is shown in Figure 5.6. Here the same nearshore significant wave height has been used in all methods. 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 4.4. The figure shows that the curves for Schleswig-Holstein, The Netherlands, Belgium and the United Kingdom lead to crest heights that are comparable. The two methods based on a run-up criterion (DK and Nds) lead to crest heights that are 1.5 to 2 m higher.

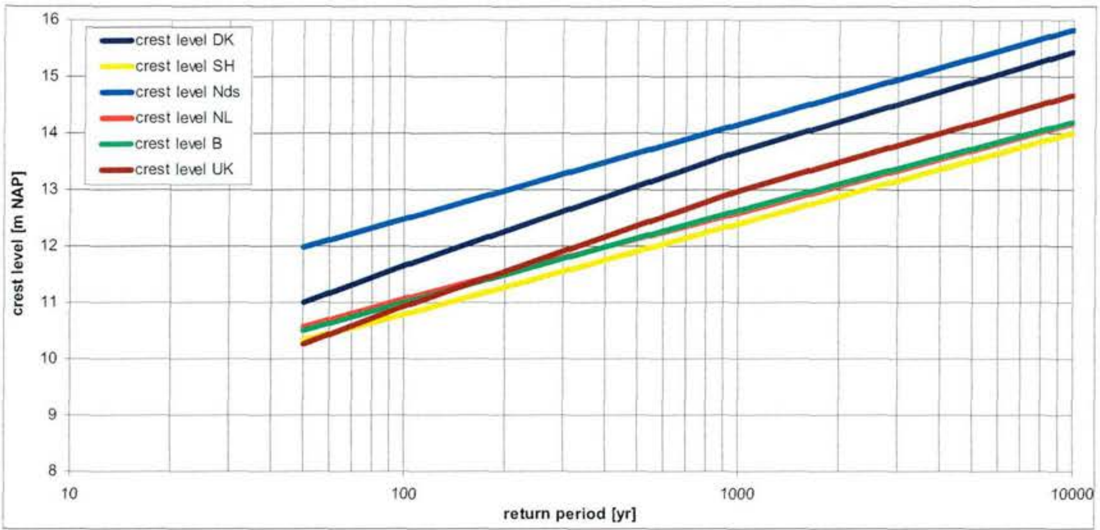


Figure 5.6 Required crest level above NAP (≈MSL) for the Pettemer Zeewering following different methods using the same water levels and wave conditions.

5.5.3.4 Uniform water levels, wave conditions and overtopping criterion

Figure 5.7 shows the result of the various methods if the water levels, wave conditions and overtopping criterion are taken the same. For the Danish method the coefficient corresponding to an overtopping percentage of 2% has been used. This has been compared with an overtopping criterion of 2 l/s/m for the other methods. Comparing this with Figure 5.6, where 1 l/s/m was used for The Netherlands and Belgium (Schleswig-Holstein, Niedersachsen and the United Kingdom are unchanged), it can be seen that the crest level for Denmark increases by about 3m whereas the crests for The Netherlands and Belgium are 0.7 to 1.0 m lower. This shows that the overtopping criterion has a significant effect on the crest height.

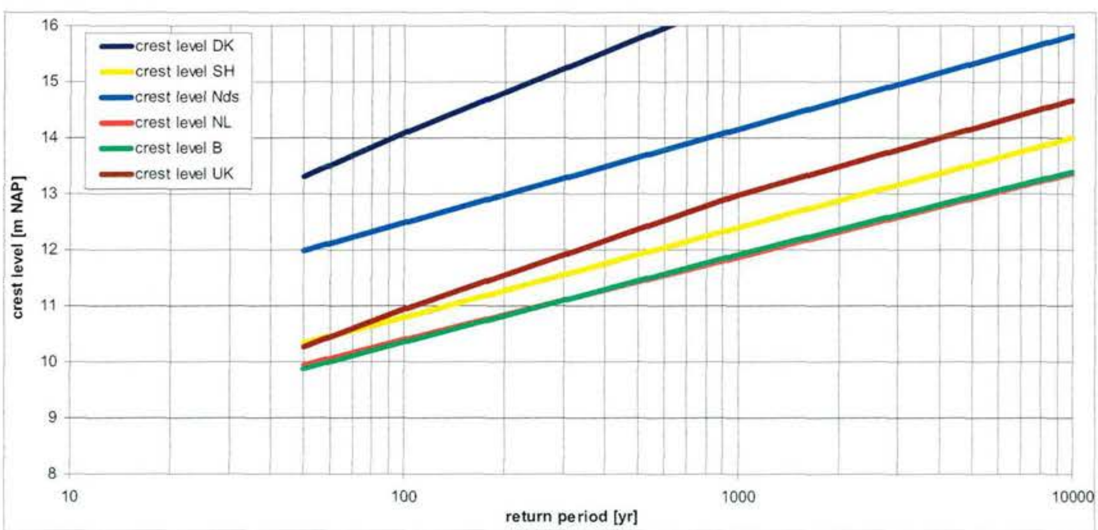


Figure 5.7 Required crest level above NAP (≈MSL) for the Pettemer Zeewering following different methods using the same water levels, wave conditions and overtopping criterion.

Remarkable in Figure 5.7 is the large difference between the results of the method of Denmark and the other methods. This is due to the fact that the Danish method does not include a reduction factor for the effect of a berm. If such a factor is included the results of the Danish method are in the same order as the other methods as can be seen in Figure 5.8. This also means that the results for Denmark with the original run-up criterion as shown in Figure 5.3 would be significantly lower than those for The Netherlands and Belgium if such a reduction factor for the berm would be included in the Danish method.

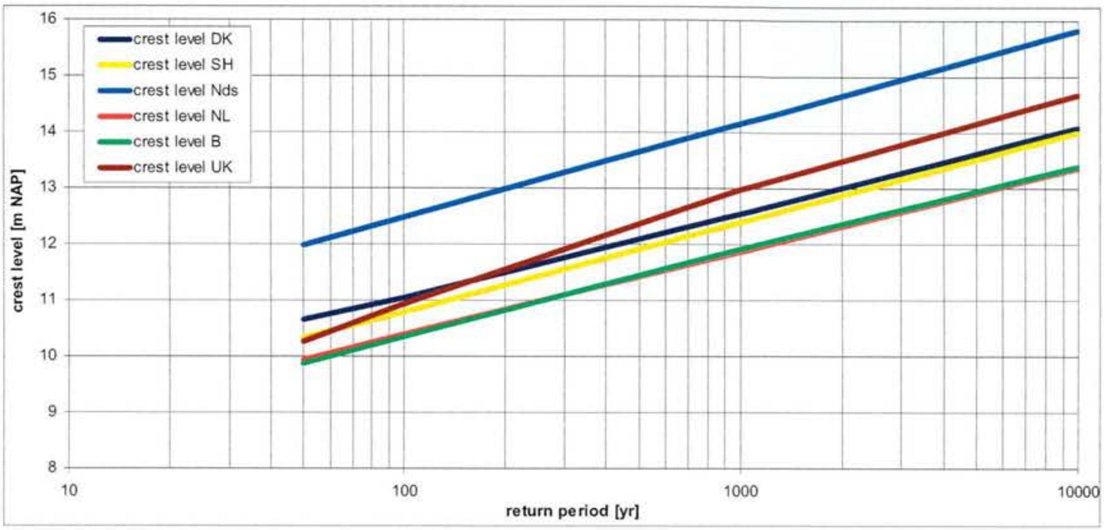


Figure 5.8 Required crest level above NAP (≈MSL) for the Pettemer Zeewering following different methods using the same water levels, wave conditions and overtopping criterion including a reduction factor for the berm in the Danish method.

5.5.4 Conclusion

The comparison of the different methods to determine the required crest height for the Pettemer Zeewering confirms the difference in crest levels of 2-3 m for the same safety level that was found in the earlier study (DWW, 2001). For the 10,000 year return period the differences between the methods of Denmark, The Netherlands and the United Kingdom, the countries also considered in the previous study, are up to 1.5 m. The difference with the German countries is larger, due to the different approach to the design water level.
If the safety level adopted in the countries is also taken into consideration, the difference in crest level at this site can be up to 4.5 m.

The difference in crest height depends of course on the step in safety level that is made. If the safety level is increased by a factor 10, the crest height must be about 1.5 m higher. This confirms the value found in the earlier study.

Difference in water level of 0.27 m leads to difference in crest height of about 0.8 m.

From the variation of the overtopping criterion it can be concluded that this has a significant effect on the crest level. Increasing the overtopping criterion from 1 l/s/m to 2 l/s/m leads to crest heights that are lower by about 1 m. Given the fairly explicit statement in e.g. the German guidelines (EAK2002) that "the criteria must be used with utmost care" and that "further research is required to complete the table and to specify the criteria with greater accuracy" this seems to be one of the larger gaps in present knowledge regarding sea dikes.

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 Pettemer Zeewering 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 shallow water.

6 Approaches to the safety assessment of dune coasts

6.1 General

Most of the countries also have stretches sandy coasts, but these are treated in different ways. In Schleswig-Holstein the sandy coast is not considered to be part of the coastal defence. Maps in the Masterplan Coastal Protection (Schleswig-Holstein, 2001, Karte 2 & 3) show that sandy coasts are fairly rare in Schleswig-Holstein. Supposing that the coastal sections marked 'unprotected' are sandy coasts, these maps show that these are only found on the islands of Sylt and Amrum.

6.2 Dune strength

Denmark uses a fairly simple criterion for the safety assessment of the dunes. These must have a minimum width of 40m at a height of 5m above MSL for unprotected dunes and 30m for dunes protected by a revetment. This is shown in Figure 6.1, where *sikkerhedbredde* is the safety width and *bufferbredde* is an allowance for erosion before maintenance or reinforcement measures will be carried out to keep the safety width. These criteria are based on investigations by the DCA in 1990 of the dune width before and after storms based on surveys. This was combined with applications of the Dutch Vellinga Model (see below) to calculate expected dune erosion during design conditions. However, the calculated dune erosion under design conditions was smaller than the observed dune erosion of up till 30 metres. The safety width is therefore based on the observations and directly applied to the existing dune profile.

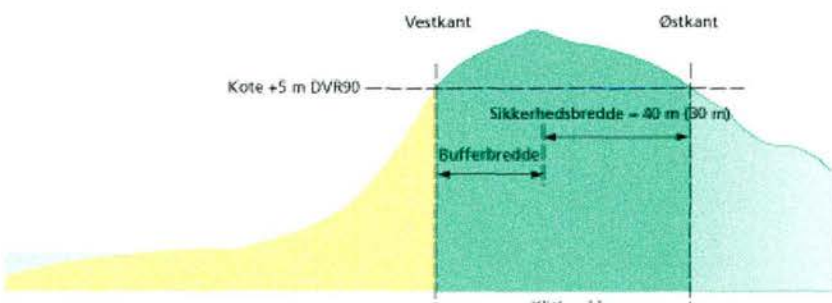


Figure 6.1 Criterion for the minimum dune with used in Denmark.

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. The model NEWDUNE is used for this purpose. This model is comparable with the model EDUNE by Kriebel (1989) which is briefly described in the EAK2002. The NEWDUNE model was developed by Newe at LWI University of Braunschweig. NEWDUNE is based on an equilibrium profile (see Figure 6.2) given by

$$h = A \cdot x^{2/3} \tag{6.1}$$

where A is a function of the fall velocity of the sand. The transition slope in deeper water of 1:2.5 and the slope of the dune above the zone of wave attack (tan δ =1) are taken after Vellinga (1983).

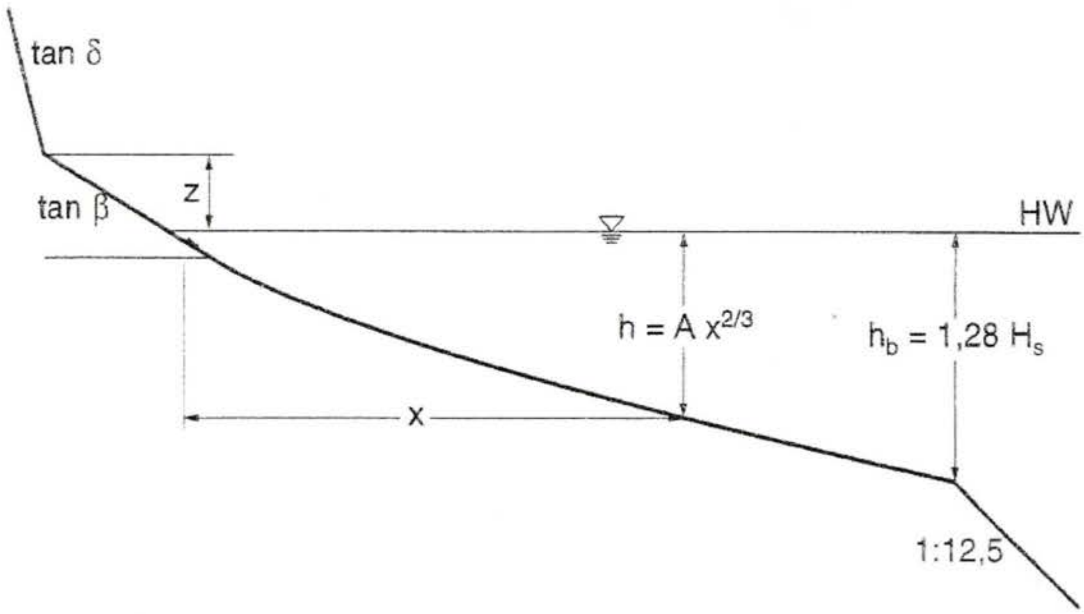


Figure 6.2 Equilibrium profile used in the NEWDUNE model applied in Niedersachsen (from LWI, 1998).

Since 2003/2004 the numerical model UNIBEST-DE (English version of DUROSTA; Steetzel, 1993) is used in addition to the NEWDUNE model. The experience shows that both models give comparable results for design conditions, but that NEWDUNE overestimates the erosion for more regular events (1/2 yr and 1/5 yr conditions; Blum, personal communication).

Input for the dune erosion simulations are time series of water level and wave conditions. The time series for the water level has a duration of about 3 tides. The water level consists of a normal tidal cycle and the storm surge profile according to Gönnert (2003). The maximum surge is assumed to last for two hours around high water. If available, design wave conditions are taken from numerical models (e.g. SWAN), otherwise the wave conditions are a function of the water depth. The design conditions are determined using the method *Einzelwert-Verfahren* described in Section 3.3.2. In the case study for Langeoog (Subproject 9), it was found that the design level has a return period of 500-1,000 years, depending on the used probability distribution. This is similar to what was found in the comparison with other methods in Section 3.3.6.

The remaining dune width at a level of NN+8m (approx. 8m 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. This value is based on surcharges on the amount of erosion above the computation level that account for model/data uncertainties and longshore transport, similar to the factors T and g in the method applied in The Netherlands (see Figure 6.4 and Figure 6.5 below). This is an average value for the present situation of beach profile and shape of the dunes.

The method to calculate dune erosion in **The Netherlands** (and also in Belgium) is based on an equilibrium profile consisting of three sections as shown in Figure 6.3.

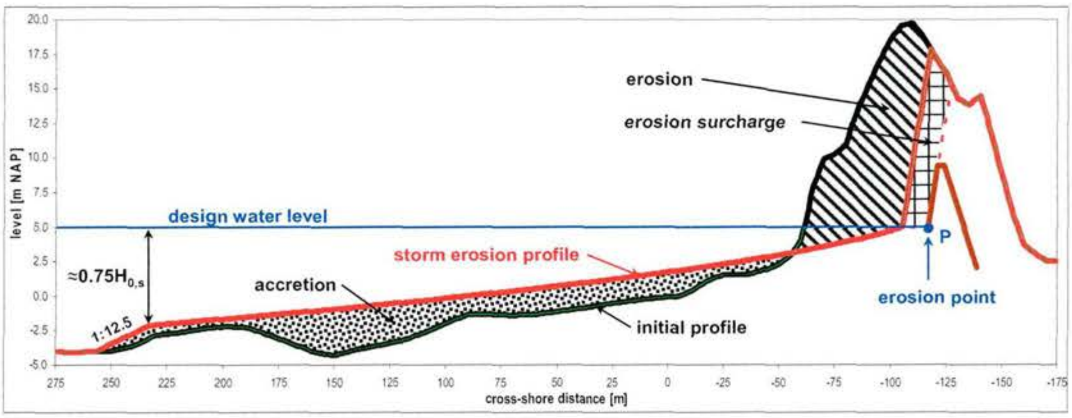


Figure 6.3 Principles of the computation of the profile after dune erosion.

The parabolic profile below the water line is given by

$$\left(\frac{7.6}{H_{0s}}\right)y = 0.4714\left[\left(\frac{7.6}{H_{0s}}\right)^{1.28} \cdot \left(\frac{w}{0.0268}\right)^{0.56} x + 18\right]^{0.5} - 2.0 \qquad (6.2)$$

where

H_{0s}	=	significant wave height at deep water	[m]
w	=	the fall velocity of the dune sand in sea water	[m/s]
x	=	the distance to the new toe of the dune	[m]
y	=	the depth below the water level	[s]

The dune front is assumed to have a slope of 1:1 and the transition to the original seabed on the seaward side has a slope of 1:12.5. This 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 T equal to the 25% of the erosion above the still water level A is added to account for the uncertainty in storm duration and the inaccuracy of the model as shown in Figure 6.4. The calculation of the above profile is implemented in the DUROS model (DUne eROSion-model).

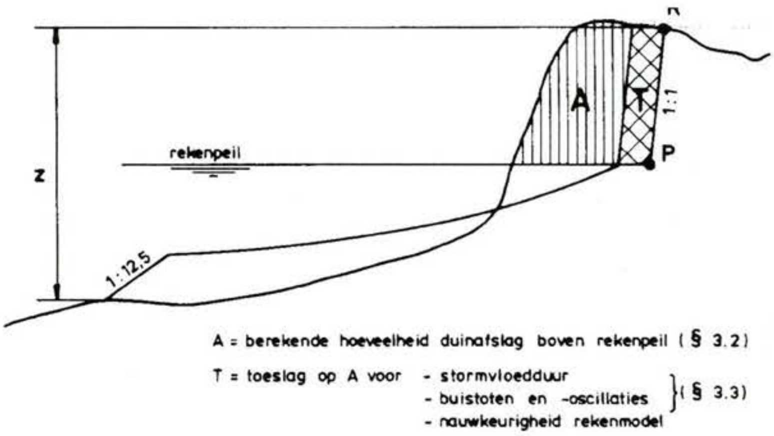


Figure 6.4 Additional erosion T and location of the toe of the dune P.

In curved parts of the coast, the longshore transport during design conditions could lead to an additional erosion of the dune. The additional retreat of the dune front to which this leads, is determined by shifting the storm profile over a distance g in such a way that the additional amount of erosion G (in m^3/m, the shaded area in Figure 6.5), is equal to

$$G = \frac{A^*}{300} \cdot \left(\frac{H_{0s}}{7.6} \right)^{0.72} \cdot \left(\frac{w}{0.0268} \right)^{0.56} \cdot G_0 \qquad (6.3)$$

in which A^* is the total amount of erosion above the water level ($A+T$ in) and G_0 is a reference value for G that depends on the curvature of the coast.

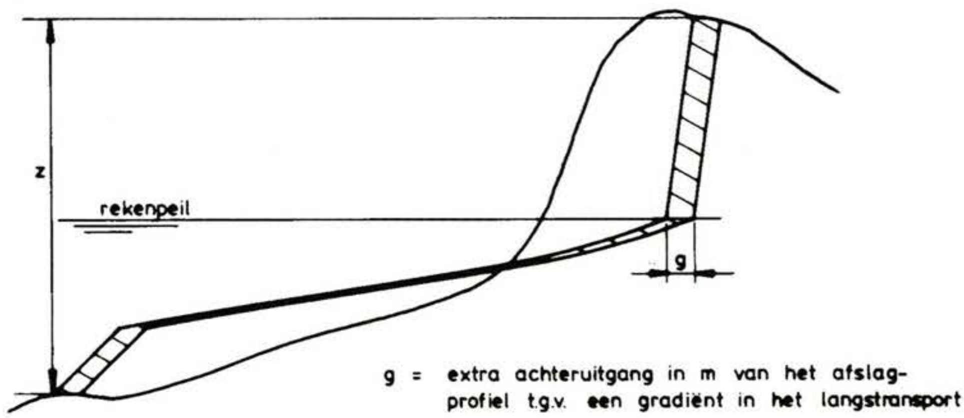


Figure 6.5 Additional retreat of the erosion profile due a gradient in the longshore transport.

Possible trends such as a slow retreat of the shoreline are included in the safety assessment of sandy coasts by calculating the position of the erosion point P for the large number of yearly measured profiles in a certain section. The regression line following from this evaluation is first shifted inshore (to account for year-to-year profile variations and the effects of longshore transport) and then extrapolated to estimate the moment that the safety level is not met any more as shown in Figure 6.6.

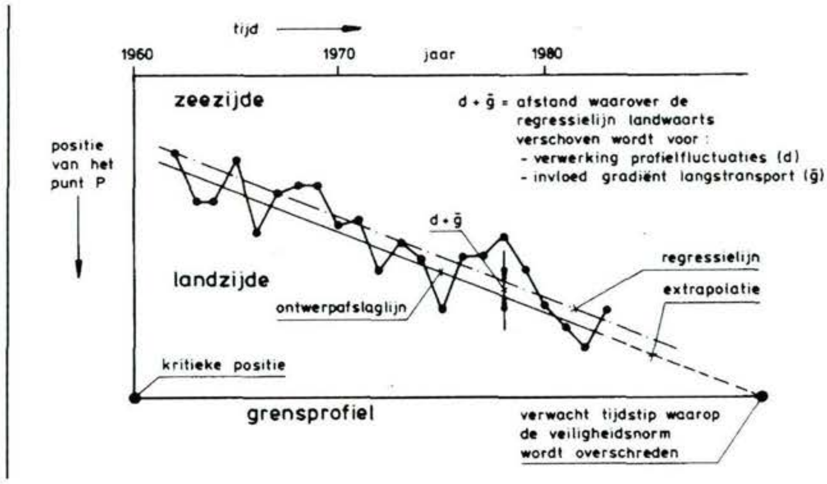


Figure 6.6 Procedure to include the effect of trends in the safety assessment of sandy coasts in The Netherlands.

Inputs to the dune erosion calculations are the still water level, the deep-water wave height and peak period and the fall velocity of the dune sand. It is remarkable, however, that the still water level used in the dune erosion calculations with DUROS is not the design water level for a certain return period. The water level *RP* (for *Rekenpeil*) used in the calculation is

$$RP = \text{design level} + 2/3 \text{ decimation height}$$

The decimation height is difference in water level between the design level and the water level with a return period that is a factor 10 longer. The significant wave height used in the calculations is the deep-water wave height corresponding to this water level. This increased water level is to account for differences in the risk of complete failure (and thus flooding of the hinterland) once the design water level is exceeded between sandy coasts and sea dikes (Den Heijer, personal communication).

The strength of sandy coasts is defined in terms of a minimum profile of the dunes that must remain after dune erosion. This minimum profile (*grensprofiel*) is shown in Figure 6.7. The minimum crest level of this profile is computed using

$$h_0 = RP + 0.12\, T_{0p} \sqrt{H_{0s}} \text{ with a minimum of } h_0 = RP + 2.5m \tag{6.4}$$

in which

h_0	=	the required height of the minimum profile	[m NAP]
T_{0p}	=	peak wave period at deep water	[s]
RP	=	the still water level under design conditions	[m NAP]

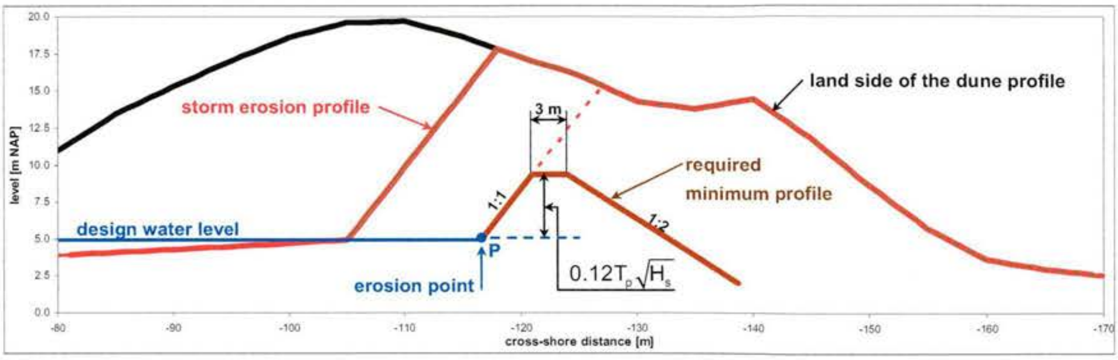


Figure 6.7 Minimum profile for dunes required to remain after storm erosion.

It is interesting to notice that the wave period is only used to assess the minimum dune profile. The wave period is not included in the equation to calculate the equilibrium profile. For the deep-water peak wave period a constant value of T_{0p}=12 s is used for the coast north of Hoek van Holland; for the coasts south of Hoek van Holland a constant value of T_{0p}=8 s

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.

6.3 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 the UCIT (Universal Coastal Intelligence Toolkit), 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. An example of the calculated erosion and required minimum profile for a return period of 10,000 years is shown in Figure 6.8. It can be seen that though a large part of the dune profile is lost by erosion, the remaining part is more than the required minimum profile.

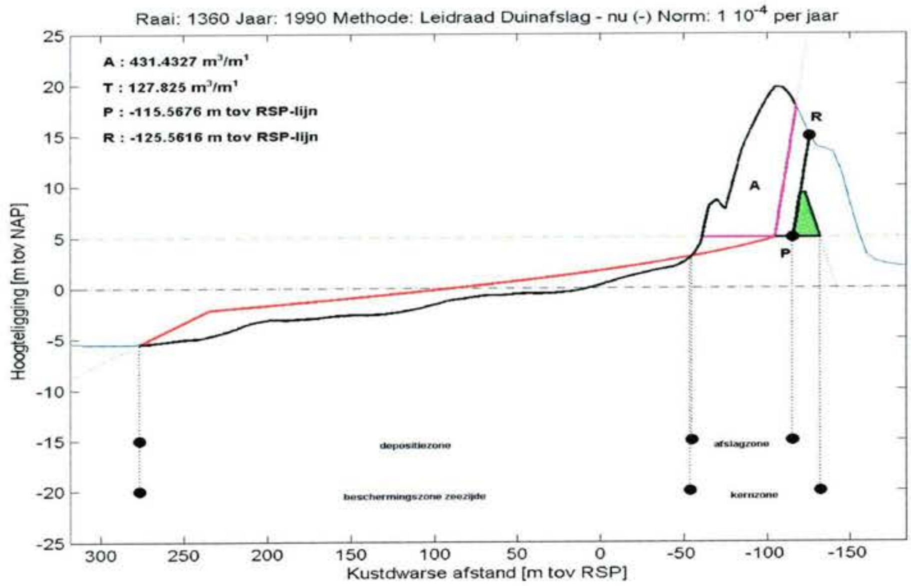


Figure 6.8 Example of dune erosion calculation using the method of The Netherlands for profile 1360 near Callantsoog; return period 10,000 years

The position of the point P was calculated for return periods of 500, 2,000, 4,000 and 10,000 years. The red line in Figure 6.9 shows the result. It can be seen that the position of P ranges from -95m (500yr) to -115m (10,000yr).

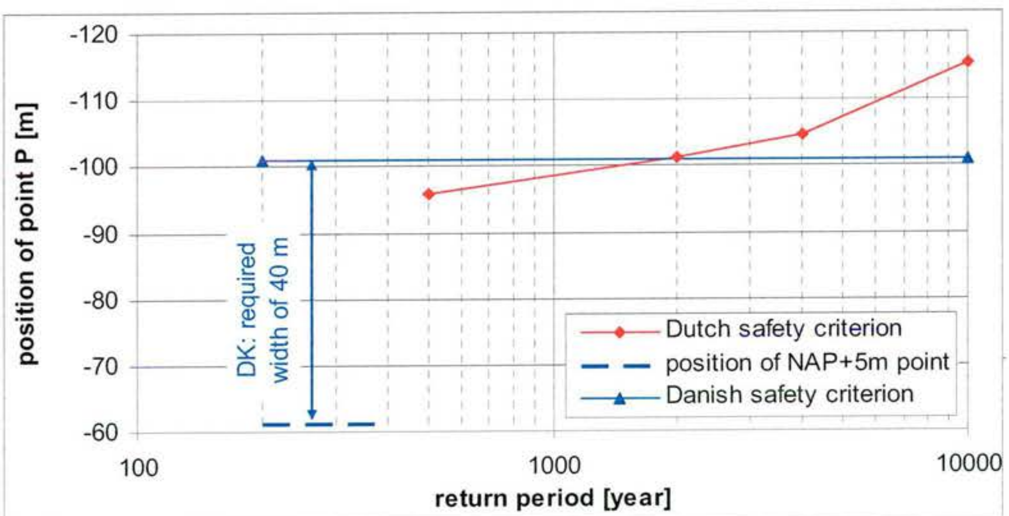


Figure 6.9 Position of the erosion point P as function of the return period for profile 1360 near Callantsoog.

