Auszug aus: Proc. of the 2nd Indian National Conference on Harbour and Ocean Engineering (INCHOE), India, 1226-1235, 1997

SAFETY VARIATION OF COASTAL DEFENSE SYSTEMS

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