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. This is shown in Figure 6.10. It can be seen that the dune width at NAP+5 m is about 95 m, which is well above the Danish criterion of 40m at a level of 5m above MSL. The Danish criterion of 40 m is just equal to the calculated retreat of the dunefront following the Dutch method. 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 levels adopted in The Netherlands (2,000 to 10,000 years) the Danish criterion would be insufficient.

Figure 6.10 shows that the remaining width at 8m above MSL, the indicator used in Niedersachsen, is for the 500 year return period close to 50m. 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.

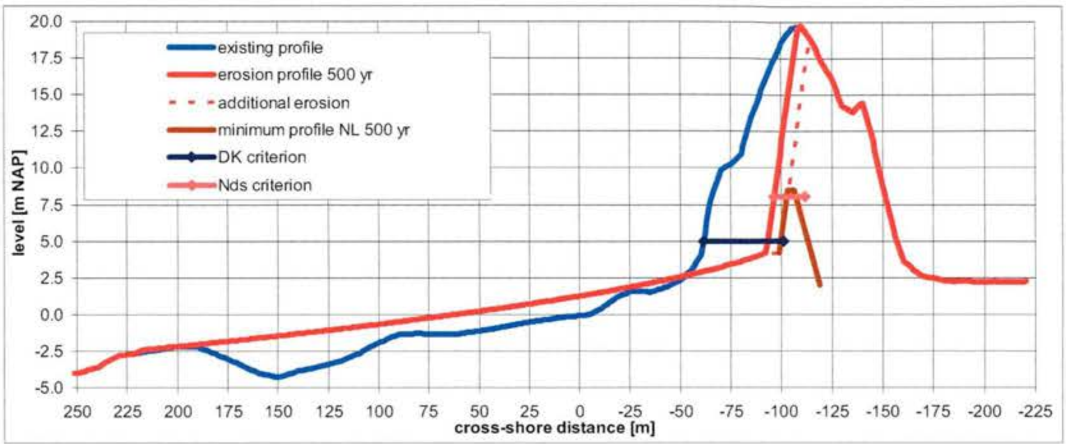


Figure 6.10 Erosion profile for a return period of 500 yr according to the method of The Netherlands compared with the criteria used in Denmark and Niedersachsen.

7 Summary and conclusion

7.1 Summary

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 parameters. 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 (1983) for wave periods up to 12 s. 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.

7.2 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.

References

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

Alkyon, 1999. **Wave computations for the coast of The Netherlands.** Report A480, November 1999.

CIRIA/CUR, 1991, **Manual on the use of rocks in coastal and shoreline engineering**. CIRIA Special Publication 83 / CUR Report 154.

DWW, 2001a. **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), April 2001, 27 pp + appendices.

DWW, 2001b. **Hydraulische Randvoorwaarden 2001, voor het toetsen van primaire waterkeringen.** 261 pp.

EAK, 2002. **Empfehlungen für Küstenschutzwerke.** *Die Küste*, Heft 65, Jahr 2002. Kuratorium für Forschung im Küsteningenieurwesen.

Gönnert, G., 2003. **Sturmfluten und Windstau in der Deutschen Bucht – Charakter, Veränderungen und Maximalwerte im 20. Jarhundert**. In: *Die Küste*, Heft 67, p.185-365.

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

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

Kystdirektoratet, 2002. **Højvandstatistikker 2002.**

Larson, M., 1988. **Quantification of Beach Profile Change.** Lund University, Institute of Science and Technology. Report No. 1008.

LWI, 1998. **Kalibrierung des Dünenabbruchmodelles NEWDUNE am Beispiel der Ostfriesischen Inseln Borkum, Juist, Langeoog und Wangerooge.** Unpublished report No. 838, November 1998. Leichtweiss-Institut für Wasserbau. Technische Universität Braunschweig.

RIKZ, 1993a. **De basispeilen langs de Nederlandse kust; Statistisch onderzoek.** Report RIKZ-93.023 Parts 1 (text) and 2 (appendices), April 1993.

RIKZ, 1993b. **De basispeilen langs de Nederlandse kust; Fysisch onderzoek.** Report RIKZ-93.025, April 1993.

RIKZ, 1993c. **De basispeilen langs de Nederlandse kust; Eindverslag.** Report RIKZ-93.026, April 1993.

RIKZ, 1995a. **De basispeilen langs de Nederlandse kust; De ruimtelijke verdeling en overschrijdingslijnen.** Report RIKZ-95.008, May 1995.

RIKZ, 1995b. **Golfrandvoorwaarden langs de Nederlandse kust op relatief diep water.** Report RIKZ-95.024 (2 vol.: text + appendices), December 1995.

RIKZ, 1996. **Randvoorwaarden voor golfperioden langs de Nederlandse kust.** Report RIKZ-96.019 (2 vol.: text + appendices), July 1996.

RIKZ, 1999. **Basis HYDRA-K; Meerdimensionale extreme-waardenstatistiek van belastingen en faalkansberekening.** Report RIKZ-99.020 (2 vol.: text + appendices), May 1999.

RIKZ, 2000. **Richtingsafhankelijke extreme waarden voor HW-standen, golfhoogte en golfperioden.** Report RIKZ / 2000.040, December 2000.

RIKZ, 2004. **Rapportage veldmetingen Pettermer Zeewering; Stormseizoen 2003-2004.** Report RIKZ/2004.032, September 2004.

Ris, 1997. **Spectral modelling of wind waves in coastal areas.** PhD Thesis, Delft University of Technology, Delft University Press. 162 pp.

SH, 2001. **Generalplan Küstenschutz; Integriertes Küstenschutzmanegement in Schleswig-Holstein.** Ministerium für ländliche Räume, Landesplanung, Landwirtschaft und Tourismus des Landes Schleswig-Holtein.

SH, 2002. **Sicherheitsüberprüfung / Bemessung der Landesschutzdeiche; Ermittlung von Wellenauflauf bzw. Wellenüberlauf an den Landesschutzdeichen der Westküste Schleswig-Holsteins für den Generalplan Küstenschutz**. Technical report of the Amt für ländliche Räume Husum, Abt. 5.2

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, 1984. **Leidraad voor de beoordeling van de veiligheid van duinen als waterkering.** Technische Adviescommissie voor de Waterkeringen. May 1984.

TAW, 1995a. **Leidraad Zandige Kust.** Technische Adviescommissie voor de Waterkeringen. July 1995.

TAW, 1995. **Basisrapport Zandige Kust.** Technische Adviescommissie voor de Waterkeringen. July 1995.

TAW, 1999. **Leidraad Toetsen op Veiligheid.** Technische Adviescommissie voor de Waterkeringen. August 1999.

TAW, 2002a. **Technisch Rapport Golfoploop en Golfoverslag bij Dijken.** Technische Adviescommissie voor de Waterkeringen. May 2002.

TAW, 2002b. **Leidraad Zandige Kust.** Technische Adviescommissie voor de Waterkeringen. December 2002 .

Vellinga, P., 1983. **Predictive Computational Model for Beach and Dune Erosion during Strom Surges.** Proc. of Coastal Structures '83, Arlington / Virginia, ASCE, pp. 806-819.

WL | Delft Hydraulics, 1993. **Golfoploop en Golfoverslag bij Dijken; Samenvatting.** Report H638; April 1993.

WL | Delft Hydraulics, 1999a. **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; July 1999; Prepared for: Commission of the European Communities.

WL | Delft Hydraulics, 1999b. **Physical model investigations on coastal structures with shallow foreshores; 2D model tests with single and double-peaked wave energy spectra.** Report H3608, author: M.R.A. van Gent; December 1999; 65p + fig, tab, ref. Prepared for: RWS-DWW.

A Situation in Denmark

A.1 Situation in Denmark

The total length of Denmark's coastline is approximately 10 times longer than the Dutch coast. However, the area at risk of flooding is much smaller. Along the Kattegat and the Baltic Sea area from Esbjerg to the German border the situation is different. Although most towns are situated on higher grounds, a few exceptions (Thyborøron, Højer, Tønder and Ribe) cause significant flooding risks. These towns are protected by major dikes, with a 1000-year (the town of Thyborøron) and a 200-year safety level (the dikes at Højer and Ribe in the Wadden Sea Area). Other important dikes have a safety level of 50 years. These safety standards were proposed by The Danish Coastal Authority (DCA) and approved by the Ministry of Transport. They are not based on scientific analysis, with the exception of the return period of 200 years for the main dikes at Ribe and Højer. This safety standard was based on a cost-benefit analysis.

The DCA is responsible for the (draft) design of sea defences. The maintenance of dikes in the southern part of Denmark (Wadden Sea area) is lying in the responsibility of the counties and dike boards. The DCA maintains sea defences in the central part of the Danish North Sea coast (groynes, breakwaters, sand nourishment, etc.).

The design is normally carried out by the DCA. In some cases a draft design is made by the DCA and consultants carry out detailed planning. The DCA defines boundary conditions and standard values.

A periodic safety assessment of the sea defences is carried out. The safety of the dikes is evaluated about each fifth year including analysing extreme water levels and wave measurements as well as surveying the dikes. The approach to design and periodic safety assessment is the same. The hydraulic boundary conditions are mainly provided by the DCA together with the Danish Meteorological Institute.

A.2 Basic data

Measurements of wind, water level, waves and currents are used as basic data. It depends on the data type how long the periods are for which data are available. The locations are indicated in Figure A.1.

Figure A.1 Location of water level measuring stations in Denmark.

A.2.1 Water level

The water level is measured at 13 stations along the Danish North Sea coast (see Table A.1). The total number of water level stations in the Danish waters is 35.

Frederikshavn, Havn (FH)	Esbjerg, Havn (KDI)
Skagen, Havn (SH)	Esbjerg, Havn (DMI)
Hirtshals, (DMI)	Højer, (KDI)
Hirtshals, Havn (HiH)	Ballum, (KDI)
Hanstholm, (DMI)	Ribe, (KDI)
Hanstholm, Havn (HaH)	Ribe, Havet (KDI)
Thyborøn, Havn (KDI)	Ribe, (KDI)
Thyborøn, Havet (KDI)	Havneby, (KDI)
Ferring, (KDI)	Brøns, (KDI)
Thorsminde, Havn (KDI)	Manø, (KDI)
Thorsminde, Fjord (KDI)	Haderslev, (KDI)
Skovlund, (KDI)	Sønderborg, (KDI)
Kloster, Havn (KDI)	Bogense, (KDI)
Hvide Sande, Havn (KDI)	Assens, (KDI)
Hvide Sande, Havet (KDI)	Fåborg, (KDI)
Hvide Sande, Fjord (KDI)	Karrebæksminde, (KDI)
Ringkøbing, Havn (KDI)	Kalvehave, (KDI)
Bork, Havn (KDI)	
KDI: Coastal directorate (Kystdirektoratet) DMI: Meteorological Institute of Denmark (Danmarks Meteorologiske Institut) SH: Harbour of Skagen FH: Harbour of Frederikshavn HiH: Harbour of Hirtshals HaH: Harbour of Hanstholm	

Table A.1 Water level stations used as basic stations for determination of design conditions for sea defences.

A.2.2 Waves

Waves are measured at 4 permanent stations along the North Sea coast and 3 variable stations in the Wadden Sea area.

Fanø Bay, (HaE)
Hanstholm, (HaH)
Hirtshals, Vest (KDI)
Fjaltring, at a depth of 16 m. (KDI)
Nymindegab, (KDI)
KDI: Coastal directorate (Kystdirektoratet) DMI: Meteorological Institute of Denmark (Danmarks Meteorologiske Institut) SH: Harbour of Skagen FH: Harbour of Frederikshavn HaE: Harbour of Esbjerg HaH: Harbour of Hanstholm

Table A.2 Wave height stations used as basic stations for determination of design conditions for sea defences.

A.2.3 Wind

Wind measurements are carried out at 12 stations, 8 of these are relevant for the North Sea coast.

Frederikshavn, (FH)	Thorsminde, (KDI)
Skagen, (SH)	Hvide Sande, (KDI)
Hirtshals, (HiH)	Blåvand, (DMI)
Hanstholm, (HaH)	Ribe (KDI)
Thyborøn, (KDI)	Havneby, (KDI)
Ferring, (KDI)	Rømø, (DMI)
KDI: Coastal directorate (Kystdirektoratet) DMI: Meteorological Institute of Denmar (Danmarks Meteorologiske Institut) SH: Harbour of Skagen FH: Harbour of Frederikshavn HiH: Harbour of Hirtshals HaH: Harbour of Hanstholm	

Table A.3 Basic stations from which wind measurements are used for the safety assessment and design of sea defences.

A.3 Data processing

Water level and wind are measured every 10 minutes. Wave height and direction are measured every 3 hours. The data are stored in an Oracle database. All hydrodynamic parameters are measured all the year around. All gathered data is quality checked before final storage in the database.

A high water statistic is worked out every fifth year based on water level data. Extrapolation is based on surge/extreme water levels.

The correlation between e.g. water level and wave height is taken into account in specific projects and/or reports. There is no general analysis of the correlation between parameters.

No confidence interval of extrapolations is taken into account in the design process.

A.4 Nearshore conditions

Design water levels are based on an analysis of a series of extreme water levels, measured at several stations. The statistics are calculated using the Weibull distribution. In the past knowledge of waves in Denmark was poor, and the calculation of design waves was based only on theory. A study of the wave climate and dike safety level has been completed a few years ago. Firstly, the wave climate was modelled using the MIKE 21 model, calibrated with 4 years of wave recordings. This resulted in a design wave at any location in front of the dikes (these design waves vary, but typical values are H = 1.8 m and T = 4-5 s). Secondly, physical model tests were conducted using the test results of the wave and water level. Several combinations of water levels, waves, slopes and roughness were used to mode the wave run-up. At locations where wave statistics based on recordings are not available, the design wave is calculated using the standard Shore Protection Manual procedures, based on long-term wind statistics.

A.5 Design / safety assessment sea dikes

A.5.1 Strength parameters

The strength of a dike or dam with regard to flood protection is characterised by the crest level and the stability of cross-section. The crest level is composed of three components: the design water level, the wave run-up and additional margins.

A.5.2 Wave overtopping

Information about the waves just in front of the dike is based on mathematic modelling carried out by DHI-Water-Environment. In Denmark wave run-up is used to determine the contribution of the waves in the required crest level of the dike. In the most recent dike design for the Rejsby Dike in 2001, the cross-section was directly based on scale model tests at DHI. The crest height has been calculated on basis of the above mentioned model studies and an overtopping criterion of 10%. The uncertainty of the design value (confidence interval) is not taken into account in the design.

In other cases the wave run-up is computed using (DWW, 2001):

$$Z_n = C_n \cdot T_m \cdot \sqrt{g \cdot H_s} \cdot \tan\alpha \tag{A.1}$$

in which

Z_n	=	wave run-up	[m]
C_n	=	$C_1 \cdot C(\varepsilon)$, see Table A.4 and Table A.5	[-]
H_s	=	significant wave height	[m]
T_m	=	wave period (not defined in detail)	[s]
$\tan \alpha$	=	slope angle	[-]

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).

The factor C_n depends on the allowed overtopping percentage, which in turn depends on the angle of the inner slope and the condition of the top layer as shown in Table A.4 and Table A.5.

Slope	unprotected dike surface	turf, sandy	turf, clayey
1:1.5	2%	10%	10%
1:2	2%	20%	50%
1:3	2%	30%	90%

Table A.4 Allowed overtopping percentages as function of the angle of the inner slope and the condition of the top layer of the inner slope (source: DWW, 2001)

Critical overtopping percentages	C_1	C_n
2%	1	0.7
10%	0.77	0.54
30%	0.56	0.39

Table A.5 Calculation factors for C_1 and C_n. (source: DWW, 2001)

In the designing of the Danish Wadden Sea dikes, in the formula of the critical overtopping percentage was set at 2% or 10% for back slope failure. The values used in Table A.5 are used to determine the strength of a dike expressed as the mean return period for the critical load.

A.5.3 Revetments

The Danish dikes are green dikes where only clay is used to strengthen the outer and inner slopes. The selected layer thickness has not been calculated by use of a formula, but is based on a compromise between economy and technical experience. No standard tools have been used.

A.5.4 Probabilistic approach to safety assessment

The design is deterministic. Probabilistic methods have not been used until now. Probabilistic methods are right now acquired and tested in COMRISK subproject SP7 and in an internal project at the DCA.

A.6 Evaluation of sandy coasts

The southern part of the North Sea coast has high rates of erosion caused by harbour breakwaters and large groyne groups. The dunes were stabilized about 100 years ago by planting marram grass. At the same time, harbours and groyne groups were built, resulting in serious downdrift erosion. The combined result of the dune stabilisation and the erosion was that the dunes had disappeared or should be implemented. The objectives of the policy were:

- to re-establish a flood safety level with a minimum 100-year return period;
- to stop erosion where towns were situated near the beach;
- to reduce erosion along parts of the coast where it would reduce the flood safety level to less than 100 years in the near future.

The dunes were reinforced and new dunes were built to re-establish flood safety to a 100-year return period. A number of measures were taken. Block supports were placed to protect the dunes (due to too little space between beach and houses). On highly exposed stretches, where erosion must be stopped, low detached breakwaters were used in combination with beach nourishment. One reason for the use of breakwaters was historical, since local politicians trusted hard structures like groynes and breakwaters. A second was the high price of nourishment sand. Nourishment was applied. but only on a small scale, mainly because the principle of beach nourishment was new to the politicians.

In 1998, the coastline retreat rate was much lower than in 1982 and a safety level of at least 10 years had been re-established for the dunes.

In the period 1993-1996 Denmark participated in a MAST project called Nourtec. The main conclusion of this project was that shore face nourishment is more stable than beach nourishment. The former is also cheaper and will be implemented in Denmark in the future (in combination with beach nourishment).

A.6.1 Strength parameters

Denmark uses a fairly simple criterion for the safety assessment of the dunes. These must have a minimum width of 40m at a height of 5m above MSL for unprotected dunes and 30m for dunes protected by a revetment. This is shown in Figure A.2, where *sikkerhedbredde* is the safety width and *bufferbredde* is an allowance for erosion before maintenance or reinforcement measures will be carried out to keep the safety width. These criteria are based on investigations by the DCA in 1990 of the dune width before and after storms based on surveys. This was combined with applications of the Dutch Vellinga Model to calculate expected dune erosion during design conditions. However, the calculated dune erosion under design conditions was smaller than the observed dune erosion of up till 30 metres.

The safety width is therefore based on the observations and directly applied to the existing dune profile.

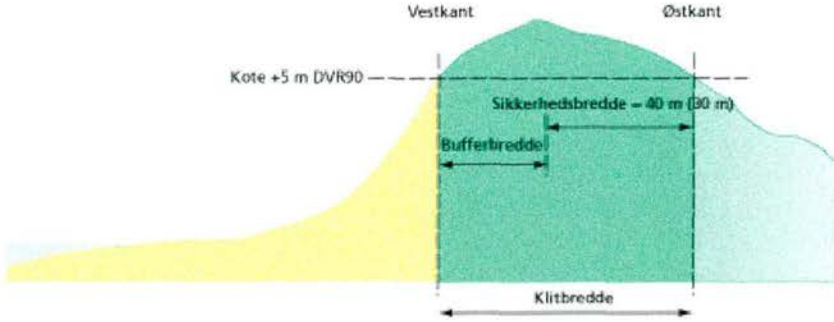


Figure A.2 Criterion for the minimum dune with used in Denmark.

An example of the way the criterion of is used, is shown in the plan view in Figure A.3. The dashed line indicates the safety width. Comparison with the 5 m height contours (dark green area) shows the sections of dune coast that are too narrow.

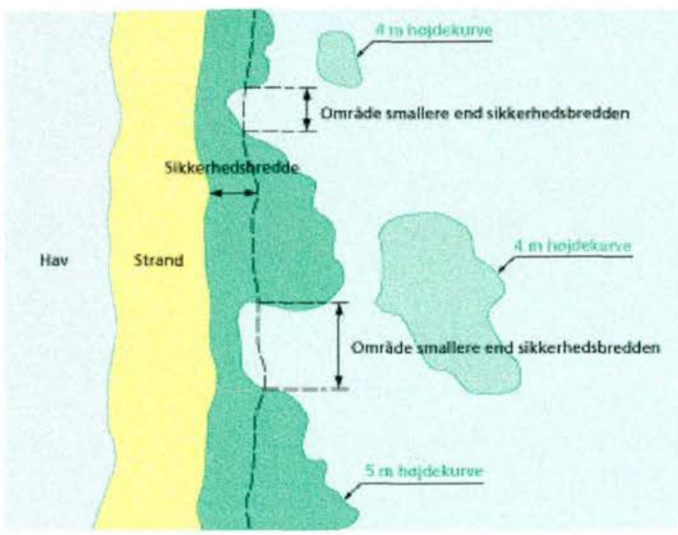


Figure A.3 Example of use of the minimum dune in Denmark.

A.6.2 Approach to safety assessment

An analysis of the shoreline variation over several years together with the variation of the cross-shore profile divided into different sections, are used to estimate dune erosion during design conditions.

The design is deterministic, probabilistic methods are not used. Probabilistic methods will be acquired in an internal project at the DCA.

Water level, wave height, wave period, wave direction, current velocity and current direction in the nearshore are required for the design process. The general shoreline and year-to-year variations in the cross-shore profile retreat are taken into account.

The cross-shore profile is measured regularly. The central stretch of the Danish North Sea coast is measured every year. The other stretches of the North Sea coast are measured every fourth year.

A.7 References

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, 2001. **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), April 2001, 27 pp + appendices.

B Situation in Schleswig-Holstein

B.1 General

The old philosophy of executing coastal defence (building sea walls) in order to reclaim fertile land already ceased in the early fifties. In Schleswig-Holstein, the last sea-wall aiming at this purpose was constructed in 1954 (Friedrich-Wilhelm-Lübke-Koog). In the masterplan coastal defence 1963 ("Generalplanes Küstenschütz 1963") for the first time a uniform high safety standard was specified for the West coast (i.e., each sea wall has the same probability of breaching). In the updating this standard was improved, since at continuous hydrologic calculation the trend of the due conditions was flattened.

Due to the impact of the storm tide of 1962, the state dikes at the West coast were the most urgent problem and therefore the central topic in the design of the dike reinforcement, shortening of the dike line and coastal protection in Schleswig-Holstein in 1963. The state dikes at the East coast were only considered as far as completion was concerned. This can be explained by the fact that catastrophic storm tides at the Baltic Sea are rarer and that the area at risk of flooding is only 8.5 % of the total flood-prone area in Schleswig-Holstein.

The sixties, seventies and early eighties were characterised by a strong belief in engineering (hard) solutions for coastal defence. However, this attitude changed into trying to use more natural techniques and material, e.g. sand nourishment to combat coastal retreat. In 1995 in Schleswig-Holstein a common salt marsh management plan was established by coastal defence and environmental authorities that aims at an ecologically sound protection and management of salt marshes. Salt marshes being both an important (natural) coastal defence structure as well as an ecologically sensible valuable area.

An important criterion was the shortening of the dike line, because this not only reduces the building and maintenance costs, but it also reduces the failure probability remaining after reinforcement. This effect became at that time designated as "Restrisiko". In this sense the length of the national defence dikes at the West coast is ca. 207 km shortened to 355 km since 1963. Between 1986 and 1990 diking-in is done only to a small extent (85 hectars: Fahretofter Koog and Ockholmer Koog). Presently there are no further plans for diking-in.

In the masterplan coastal defence 2001 are within the framework of the so-called dynamic dike safety, safety checks for the "state dikes" prescribed, which in future must be updated after the newest insights at least every 10 year. The dikes that do not fulfil the safety standards are placed on a list of investments, ordered by priority. The calculation of the dikes that need to be reinforced, has to include an investigation of the profile after the respective technological and scientific insights. During the definition of this list also further technical and socio-economic data are considered. Small measures with expenditures under 0.5 millions euro are not occupied with priorities. They are to be flexibly accomplished in the context of the annual household completion.

The state coastal defence authorities (two regional offices in Husum and Kiel and the "Innenministerium" of Schleswig-Holstein) are responsible for design and maintenance of the state dikes (413 km along the North Sea and the Baltic Sea). Local water boards are responsible for so-called "other dikes" (about 61 km with a lower standard). These other high water protection structures usually have lower safety standards and other legal foundations than the state dikes. A distinction is made between overtopping dikes and other high water protection structures such as walls, dams etc.

The entire coastline measures about 1.190 km.

B.2 Basic data

Hydraulic boundary conditions are provided (measured) partly in-house, partly by the Federal water authorities.

The dynamic dike safety system applied in Schleswig-Holstein yields to a differentiated calculation procedure. The target dimension of national protection dikes are constituted of:

- the normative flood level (calculation water level),
- the normative wave height, and
- a safety margin (appr. 0.5 m).

B.2.1 Water level

The design water level has to meet the following three criteria:

- it should have a return period of at least 100 year (probability lower than n = 0,01),
- it should not be lower than the highest storm surge recorded so far (reference value method), and
- it should not be lower than the sum of mean high water, spring tide set up and the highest surge height recorded (single value method).

For the state dikes along the Elbe the normative water levels were determined by the land working group Elbe on the basis of a deterministic numeric model, which results in a definition of the respective reference and normative water levels at the individual gauge locations.

B.2.2 Waves

The wave height is based on field measurements, forecast procedures and physical model tests.

West coast

Deep water waves (and wind) are measured at a location off Sylt as a basis for sand nourishment planning since 1984 (21 years). For the investigations of field measurements of the wave parameters and partly the wave run-up, the following series of measurement locations are present:

- Nordfrisian coast: 4 stations with wave measurements only (Sönke-Nissen-Koog, Holmer Siel, Strucklahnungshörn, Everschoopsiel; KFKI-Projekt Wattseegang).

- Ditmarscher coast: 4 stations, of which 2 stations have additional wave run-up measurements (Eiderdamm, Heringsand, Stinteck, Speicherkoog-Süd; ALW Heide/FZK).
- Elbe: 2 stations additionally wave run-up measurements (Brünsbüttel-Hermannshof, Neuendeich; ALW Heide/FZK).

Additionally some wave measurements from the KFKI- project "Estuary Sea Waves Elbe" and individual wave measurements in the Ausseneider, Piep and Elbe can be used.

East coast

For the Baltic Sea coast wave measurements were performed only at two locations. For Probstei the wave characteristics were measured during a long period, while for the region of the inner Lübecker Bucht measurements were done for a short term only. Comparison of these measurements (also for intermediate storm tides) with wave forecast procedures for shallow water areas resulted in justifiable agreements, so that for the region of Probstei relatively good characteristic values for maximum wave heights could be established theoretically. Usable measurements on wave run-up are not present for the coastal region of the Baltic Sea. This leaves only theoretical and/or estimated values for wave run-up to perform the safety checks and calculations. In the interest of continuity the values of the past masterplan were, after appropriate evaluation, taken over. In some places reduced values could be set because of the protected situation. Because of the uncertain knowledge the reference height of 2010 has been used in stead of the permissible wave overflow values. The former consists of the storm surge water level of 1872, added with the expected global sea level rise for the period 1872 to 2010 and the expected wave run-up.

B.2.3 Wind

Wind (and deep water waves) are measured at a location off Sylt as a basis for sand nourishment planning since 1984 (21 years)..

B.3 Data processing

B.3.1 Water level

The design water level has to meet the following three criteria:

- it should have a return period of at least 100 year (probability lower than n = 0,01),
- it should not be lower than the highest storm surge recorded so far (reference value method), and
- it should not be lower than the sum of mean high water, spring tide set up and the highest surge height recorded (single value method).

Three values are thus determined, from which on the basis of plausibility a design water level was determined. This applies in principle both to the North Sea coast (West coast) and to the Baltic Sea coast (East coast). The results are however different. For the Baltic Sea coast the storm tide water level of 1872, increased with a value of approximately 0.5 m for the expected sea level rise, determines the design water level. At the West coast the normative water level has been established through statistical analyses of the yearly highest high water levels registered at coastal gauge stations.

Probabilistic methods are used only to establish the design water level in Schleswig-Holstein. For the original masterplan of 1963, the yearly highest water levels from different tidal gauges for the period 1950-1991 were ordered according to Weibull. Through this Weibull distribution a line was drawn and a 100-year water level was extracted from the diagram. In the eighties, this very simple method was checked using different functions (Gumbel, Jenkinson, etc.) fitted through the Weibull distribution as well as updated time series.

For the new masterplan, the validity of the existing design water levels was reconsidered using a time series from 1950 to 1999, and the same methods as in the eighties. See below for details.

West coast

As an examination year the year 2010 was specified. At nine gauges the reference water level for this year was determined. The statistic value with n=0.01 was determined from time series over the last 50 years (1950 - 1999). Such a sample is available for substantially more locations than longer time series. Since an extrapolation into the future is necessary only for 10 years only, a 50-year sample is sufficient. The fact that the time series is characterized by the increase of the storm tides in the last decades, gives additional security to the value. The determined reference water levels are about 0.2 to 0.4 m higher than the normative storm tide water levels that were used in the past. Subsequently a reference water level has been determined for each dike section.

The normative water level, previously described, is related to the time year 2100 (=year of construction + 100). Since the lifetime of the structures is relatively long (100 year on average) the time series were taken as long as possible. This resulted that, with longer time series (75 year), the reference water levels were on average 0.2 m lower than the reference water levels for the year 2010, calculated on basis of a 50-year time series. Considering a uniform treatment of all dike sections, this value was taken off from the reference water levels. To the storm surge conditions determined accordingly, a value of 0.5 m was added for the expected sea level rise up to the year 2100. As a result, the established water level is generally 0.3 m higher than the reference water level 2010, and 0.3 to 0.65 m higher than the normative storm tide water levels specified in the old masterplan.

East coast

As said before, the design water level is based on the storm tide of 1872. The method applied is in principle the same as the one applied at the West coast. However, since there are only comparatively few field measurements available, especially with regard to waves and wave height, the data must to a large extent be theoretically determined. In future the database must be filled up.

Storm tides with very high water levels are relatively seldom at the East coast, which makes it difficult to determine probability of occurrence. The storm tide of November 12 and 13 1872 is, within living memory, known as the most enormous storm surge afflicting the Baltic Sea. Most other unusual storm that are historically passed on, had clearly lower water levels in Schleswig-Holstein. For the Mecklenburger and Pommersche Bucht extreme floods are well-known from the years 1625 and 1044. Although the corresponding water levels were in the same order of magnitude as the one measured in 1872, they can not be transferred directly to the Baltic Sea coast, but should be considered as having a water level at least 0.5 lower. A storm that can be considered acceptable and with a comparable extreme height, is the storm of 1320.

It is possible to determine a probability for these events. However, the two extreme floods deviate substantially from the other data and do therefore not fit into the whole of the remaining values. This reduces the usability of the statistic method substantially and makes the results extremely doubtful.

The water level with the statistically determined probability of occurrence of once in 100 years is, depending on the exact location, about 0.8 to 1.0 m lower than the storm surge conditions of 1872. The determination of a corresponding water level from the largest observed spring tide set-up over mean high water is possible only if the set-up can be determined separately. This is extremely difficult, because the water level at the Baltic Sea coast caused by storm events, is encompassed frequently and strongly by oscillations.

Therefore, for the range of the Baltic Sea, the storm tide in 1872 is used for the determination of the reference water level. Because of the small number of reliable gauges from this time, some values for the state dikes were established through calculations and interpolation. For the determination of the reference water level 2010, the observed water levels are increased with a margin to account for the global sea level rise to increase. New trend analyses confirm the results of the older investigations, according to which the sea level rise at the Baltic Sea coast was on average approximately 15 cm per century. For the period 1872 – 2010 this resulted in a sea level rise of 21 cm.

The normative water level, previously described, is related to the time 'year of construction + 100'. A preliminary determination is accomplished for the year 2100 and it must be updated in the respective building design. The calculation water level 2100 consists of the storm tide water level 1872, the sea level rise between 1872 and today, and the sea level rise expected to 2100. The prognosis of an accelerated rise was considered with a measure by approximately 30 cm, so that for the period from 1872 to 2100 on the peak value of the storm tide altogether 0.5 m is added as sea level rise. The smaller value than at the west coast is justifiable, because the storm tide of 1872 has a substantially lower probability of occurrence than 0.01. A comparative calculation with a statistically determined 100-yearly water level and an addition of 0.5 m for the next 100 years gave clearly lower values in the comparison to the water level based on the single value of 1872.

B.3.2 Waves

The wave run-up values were established deterministically using the formula of Hunt. This formula was validated by levelling of the flotsam data on outer dike slopes during extreme storm surges. This resulted in values, which in general were on the save side. In the determined normative wave heights a safety margin was added to cover up general uncertainties in the calculation. For the Baltic Sea coast only few wave measurements were present. From these data as well as from forecast procedures and physical model tests the normative wave heights were determined.

For the new masterplan, the formula of Hunt was, again, used as a basis for the establishment of the respective wave run-up. However, the general formula of Hunt was modified with specific coefficients to fit local wave characteristics and dike geometries. The coefficients were established on the basis of field measurements and physical model results.

For the safety assessment of the sea defences no directional information is used.

B.4 Nearshore conditions

B.4.1 Water level

For the west coast, the design water level is, with consideration of the above mentioned three conditions, the water level with a probability of entrance of n=0.01 related to the examination year. For the Baltic Sea coast this is the storm tide water level of 1872 plus the sea level rise up to the examination year.

B.4.2 Waves

The procedure to derive general wave conditions from regulation parameters, is based on formulae, established through numerous field measurements of wind-generated wave conditions in coastal zones with small water depths (Wadden Seas). The measured wave heights appeared to be primarily determined by the water depth (depth-limited wave growth). Thus the stationary relationship for the wave height $H_{1/3}$ can be described completely with two regulation parameters (DZ, GR):

$$H_{1/3} = (SWL - DZ) * Gr \tag{B.1}$$

It showed that the correlation between the wave period and the wave height described by two regulation parameters (a, b) as follows:

$$T_m = a + b * H_{1/3} \tag{B.2}$$

With the regulation parameters the wave height $H_{1/3}$, as function of the water level SWL, and the wave period T_m, as function of the wave height, can be determined. For some regions in the Elbe estuary a parabola-shaped curve is used as an alternative. For high water conditions (NN + 4.0 to NN + 7.0 m), the wave parameters in the Elbe are established through a linear relationship between the wave height and the water level, similar to the Wadden Sea. Wave heights at higher depths – necessary for, e.g., numerical simulations – are determined through linear extrapolation.

In the Math-CAD model for the determination of the wave run-up and wave overtopping, which is used both for the safety checks of the dikes and in the reversal process for the determination of the wave parameters from flotsam data, the normative wave conditions are calculated with the regulation parameters. The results for the respective location-dependent relations for $H_{1/3}$ (water depth - wave height relation) and T_m (wave height - wave period relation), can be generalized in linkage with the morphologic area characteristics in such a way that thereby relations between local swell and local area characteristics can be described. Typical morphologic area characteristics are , e.g., the distance to deep water, size of trenches, significant heights of the shoals, influence of submarine contours on refraction and shoaling of the waves.

The Math-CAD model iteratively determines the wave parameters from the results of the flotsam data and the associated dike profiles at the flotsam measuring locations in the reverse process. The calibration of this procedure took place at the measuring locations, at which additionally wave height and wave run-up measurements were done. Altogether for 143 flotsam stations computations of the swell parameters were done (38 stations at the North Frisian mainland coast, 58 stations on the North Frisian Islands (Pellworm, Foehr, Sylt), stations at the Dithmar coast and 33 stations for the region of the Elbe).

In the context of the determination of the normative wave conditions, the wave forecast procedure of Sverdrup-Munk-Bretschneider (CERC 1984 for shallow water conditions, Sverdrup Munk Bretschneider SMB) was also used. A comparison of the results from the SMB-computations with the results of gives substantial differences. This can to a large extent be explained by the data used in the forecast procedure. The wave parameters are derived from wind data (fetch, wind velocity, wind duration), assuming constant water depths. However, the Wadden Sea has a very complex topography (multiple shallow water areas) with friction, refraction and diffraction effects and interaction between waves and morphology. Therefore, the application of the wave forecast procedure in the Wadden Sea areas is only limitedly valid. In an estuary like the Elbe (i.e., in the region of Brunsbüttel), the SMB procedure can reliably be applied.

For the determination of the normative wave conditions, numerical wave models, such as, e.g., the SWAN model, are used more and more. The model SWAN was used, as an experiment, in the region of Brünsbuttel Altenhafen for the determination of the wave conditions along the dike line. In order to apply the numerical models reliably, sufficient field data should be available for the provision of correct boundary conditions and for verification and calibration of the model.

B.5 Design / safety assessment sea dikes

In the old masterplan coastal defence from 1963, it was assumed that the design water level is valid up to the year 2000. Hence, the yearly high water levels were homogenised (e.g., corrected for sea level rise) to the year 2000. In the new masterplan, the validity of design components will be checked every 10 years (2010, 2020, etc.) to accommodate sea level rise and new technical developments (especially for the computation of wave run-up).

The instrument/method applied for design and safety assessment is the same, the data used differ.

B.5.1 Strength parameters

The design of the main dikes comprise of three components:

- design water level;
- wave run-up;
- design slope.

B.5.2 Dike height

For the safety assessment of the dikes in Schleswig-Holstein both wave run-up and wave overtopping are evaluated (SH, 2002). The formula used for wave run-up, which is a modification of the formula from Hunt (1959), reads

$$R_{98} = \prod_{i=0}^{9} k_i \cdot \sqrt{H_{1/3}} \cdot T_m \tag{B.3}$$

in which R_{98} is the 2% wave run-up level, $H_{1/3}$ the significant wave period (=H_s) and T_m the mean zero-crossing wave period (=T_z). The factors k_i describe various influences related to the considered statistical parameter for wave run-up (k1), the wave conditions (k2 to k6) and the dike geometry (k7 to k9) according to Table B.1.

influence from	factor	Description
wave run-up	k0	dimensionless representation of the formula
	k1	statistic run-up parameter (e.g., R_{98})
wave characteristics	k2	wave characteristic
	k3	statistic wave height parameter (e.g., $H_{1/3}$)
	k4	statistic wave period parameter (e.g., T_m)
	k5	relative water depth conditions (= γ_h)
	k6	diagonal direction of wave propagation (= γ_β)
Dike geometry	k7	factor for outer slope (uniform, compound or irregular outer slopes; = tan α)
	k8	factor for additional berms (= γ_b)
	k9	factor for surface roughness of the dike (= γ_f)

Table B.1 Parameters for computation of wave run-up used in Schleswig-Holstein.

The values of the individual factors k_i are established using laboratory results as well as field data. The individual factors do not necessarily have a constant value, but can also be a function of several variables.

The documentation (SH, 2002) mentions that if the 2% wave run-up level exceeds the crest level, wave overtopping is calculated according to the formulae of Van der Meer (without detailed reference). This probably refers to a study carried out by WL | Delft Hydraulics (1993; see also TAW, 1999). These formulae read:

$$q = 0.06 \cdot \sqrt{gH_s^3}\,\frac{\sqrt{\tan\alpha}}{\sqrt{H_s/L_{0p}}} \cdot \exp\left(-5.2\frac{R_c}{H_s}\frac{\sqrt{H_s/L_{0p}}}{\tan\alpha}\frac{1}{\gamma_b \cdot \gamma_h \cdot \gamma_f \cdot \gamma_\beta}\right) \tag{B.4}$$

with a maximum of

$$q = 0.2 \cdot \sqrt{gH_s^3}\,\exp\left(-2.6\frac{R_c}{H_s}\frac{1}{\gamma_b \cdot \gamma_h \cdot \gamma_f \cdot \gamma_\beta}\right) \tag{B.5}$$

From the coefficients in the equations above it seems that the formula for the average relation is used, not the formulae recommended by the Dutch TAW (see also TAW, 1999). for design purposes. These are implemented in the MATH-CAD program. The documentation (SH, 2002) shows that peak wave period is estimated from the mean period using $T_p = 1.25T_m$.

Wave overtopping is of increasing importance for safety analysis of existing dikes and the calculation of dikes that need to be reinforced, since a modern dike can in principle withstand a certain overtopping rate without loss. In the past the determination of the crest height was on the basis of not allowing any wave overtopping. However, it is not feasible to hold this principle nor can an appropriate increase of crest height be accomplished for economic reasons. Therefore an average wave overtopping rate in the order of magnitude of 2 l/s/m is allowed, after the current soil mechanical insights and based on a clay dike with a 1:3 inner slope (cf. EAK2002).

An additional safety reserve of 50 cm was defined in the original masterplan coastal defence to account for sea level rise and uncertainties in the design. The design dike heights were established for about 60 flood units, ranging between NN +6.6 m and NN +8.8 m. On top of this, an individual margin for sagging and sinking after construction, depending on geotechnical analyses, is added.

B.5.2.1 Design slope

Apart from the normative water level and wave run-up, the dike profile and the characteristics of the soil material also have a separate influence on the resistance of the dike. Therefore, a standard design slope was established in Schleswig-Holstein with the following gradients:

- inner slope: 1:3.
- outer slope
  - lower parts: 1:15 to 1:10
  - near design water level: 1:8
  - upper parts 1:6

These slopes are prescribed in order to have sufficient resistance to withstand the wave attack (outer slopes) and not to endanger the erosion stability in case of wave overtopping and wave overflow (inner slope).

Sea walls that are fronted by salt marshes do not have stone revetments at their foot. Instead, the gradient in the lower parts is a bit flatter. A last requirement concerns the material. The outermost bottom layer of a dike must consist of bound material with high erosion resistance, for example boulder clay.

B.5.3 Numerical modelling

For the simulation of hydrodynamic processes at dikes that are under wave attack, the use of numerical models is also increasing. The 1-dimensional flow model ODIFLOCS, which has been developed to compute wave interaction on coastal structures, is used for functional investigations on dike profiles. Results of experimental applications showed that numerical models cannot correctly be calibrated, if friction coefficients are used, which in turn are necessary for a reliable reproduction of outer slopes of flat dikes. Therefore, the model ODIFLOCS is only applied in comparison, whereby both the cross-section and the wave parameters can be varied. In addition, the application of this model for comparison studies is also bound to certain conditions, since only profiles with small geometrical variations in the outer slope can be examined. For the application of numerical models in the process of optimization of the dike profiles still substantial research is need.

B.6 Evaluation of sandy coasts

The sandy coasts are not included in the regular safety assessment 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.

References

DWW, 2001. **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), April 2001, 27 pp + appendices.

EAK, 2002. **Empfehlungen für Küstenschutzwerke.** *Die Küste*, Heft 65, Jahr 2002. Kuratorium für Forschung im Küsteningenieurwesen.

SH, 2001. **Generalplan Küstenschutz; Integriertes Küstenschutzmanegement in Schleswig-Holstein.** Ministerium für ländliche Räume, Landesplanung, Landwirtschaft und Tourismus des Landes Schleswig-Holtein.

SH, 2002. **Sicherheitsüberprüfung / Bemessung der Landesschutzdeiche; Ermittlung von Wellenauflauf bzw. Wellenüberlauf an den Landesschutzdeichen der Westküste Schleswig-Holsteins für den Generalplan Küstenschutz**. Technical report of the Amt für ländliche Räume Husum, Abt. 5.2

WL | Delft Hydraulics, 1993. **Golfoploop en Golfoverslag bij Dijken; Samenvatting.** Report H638; April 1993.

TAW, 1999. **Leidraad Toetsen op Veiligheid.** Technische Adviescommissie voor de Waterkeringen. August 1999.

C Situation in Niedersachsen

C.1 General

Coastal (flood) defence in Germany is in the responsibility of the coastal states. For the North Sea these are Niedersachsen, Bremen, Hamburg and Schleswig-Holstein. In Niedersachsen the waterboards (Deichverbände) are responsible for the design and maintenance of the sea defences. These boards cover the biggest part of sea dikes at the Niedersachsen mainland coast. On the Eastfrysian Islands the federal state of Lower Saxony is the responsible party and the Niedersächsische Landesbetrieb für Wassenwirtschaft und Küstenschutz (NLWK) is the responsible state authority.[1] The NLWK is also responsible for some parts of the sea dikes at the mainland coastline and for the storm surge barriers. The state of Lower Saxony is also responsible for the maintenance of the foreland and dikefoot protection construction. For the coastal engineering point of view the design is done by NLWK/NLÖ-FSK (Coastal Research Station). The dike height and shape is officially approved by the county government.

In most cases the design is done in house (NLWK) or by the NLWK on behalf of the waterboards and the NLÖ-FSK. For special design tasks consultants are asked to carry out special investigations or parts of the projects (e.g. soil examination, structural design).

The defence system is surveyed and inspected regularly. These data are analysed to detect weak points. The survey and inspection is according to § 5(4) of the Niedersachsische Deichgesetz (Lower Saxony Dike Law).

Evaluation of the dike will be carried out in case of:

a) occurrence of higher storm surge levels than observed until now,
b) Other basic changes of design parameters (e.g. H_s, T), or
c) Changes in bathymetry.

The approach to design and periodic safety assessment is the same.

The hydraulic boundary conditions are provided by

- NLÖ-FSK (wave conditions (HN-model) and measurements of wave climate)
- University of Hannover (wave conditions (HN-model) and physical model tests)
- Leichtweiss Institut of the Technical University Braunschweig (physical model tests)

[1] *The Niedersächsische Landesbetrieb für Wassenwirtschaft und Küstenschutz (NLWK) has been founded on January 1 1998 after dissolution of the eleven of national offices for water and waste as well as the national office for island and coastal protection, whose tasks as far as possible it took over. As part of the landesverwaltung the NLWK is responsible for wide ranges of the water management and the coastal protection in Lower Saxony. As national upper authority it comes directly under the Niedersächsischen Umweltministeriums and is responsible for:*

- *Maintenance of local waters and water-economical plants;*
- *Planning and building of coastal protection mechanisms, waters and water-economical plants;*
- *Hydraulic engineering national service.*

- BSH Bundesamt für Seeschiffahrt und Hydrographie (wave and wind data),
- WSV Wasser und Schiffahrtverwaltung (water level data provided by a gauge net)
- DWD Deutscher Wetter Dienst (predicted and measured wind fields (velocity and direction)).

C.2 Basic data

C.2.1 Water level

Both measurements and hindcast data are necessary for the design method *Einzelwehrt-Verfahren* (single value procedure, see). This method uses:

- Mean High Water; 10-year mean value (in German: MThw);
- Maximum water level offset caused by a spring tide (= Spring tide – Mean High Water);
- Highest occured storm surge (= Highest occurred high water – Predicted astronomical tide).

The measured water levels are provided by WSV Wasser und Schiffahrtverwaltung. The gauges in the local region are maintained by WSA Emden (office of WSV).

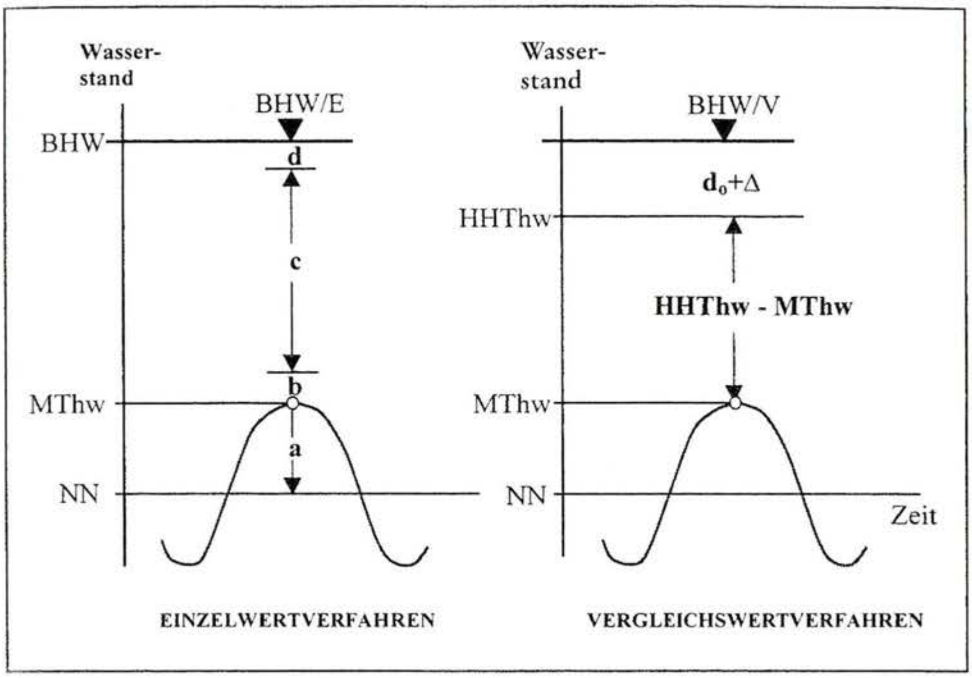


Figure C.1 Principle of the method *Einzelwet-Verfahren* used in Niedersaschsen.

C.2.2 Waves

The basic wave data are provided by NLÖ-FSK (wave conditions (HN-model) and measurements of wave climate), BSH Bundesamt für Seeschiffahrt und Hydrographie (wave und wind data).

C.2.3 Wind

Wind data are obtained from the BSH (Bundesamt für Seeschiffahrt und Hydrographie), comprising both measured and forecasted wind data.

C.3 Data processing

No detailed information on processing of the data was available.

C.4 Nearshore conditions

C.4.1 Water level

Both measurements and hindcast data are necessary for the design method "Einzelwehrt-Verfahren" (single value procedure). This method uses:

- Mean High Water (MHW) above GOL (German Ordnance Level); 10-year mean value;
- Maximum water level offset caused by a spring tide (= Spring tide – Mean High Water);
- Height difference between MHW and the highest observed (storm surge) water level (= Highest occurred high water – Predicted astronomical tide).

The following numerical models are used: TRISULA and DELFT3D.

TRISULA
TRISULA is a flow simulation model which includes tide, wind, wave and density driven flow capabilities. Various turbulence closure models, among which a k-e model are available. TRISULA solves the three dimensional shallow water equations by an implicit finite difference method (ADI) on a staggered (spherical or curvilinear) grid.

DELFT3D
DELFT3D is a fully-integrated two or three-dimensional compound modelling system. It simulates the flows, waves, sediments, morphological developments and water quality aspects. The *sediment transport module* models bottom and suspended transport of sediment separately using a variety of formulae. The effects of wave motion on transport magnitude and direction are included. The *morphological module* computes bottom changes due to transport gradients and various types of boundary conditions. It can be run in a time-dependent way (coupling of hydrodynamics with computed bottom changes) or in a time-independent mode. In the time dependent mode, animation's can be made of the bottom development over several years.

C.4.2 Waves

To determine the maximum wave run-up, hindcasting is used. The following numerical models are being used:

SWAN
SWAN is a two dimensional full spectral wave model for wave propagation in shallow water including refraction and shoaling, growth due to wind action, non-linear wave interactions (Triad and Quadruplet) and dissipation by bottom friction and breaking.

MIKE 21
MIKE 21 is a 2D engineering modelling tool for rivers, estuaries and coastal waters. MIKE 21 is applicable to the simulation of hydraulic and related phenomena in lakes, estuaries, bays, coastal areas and seas where stratification can be neglected. The package consists of more than twenty modules covering the following areas:

- Coastal hydrodynamics;
- Environmental hydraulics;
- Sediment Processes;
- Wave processes.

C.5 Design / safety assessment sea dikes

The design of the main dikes comprises three components:

- design water level;
- wave run-up;
- design slope.

C.5.1 Strength parameters

The strength of a dike or dam with regard to flood protection is characterised by the crest level and the stability of cross-section. The failure mechanisms to be evaluated are prescribed in the Recommendations for the Execution of Coastal Protection Works (EAK, 2002). These include both hydraulic and geotechnical aspects. The most relevant hydraulic aspects are:

- failure of the crest level due to wave overtopping;
- failure of the stability of the revetment / armour.

C.5.1.1 Crest level

EAK 2002 is used to assess the crest height. There is no probability of exceedance that is used for the dike design since the design formula (Einzelwert-Verfahren) aims to avoid any exceedance. The design is deterministic. In Niedersachsen wave run-up is used to calculate the required crest height.

The formulae for run-up and overtopping adopted in Niedersachsen are given in the EAK2002. The equations for wave run-up used in Niedersachsen are:

$$z_{2\%} = 1.6 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \sqrt{\frac{g}{2\pi}} \cdot \sqrt{H_s} \cdot T_p \cdot \tan\alpha \tag{C.1}$$

with a maximum of

$$z_{2\%} = 3.2 \cdot \gamma_f \cdot \gamma_\beta \cdot H_s \tag{C.2}$$

in which $z_{2\%}$ is the run-up level exceeded by 2% of the waves, H_s the significant wave height, T_p the peak wave period. and $\tan\alpha$ the angle of the slope.

The formula in the EAK2002 to determine the average overtopping discharge (volume per meter length) reads:

$$q = 0.038 \cdot \gamma_b \cdot \sqrt{2gH_s^{\,3}}\, \frac{\tan\alpha}{\sqrt{H_s/L_0}} \cdot \exp\left(-3.7 \frac{R_c}{H_s} \frac{\sqrt{H_s/L_0}}{\tan\alpha} \frac{1}{\gamma_b \cdot \gamma_f \cdot \gamma_\beta}\right) \tag{C.3}$$

with a maximum of

$$q = 0.096 \cdot \sqrt{2gH_s^{\,3}}\, \exp\left(-1.85 \frac{R_c}{H_s} \frac{1}{\gamma_f \cdot \gamma_\beta}\right) \tag{C.4}$$

in which

q	=	mean overtopping discharge	[m^3/m/s]
g	=	gravitational acceleration	[m/s^2]
H_s	=	significant wave height at the toe of the dike	[m]
ξ_0	=	breaker parameter = $\tan\alpha / \sqrt{H_s / L_0}$	[-]
L_0	=	wave length at deep water = $gT_m^2 / 2\pi$	[m]
T_m	=	mean wave period	[s]
$\tan\alpha$	=	slope	[-]
h_k	=	crest level above the still water line	[m]
γ_β	=	reduction factor for the angle of wave attack $\gamma_\beta = 0.35 + 0.65 \cdot \cos\beta$	[-]
γ_b	=	reduction factor to account for influence of a berm	[-]
γ_f	=	reduction factor for surface roughness	[-]

For high water protection walls, the mean overtopping discharge can be computed with the following formula (Franco *et al.*, 1995; Oumeraci *et al.*, 1995).

$$\frac{q}{\sqrt{gH_s^3}} = 0.082 \cdot \exp\left(-3.0 \frac{h_k}{H_s \cdot \gamma_\beta}\right) \tag{C.5}$$

The EAK2002 provides also indicative values for the allowable mean overtopping discharge (see Table C.1), but is very explicit in stating that these must be used with utmost care and that further research is required to complete the table and to specify the criteria with greater accuracy.

	mean overtopping discharge					damages
buildings			q	<	0.001	Mostly no damage
	0.001	<	q	<	0.03	Small damage to building parts
			q	>	0.03	Massive damage
stone/concrete cover			q	<	50	Mostly no damage
	50	<	q	<	200	Damage for unprotected crest
			q	>	200	Damage possible
grass dike			q	<	1	No damage
	1	<	q	<	10	Damage if crest not protected
			q	>	10	Damage

Table C.1 Some criteria for the mean overtopping discharge for structural safety (in l/s/m; after EAK2002).

C.5.1.2 Revetments

EAK 2002/1993 provides the procedures and formulas to evaluate the strength of the revetments and rock armour.

There are numerous empirical formulae to find dimensions for the rock armour revetments, describing the relationship between the waves and the necessary weight of the armour rocks. The EAK focuses on the Hudson formula (CERC, 1984), which is based on the work of Irribarren, and the formula that is derived from the investigations of Van der Meer (1988). It is common to both formulas that the choice of the empirical factors contained in the equations affects the necessary stone size considerably. Therefore, the application of the equations as a rigid 'recipe' cannot lead to the optimal stone dimensions. Comparative calculations and in particular the local engineering experience are an elementary basis for the final dimensions of the stones as well as for the choice of the strength of the surface layer.

Hudson formula

The Hudson formula is based on a large series of physical scale model tests with regular waves. The formula yields:

$$W_{nec} = \frac{\rho_s \cdot g \cdot H}{K_D \left(\frac{\rho_s}{\rho_w} - 1 \right)^3 \cdot n} \tag{C.6}$$

in which

W_{nec}	=	necessary weight of one block in surface layer	[kN]
g	=	gravitational acceleration	[m/s^2]
H	=	measured wave height	[m]

K_D	=	dimensionless, experimentally determined parameter	[-]
ρ_s	=	density of the rocks	[t/m^3]
ρ_w	=	density of the water	[t/m^3]
n	=	cot θ	[-]
θ	=	angle of the structure slope measured from horizontal	[°]

In this formula, the slope, de design wave height, the density of the rocks as well as the density of the surrounding liquid are contained. In the value of the dimensionless K_D-parameter all other factors are contained, e.g. the form of the surface layer rocks, the sharpness of the edges of the armour units (degree of interlocking) and the form of the attacking wave (breaking or non-breaking waves). In the Shore Protection Manual (CERC, 1984) K_D-values varying between 1.1 for breaking waves on randomly placed smooth rounded quarry stones, to 31.8 in case of non-breaking waves on structures with randomly placed dolosses, are suggested (see Table C.2).

armour units	n*)	place-ment	structure trunk K_D^2		structure head K_D		slope
			breaking	non-breaking	breaking	non-breaking	cot θ
quarry stone – smooth rounded	2	random	1.2	2.4	1.1	1.9	1.5-3.0
quarry stone – smooth rounded	>3	random	1.6	3.2	1.4	2.3	1.5-3.0
quarry stone – rough angular	1	random	-	2.9	-	2.3	1.5-3.0
quarry stone – rough angular	2	random	2.0	4.0	1.9 1.6 1.3	3.2 2.8 2.3	1.5 2.0 3.0
quarry stone – rough angular	>3	random	2.2	4.5	2.1	4.2	1.5-3.0
quarry stone – rough angular	2	special	5.8	7.0	5.3	6.4	1.5-3.0
quarry stone – parallelelipided	2	special	7.0 -20.0	8.5 - 24.0	-	-	
tetrapod and quadripod	2	random	7.0	8.0	5.0 4.5 3.5	6.0 5.5 4.0	1.5 2.0 3.0
tribar	2	random	9.0	10.0	8.3 7.8 6.0	9.0 8.5 6.5	1.5 2.0 3.0
dolos	2	random	15.8	31.8	8.0 7.0	16.4 14.0	2.0 3.0
modified cube	2	random	6.5	7.5	-	5.0	1.5-3.0
hexapod	2	random	8.0	9.5	5.0	7.0	1.5-3.0
toskane	2	random	11.0	22.0	-	-	1.5-3.0
tribar	1	uniform	12.0	15.0	7.5	9.5	1.5-3.0
quarrystone – graded angular	-	random	2.2	2.5	-	-	1.5-3.0

*) n is the number of units comprising the thickness of the armour layer.

Table C.2 Suggested K_D values for use in determining armour unit weight (CERC, 1984)

Van der Meer (1988) formulae

On the basis of an extensive set of both large-scale and full-scale physical model tests with wave spectra Van der Meer (1988) developed a relationship for the dimensioning of rock armour units, in which the measured variables, which are summarized in the K_D-value of Hudson, are separately specified. These are :

- the wave steepness,
- the permeability of the surface layer and the filter layer,
- the permissible degree of destruction, and
- the storm duration.

The formula of Van de Meer (1988) suggests an increased accuracy in comparison to the Hudson formula, since the numerous measured variables are apparently exactly taken into account. However, large uncertainties arise in the determination of the measured variables (e.g. porosity and storm duration) such that the accuracy of the calculation is not increased compared to the Hudson formula.

As dimensionless characteristic value Van der Meer (1988) uses the stability number N_s, which represents the relationship of the attacking forces (wave height H_s) to the resisting forces ($\Delta \cdot D_{n50}$):

$$N_s = \frac{H_s}{\Delta \cdot D_{n50}} \tag{C.7}$$

in which

N_s	=	dimensionless stability number	[-]
H_s	=	significant wave height	[m]
D_{n50}	=	mean stone diameter	[m]
Δ	=	relative stone density ($= \rho_s/\rho_w - 1$)	[-]
ρ_s, ρ_w	=	density of the stones and the water	[t/m^3]

Since the weight of a stone is proportional to the third power of the stone diameter (i.e., $W \sim D_{n50}^3$), the wave height is, similar to the Hudson Formula, comprised with a third power in the computation of the surface layer. Van der Meer (1988) distinguishes between deep water and shallow water at the toe of the construction. In the latter case, wave breaking occurs and hence a change of the wave height. In this case Van der Meer (1988) advises to use $H_{2\%}$ (the wave height with an exceedance probability of 2%) instead of H_s. In practice it is hard to determine the value for $H_{2\%}$ since in shallow water the waves are not Rayleigh-distributed. Therefore, the relation $H_{2\%}/H_s = 1.4$ does not hold in shallow water. So, if it is not known whether the waves break at the toe of the structure, it is best to stay on the conservative side and assume a situation of deep water. Furthermore, Van der Meer (1988) distinguishes between plunging and surging waves.

For deep water the relation between the stability number N_s on the one hand and the wave steepness, the permeability, the storm duration and the damage number on the other can be written as:

$$N_s = 6.2 \cdot P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \xi_m^{-0.5} \tag{C.8}$$

for plunging waves, and

$$N_s = 1.0 \cdot P^{-0.13} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \sqrt{\cot \alpha} \cdot \xi_m^{\ P} \qquad \text{(C.9)}$$

for surging waves, and in which

S	=	dimensionless damage = A_e/D_{n50}^2	[-]
A_e	=	erosion area on profile around still water level	[m^2]
P	=	notional permeability factor	[-]
N	=	number of waves in a storm, record or test	[-]
ξ_m	=	breaker parameter or surf similarity parameter = tan $\alpha/\sqrt{s_{om}}$	[-]
s_{om}	=	wave steepness based on H_s and T_m = $2\pi H_s / gT_m^2$	[-]

For shallow water the relationship is:

$$N_s = 8.7 \cdot P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \xi_m^{\ -0.5} \qquad \text{(C.10)}$$

for plunging waves, and

$$N_s = 1.4 \cdot P^{-0.13} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \sqrt{\cot \alpha} \cdot \xi_m^{\ P} \qquad \text{(C.11)}$$

for surging waves.

The transition from plunging to surging waves can be calculated using a critical value for ξ_m:

$$\xi_m = \left[6.2 \cdot P^{0.31} \cdot \sqrt{\tan \alpha} \right]^{\frac{1}{P+0.5}} \qquad \text{(C.12)}$$

For cot $\alpha > 3.0$ the critical value for ξ_m can be calculated as follows:

$$\xi_m = \left[3.58 \cdot P^{0.31} \cdot \sqrt{\tan \alpha} \right]^{\frac{1}{P+0.5}} \qquad \text{(C.13)}$$

For cot $\alpha \geq 4.0$ the transition from plunging to surging does not exist and for these slope angles only Eq. (C.8) and (C.10) should be used.

The notional porosity should have a value between 0.1 (surface layers with filter layer on impermeable subsoil) and 0.6 (homogeneous structure). From the Equations (C.8) – (C.11) it is clear that with increasing P, the necessary dimensions decrease. Physically this can be explained by the fact that on impermeable layers the water motion, and especially the wave run-up, is concentrated on the surface layer and this causes large forces on the single elements of the surface layer.

For the definition of damage three stages are discerned:

- beginning of damage,

- transition range, and
- failure of the structure.

The first step (beginning of damage) can be considered equal to the no-damage criterion in the Hudson formula. As design values for the two-layer surface the values as listed in Table C.3 are advised.

Slope	lightly	intermediate	completely
1:1.5	2	3-5	8
1:2	2	4-6	8
1:3	2	6-9	12
1:4	3	8-12	17
1:6	3	8-12	17

Table C.3 Degree of destruction.

Permeability	surface layer	filter	permeable core
P=0.1	x	x	no
P=0.4	x	x	yes
P=0.5	x	-	yes
P=0.6	homogeneous	-	yes

Table C.4 Composition of the structure.

The formulae of Van der Meer (1988) are valid for storm durations of 1000-7000 waves. The experiments showed that after 8000-9000 waves the degree of destruction had a maximum and that afterwards an equilibrium arose. The procedure after Van der Meer (1988) has the essential advantage that the degree of destruction can be estimated directly.

C.6 Evaluation of sandy coasts

C.6.1 Strength parameters

Strength is defined as the presence of a minimum width of the dune based on the morphological and hydrodynamic situation. The remaining dune width at the level NN+8m after numerical simulation of a storm flood with the design water level is taken as an indicator. The minimum required width at this level is 15 m. The uncertainties of the model and the data are taken into account by adding a defined width which is added to the calculated loss of width.

C.6.2 Approach to safety assessment

Numerical simulations are used to estimate dune erosion during design conditions. The design and safety assessment is deterministic and it is based on the following hydraulic input parameters: water level, T_p and H_s as a function of time, D_{50}.

The model NEWDUNE is used for this purpose. This model is comparable with the model EDUNE by Kriebel (1989) which is briefly described in the EAK2002. The NEWDUNE model was developed by Newe at LWI University of Braunschweig. NEWDUNE is based on an equilibrium profile (see Figure C.2) given by

$$h = A \cdot x^{2/3} \tag{3.1}$$

where A is a function of the fall velocity of the sand. The transition slope in deeper water of 1:2.5 and the slope of the dune above the zone of wave attack (tan δ =1) are taken after Vellinga (1983).

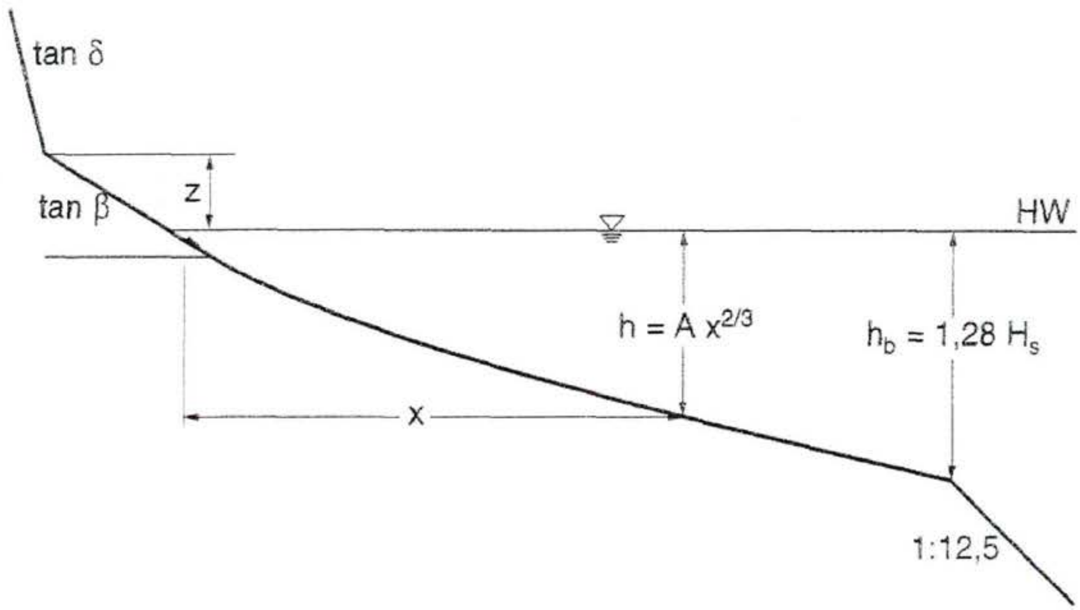


Figure C.2 Equilibrium profile used in the NEWDUNE model applied in Niedersachsen until 2003 (from LWI, 1998).

Since 2003/2004 the numerical model UNIBEST-DE (English version of DUROSTA; Steetzel, 1993) is used in addition to the NEWDUNE model. The experience shows that both models give comparable results for design conditions, but that NEWDUNE overestimates the erosion for more regular events (1/2 yr and 1/5 yr conditions; Blum, personal communication).

The year-to-year behaviour of a cross-shore profile is observed by the spring and autumn surveying. The normal seasonal-caused behaviour of a cross-shore profile is checked in the analysis. The shifting of specific bathymetry-lines (e.g. NN+0, NN+3m (= dune foot)) is documented in time-position-plots.

If a beach section is detected as an very erosive zone, the investigations may lead to calculations of a trend of erosion (shift of shoreline or volume-based trend). So this general shoreline behaviour could be used to determine the remaining "safe" time. For these beach sections numerical simulations (for the weak points) are carried out regularly (period of 1-2 years).

The cross-shore profile is measured twice a year. The first surveying data are dated of the 1930. Since the end of the 1990's differential GPS surveying is standard use. For every erosive zone LIDAR surveys of the dune area are carried out regularly.

References

CERC, 1984. **Shore Protection Manual**, Vol. I-III, Coastal Engineering Research Center, Vicksburg.

DWW, 2001. **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), April 2001, 27 pp + appendices.

EAK, 2002. **EAK 2002 Empfehlungen für die Ausführung von Küstenschutzwerken. Die Küste**. Kuratorium für Forschung im Küsteningenieurwesen.

Franco, C., L. Franco, C. Restano, J.W. van der Meer, 1995. **The Effect of Wave Obliquity and Short Crestedness on the Overtopping Rate and Volume Distribution on Caisson breakwaters**. MAST II-MCS Proceedings. Final Workshop.

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

LWI, 1998. **Kalibrierung des Dünenabbruchmodelles NEWDUNE am Beispiel der Ostfriesischen Inseln Borkum, Juist, Langeoog und Wangerooge.** Unpublished report No. 838, November 1998. Leichtweiss-Institut für Wasserbau. Technische Universität Braunschweig.

Oumeraci, H., H. Schüttrumpf. J. Möller, M. Kudella, 1995. **MCS-Project Multi-disciplinary Research on Monolithic Coastal Structures**, MAST II-MCS Proceedings. Final Workshop.

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.

Van der Meer, J.W., 1988. **Rock Slopes and Gravel Beaches under Wave Attack**. Delft Hydraulics Publications No. 396.

Vellinga, P., 1983. **Predictive Computational Model for Beach and Dune Erosion during Strom Surges.** Proc. of Coastal Structures '83, Arlington / Virginia, ASCE, pp. 806-819.

D Situation in The Netherlands

D.1 General

In The Netherlands the safety standards for all flood-prone low-lying areas are set in the Flood Protection Act (Wet op de waterkering, 1996). This law requires that the strength of the defences against flooding (river and sea dikes, dunes and constructions therein) is evaluated every five years. This safety assessment is carried out by the waterboards that are in charge of the flood protection in specific areas. This process is supported by the national government (Ministry of Transport and Public Works) by providing hydraulic boundary conditions and guidelines for the safety assessment.

This first set of boundary conditions was prepared in 1996 and issued simultaneously with the Flood Protection Act. The current version of the "Book of Boundary Conditions" is from 2001 (DWW, 2001b), but the hydraulic data in this version are not essentially different from the previous version. This publication, which is rather a compilation of all readily available data, contains for each dike section the water level and wave height condition to be used for the safety assessment of the flood defences. The data can be used in a deterministic approach to evaluate the strength of the sea defences for a number of aspects.

Along with the preparation of the Flood Protection Act, the Ministry of Transport and Public Works set out in a series of studies that must lead to a probabilistic approach to the safety of the sea defences (see e.g. RIKZ, 1999 for more background). For the probabilistic approach special software is being prepared such as HYDRA-K for the safety assessment of sea dikes (see e.g. HKV, 2000). A step further is the probabilistic assessment of the risk of flooding for entire "dike ring areas", regions within one system of primary flood defences, which is presently being developed within the VNK project (Veiligheid Nederland in Kaart, "Mapping the Safety of the Netherlands"). In this approach the total risk of flooding of a region is evaluated by integrating the risk of failure of all of the dike segments, dune sections or structures making up the dike ring, while taking into account the correlation between the occurrence of extreme hydraulic events for each of the segments.

This appendix rather describes the items of this probabilistic approach, which is soon expected to be adopted as the standard approach for the safety assessment, than the current set of boundary conditions, which are for some dike sections the wave conditions used for the design in the 1960-ies.

The procedure used to design new cross-sections for dikes that need to be upgraded is similar to procedure used for safety assessment. The difference is the longer time-horizon needs to be taken into account for factors such as sea level rise.

D.2 Basic data

D.2.1 Water level

The water levels used for the design and safety assessment of sea defences are based on the reference levels derived for the situation in 1985 ("Basispeilen 1985", "basic levels 1985"). These are the water levels with a probability of exceedance of 10^{-4} were determined in extensive studies in the late 1980-ies, early 1990-ies (RIKZ, 1993a,b,c, 1995a). The probability of exceedance to be used for the design and safety assessment of the sea defences depends on the location and can range from $5*10^{-4}$ to 10^{-4} (1/2,000 to 1/10,000). The water levels with larger probability of exceedance are derived from the "basic levels".

D.2.1.1 Measurements

Water level measurements are carried out for some 31 locations in the Dutch coastal waters. The locations are indicated in Figure D.1.

The data from 12 of these water level stations have been used in various studies for the determination of design conditions for the sea defences (RIKZ, 1993a, 1995a, 2000). These stations are given in Table D.1.

Station	Code	Period of data	Latitude	Longitude
Delfzijl	DFZ	1 Mar 1881 - 1985	53°20'	6°56'
Huibertsgat	HBG			
Lauwersoog	LWO			
Harlingen	HRL	1 June 1932 – 1985	53°10'	5°25'
West-Terschelling	TSW		53°22'	5°13'
Den Oever	DOV			
Den Helder	HLD	1 June 1932 – 1985	52°58'	4°45'
IJmuiden	YMB		52°28'	4°35'
Hoek van Holland	HVH	1 Aug 1887 – 1985	51°59'	4°07'
Vlissingen	VLS	1 July 1881 – 1985	51°27'	3°36'
Terneuzen			51°20'	3°50'
Hansweert	HSW			

Note: A blank cell indicates that information was not available in the reports used for this summary.

Table D.1 Water level stations used as basic stations for determination of design conditions for sea defences.

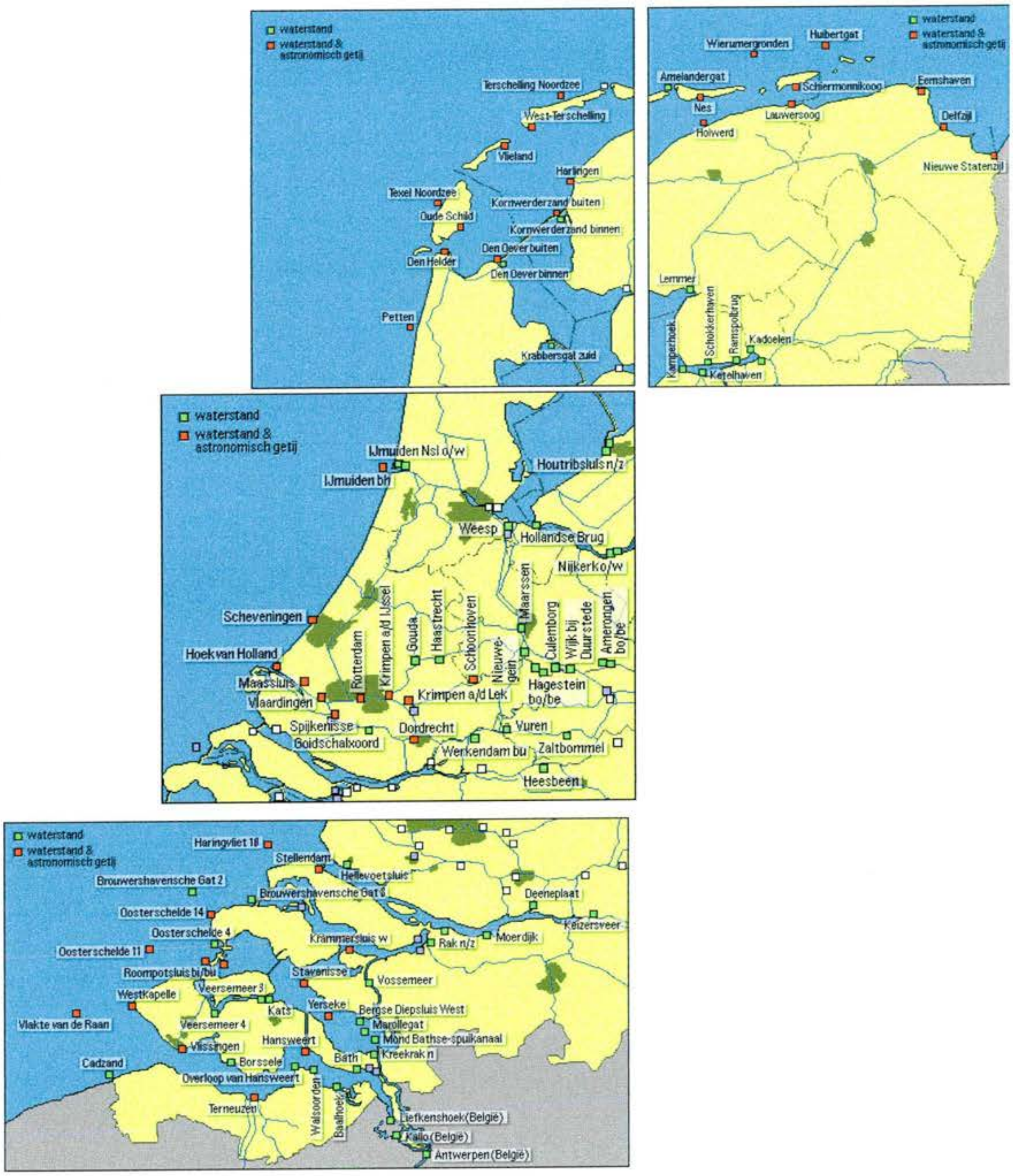


Figure D.1 Location of water level stations in the Netherlands.

For the stations Harlingen and Den Helder only data after 1932 have been used, because of the closure of the Afsluitdijk in that year changed the hydrodynamics in the western part of the Wadden Sea considerably, so that inclusion of data before that year would lead to an inconsistent data set. The observed high waters in these stations served as input for a statistical analysis.

D.2.1.2 Hindcast

The results of the statistical analysis are combined with the results of model simulations for extreme storm conditions (RIKZ, 1993a,b,c). The purpose of this hindcast study was to obtain more insight in some physical processes that are of relevance for the water levels in extreme conditions, in particular for the Wadden Sea area in the north of The Netherlands:

- the limited flow capacity of the tidal inlets may, depending on the storm duration, reduce the water levels that are reached in the western part of the Wadden Sea;
- the increased water depth on the tidal flats during extreme water levels may affect the hydrodynamics in the area, especially with regard to wind set-up.

In the hindcast study four historical storms were selected, which were manipulated in such a way that water levels were obtained in the range of the extreme probability of exceedance. To this end the following modifications were made to the original storm data:

- increase the wind speeds by a factor ranging between 1 and 1.75;
- increase the duration of the storms by a factor 1.25 and 1.5;
- shift the storms in time so that the storm peak coincides with different phases of the tide;
- shift the storms in space so that the area of maximum wind speeds affects different parts of the Dutch coast.

The water levels during the storms were hindcast using three models of different resolution made using the WAQUA package: the Continental Shelf Model, the Southern North Sea Model and a model of the Wadden Sea.

D.2.2 Waves

D.2.2.1 Measurements

Wave conditions are measured at the 9 locations in the Dutch coastal waters given in Table D.2. The position of the stations is indicated in Figure D.2. Table D.2 also shows the water depth and the date of the start of the measurements. The start date for measurements with a directional wave buoy is shown in parentheses.

The stations SON, ELD, YM6, K13 and EUR are located further offshore in deeper water and are used to derived the deep water statistics with regard to the wave conditions for the sea defences.

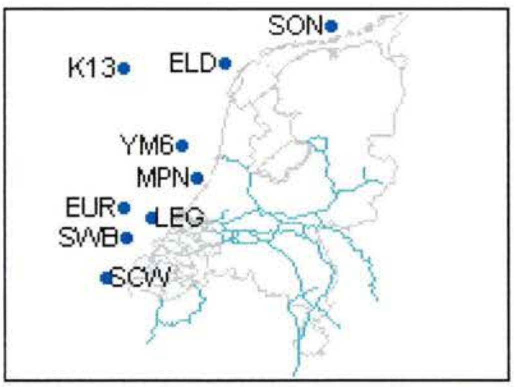


Figure D.2 Location of wave measuring stations in the Netherlands.

Station	Code	Start of data	Lat	Long	Water depth
Platform K13a	K13	18 Jan 1979 (21 Mar 1985)	53°13'04"	3°13'13"	30
Schiermonnikoog-Noord	SON	24 Nov 1979	53°35'44"	6°10'00"	19
Eierlandse Gat	ELD	13 Sep 1979	53°16'37"	4°39'42"	26
IJmuiden Munitiestort.	YM6	12 Jan 1979	52°33'00"	4°03'30"	21
Meetpost Noordwijk	MPN	1 Jan 1979	52°16'26"	4°17'46"	18
Euro Platform	EUR	19 Nov 1982	51°59'55"	3°16'35"	32
Lichteiland Goeree	LEG	1 Jan 1979	51°55'33"	3°40'11"	21
Schouwenbank	SWB	1 Jan 1979	51°44'48"	3°18'24"	20
Scheur West	SCW	1 Jan 1985	51°23'32"	3°02'57"	15

Table D.2 Locations where wave measurements are carried out. Stations SON, ELD, YM6, K13 and EUR are used as basic stations for determination of design conditions for sea defences.

D.2.2.2 Hindcast

In the late 1980-ies a consortium of oil companies decided to carry out a hindcast study to obtain more reliable data for the design of structures in the North Sea. Rijkswaterstaat participated in this North European Storm Study (NESS). In this studies the wave conditions wave been hindcast using the HYPA model of GKSS Forschungszentrum (Germany) for all winter seasons in the period October 1964 – March 1989, the summer seasons of 1977, 1978 and 1979 and all other summer storms in the period 1964-1989. RIKZ obtained data for 130 points of the 30*30km grid that are located in the southern North Sea.

D.2.3 Wind

The probabilistic approach that is presently being developed for the design of sea dikes requires directional extreme value distributions for the water level, wave height and period. This has been determined by analysing the joint occurrence of these parameters with the wind direction. The wind data are taken from a few coastal stations that are considered to be representative for a certain section of the coast (RIKZ, 2000). These wind stations are given in Table D.3. Wind data have been used for the period 1981 – 1996, because in this period simultaneous data of waves and water levels were available.

Station	Code	Lat	Long
Terschelling West	TSW	53°21' 8.8"	5°11' 6.26"
De Kooy (Den Helder)	KOY	52°55'42.39"	4°47' 7.94"
IJmuiden	YMS	52°27'45.59"	4°33'23.27"
Hoek van Holland	HVH	51°59'14.77"	4° 5'10.71"
Euro Platform	EUR	*)	*)
Lichteiland Goeree	LEG	51°55' 4.48"	3°40' 6.16"
Vlissingen	VLS	51°26'32.22"	3°35'50.46"

*) Lat, long not available; easting 10044, northing 447580 in local system of The Netherlands

Table D.3 Basic stations from which wind measurements are used in the probabilistic approach to the safety assessment and design of sea defences.

D.3 Data processing

D.3.1 Water level

D.3.1.1 Data treatment

For the statistical analysis the data regarding observed high water (HW) and the surge at high water (HW surge) from the selected stations have been used. Before carrying out the statistical evaluation, an extensive process of data treatment and selection was carried out on data for the 9 reference stations. This data treatment included:

- filling in gaps by multiple linear regression based on data from other stations;
- determination of the HW surge by subtracting the astronomical high water from the observed high water level (irrespective of any shift in time between actual and predicted high water);
- select high waters for which HW surge is larger than 30cm;
- select only values in the representative storm season (1 October – 15 March) to obtain homogeneous data;
- selection against auto-correlation: secondary maxima within 4 high waters from the real maximum are discarded;
- correction for trends (especially increase of HW) so that all data are representative for the situation in 1985.

D.3.1.2 Statistical evaluation

The number of events per storm season was described using a Poisson distribution. This was combined with five different extreme-value distributions for the water level of the selected events:

- "distribution free method" - c (VVM-c, Verdelings Vrije methode; similar to GPV);
- "distribution free method" - 0 (VVM-0, Verdelings Vrije methode; similar to GPV);
- Generalised Pareto distribution (GPV);
- a method based on the convolution model (CON, a variant of the GPV method);
- Generalised Extreme Value distribution (GEV).

The last distribution was not applied to all selected maxima but to yearly maxima only.

All distributions were applied to the data of 5 selected stations (VLS, HVH, HLD, HRL and DFZ, see Table D.1) to estimate the water level with a probability of exceedance of 10^{-4} and the corresponding 95% confidence interval. The VVM-0 method was adopted based on theoretical consideration and because of the results for the 5 stations. Further tests with regard to sensitivity and confidence intervals were carried out. The selected method was also applied to the stations TSW and YMB (RIKZ, 1993a; see Table D.1 for name and position of the stations).

D.3.2 Waves

D.3.2.1 Data treatment

Measurements

The measured wave data from the five deep water stations SON, ELD, K13, YM6 and EUR (see Table D.2) have been extensively analysed to estimate the deep water wave heights with a probability of exceedance of 10^{-4} and their associated wave periods (RIKZ, 1995b, 1996). The wave data have been measured using different instruments: instruments measuring only wave heights (step gauge, wave rider) have been replaced by directional wave buoys (wavec, directional wave rider). The data from the instruments have further been processed in different ways. The first step in the data processing was therefore an extensive validation of the data. This included a thorough scrutiny of the quality of all available records and an analysis of the different procedures used for initial processing. Simultaneous measurements of different instruments at the same location were also compared to allow correction of systematic errors.

Gaps in the data up to 3 months in duration were filled based on wind speed and direction dependent relations with other stations. These relations were established from periods of simultaneous measurements. The accuracy of these estimations ranged from similar to the measuring accuracy (about 5%) to approximately 25% under unfavourable conditions. In this way uninterrupted time-series of 13 years long were obtained for the five main stations. These time-series have been used to assess the extreme value statistics and for other climatologic studies.

The extreme wave height statistics were determined using a 'peak-over-threshold' (POT) method based on the wave height parameter H_{m0} (spectral significant wave height). In the data selection the following criteria were used:

- restricted to the winter period (1 October – 31 March);
- minimum distance of 2 days between two storm peaks;
- threshold selected in such a way that about 20 maxima were selected in each year.

This yielded between 200 and 260 events for each of the five stations to which the following corrections were made:

- a correction of values from different sensors so that all data correspond to "wavec" measurements;
- a reduction of 2% on all maxima to correct for the systematic over-estimation due to the fairly short sampling period (standard about 20 minutes every 3 hours);
- an improvement of the value of the highest 20 values in each station by careful analysis of the time series for these storms.

NESS hindcast

From the NESS dataset the results from grid points near the 5 measuring stations were selected. The hindcast results were compared with the measured data for the periods were simultaneous data were available. The accuracy of the model results was about 15% and for larger wave heights the values were systematically underestimated. The NESS data were treated in a similar way as the measurements:

- restriction to the winter period (1 October – 31 March);
- minimum distance of 2 days between two storm peaks;
- selection of values above certain threshold.

The threshold was taken 10cm higher than for the wave measurements, but the average number of events per storm season was nevertheless larger (22 to 26 depending the station).

D.3.2.2 Statistical evaluation

Five different extreme-value distributions were used in the statistical evaluations:

- Generalised Pareto distribution (GPV);
- "distribution free method" (VVM-0, Verdelings Vrije methode; similar to GPV);
- Weibull-distribution;
- Gumbel-distribution;
- Generalised Extreme Value distribution (GEV).

The two last distributions were applied to yearly maxima only.

Various tests were carried out to compare results and to evaluate the sensitivity of the distributions to outliers in the extremes. It was concluded that the Weibull-distribution was the most appropriate because of its robust results (not very sensitive to a change in the value of the highest observation) in combination with the fairly short time-series that were available. With this distribution detailed studies were carried out into the sensitivity of the results for the threshold and the way the parameters of the distribution were estimated. This was carried out both for the measurements only and for a combination of the measurements with the NESS hindcast results. The adopted method is based on measurements and NESS data (because the longer observation period outweighs the larger inaccuracy of the hindcast results) and uses a Weibull-distribution with a fixed shape parameter of 2.62 and the threshold significant wave height ranging from 3.7 to 4.3m, depending of the station. The significant wave height with a probability of exceedance of 10^{-4} ranges from 8.4m (EUR) to 10.0m (ELD).

D.3.3 Directional extremes

For the probabilistic approach for the safety assessment of the sea defences the individual statistics of water level, wave height and wave period are not sufficient. For such a probabilistic assessment data on the dependency of these parameters per directional sector is required.

This has been determined by deriving the extreme value distributions (exceedance curves) for the high water level, the wave height H_{m0} and the wave periods T_{m02} and T_p for directional sectors (RIKZ, 2000). This has been carried out for based on simultaneous measurements for the period 1981 – 1996. It has been verified that this fairly short period is representative by comparison with results for longer in a single station where more data were available.

The directional extreme value distributions have been determined adopting a Weibull-distribution, the same distribution as adopted for the wave height (and wave periods). The derived directional extremes have been adjusted in such a way that the sum of the probabilities of exceedance for a certain value is the same as the value that was derived based on the reference water level or from the marginal statistics of wave height and period. The directional extremes were derived for direction sectors of 10 degrees. This will not be used in practice, but allows for a large flexibility for later use. The results are summarized in tables presenting the parameters of all the distributions (threshold, frequency of exceedance of the threshold per year, scale parameter and shape parameter). Table D.4 summarizes the stations used in the derive the directional extreme value distributions.

Station	Code	Used with water levels for	Used with waves for
Terschelling West	TSW	DFZ, HBG, TSW, LWO, HRL, DOV	SON
De Kooy (Den Helder)	KOY	HLD	ELD, K13
IJmuiden	YMS	YMB	YM6
Hoek van Holland	HVH	(HVH)	
Euro Platform	EUR	(HVH)	EUR
Lichteiland Goeree	LEG	HVH	
Vlissingen	VLS	VLS, HSW	

Table D.4 Basic stations from which wind measurements are used in the probabilistic approach to the safety assessment and design of sea defences.

D.4 Nearshore conditions

D.4.1 Water level

Based on the water levels for the basic stations the design water levels for all sea defences have been determined (RIKZ, 1995a). This is carried out for four regions:

- the Wadden Sea and Eems-Dollard estuary;
- the coast of Holland between Hoek van Holland en Den Helder;
- the North Sea coast of the (former) islands in the south-west of the Netherlands (the Delta area);
- the Western Scheldt and the coast south thereof towards Belgium (Zeeuws-Vlaanderen).

The spatial interpolation of the reference level (10^{-4} exceedance value) was carried out in two steps. First the reference level for other measuring stations and output points from the hindcast study (see Section 2.1.2) was determined by determining the relation between the water level in these locations and the water level in one of the basic stations. Hoek van Holland was taken as basic station for the western part of the Wadden Sea, the coast of Holland and the North Sea coast of the Delta area, West-Terschelling for the eastern part of the Wadden Sea and the Eems-Dollard estuary and Vlissingen for the Western Scheldt.

In the second step the reference level for each part of the sea defences was determined by interpolation based on the data obtained in the first step. The method adopted for the interpolation differed by region. For the Wadden Sea it appeared to be reasonable to assume that the 10^{-4} value was a plane surface. For the coast of Holland and the Delta area the values are based on the results of one of the hindcast storms that yields results that match the water level relations between the various locations well. In the Western Scheldt sufficient output locations were available to allow direct spatial interpolation. Small corrections in the spatial distribution were made based on the results in the basic water level stations IJmuiden, West-Terschelling, Terneuzen and Hansweert.

Based on the reference level (10^{-4} exceedance probability) and the values with an exceedance probability of $5*10^{-1}$ and 10^{-1} as derived directly from the measurements, the water level exceedance curves for the basic stations were determined. The spatial distribution of the water level for exceedance frequencies as adopted for design and safety assessment was derived in the same way as for the reference level.

D.4.2 Waves

To obtain wave conditions at the toe of the sea defences a series of computations have been carried out using the program SWAN. SWAN is a fully-spectral 2-dimensional wave propagation and generation program based on the energy balance equation which models all relevant physical processes of wave propagation such as:

- fully discrete direction and frequency spectrum;
- refraction and shoaling due to variations in depth and currents;
- wave growth due to wind;
- wave dissipation due to white-capping, surf-breaking and bottom friction;

- non-linear wave-wave interactions in deep and shallow water (quadruplets, triads);
- transmission and reflection at obstacles.

The SWAN program was developed a few years ago by the Delft University of Technology and is further described in a number of references (Ris, 1997; Booij, Ris and Holthuijsen, 1999). The model is public domain and has been downloaded by over 300 institutes, universities and consultants.

SWAN models were set up for various parts of the coast including the Wadden Sea and the Eastern and Western Scheldt estuaries (e.g. Alkyon, 1999). With these SWAN models (overall, intermediate and detailed nested models) for each part of the coast simulations were carried out for 3 or 4 fixed water levels, 14 wind directions and wind speeds ranging from a value of about 15 m/s to a value well above the wind speed with a probability of exceedance of 10^{-4}. Simulations were carried out for more that 200 conditions. At the seaward boundaries corresponding wave conditions were applied. Output was generated at locations near the toe of the dikes at a typical distance of 200m in alongshore direction. This provides a set of results that can be used as a basis for interpolation in the probabilistic approach to the safety of the sea defences.

D.5 Design / safety assessment sea dikes

D.5.1 Strength parameters

The strength of a dike or dam with regard to flood protection is characterised by the crest level and the stability of cross-section. The failure mechanisms to be evaluated are prescribed in the Guideline Evaluation on Safety (Leidraad Toetsen op Veiligheid; TAW, 1999). These include both hydraulic and geotechnical aspects. The most relevant hydraulic aspects are:

- failure of the crest level due to wave overtopping;
- failure of the stability of the revetment / armour.

D.5.1.1 Wave overtopping

In the Netherlands the dikes are evaluated and designed using a wave overtopping criterion as prescribed in the Guideline Evaluation on Safety (*Leidraad Toetsen op Veiligheid*; TAW, 1999). The presently used formulae are given in a technical report (TAW, 2002) as:

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b \cdot \xi_0 \cdot \exp\left(-4.3\frac{h_k}{H_{m0}}\frac{1}{\xi_0 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \tag{D.1}$$

with a maximum of

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \cdot \exp\left(-2.3\frac{h_k}{H_{m0}}\frac{1}{\gamma_f \cdot \gamma_\beta}\right) \tag{D.2}$$

in which

q	=	mean overtopping discharge	[m^3/m/s]
g	=	gravitational acceleration	[m/s^2]
H_{m0}	=	significant wave height at the toe of the dike	[m]
ξ_0	=	breaker parameter = $\tan\alpha / \sqrt{s_0}$	[-]
s_0	=	wave steepness = $2\pi H_{m0} / \left(gT_{m-10}^2\right)$	[-]
T_{m-10}	=	spectral mean period at the toe of the dike	[s]
$\tan\alpha$	=	slope	[-]
h_k	=	crest level above the still water line	[m]
γ_β	=	reduction factor for the angle of wave attack $\gamma_\beta = 1 - 0.0033 \cdot \lvert\beta\rvert \quad \text{for } \beta < 80$	[-]
γ_b	=	reduction factor to account for influence of a berm	[-]
γ_f	=	reduction factor for surface roughness	[-]

The first equation gives wave overtopping for breaking waves ($\gamma_b\xi_0 < \approx 2$), the second for non-breaking waves ($\gamma_b\xi_0 > \approx 2$). An advantage of the use of the spectral mean period T_{m-10} compared to the peak period T_p that was used earlier, is that the same formulas are also applicable for double-peaked wave spectra (WL | Delft Hydraulics, 1999).

The criterion to be used for wave overtopping depends on the condition of top layer of the inner slope of the dike. The Guideline specifies the following criteria for the mean overtopping discharge:

- 0.1 l/s/m for a sandy soil with an unsatisfactory grass cover;
- 1 l/s/m for a clayey soil with a reasonably good grass cover;
- 10 l/s/m for a clay layer and grass cover according to the standards of the outer slope or in case of a revetment cover.

D.5.1.2 Revetments

The Guideline Evaluation on Safety also provides procedures and formulas to evaluate the strength of the cover layer of dikes consisting of a stone revetment, asphalt layer, grass cover or concrete slabs. Rock armour is not treated as it is hardly used for sea defences in the Netherlands. This section describes as an example parameters and formulas used for the safety assessment of a placed block revetment.

Four inter-related types of failure are discerned:

- loss of blocks from the top layer (due to overpressure under the top layer);
- loss of core material due to malfunction of filter layers or geotextile;
- sliding of the top layer;
- erosion of filter or clay-under-layers after failure of the top layer.

The stability of elements in the top layer is carried out in two steps. In the first step the stability is assessed in terms of *good*, *questionable* and *insufficient* (*"goed"*, *"twijfelachtig"*, *"onvoldoende"*) using a graphical method for 6 different types of composition of the structure. Figure D.3 gives an example for pitched blocks on a geotextile on sand or clay.

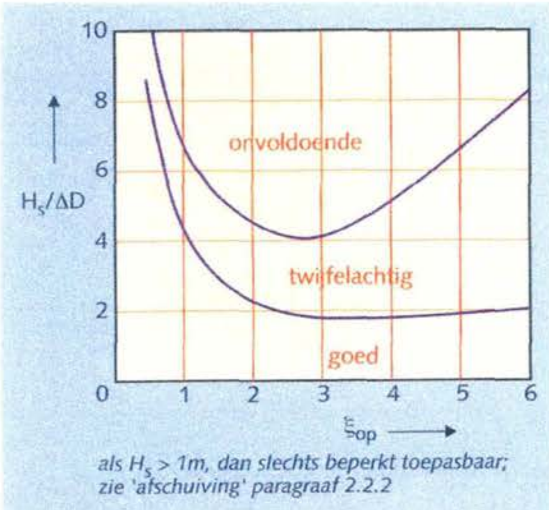


Figure D.3 Example of graphical method for stability evaluation for a block revetment of type **a**

If the result is *questionable* a more detailed analysis can be carried out for some types of revetments. The design tool ANAMOS2.1 was developed by WL | Delft Hydraulics for the TAW to support the design of pitched block revetments. It evaluates the three first failure mechanisms based on the characteristics of the structure as provided by the user of the program.

D.5.2 Probabilistic approach to safety assessment

As mentioned in the introduction, a new method for the probabilistic safety assessment of the sea defences is at present being developed. As this method is rather complex, it is implemented in the software tool Hydra-K. The procedure to evaluate the probability of failure is based on the "Method de Haan" (De Haan and Resnick, 1978). This method is based on the assumption that the correlation between two or more variables that has been observed during a certain measurement period, remains intact for extreme events (the range of events so extreme that they have not been observed yet). Keeping the correlation intact means in practice a translation along the diagonal of the standard exponential plane as illustrated in Figure D.4 below, where the black dots are observed values and the circles are hypothetic events obtained by scaling up the observed values.

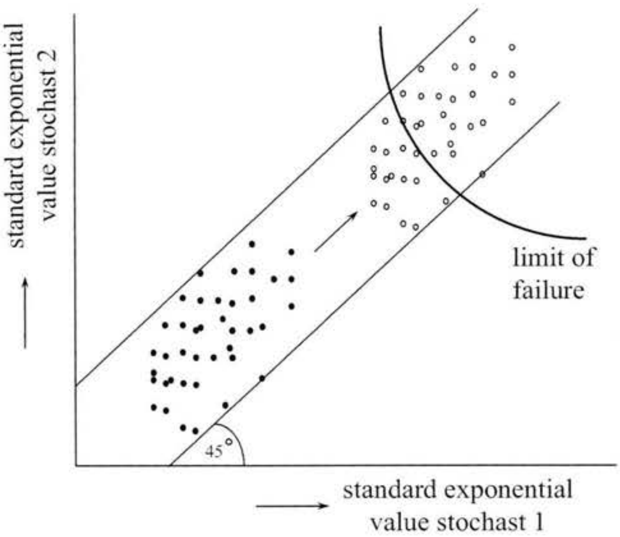


Figure D.4 Principle of the "Method de Haan" for scaling-up of observed events (dots) to events near the limit of failure (circles)

If the two stochastic parameters are e.g. the water level and the wave height in front of the sea dike, the strength of the dike against e.g. overtopping can be determined by evaluating whether the amount of overtopping for each of the scaled-up events is acceptable or not. The probability of failure can be determined from the number of events for which failure occurs, the length if the period of measurements and the factor used to scale up the events.

This procedure is being implemented for events that are based on the simultaneous data on water level, wind speed and wave conditions offshore.

D.6 Evaluation of sandy coasts

A large part of the sea defences of the Netherlands are dunes. These are evaluated according to the Guideline for Dune Erosion (TAW, 1984). The use of this guideline and the management and maintenance of sandy coasts are described in the Guideline for Sandy Coasts (TAW, 1995a) and the Technical Report Sandy Coasts (TAW, 1995b). The final version of the updated Guideline Sandy Coasts is in print (TAW, 2002b).

D.6.1 Strength parameters

The safety assessment of the sandy coasts is based on a standard cross-shore profile after dune erosion. This standard profile after dune erosion is defined by three sections (see Figure D.5):

- the dune front, this has a 1:1 slope starting at the toe of the dune (x=0, y=0) which supposed to coincide with the design water level ("rekenpeil")
- a parabolic profile seawards from the toe of the dune (x=0, y=0) given by

$$\left(\frac{7.6}{H_{0s}}\right)y = 0.4714\left[\left(\frac{7.6}{H_{0s}}\right)^{1.28}\cdot\left(\frac{w}{0,0268}\right)^{0.56}x+18\right]^{0.5} \tag{D.3}$$

up to the point where

$$x = 250\left(\frac{H_{0s}}{7.6}\right)^{1.28} \cdot \left(\frac{0.0268}{w}\right)^{0.56}$$
$$y = 5.717\left(\frac{H_{0s}}{7.6}\right) \approx 0.75H_{0s} \qquad \text{(D.4)}$$

- seaward from this point the standard profile has a straight 1:12.5 slope until the original seabed is reached.

The parameters in the above formula are defined by

H_{0s}	=	significant wave height at deep water	[m]
w	=	the fall velocity of the dune sand in sea water	[m/s]
x	=	the distance to the new toe of the dune	[m]
y	=	the depth below the water level	[s]

The calculation of the above profile is implemented in the DUROS model (DUne eROSion-model.

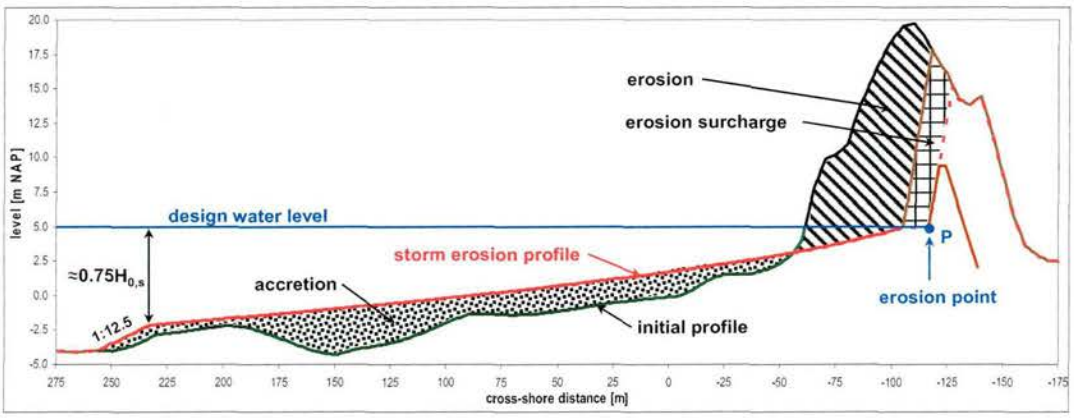


Figure D.5 Principles of the computation of the profile after dune erosion.

An additional amount of erosion is added to account for the uncertainty in storm duration and the inaccuracy of the model as shown in Figure D.6.

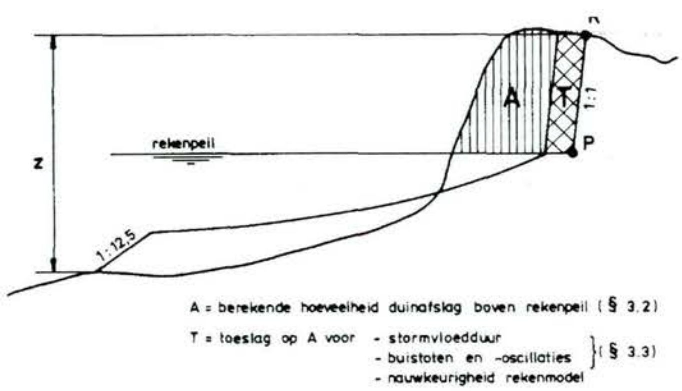


Figure D.6 Additional erosion T and location of the toe of the dune P.

In curved parts of the coast, the longshore transport during design conditions could lead to an additional erosion of the dune. The additional retreat of the dune front to which this leads, is determined by shifting the storm profile over a distance g in such a way that the additional amount of erosion G (in m^3/m, the shaded area in Figure D.7), is equal to

$$G = \frac{A^*}{300} \cdot \left(\frac{H_{0s}}{7.6}\right)^{0.72} \cdot \left(\frac{w}{0.0268}\right)^{0.56} \cdot G_0 \qquad \text{(D.5)}$$

in which A^* is the total amount of erosion above the water level ($A+T$ in Figure D.6) and G_0 is a reference value for G that depends on the curvature of the coast.

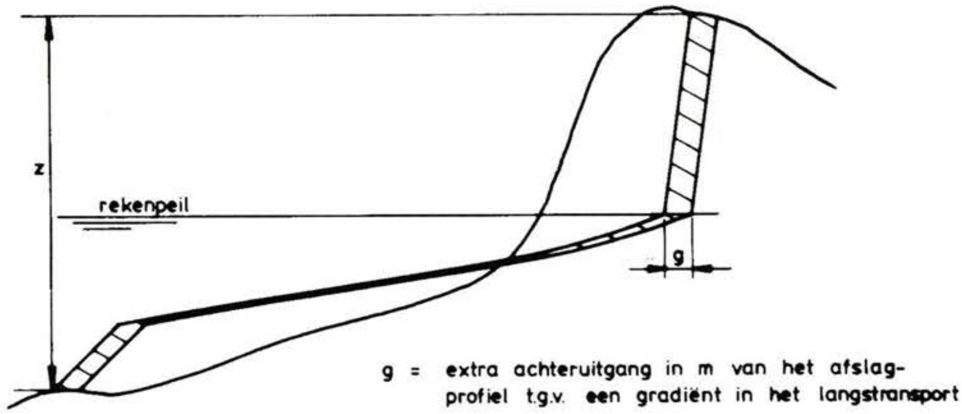


Figure D.7 Additional retreat of the erosion profile due a gradient in the longshore transport.

Possible trends such as a slow retreat of the shoreline are included in the safety assessment of sandy coasts by calculating the position of the erosion point P for the large number of yearly measured profiles in a certain section. The regression line following from this evaluation is first shifted inshore (to account for year-to-year profile variations and the effects of longshore transport) and then extrapolated to estimate the moment that the safety level is not met any more as shown in Figure D.8.

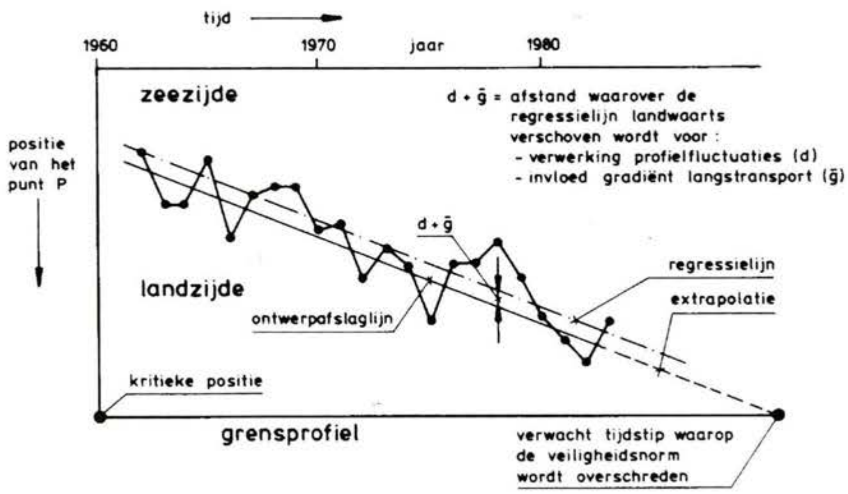


Figure D.8 Procedure to include the effect of trends in the safety assessment of sandy coasts.

Inputs to the dune erosion calculations are the still water level, the deep-water wave height and peak period and the fall velocity of the dune sand. It is remarkable, however, that the still water level used in the dune erosion calculations with DUROS is not the design water level for a certain return period. The water level *RP* (for *Rekenpeil*) used in the calculation is

$$RP = \text{design level} + 2/3 \text{ decimation height}$$

The decimation height is difference in water level between the design level and the water level with a return period that is a factor 10 longer. The significant wave height used in the calculations is the deep-water wave height corresponding to this water level. This increased water level is to account for differences in the risk of complete failure (and thus flooding of the hinterland) once the design water level is exceeded between sandy coasts and sea dikes (Den Heijer, personal communication).

D.6.2 Approach to safety assessment

The strength of sandy coasts is defined in terms of a minimum profile of the dunes that must remain after dune erosion. This minimum profile (*grensprofiel*) is shown in Figure D.9.

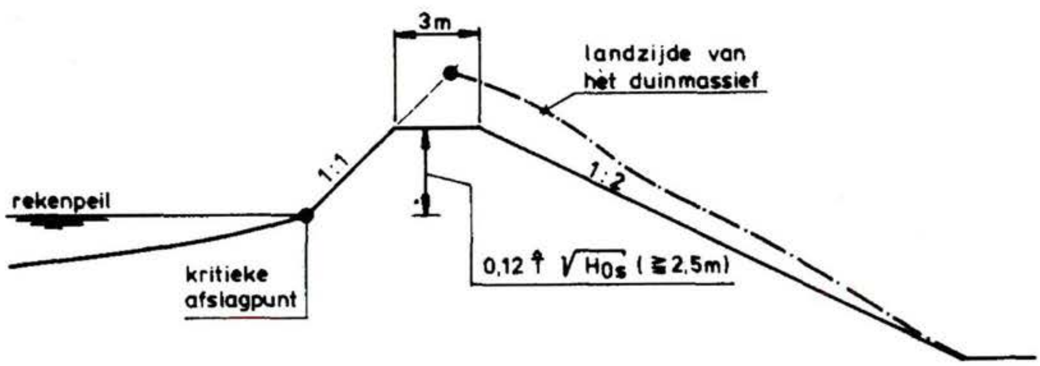


Figure D.9 Minimum profile for dunes required to remain after storm erosion (note: $\hat{T} = T_{0p}$).

The minimum crest level of this profile is computed using

$$h_0 = RP + 0.12\, T_{0p}\sqrt{H_{0s}} \text{ with a minimum of } h_0 = RP + 2.5m \qquad \text{(D.6)}$$

in which

H_{0s}	=	significant wave height at deep water	[m]
T_{0p}	=	peak wave period at deep water	[m]
w	=	the fall velocity of the dune sand in sea water	[m/s]
x	=	the distance to the new toe of the dune	[m]

In this safety assessment other factors such as the general trend of shore line retreat and the variation in time of the profile are taken into account.

References

Alkyon, 1999. **Wave computations in the Waddenzee.** Report A352, November 1999. Prepared for Rijkswaterstaat RIKZ.

De Haan and Resnick, 1978. **Limit theory for multivariate sample extremes.** *Z. Warscheinlichkeitstheorie, verw. Gebiete*, 40, p. 317-337.

DWW, 2001a. **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), April 2001, 27 pp + appendices.

DWW, 2001b. **Hydraulische Randvoorwaarden 2001, voor het toetsen van primaire waterkeringen.** 261 pp.

HKV, 2000. **Hydra-K; Functioneel Ontwerp.** Report PR313, HKV Lijn in water, december 2000. Prepared for Rijkswaterstaat RIKZ.

RIKZ, 1993a. **De basispeilen langs de Nederlandse kust; Statistisch onderzoek.** Report RIKZ-93.023 Parts 1 (text) and 2 (appendices), April 1993.

RIKZ, 1993b. **De basispeilen langs de Nederlandse kust; Fysisch onderzoek.** Report RIKZ-93.025, April 1993.

RIKZ, 1993c. **De basispeilen langs de Nederlandse kust; Eindverslag.** Report RIKZ-93.026, April 1993.

RIKZ, 1995a. **De basispeilen langs de Nederlandse kust; De ruimtelijke verdeling en overschrijdingslijnen.** Report RIKZ-95.008, May 1995.

RIKZ, 1995b. **Golfrandvoorwaarden langs de Nederlandse kust op relatief diep water.** Report RIKZ-95.024 (2 vol.: text + appendices), December 1995.

RIKZ, 1996. **Randvoorwaarden voor golfperioden langs de Nederlandse kust.** Report RIKZ-96.019 (2 vol.: text + appendices), July 1996.

RIKZ, 1999. **Basis HYDRA-K; Meerdimensionale extreme-waardenstatistiek van belastingen en faalkansberekening.** Report RIKZ-99.020 (2 vol.: text + appendices), May 1999.

RIKZ, 2000. **Richtingsafhankelijke extreme warden voor HW-standen, golfhoogte en golfperioden.** Report RIKZ / 2000.040, December 2000.

Ris, 1997. **Spectral modelling of wind waves in coastal areas.** PhD Thesis, Delft University of Technology, Delft University Press. 162 pp.

TAW, 1984. **Leidraad voor de beoordeling van de veiligheid van duinen als waterkering.** Technische Adviescommissie voor de Waterkeringen. May 1984.

TAW, 1995a. **Leidraad Zandige Kust.** Technische Adviescommissie voor de Waterkeringen. July 1995.

TAW, 1995. **Basisrapport Zandige Kust.** Technische Adviescommissie voor de Waterkeringen. July 1995.

TAW, 1999. **Leidraad Toetsen op Veiligheid.** Technische Adviescommissie voor de Waterkeringen. August 1999.

TAW, 2002a. **Technisch Rapport Golfoploop en Golfoverslag bij Dijken.** Technische Adviescommissie voor de Waterkeringen. May 2002b.

TAW, 2002b. **Leidraad Zandige Kust.** Technische Adviescommissie voor de Waterkeringen. December 2002 (being printed).

E Belgium

E.1 General

In Belgium the Flemish government is responsible for the design and maintenance of sea defence structures. The Region owns all the coastal defence structures. A part of the natural defences, some beaches above the high-water line and part of the sea-front dunes are owned by municipalities or private land-owners.
The responsible administration is AWK (Afdeling Waterwegen Kust; Coastal Waterways Division), which is part of AWZ (Afdeling Waterwegen en Zeewezen, Administration of Waterways and Maritime Affairs). The regionalisation act of 1988 says that all powers with regard to flood and coast defence are transferred to the region. The Flemish Region is fully responsible for flood and coastal defence. All coastal and flood defence measures are paid by the Region.

In Belgium (Flanders) there is currently no statutory level of coastal defence. The selection of designs is not based on cost/benefit analyses. In the recent past safety levels for beach nourishments were calculated using two successive storms with return periods of 100 years. This is approximately equivalent to at least a 1000-year safety level. At the moment a minimum required safety level of a 1000 years is prescribed according to the Dutch methodology. However, a comprehensive study has been started with regard to safety levels and coastal protection in general along the Flemish coast. One of the aims of this study is to determine statutory levels of protection.

The design is carried out by consultants. Up to now no periodic safety assessment of the sea defences is carried out. At this moment a safety assessment is carried out. In two other projects a risk assessment is done. These 3 projects together will point out which further actions are necessary.

Up to now, for both the design and the safety assessment the Dutch methodology, with adaptations where necessary is followed. In the design the sea level rise over the next 50 years is incorporated.

The method to provide hydraulic boundary conditions is developed by IMDC. The statistics at deep water and at the nearshore are also carried out by IMDC. Flanders Hydraulics prepared the SWAN calculations to transform waves from deep water to the nearshore.

E.2 Basic data

The basic data are:

- water levels at three harbours (Zeebrugge, Oostende and Nieuwpoort);
- wave heights measured at deep water and nearshore. The latter are only used for the calibration of the SWAN-model. These nearshore data consist of shorter time series;
- wind measurements at on- and offshore locations.

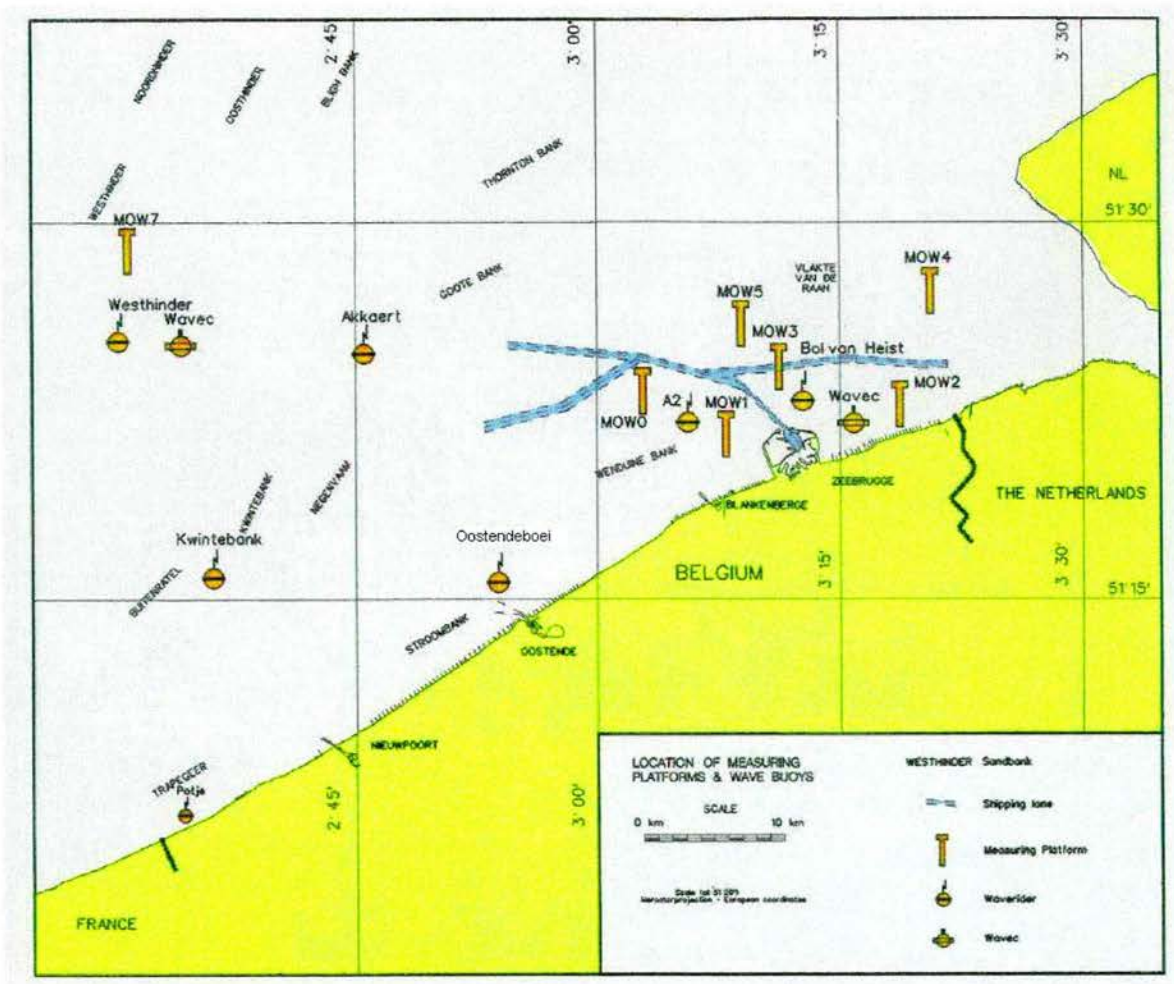


Figure E.1 Overview of available data (obtained from Flemish Community, AWK)

To obtain output at each location along the coast, interpolation and models are used.

E.2.1 Water level

Registrations of High Water levels are available since 1929 in Ostend. Since 20 years registrations with an interval of 10 minutes are available. Other water level registrations exist in Zeebrugge and Nieuwpoort.

For the water levels it is assumed that the storm surge does not depend on the location. Differences in water levels occur due to differences in tidal elevation at spring tide. This method is also verified with computations of the water level during a storm with return period 100 year.

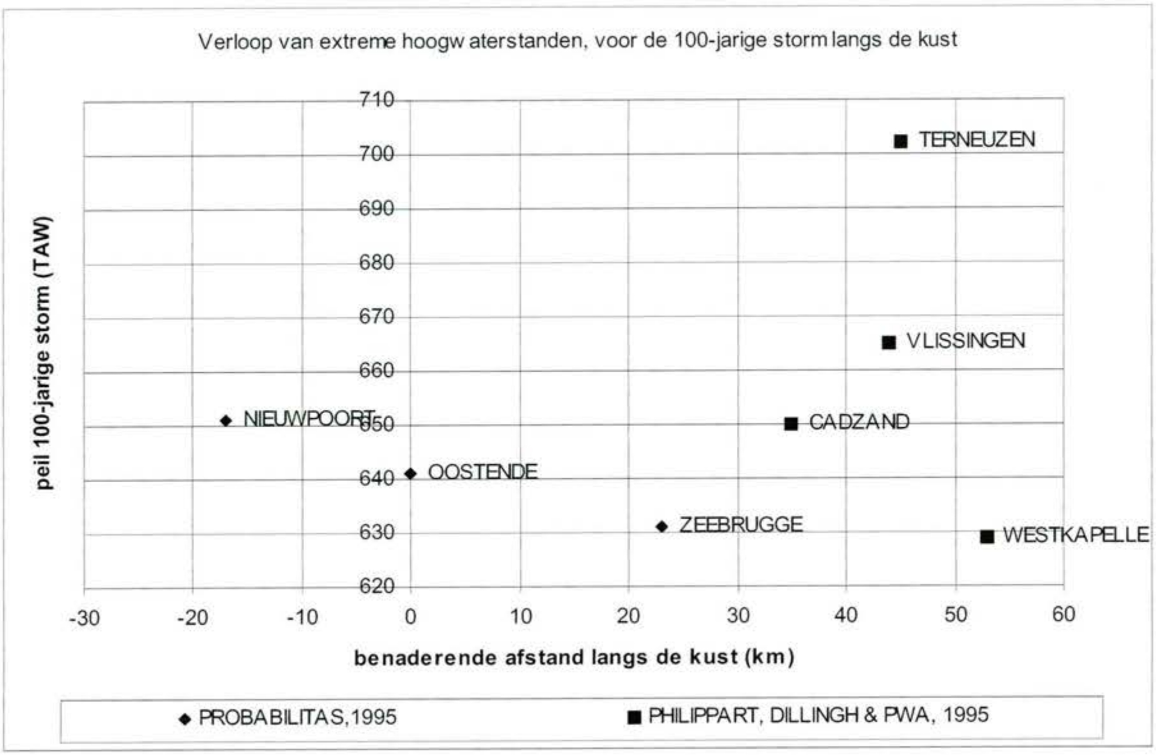


Figure E.2 Course of extreme water levels for 100-year storm condition along the coast.

E.2.2 Waves

Since about 25 years wave data are measured at deep water (at a distance of about 25 – 30 km from the coast and at a water depth of about 30 m) at Westhinder and Akkaert. These data are measured with non-directional Waveriders. Since 10 years a directional wave buoy and a wave gauge (MOW7) are also measuring at Westhinder. A detailed analysis of these data made clear that it is reasonable to assume that wind and wave direction are the same at deep water. For the last 5 years also non-directional (Waverider) wave data near Ostend are available.

For waves it was observed that the wave height did not differ much at deep water (e.g. comparison between Westhinder and Akkaert). Therefore it can be concluded that it is sufficiently accurate to apply only one (constant) wave condition at the offshore boundary of the computational domain in SWAN.

E.2.3 Wind

Wind data are available for about the same period as the wave measurements. These data are measured at 2 offshore locations and 1 location on land (Zeebrugge).

E.3 Data processing

E.3.1 Measurements

All analysis of the different data is done for the full year.

E.3.1.1 Wave heights

For the time series of the wave height a relation was sought between registrations made by the different wave buoys at deep water. This relation allows to fill registration gaps in the wave buoy data.

As basic wave buoy the Waverider at Westhinder has been chosen, since this buoy does measurements at the deepest water depth and since its recording length is the longest of all.

Relations are derived between the POT values in the wave buoy data. Since the statistical analysis will be a Peak Over Threshold analysis, only a relation between the peaks in wave heights during a storm are important. Time lags between the two location registrations are not important and would scatter the relation.

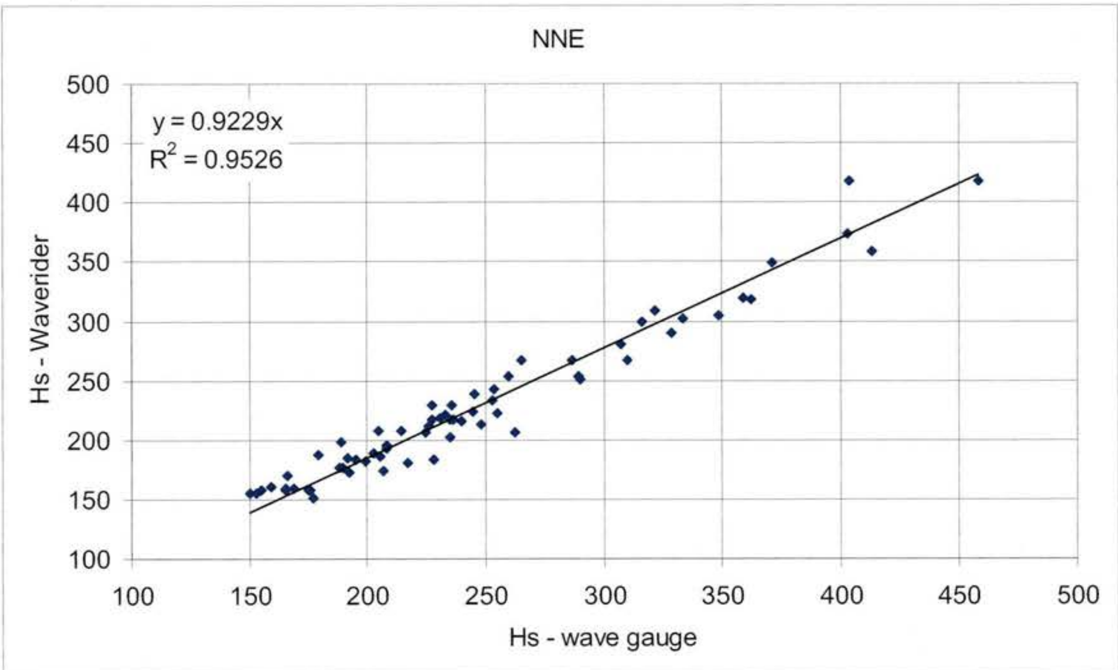


Figure E.3 Relation between wave height at the wave gauge and the wave rider.

Figure E.3 shows the relation between the wave height at the wave gauge and the Waverider for waves from NNE direction. The values for the wave gauges are 8% higher compared to these of the Waverider. Two causes are possible a) the wave gauge is located at the crest of a (large) sand bank (Westhinder bank) which might give some shoaling effects b) wave gauges give usually higher wave heights during storms compared to wave buoys.
The relation between the wave heights for the different equipment seemed to be fairly independent from wave directions (variations of about 1%), except of course for winds coming from land (which are not important for this study).

For the basic location, a relation between the significant wave height of the time series (H_s) (average of the 33% highest individual waves) and the significant wave height obtained via the wave spectrum (H_{m0}) was examined. It was found that H_{m0}=1.055 H_s (with a standard deviation of 1%) where in literature a ratio of 1.06 is found.

With these relations, the time series of the basic location could be filled up with data from other locations. Remaining gaps where examined individually. One should not worry about such a gap if a) the corresponding wind speed is low b) the wind is coming from land c) the wave height is clearly building up (or decreasing) (since in that case the POT value is not missed. After this detailed examination, 3 gaps were still open. This is relatively small compared with the 200 POT values, which were selected from the time series.

E.3.1.2 Wave period

Two relations between wave height and wave period are found in literature : $T_p=aH_{m0}^{0.5}$ and $T_p=aH_{m0}^{b}$. Both relations were fitted (Figure E.4) and no differences are found (b±0.5). The standard deviation was about 7%.

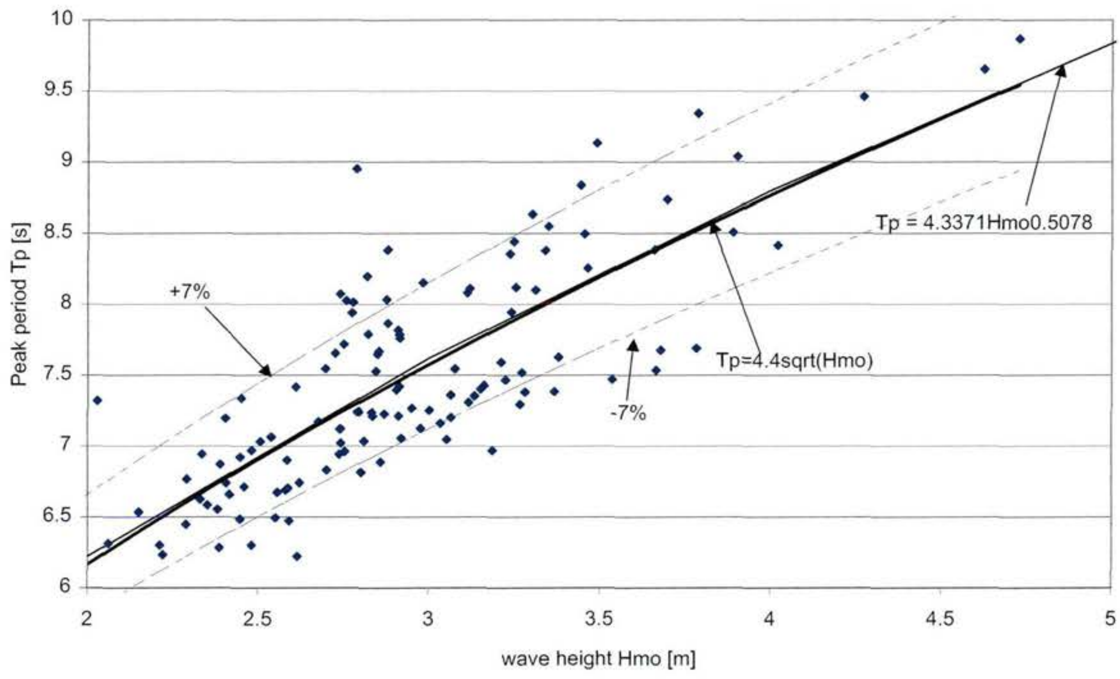


Figure E.4 Relation between wave height at the wave gauge and the peak period.

E.3.1.3 Wind time series

For the wind velocity a similar analysis as for the wave height has been done: different stations are combined to get a complete time series at one location. However, for the wind time series the number of gaps that had to be filled up was much lower.

E.3.1.4 Storm surge

The tidal height at the Flemish coast is important, with a difference between neap tide and spring tide (tidal heights of resp. 3.77 m and 4.69 m, a difference which is comparable with a storm surge occurring once every 8 years). Accordingly, the same measured (high) water level can occur both due to a spring tide combined with a moderate storm or due to a neap tide combined with an exceptional storm. So it was decided to perform an extreme analysis on the storm surge and than combine these results with the (known) astronomical tide to get an extreme water level distribution.

The astronomical tide was obtained with an harmonical analysis of the past 20 years for which the complete time series were available. The analysis resulted in 94 tidal components with their resp amplitude and phase. These components were used to calculate the astronomical tide during a period of 75 years for which high water levels are available, of course accounting for sea level rise in this period (about 0.015 m/year) (variation of the mean water level component in time).

Once the astronomical tide is known, the storm surge (here defined as the difference between maximum water level and maximum astronomical water level) can also be determined. These do not necessarily occur at the same moment, since the storm surge will delay the time of the maximum water level.

E.3.2 Extreme value analysis

The method presented here is based on previous extreme value statistics (IMDC 2001a; IMDC, 2001b) and theoretical work of Beirlant *et al.* (1996).

For this method no theoretical distribution is assumed. Three typical distributions are examined each time, distinguished by the extreme value index γ. An increasing γ stands for a higher frequency of occurrence for extreme values.

- $\gamma=0$ are the exponential, lognormal and Weibull distributions,
- $\gamma>0$ are the Pareto distributions, and
- $\gamma<0$ (beta-distributions) occur rather seldom, except for depth limited waves.

The statistical analysis consists of :

- Derivation of the POT-values
- Generation of Quantile-Quantile-plots (QQ)
- Derivation of the γ-index and optimal threshold
- Derivation of the parameters of the chosen distribution
- Derivation of the return periods

Derivation of POT values

Peak over Threshold values are independent peak values in the time series of wave height, wind speed and storm surge. A value is selected if resp. the storm surge is higher than 40 cm, the wave height is higher than 250 cm and the wind speed is more then 10 m/s (values which occur on average 5 to 10 times in a year). The minimum time interval between two POT's has to be larger than 24 hours and between two POT values, the minimum measured value had to be smaller than 50% of the smallest POT-value. These criteria assure independency of the POT-values.

QQ plots

A QQ-plot (Quantile-Quantile plot) is a plot where empirical quantile functions are represented against the theoretical quantile functions.

Extreme value index γ and optimal threshold

The estimation of γ is based on the different QQ-plots and the alternative QQ plot (Beirlant *et al.*, 1996) (UH-plot). The gradient of the UH plot converges to γ. The gradient is obtained with a linear regression. Consequently the highest POT-values have the biggest influence, since the highest values have the highest uncertainty and thus the largest deviation of the regression. For this reason, weight factors are introduced. The optimal threshold is obtained if the mean square error in the linear regression is minimal.

Parameters of the chosen distribution

Once the distribution is chosen, the parameters of this distribution can be examined, including a refinement of γ. The examination is based on the QQ-plot for the corresponding distribution.

Return periods

In the last step the return period T_X for a peak value X can be obtained.

In coastal engineering applications often Weibull distributions are used. Two kind of Weibull distributions exists: a 2 parameter and a 3 parameter distribution:

$$P(H > h | h > h_{\ddot{a}}) = \exp(-\frac{h^k - h_0^k}{u^k}) \tag{E.1}$$

and

$$P(H > h | h > h_{\ddot{a}}) = \exp(-(\frac{h - loc}{u})^k) \tag{E.2}$$

with h the extreme value, h_0 the threshold u, k and loc the Weibull parameters.

In this study the 3 parameter distribution is used, following the conclusions of the Working Group on Extreme Wave Statistics ordered by the International Association for Hydraulic Research (PIANC, 1992). The conclusions are also used by Van Vledder *et al* (1994) , who proposed that the 2 parameter distribution should be used for marginal distributions and the 3 parameter distribution for POT-analysis.

E.3.3 Uncertainties on the distributions

Three kind of statistical uncertainties exists:

- The uncertainty on the sample: the analysis is done on a time series (a sample) of 25 years (or 75 for storm surge). Another 'sample' in time (eg. the period 1950-1975) would give a (small) difference in the extreme value distribution. This is the most important source of uncertainty. The uncertainty is estimated by sampling the used complete sample, and doing a analysis on (reduced) samples, from which the variation in distribution is estimated. 3 techniques can be used : Jacknife (used in this study), Monte Carlo (tested in this study) and Bootstrap (not used). In this study it appeared that the two first methods give the same results.

- The uncertainty caused by the choice of the type of distribution. This uncertainty is minimised by comparing all kind of distributions
- Uncertainty on the parameters of the distribution. This uncertainty is obtained with the Maximum Likelihood method. This method minimises the shape and scale factor and it is generally assumed that the resisting uncertainty is small compared to the first uncertainty.

Also the uncertainty on the measured values (measuring error) will influence the total uncertainty. The uncertainty on the data is estimated at 5 to 10 % of the measured value. This value was generally considerably smaller than the uncertainty on the extreme value analysis.

E.3.4 Results

Extreme value distributions are obtained both for the wave height, wind speed and storm surge for different (wind) directions. The distributions give the probability that a value is exceeded, knowing that the value is a POT (*i.e.* the threshold is exceeded and the value is a peak value). The return period of this value is the reciprocal of the probability multiplied by the averaged number of POT values in 1 year.

E.3.4.1 Extreme wind speed

The wind speed is the energy source for waves travelling from deep water to Ostend. For this reason it is important to select during a storm a POT value for each occurring wind direction. It is possible that a storm reaches its maximum wind speed seldom for a specific direction, although the wind speed reaches very high values at that direction, while the wave height reaches its maximum for that specific direction. This would lead to a combination of a high wave height and a small wind speed. Transforming this combination to Ostend would give a small wave height, since energy dissipation by bottom friction would not be compensated with energy input from wind. In reality, the wind speed would be high for that direction (although not a maximum during the storm) and thus the wave height in Ostend would be underestimated. By selecting a POT value for each direction that occurred, this problem is avoided (but the wind speed is probably overestimated, resulting in slightly conservative wave heights in Ostend).

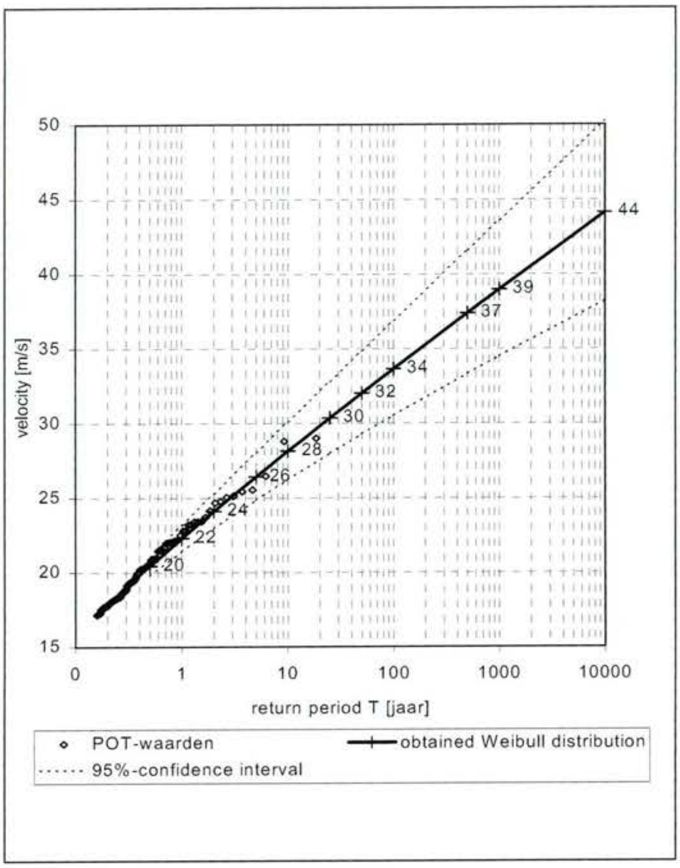


Figure E.5 Extreme wind speed distribution for direction West (and duration of 15 minutes)

Figure E.5 gives an example of an obtained distribution for a particular direction together with the POT-values, while
Figure E.6 gives the distribution for all directions (without POT values).

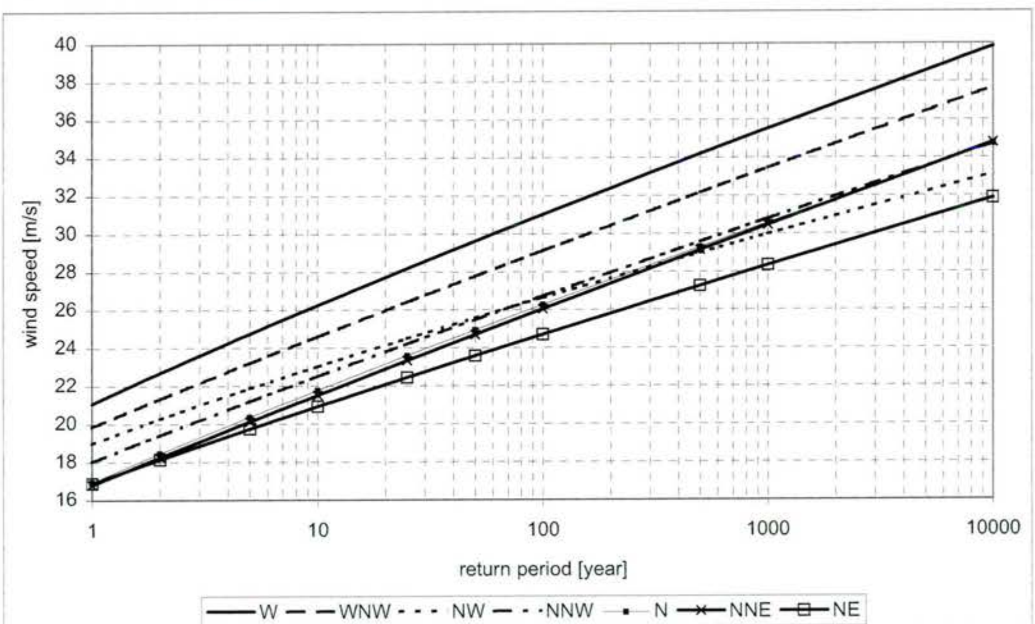


Figure E.6 Wind speed distributions (2h averages) for different directions

E.3.4.2 Waves

For the wave height, which is the basic variable, only 1 POT value per storm is selected instead of a POT-value for each direction as is done for the wind speed. This prevents that storms are counted twice, trice…which would lead to an overestimation of the occurrence of severe storms. (while selecting more POT values for wind speed can only lead to an overestimation of an energy source (wind). After selection the corresponding direction can be obtained from the data series. As an example the wave height distributions for a time interval of 2 hours is shown (Figure E.7)

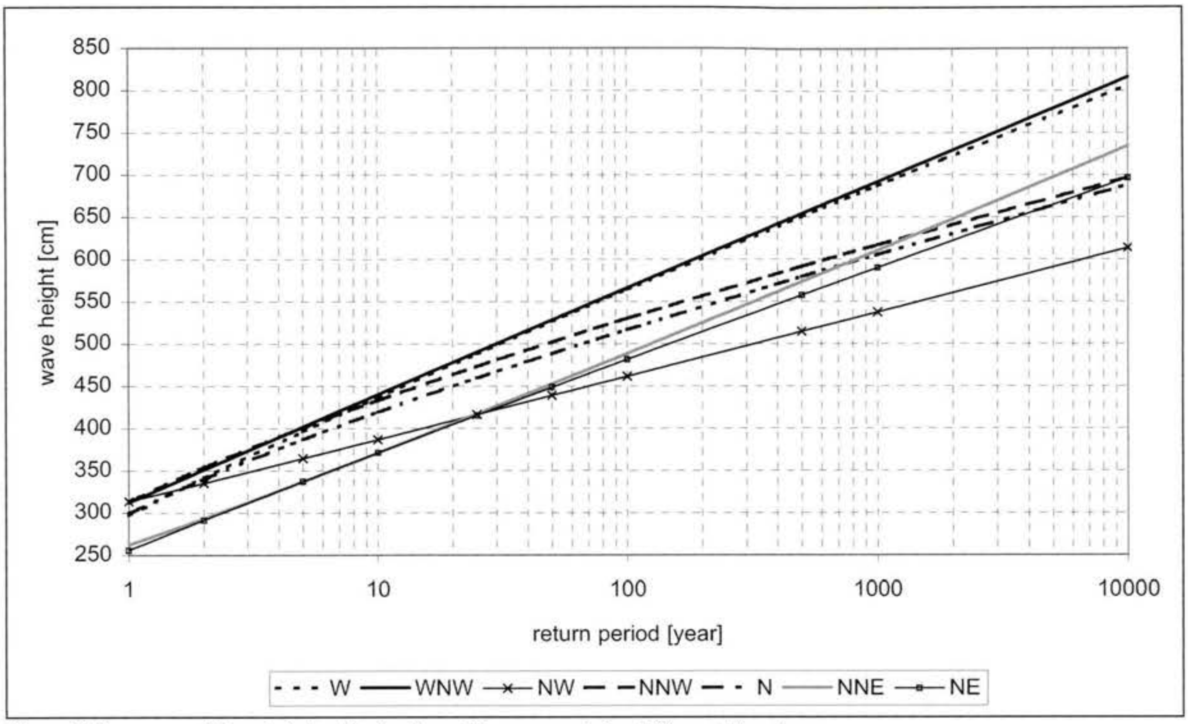


Figure E.7 Wave height distributions (2h averages) for different directions

E.3.4.3 Storm surge

Storm surge is a process with a relatively large time scale compared to wind and waves. Consequently, it is rather risky to link 1 (wind) direction to the storm surge. It is necessary to classify the water levels/storm surges per (wind) direction since for each wave height for each direction, the corresponding water level has to be known (both for the wave model as for the design). The storm surge is the value (with corresponding direction) obtained at high water, which is 1 moment in the storm, while the storm might last for 12 hours. If the (wind) direction classes are small (eg 22.5 degrees as for wind and waves), the attribution is rather random.

This can be shown for the storm of 1953 (largest storm during the past 125 years, with an estimated return period of 700 years (based on Figure E.9). At the time of maximum high water, the (wind) direction was 320 degrees to the North, while 1 hour before, the direction was 290 degrees. It would be risky to neglect this storm in the analysis of the direction WNW (290 degrees), since it is very well possible that high water occurred for this wind direction. It was decided to analyse the storm surges in direction intervals of 3 x 22.5 degrees. Eg for NNW the directions NW, NNW and N were considered, for N the directions

NNW, N, NNE, … Since all classes contain in that case 3 times more storms as necessarily, the probability of occurrence is divided by 3 afterwards.

With the extreme value distribution of the storm surge, the extreme value distribution of the water levels can be obtained as follows:

$$p(h > h_1) = \int p(h_a).f(s > (h_1 - h_a))dh_a \tag{E.3}$$

with

h_1	=	the water level for which the probability is needed	[m]
h_a	=	the astronomical high water level	[m]
s	=	the storm surge	[m]
$p(h_a)$	=	the probability density of h_a	[-]
$f(s)$	=	the probability distribution of the storm surge	[-]

In practice this results in:

$$p(h > h_1) = \sum_{i=1}^{N} f(s > (h_1 - h_{ai}))/N \tag{E.4}$$

with:

N	=	the number of high waters in 18.6 year (nodal period, after which the astronomical tide repeats itself more or less)	[-]
h_{ai}	=	the height of the i^{th} astronomical high water level	[m]

For a number of water levels the probability of occurrence is determined, and intermediate values are obtained with interpolation.

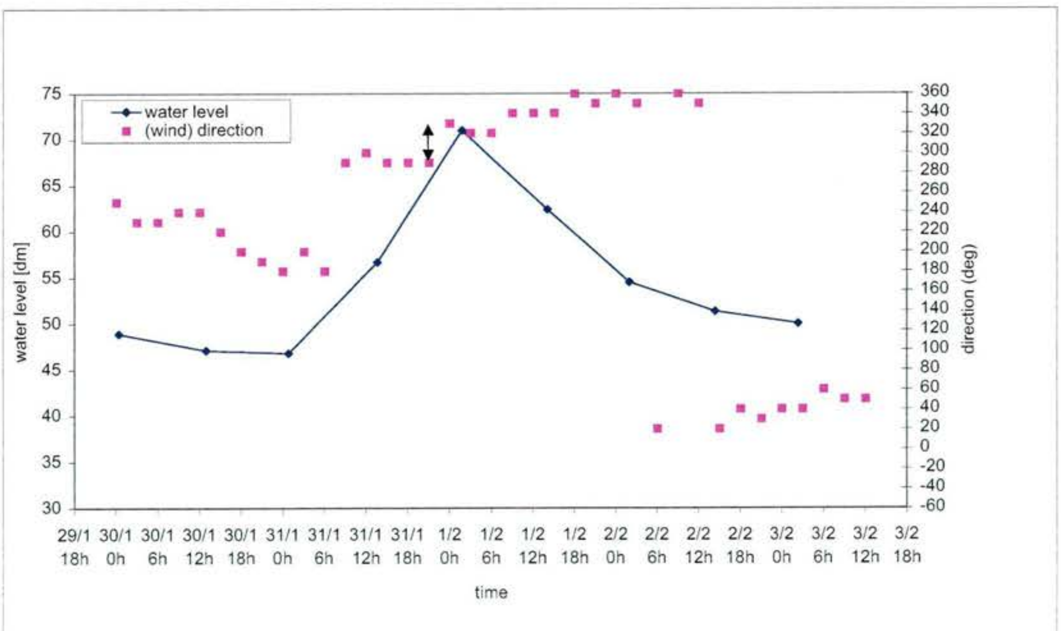


Figure E.8 Storm of 1953

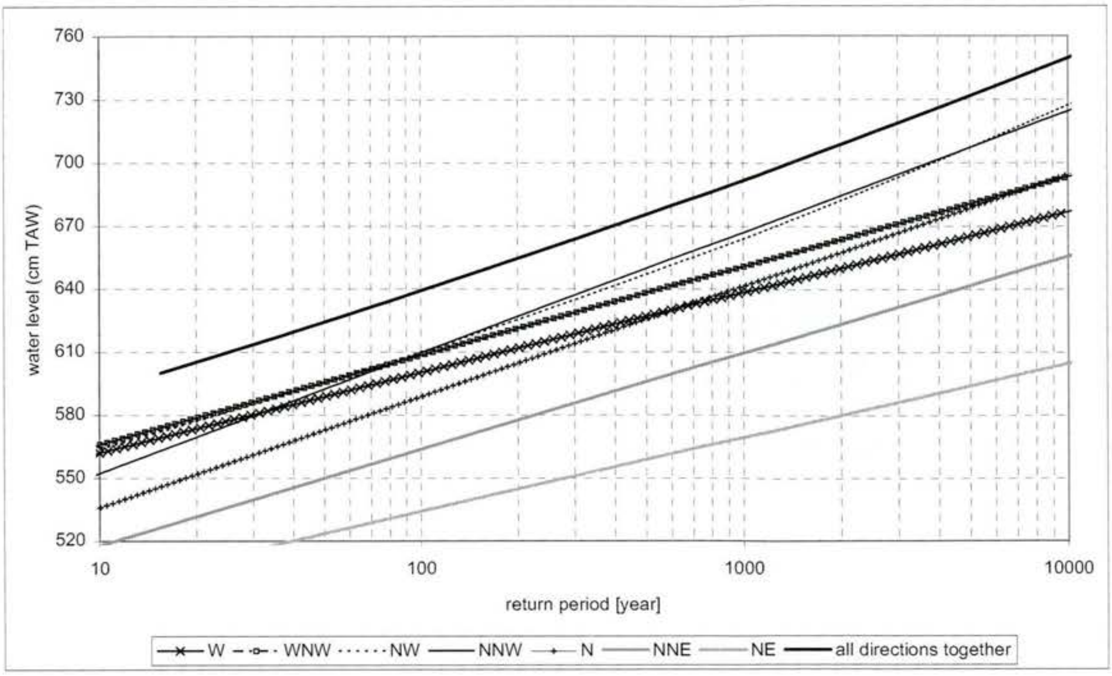


Figure E.9 Extreme water level distributions for different directions

Note on the correlation between parameters. It is assumed that, for each direction separately, that wind speed, wave height and water level are perfectly correlated. It was found that instead of water level, storm surge was a better parameter to correlate wave height, but this would make the analysis of extreme values at the nearshore much more complicated.

The confidence interval is taken into account for the design where necessary (e.g. design of breakwaters). E.g. for overflow, the Dutch guidelines do not take into account uncertainty.

E.4 Nearshore conditions

First, the offshore wave conditions are transformed to a nearshore point with a water depth of about 12 – 15 m (during storms). In the second step resp. DUROS and DUROSTA are used for dunes and dikes.

E.4.1 Transformation to the nearshore

E.4.1.1 Method

For each direction wave heights are classified with intervals of 0.25 m. Each of those classes have a known probability of exceedance (p). The corresponding water level (h) and wind speed (w) are calculated using the same p in the probability of exceedance functions of h and w. The obtained parameter combinations (Hs, w, h, θ, p) in deep water conditions are transformed to Ostend by means of the numerical wave model SWAN (Ris, 1997).

All this led up to new nearshore parameter combinations (Hs', w, h, θ', p), whereas the wind speed, the water level and the probability of occurrence do not change. Those nearshore combinations are again subdivided in directions (with θ'= N, NNW,...), sorting the wave heights. Finally the probability of exceedance is estimated for each wave height and for each direction as the summation of p's give the probability of exceedance.

E.4.1.2 Wave transformation

The SWAN model has been validated based on a 5 years time series of wave measurments in front of Ostend (nearshore). This time series has been simulated by using transformation tables (developed by Alkyon). Each basic parameter (wave height, peak period, direction, water level and wind speed) is divided in classes with a representative value (Hm01, Hm02, ...). All possible combinations of these parameters (4 wave heights, 3 wave steepnesses (representing the peak period), 7 directions, 4 water levels and 4 wind speeds) (=1344 combinations) are simulated resulting in a table, which gives the link between deep water conditions in the open sea and local conditions in Ostend. Each point in the deep water time series can be linked with a corresponding condition in Ostend by interpolating in this table.

After a detailed analysis of some storms, the basic parameters for SWAN were obtained and with these parameters, the simulations are done to get the table. Then the time series were transformed with the table and the results in Ostend were compared with the measurements.

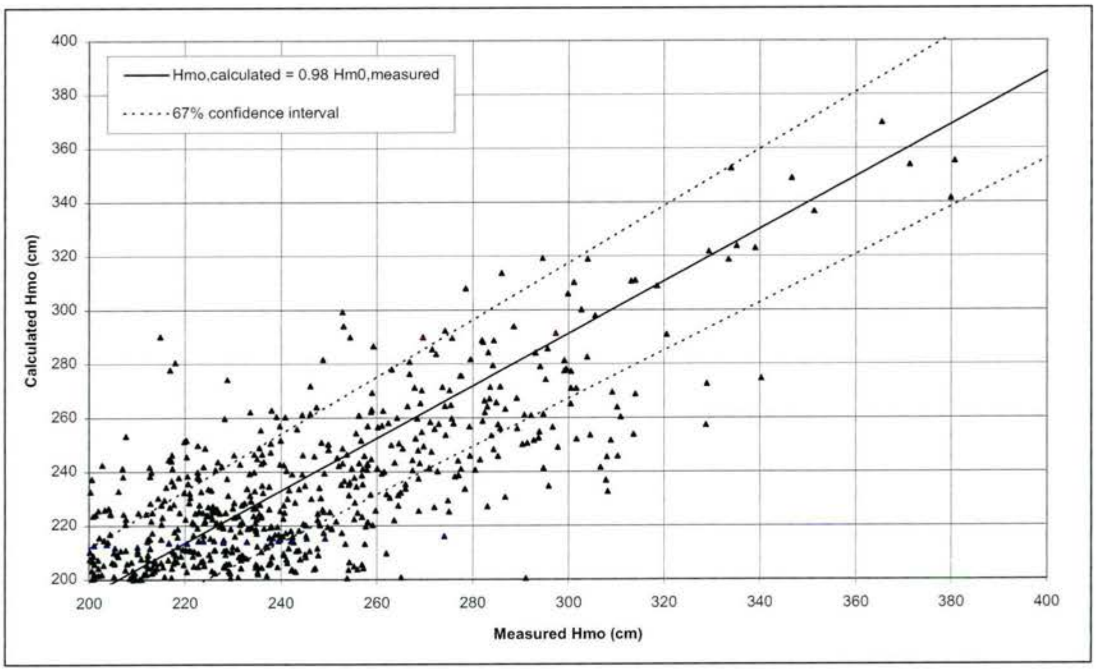


Figure E.10 Comparison between calculated and measured wave height in Ostend

Figure E.10 compares measured and calculated wave heights (only wave heights higher than 2 m were selected to be representative for storm conditions. From this exercise it can be concluded that the model underestimates the wave height with 2%, this underestimation is compensated to get the final values). The standard deviation is about 9%. One reason for these deviations is the Swan model that runs steady state. This introduces some errors due to time lags. Comparing 2h averaged measurements and calculated values reduces the standard deviation to 6.5%. However, it was decided to maintain the standard deviation at 10%, a value recommended by PIANC (1992).

For the wave period, the same analysis is done, giving an underestimation of 3% and a standard deviation of 10%.

E.4.2 Statistical analysis at the nearshore

All combinations X(Hs, w, h, θ, p) (on the extreme value distributions) in deep water conditions are now transformed to Ostend resulting in new nearshore parameter combinations X'(Hs', w, h, θ', p), whereas the wind speed, the water level and the probability of exceedance do not change. The use of the probability p is actually replaced by a better method since a POT analysis is used.

Consider a fictive period D (eg 1000 years). The number of times that a combination occurs during D is given by:

$$\left(p(X)-p(X+dX)\right)\cdot N\cdot D \tag{E.5}$$

with:

N	=	the averaged number of POT values per year	[-]
$N.D$	=	the number of POT values in the period D	[-]

Consequently, this is also the number of times that the combination X' occurs in the considered location.

New classes are defined in each relevant location (different directions, different wave height classes, ...). One can count the number of times that conditions belong to a certain class. With these numbers again exceedance curves/return periods can be derived. This analysis is done for all necessary locations.

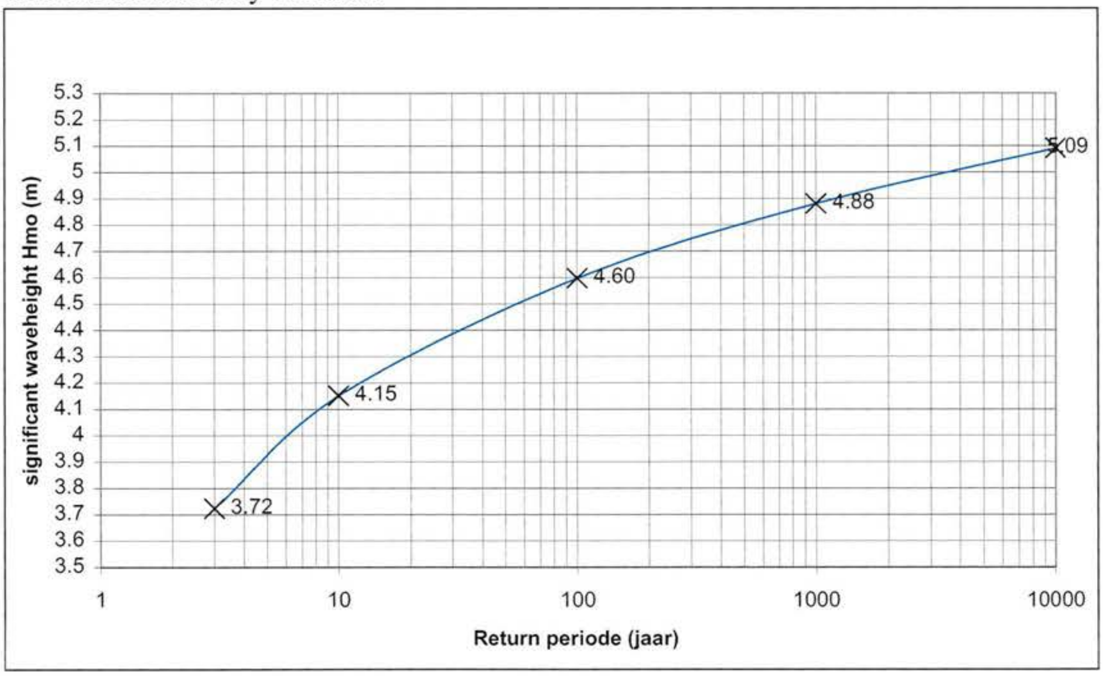


Figure E.11 Wave height distribution for a point at the groin, considering all waves with a direction in the range perpendicular +- 80 degrees on the breakwater

Figure E.11 gives an example. For a point the wave heights and corresponding water levels and peak periods are calculated for all relevant directions. The table should be interpreted as (e.g. 'once in 1000 years waves are coming from NW AND exceed the height of 4.94 m'. This wave condition in Ostend can be caused by a storm with wind direction WNW (in which case the corresponding water level is 6.6 m and the corresponding wave period 12.4s) or by a storm with direction NW (and water level of 6.75, wave period of 11.8 s).

Direction	Significant wave height H_{mo} (m)	Water level (m)	Peak period (s)	Wave direction at deep water
WNW	4.83	6.4	12.2	W
NW	4.94	6.6/6.75	12.4/11.8	WNW/NW
NNW	4.94	6.7	11.8/12.2	NNW/N
N	4.45	6.1	11.6	NNE
NNE	3.38	5.55	11.1	NE
All directions	5.03			

The standard deviation on the results is the combination of the standard deviation due to the transformation of the wave climate and the (transformed) standard deviation on the different extreme value distributions (both for water level, wave height and wave period). The two most important contributions are from the standard deviation of the water level and from the transformation.

Dunes
For dunes, DUROS assumes input from deep water. However, at the Flemish coast sand banks have an important influence on the nearshore wave height. It was decided to use the wave height just in front of the beach/dune (at water depth of 12 – 15 m), corrected for the shoaling coefficient between deep water and the output location. So, a kind of fictive deep water wave height is used.

Dikes
In front of dikes the beach erodes during the storm which leads to a higher water depth at the toe of the dike, and thus also to higher waves. For this reason, DUROSTA is used to calculate the erosion profile. At the moment, we are evaluating if SWAN/DUROSTA can be used for the transformation of the wave height to the very shallow beach in front of the dike.

E.5 Design / safety assessment sea dikes

For overtopping/ run up and revetment the Dutch guidelines are followed. In Flanders dikes occur in front of towns and the dike is completely covered with material (stones, asphalt, ...). The maximum overtopping discharge is equal to 1l/s/m, and is based on the damage on buildings, rather than on slope stability (since for the coverages 1 l/s/m is still save).

The design is carried out for a return period of 1000 years.

In Ostend the harbour groin is designed using a Monte Carlo Simulation.

E.6 Evaluation of sandy coasts

The strength of the dunes is evaluated with the Dutch methodology (DUROS). DUROS is semi-probabilistic. In Flanders, buildings are constructed in the dunes. The situation is judged to be save, if no buildings occur in the zone of dune erosion and in the zone of the minimum profile (*grensprofiel*). Some overtopping calculations will be done to evaluate if the overtopping discharge is smaller than 1 l/s/m.

The year-to-year variations in the cross-shore profile are taken into account during the evaluation: if a profile is safe, but without much margin and the profile has an erosive trend, it will be evaluated as 'dangereous'. The cross-shore profiles are measured yearly.

References

Beirlant, J., J.L. Teugels and P. Vynckier, 1996. **Practical Analysis of Extreme Values**. Leuven University Press.

DWW, 2001. **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), April 2001, 27 pp + appendices.

IMDC, 2001a. **Opmaak van hydrologische modellen in het IJzerbekken en opstellen van debiet-duur-frequentie (QDF) relaties en composiethydrogrammen,** Deelrapport I : inventarisatie en statistische analyse van de beschikbare tijdreeksen, I/RA/11206/01.040/JBL

IMDC, 2001b. **Leveren van numerieke hydrologische en hydraulische modelleringssoftware, met de implementatie van het Demerbekken** Deelrapport II : statistische frequentie-analyse I/RA/11193/01.037/JBL

PIANC, 1992. **Analysis of rubble mound breakwaters, sub group B, Uncertainty related to environmental data and estimated extreme events**, by H.F. Burcharth, report of PIANC working group no. 12 of Permanent Technical Committee II

Ris, 1997. **Spectral modelling of wind waves in coastal areas.** PhD Thesis, Delft University of Technology, Delft University Press. 162 pp.

Van Vledder, G.Ph., Y. Goda, P. Hawkes, E. Mansard, M.J. Martin, M. Mathiesen, E. Peltier, and E. Thompson, 1994. **Case studies of extreme wave analysis: A comparative analysis**, Proc. WAVES'93, New Orleans

F United Kingdom

F.1 General

In the United Kingdom, the flood defences at the coast and in tidal areas are generally the responsibility of the Environment Agency, although some local authorities have responsibility for certain defence lengths for historic reasons (they may have installed the defences originally). Policy for flood defence is set by the UK Governments Department for Environment, Fisheries and Rural Affairs (Defra). The Environment Agency works within the policy framework and guidance produced by Defra to ensure sea defences are technically, economically and environmentally sound and sustainable.

The Environment Agency and local authorities are also responsible for maintaining defences. Most of these are 'owned' by the Agency, and the Agency also has an overall 'supervisory duty' over privately owned defences, for example those owned by the railways, the Ministry of Defence, the National Trust and private individuals.

Most design is carried out by consultants. On 10th October 2000 the Agency awarded the five year National Framework Agreement for Engineering & Environmental Consultancy Services (NEECA) for provision of engineering and environmental consultancy services to four consultants. The work may cover any aspect of the project cycle from project identification, option appraisal and feasibility studies through detailed design and contract documentation to contract supervision and completion of projects including post project appraisals. Non-project work may also be requested, for example, advising on processes and systems and added value work.

Operation Public Safety (OPUS) has been instigated to determine the safety of all defence structures owned or operated by the Agency. This has enabled a classification of health & safety risk, and is linked to a major capital programme of improvements from deployment of signage to repairs and replacement works. There is no target risk or flood defence standard. Rather, the national policy is defined in terms of the general aim of reducing risks to people and the natural environment, and the requirement to achieve value for money.

Structural inspections are carried out at intervals depending on the risk. These provide a 'condition grade', which is stored in a central database alongside other information about the flood defence system (the National Flood and Coastal Defence Database (NFCDD). The Flood Defence Management Manual and System (FDMM / FDMS) is used by the Agency to prioritise and justify maintenance work - actual 'standard of protection' (SoP) is compared with a target (depending on land use) to establish need for works. For major schemes and strategies (e.g. Thames or Humber), more detailed analysis may be carried out, including assessment of structure reliability, and associated risk.

Periodic review of the flood defence 'policy' for different lengths of the coast is carried out within the Shoreline Management Plan programme (typically reviewed every 5 years). A countrywide assessment of risk has been carried out using the RASP (Risk Assessment for Strategic Planning) approach.

The approach to design and periodic safety assessment of the flood defence is not necessarily the same; the level of data and degree of detail varies depending on the size of scheme, and the risk. In the 'Source - Pathway - Receptor' model of environmental risk adopted in the UK Government and the Environment Agency, the hydraulic boundary conditions are essentially seen as 'source' terms. The use of hydraulic boundary conditions is then seen within the context of risk of flooding which depends on the properties and response of the whole system.

F.2 Basic data

Much of this information concerning the basic data could be obtained from the Beach Management Manual (Simm et al., 1996), the Overtopping Manual (HR Wallingford, 1999) and to some extent the Rock Manual (CIRIA/CUR, 1991).

The last two years has seen a developing trend towards regional monitoring programmes, based on the SMP boundaries. The most advanced of these is the Southern Regional Monitoring Programme (Portland Bill to the Isle of Dogs), where data is gathered, stored and analysed in a consistent fashion across more than 20 administrative boundaries. More recently, this model has been applied to the north-west and the north-east.

F.2.1 Water level

Tide levels are available from tide gauge records. Surge heights are derived from measured tide levels and harmonic analysis of the astronomical component. Extreme levels from Weibull, Gumbel, Generalised Pareto or other suitable EV distributions are commonly used. Levels at intermediate locations can be interpolated on the basis of 2-D numerical modelling. There is no national standard here.

F.2.1.1 Measurements

The national tide gauge network consists of around forty gauges installed and maintained to a common high standard. Following a recent change in policy, these data are now freely accessible back to 1990 for use in coastal engineering studies.

F.2.1.2 Hindcast

Because of the longer periods of measured data available, tidal models are less frequently used than wave models. They are mainly used to provide detail between measurement stations.

F.2.2 Waves

F.2.2.1 Measurements

Until recently, the national tide gauge network did not have a counterpart for waves, but over the last year or so, a national network and a number of regional observatories have been established. The national network currently has a few deep water wave recorders of its own and access to a few others, making a total of about ten wave buoys, data from which is freely available in near real-time via the internet (http://www.cefas.co.uk/wavenet). Figure E.1 shows the locations of different wave buoys. The exact location is given in .

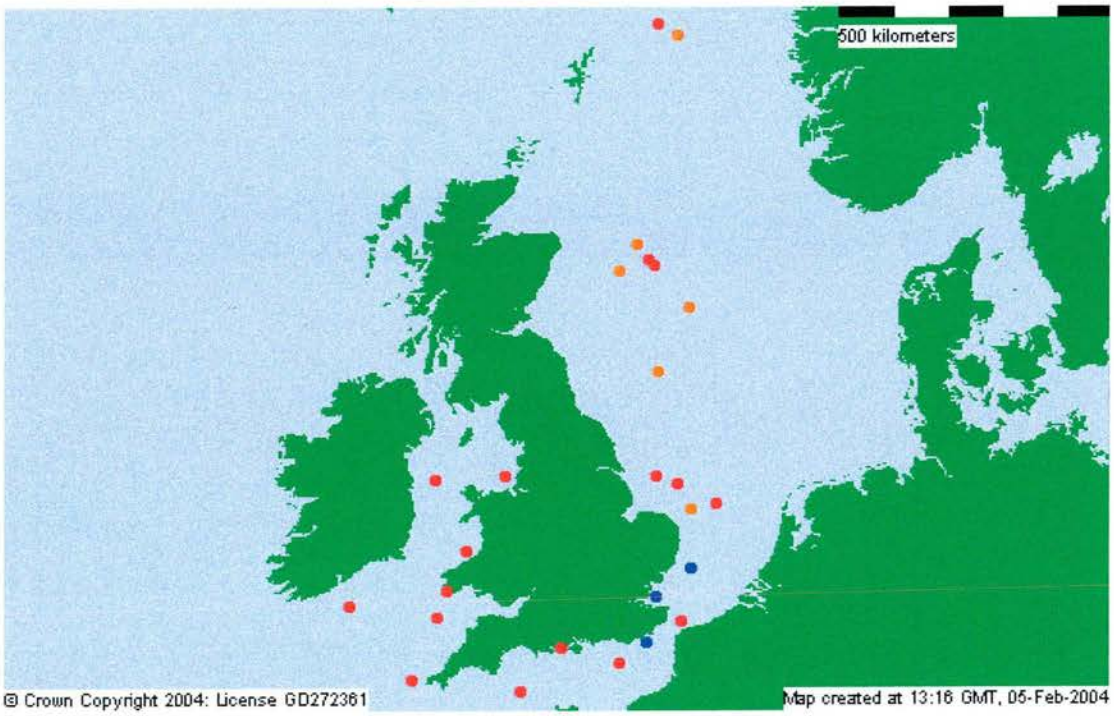


Figure F.1 Wave measuring sites used in the United Kindom. Blue dots denote historic buoys; red buoys correspond to active buoys, whereas yellow dots correspond to buoys which were not active at the time of writing this report.

Name (Provider)	Position (N)	Position (E/W)
Poole Bay WaveNet Site; (CEFAS)	50°38'.02 N	001°43'.10 W
Anasuria; (Shell UK)	57°12'.00 N	000°48'.00 E
Clipper Field; (Shell UK)	53°24'.00 N	001°42'.00 E
Gannet; (Shell UK)	57°6'.00 N	001°0'.00 E
North Cormorant; (Shell UK)	61°12'.00 N	001°6'.00 E
FS1; (The Marine Institute)	51°22'.26 N	007°56'.70 W
M2 Buoy; (The Marine Institute)	53°28'.80 N	005°25'.50 W
Aberporth Buoy; (UK Met Office)	52°18'.00 N	004°30'.00 W
Channel Lightship; (UK Met Office)	49°54'.00 N	002°54'.00 W
Greenwich Lightship; (UK Met Office)	50°24'.00 N	000°0'.00 W
Pembroke Buoy; (UK Met Office)	51°36'.00 N	005°6'.00 W
Sevenstones Lightship; (UK Met Office)	50°6'.00 N	006°6'.00 W
Sandettie Lightship; (UK Met Office)	51°6'.00 N	001°48'.00 E
Dowsing WaveNet Site; (CEFAS)	53°31'.96 N	001°3'.07 E
West Lundy WaveNet Site; (CEFAS)	51°10'.29 N	005°21'.35 W
Liverpool Bay WaveNet Site; (CEFAS)	53°32'.16 N	003°21'.54 W
Sean P; (Shell UK)	53°6'.00 N	002°48'.00 E

Table F.1 Locations of active buoys in the United Kindom

The Southeast Observatory (Kent, Sussex, Hampshire and parts of Dorset, http://www.channelcoast.org) has seven wave recorders of its own, generally in shallower water than the national network (see also Figure F.2).

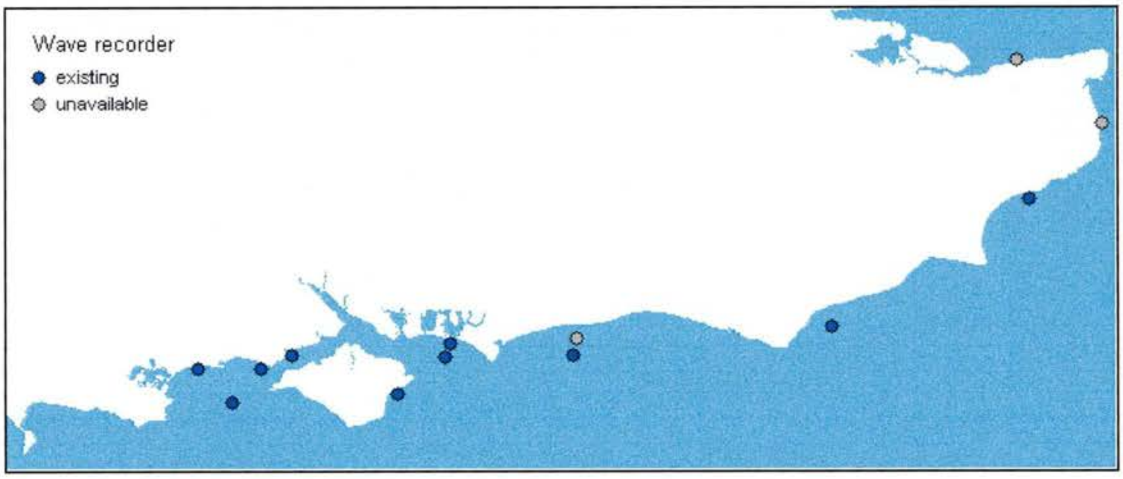


Figure F.2 Wave recorders at the southeastern coast (including 4 recorders not owned by the Southeast Observatory.

Directional wave buoys are the most common type of wave recorder used in the UK, but bottom mounted pressure sensors are used in shallow water up to about eight metres depth.

Until recently, wave data tended to be measured on an *ad hoc* basis as the need arose, with little effort at consistency between methods used, and a typical recording length of one year. With the recent development of national and regional wave networks, this situation may change. Typically, ten years of data are available from tide and wind gauges, and in some cases several tens of years, although prior to about 1970 data may not be available in digital format.

F.2.2.2 Hindcast

Many different wave hindcast and wave transformation models are used, often in conjunction with a short period of wave measurements for calibration, and a longer period of wind records to provide duration.

Wave heights are generally hindcasts from winds at offshore locations. Typical record length for wind data is 20 years.

Extreme wave heights are obtained by fitting an appropriate distribution. Wave periods associated with extreme conditions often use a standard wave steepness based on the average steepness of the highest few percent of the wave conditions in the source data set.

F.2.3 Wind

There are about forty wind measurement stations close enough to the coast to be value in UK coastal engineering studies and, although wind statistics can be purchased from the UK Met Office, the time series data are often prohibitively expensive.

F.3 Data processing

F.3.1 Data treatment

Pre-processing of the source data to extract appropriate records (annual maxima, peaks over threshold, removal of dependent records, checking of outliers) and perhaps to transform to the site of interest, is often more important than the particular extrapolation method chosen.

In the case of regional monitoring packages, data has been stored and analysed on the Shoreline and Nearshore Data System (SANDS) developed by Halcrow. This facilitates storage and analysis of wave, water level, beach profile and other data (photos, structures etc) in GIS-type of database.

Where only high return periods are required, annual maxima are often used, otherwise peaks over threshold values are used to retain about ten records per year, or sometimes all records are used. Occasionally data sets will be divided into populations, e.g. southeasterly and westerly, or windsea and swell, but division by season is unusual.

The validity of the source data can be checked by comparison with neighbouring (either spatial or temporal) records and/or contemporary reports. A longer period of data may be possible by merging with other data sets and/or numerical modelling. Separate analyses can be undertaken with separate parts of the data set to test the sensitivity to outliers. If all else fails, accept them, they might be truly representative of a higher risk than expected.

F.3.2 Statistical evaluation

Several different distributions can be fitted and extrapolated to extremes. For joint probability analysis, with dependence, a mathematical extrapolation may be too difficult and hence a Monte Carlo long-term simulation is preferred.

The national authority on sea levels, the Proudman Oceanographic Laboratory, favours division of sea level into its separate astronomical and surge components, before extrapolation to extremes. Other organisations usually use total water level, sometimes limiting themselves to annual maxima, depending upon the return periods of interest.

Joint Probability approaches are preferred over the Structure Variable method (e.g. HR Report SR 537, The joint probability of waves and water levels JOIN-SEA, and a subsequent best practice guide, in preparation). Joint Probability methods establish a probability density function for the loading variables (e.g. wave height and tide level) and use this multivariate distribution for design / assessment. These methods mostly assume stationarity - there may be a need for trend analysis and adjusting for trend. The correlation is calculated by analysis of concurrent data sets (note this can be limited length so often the full JP distribution needs to be re-scaled to match the marginal extremes). There are different ways of describing the correlation, including the conventional statistical correlation coefficient varying from -1 to +1, to correction factors which can be used as 'desk' methods to adjust the basic design parameters.

Although is is possible, the confidence interval of extrapolations is not often taken into account in the design process explicitly. The JOIN-SEA joint probability method includes an assessment of statistical uncertainty accounting for record length. It does not, however, include full knowledge uncertainty.

F.3.3 Sea level rise

Climate change is an essential component of design and assessment. Standard allowances for sea level rise are provided by Defra, UKCIP02 climate scenarios, and associated R&D. Table F.2 shows the global-average sea level change according to different emission-scenarios, as provided by UKCIP02.

UKCIP02 Scenario	2020s (cm)	2050s (cm)	2080s (cm)
Low Emissions	6 (4 – 14)	14 (7 – 30)	23 (9 – 48)
Medium-Low Emissions	7 (4 – 14)	15 (7 – 32)	26 (11 – 54)
Medium-High Emissions	6 (4 – 14)	15 (8 – 32)	30 (13 – 59)
High Emissions	7 (4 – 14)	18 (9 – 36)	36 (16 – 69)

Table F.2 Global-average sea-level change (cm) relative to the 1961-1990 average for the four UKCIP02 scenarios as calculated by the Hadley Centre models. Figures in brackets are the IPCC range associated with the same SRES emissions scenarios we have used in UKCIP02; we term these our 'low' and 'high' estimates for each scenario, with the HadCM3-derived values adopted as our 'central' estimates. Note that the values we cite for the 2080s are somewhat less than the quoted IPCC values for 2100 since we are averaging over the period 2071-2100.

Other aspects such as storm sequencing are assessed on an 'ad hoc' basis.

F.4 Nearshore conditions

Wave transformation models, and sometimes also tidal flow models, are usually used to determine the conditions near the sea defences. Many different models are used by different consultants and others.

F.4.1 Periods of data

The strategy of selecting the proper period of the data can be summarized as follows: use short periods of measured wave data and usually longer periods of measured wind data. Typically around the UK, about one year of sequential measured wave data will exist within a reasonable distance of the location of interest. Usually neither the length nor the location of the data will be adequate for the purpose, and usually urgency will dictate that the project will not wait for new wave data to be measured.

Typically around the UK, 20-30 years of sequential wind data will exist within a reasonable distance of the location of interest. Usually the length of the data series will be adequate for reliable extremes predictions and, if necessary, realistic adjustments can be made to reflect differences in exposure between the measurement point and the location of interest.

The most efficient use of a short period of measured wave data and a long period of measured wind data consists of the following steps:

- set up a numerical wave hindcasting model, to be driven by sequential wind data, to cover the location of the wave measurements and any other wave prediction points of interest
- run the wave model for the duration and location of the wave measurements
- calibrate the wave model to achieve best agreement with the measurements
- re-run the wave model for all prediction points of interest using the longer period of wind data available

The benefits of the accuracy of local wave measurements and the duration of local wind measurements are thus incorporated into the predictions.

F.4.2 Extreme predictions

Depending upon the intended use of the wave predictions, it may be necessary to predict wave period and/or wave direction in addition to wave height, and it may be necessary to know the range and distribution of wave conditions in addition to the extremes.

Where required, extreme wave conditions will be derived by extrapolation from a distribution of wave conditions. In most applications it is useful to determine extreme conditions for a number of different direction sectors, as they may propagate differently, may cause different impacts on structures, and may have different dependencies with other key variables.

Extremes can be predicted offshore and then be transformed to inshore locations of interest, or can be predicted directly inshore. If extremes are predicted offshore, for subsequent application inshore, then it is necessary that they should still be extremes on arrival inshore and, for example, that they would not be blocked by headlands. Conversely, it may be better not to predict extremes too far inshore, where depth limitation may result in a truncated distribution of wave height not amenable to fitting with a standard statistical model.

F.4.3 Validation of hindcast models against field data

The location map of the validation sites around or beside the UK is presented in Figure F.3. Table F.5 at the end of this appendix presents about fifty HR Wallingford wave modelling studies in which validation against field data has been undertaken.

Each of the major consultants in the UK operate different models of wave disturbance, and similarly have calibrated their methods at numerous different sites around the world. Halcrow has developed in-house models, whilst other consultants rely upon commercially available software.

Figure F.3 Location map of validation sites around or beside UK

F.5 Design / safety assessment sea dikes

F.5.1 Strength parameters

The sea defence is described by the profile and the surface roughness, key parameters being the toe elevation, the crest elevation, the surface steepness, the surface roughness and the material. The hydraulic input is the wave height, wave period and water level, and occasionally the wave direction and wind speed. The results are given in terms of a mean overtopping rate and a high (say 2% exceedance) run-up level. Other run-up parameters may be calculated, as may peak overtopping rate and overtopping volume.

If the crest height, or some other aspect of the defence, is to be designed in this way, it is usually based on a target acceptable mean overtopping rate, which itself will depend on the land-use immediately behind the defence.

The probability of exceedance that is used in the design depends on the risk and cost as expressed through the economic benefit cost assessment, and in line with Defra's published Indicative Standards. This also takes account of joint probability where appropriate.

Special conditions apply for temporary works. Here contractors may make use of risk-based designs and decisions to design temporary works and to manage risks of storms which could damage plant and partly built structures.

F.5.1.1 Wave overtopping

This is well described in the Rock Manual, although some consultants (including Halcrow) do not subscribe to all of the recommendations where they deviate from practical experience. For example, rock layer thickness, and recommended rock grading derivation are not always applied directly as set out in CIRIA/CUR.

In the United Kingdom the design or safety assessment of sea defences is usually carried out by consultants, which are free to use their own methods. The criteria used may depend on the local circumstances and can be given in terms of a mean overtopping rate and a high (e.g. 2% exceedance) run-up level. Other run-up parameters may be calculated, as may peak overtopping rate and overtopping volume. If the crest height is to be designed, it is usually based on a target acceptable mean overtopping rate, which itself will depend on the land-use immediately behind the defence.

Though no strict guidelines apply, the Rock Manual (CIRIA/CUR, 1991), the Overtopping Manual (HR Wallingford, 1999) and the Beach Management Manual (CIRIA, 1991) are often used, although some consultants (including Halcrow) do not subscribe to all of the recommendations where they deviate from practical experience.

The Rock Manual (CIRIA/CUR, 1991) presents the following formulae for wave overtopping:

$$\frac{q}{\sqrt{gH_s^3}}\sqrt{s_m/2\pi} = a\cdot\exp\left(-b\frac{R_c}{H_s}*\frac{\sqrt{s_m/2\pi}}{r}\right) \tag{F.1}$$

in which

q	=	mean overtopping discharge	[m^3/s.m]
R_c	=	freeboard parameter	[m]
g	=	gravitational acceleration	[m/s^2]
H_s	=	significant wave height	[m]
s	=	wave steepness = $2\pi H_s/\left(gT_m^2\right)$	[-]
T_m	=	mean wave period	[s]
r	=	a correction factor for slope roughness	[-]

The coefficients a and b depend on the slope. The same equation is used for slopes with a berm with different values for the coefficients a and b.

The Overtopping Manual (HR Wallingford, 1999) gives a slightly different formula, which after some rewriting reads:

$$\frac{q}{\sqrt{gH_s^3}}\sqrt{s_m/2\pi} = a\cdot\exp\left(-b\frac{R_c}{H_s}*\frac{\sqrt{s_m/2\pi}}{\sqrt{g}\,r}\right) \tag{F.2}$$

It can be seen that the difference is in the factor $\sqrt{g}$ on the right hand side of the equation. The coefficients a and b for this equation are slightly different than those in the Rock Manual. Values for a and b for Eq. (F.1) and (F.2) are shown in Table F.3. For seawalls with a berm different sets for a and b apply depending on slope, berm elevation and berm width.

Slope	**Rock Manual (Eq. F.1)**		**Overtopping Manual (Eq. F.2)**	
	a	**b**	**a**	**b**
1:1	0.00794	20.12	0.00794	20.1
1:1.5	0.0102	20.12	0.00884	19.9
1:2	0.0125	22.06	0.00939	21.6
1:3	0.0163	31.9	0.0109	28.7
1:4	0.0192	46.96	0.0116	41.0
1:5	0.025	65.2	0.0131	55.6

Table F.3 Empirical coefficients for the computation of wave overtopping for simply sloping seawalls used in the UK (after CIRIA/CUR, 1991 and HR Wallingford, 1999).

The Overtopping Manual presents also tolerable overtopping discharges based on guidelines in the Rock Manual (CIRIA/CUR, 1991). The tolerable mean overtopping discharges for an embankment seawall (with a back slope, thus a dike) are given in Table F.4.

mean overtopping discharge					damages
		q	<	2	No damage
2	<	q	<	20	Damage if crest not protected
20	<	q	<	50	Damage if back slope not protected
		q	>	50	Damage even if fully protected

Table F.4 Criteria for mean overtopping discharge for an embankment seawall (in l/s/m; after HR Wallingford, 1999).

F.5.2 Probabilistic approach to safety assessment

The basic design is generally 'deterministic' in that it is based on a single probability of exceedance. But probabilistic methods are widely used: examples include the assessment of structure performance above and below the design standard, and the risk-based design for a full range of loads, not just for a single 'design' load. Probabilistic approaches are explicitly used for 'soft' and natural structures such as beaches and dunes where uncertainties in response may be particularly important.

For assessment, probabilistic approaches are widely used. For example, the use of 'fragility curves' to describe structural reliability over a range of loads is an essential component of flood risk assessment.

Sea dykes often incorporate discrete components such as gates, sluices, pumping schemes. Risks associated with these are linked to their reliability - including the probability of failure 'on demand' i.e. during an extreme storm event.

A range of probabilistic methods exist, but the outcome of many of these can be encoded in the form of a fragility curve. It is probably fair to say that there is limited use of the 'hard core' reliability methods such as FORM, Monte Carlo etc, although many of these have been demonstrated in research programmes.

Usually the uncertainty of the design value (confidence interval) is not taken into account explicitly, but is implicit in the design formulae used. It would be unusual for a consultant to do more than give a subjective estimate of the accuracy of predictions, or for a client to ask for explicit information on uncertainty.

F.6 Evaluation of sandy coasts

Generally, there are little data on dune composition and they tend to be treated as consisting entirely of the superficial sand deposits. Dunes are considered as part of the beach profile response to storms, which can be predicted with numerical or physical models, with a certain length of retreat implying failure. Incidentally, this is about the only type of model capable of predicting the onset (as opposed to the development) of breaching.

F.6.1 Hydraulic input parameters

The wave height, wave period, wave direction and water level are all important, the general climate usually being more important than the extreme values. Although several sediment parameters may be used, they are often chosen based on a mean grain size. Development of beach profile and plan shape can be taken into account, as can the existence of mixed (i.e. sand plus shingle) beaches.

F.6.2 Approach to safety assessment

In the safety assessment, general shoreline and year-to-year variations in the cross-shore profile retreat are taken into account. The cross-shore profile is measured regularly in many areas of local interest, although not as part of a national programme.

F.6.3 Statistical method

Probabilistic methods have been demonstrated, to test the sensitivity to assumptions in the modelling and the order in which storms arrive, and to look at inter-annual variability, but there is no consistent approach to this. The most common approach is still currently to undertake a deterministic design approach.

Many years of wave (and possibly water level) data will be used. Differences in beach evolution from one year to the next illustrate inter-annual variability. Long periods of data can be played back in different orders to see the range of responses predicted.

References

CIRIA/CUR, 1991, **Manual on the use of rocks in coastal and shoreline engineering**. CIRIA/CUR, 1991.

DWW, 2001. **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), April 2001, 27 pp + appendices.

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

Simm, J.D. (ed), A H Brampton, N W Beech et al, 1996. **R153 - Beach management manual**, CIRIA, 1996

Table F.5 List of validations against field data (Column 'Model' gives the name of the wave model; Column 'Location' lists the location of the field data; Column 'Validation data' presents the quantity and survey method of the wave data; Column 'Reference and data' shows the sequential number of the HRW technical report and the year of the report; and Column 'Remarks' tries to give more information about the wave data or the validation)

Model	Location	Validation data	Reference and date	Remarks
JONSEY	Loch Glascamoch	7 months sequential waverider data	EX 1809 (1988)	Deep long thin loch
	Megget Reservoir	12 months sequential waverider data	EX 1477(1986) EX 1809(1988)	Deep long thin reservoir
	West coast of Eday, Orkneys	3 months directional waverider sequential data	EX 4225 (1999-2000)	Manual diffraction; local wave and swell
JONSEY with COWADIS	Mina Al Ahmadi refinery, Kuwait	3-hourly waverider storm data	EX 4325 (2001)	Shallow water location
DONELAN	Loch Glascamoch	7 months sequential waverider data	EX 1809 (1988)	Deep long thin loch
	Megget Reservoir	12 months sequential waverider data	EX 1809 (1988)	Deep long thin reservoir
SAVILLE	Loch Glascarnoch	7 months sequential waverider data	EX 1809 (1988)	Deep long thin loch
	Megget Reservoir	12 months sequential waverider data	EX 1477(1986) EX 1809 (1988)	Deep long thin reservoir
HINDWAVE	Cromer	12 months sequential directional waverider data	EX 1665 (1988) SR 218 (1989)	Protected by offshore banks; Deep and shallow water versions
	Dowsing LV	30 months sequential shipbome data, and 16 years of observations	EX 1665 (1988) SR 218 (1989)	Deep and shallow water versions tested
	Dyck LV, France	Extremes from several years observations	EX 1309 (1985)	
	Flamborough	3 months sequential directional waverider data	EX 1665 (1988)	
HINDWAVE	Galloper	12 months sequential shipborne data, and 27 years observations	EX 1665 (1988) EX 1750 (1988) SR 218 (1989)	Deep and shallow water versions tested
	Holderness	15 months sequential waverider data	EX 1665 (1988)	Two recording sites used

Model	Location	Validation data	Reference and date	Remarks
	Irish Sea, St George's Channel, and Dublin Bay	1 year sequential shipborne data	EX 3585 (1997)	
	Kentish Knock	28 months sequential waverider data	EX 1665 (1988)	
	Littlehampton	12 months sequential waverider data	EX 1871 (1989)	HINDRAY also tested, with different wind data
	Loch Linnhe	3 months sequential waverider data	EX 1984 (1990)	Local generation within the loch only
	Maplin	12 months sequential waverider data	EX 1641 (1987)	Two shallow recording sites and bank-limited HINDWAVE used
	Morecambe	6 months sequential waverider data	EX 1721 (1988)	Well offshore from Morecambe Bay
	Perranporth	4 months sequential waverider data (depth 44m), 25 months sequential waverider data (depth 25m)	SR218 (1989) EX 2401 (1991)	Deep coastal site
	Pevensey	6 monthly sequential directional waverider data	EX 2119 (1990)	
	Prestatyn	12 months sequential waverider data	EX 1369 (1986) Hawkes (1987)	
	St Ouen's Bay, Jersey, France	6 months sequential waverider data	EX 4020 (1999) (updated 2001)	
	Scapa Flow	3 months sequential waverider data	EX 4187 (2000)	Deep water
	Seaford	12 months sequential waverider data	EX 1345 (1985) Hawkes (1987)	
	Severn B	35 months waverider data: some sequential	EX 1736 (1988)	Mid-channel, just west of Steep Holm
	Smiths Knoll LV	37 years of observations	EX 1665 (1988)	
	Solway Firth	12 months sequential waverider data	EX 2719	Mid-channel, north-west of Workington

Model	Location	Validation data	Reference and date	Remarks
	Varne LV	Extremes for several years observations	EX 1309 (1985)	Well offshore
	West Sole	12 months sequential wavestaff data	EX 1665 (1988)	Well offshore
BRISTWAVE	Cromer	4 months sequential waverider data	SR 218 (1989)	Well offshore, but near shallow banks
	Perranporth	4 months sequential waverider data	SR 218 (1989)	Deep coastal site
	Severn A, B and C	sequential waverider data for 9 storms simultaneously recorded at three locations, off Nash Bank, Steep Holme and Flat Holme	EX 978 (1981)	Deep water, mid channel three locations
OUTRAY	Perranporth	26 storms simultaneously recorded at depths 23m and 48m	SR 194 (1989)	Parallel contoured OUTRAY (PCM) also tested
	South Uist	35 storms simultaneously recorded at depths of 15m and 44m	SR 253 (1991)	Swell and storm waves over rocky nearshore zone
	Egypt	Sequential waverider data	EX 4442 (2003)	
	Perranporth	10 storms waverider data	SR 450 (1996)	Deep water location; shallow water location (20m)
	Port Qasim, Pakistan	Extremes derived from waverider data recorded simultaneously offshore and in the dredged channel	EX 1841 (1988)	Mainly swell over shallow muddy nearshore zone – adjust for friction
	South Uist	10 storms waverider data	SR 450 (1996)	Deep water location; shallow water location (20m)

Model	Location	Validation data	Reference and date	Remarks
OUTURAY	St Sampson, Guemsey	12 months sequential waverider data	EX 2099 (1991)	Tested in conjunction with HINDWAVE and water level data - nearshore near Herm
	Shakespeare Cliff	16 months sequential waverider data	SR 236 (1990)	Tested in conjunction with HINDWAVE
INRAY	South Uist	35 storms simultaneously recorded at depths of 15m and 44m	SR 253 (1991)	Swell and storm waves over rocky nearshore zone
FDWAVE	Holderness, Humberside	6 months sequential waverider and directional waverider data	TR 30 (1997)	Shallow water locations of 5m – 20 m
FDWAVE	Great Yarmouth	13 months synoptic waverider sequential wave data	EX 3726 (1998)	Deep water location; shallow water location
	Port Qasim, Pakistan	16 storms simultaneously recorded offshore and in the dredged channel	EX 2151 (1992)	Mainly swell
	South Uist	35 storms simultaneously recorded at depths of 15m and 44m	SR 253 (1991)	Swell and storm waves over rocky nearshore zone
NPM	Aberdeen	Current velocities	EX 1759 (1988)	No comparison of waves
	Cromer	Small number of measured storm wave conditions	SR 247 (1990)	Measurements landward of offshore bank
	Dunwich	Small number of measured storm wave conditions	SR 247 (1990)	Measurements each side of a submerged bank
	Perranporth	26 storms simultaneously recorded at depths of 23m and 48m	SR 194 (1989)	Measurements at two locations inshore and offshore
HINDRAY	Aberdeen	14 months sequential waverider data	EX 1759 (1988)	

Model	Location	Validation data	Reference and date	Remarks
	Aldeburgh	24 months sequential pressure wave recorder data	EX 1465 (1986) Hawkes (1987)	Shallow water location
	Barrow	6 months sequential pressure wave recorder data	EX 1840 (1989)	
	Cardiff	12 months sequential pressure wave recorder data	EX 1850 (1988) EX 1857 (1989)	Very shallow and site-specific: dries at low water
	CTR Tower, northern Adriatic Sea	4 months sequential waverider data	EX 4607 (2002)	Deep water location
	Dover	13 months sequential waverider data	EX 1470 (1986)	
	Guernsey	12 months sequential waverider data	EX 2099 (1991)	Very site-specific variable depth and OUTURAY used (HINDRAY)
	Jersey	Report on several years waverider measurements	EX 1961 (1989)	Inshore site-specific shallow water location
	Kentish Flats Wind Farm	Sequential waverider data	EX 4725 (2003)	
	Littlehampton	12 months sequential waverider data	EX2371 (1991)	HINDWAVE also tested with different wind data
HINDRAY	Pittenween	6 months sequential pressure wave recorder data	EX1611 (1987)	Site-specific shallow water location
	Poole Bay	6 months sequential waverider data	EX 2406 (1991) EX 2508 (1992)	OUTDIF used local generation included
	St Ouen's Bay, Jersey, France	6 months sequential waverider data	EX 4020 (1999) (updated 2001)	Shallow water location
	Shakespeare Cliff	24 months sequential waverider data	SR 236 (1990) EX2111 (1990)	With and without current versions of HINDRAY used
	Southboume	3 months sequential waverider data	EX 1460 (1986)	
	Swanage	3 months sequential pressure wave recorder data	EX 1573 (1987)	Very shallow and site-specific

Model	Location	Validation data	Reference and date	Remarks
	Tees Estuary	16 months sequential waverider data	EX 2064 (1990)	
	Wylfa	10.5 months sequential waverider data	EX 3351 (1996)	Shallow water location
WINDWAVE	Morecambe Bay	sequential waverider data during two severe storms	SR 295 (1992)	Model run with and without refraction
	North Sea	18 storms validated running NORSWAM studies	SR 295 (1992)	Validation data from Famita and Stevenson
	Prestatyn	sequential waverider data during three severe storms	SR 295 (1992)	Model run with and without refraction, and for different water levels
PARAB	Perranporth	10 storms waverider data	SR 450 (1996)	Deep water location; shallow water location (20m)
	South Uist	10 storms waverider data	SR 450 (1996)	Deep/shallow water locations
SWAN	Bathside Bay, Stour Estuary	Winter 2000-2001 using pressure sensor	EX 4407 (2001)	Shallow water
PORTRAY*	Perranporth	10 storms waverider data	SR 450 (1996)	Deep water location; shallow water location (20m)
	South Uist	10 storms waverider data	SR 450 (1996)	Deep water location; shallow water location (20m)

*PORTRAY Many validations against wave heights recorded in physical models, but all are confidential to clients.

WL | Delft Hydraulics

Rotterdamseweg 185
postbus 177
2600 MH Delft
telefoon 015 285 85 85
telefax 015 285 85 82
e-mail info@wldelft.nl
internet www.wldelft.nl

Rotterdamseweg 185
p.o. box 177
2600 MH Delft
The Netherlands
telephone +31 15 285 85 85
telefax +31 15 285 85 82
e-mail info@wldelft.nl
internet www.wldelft.nl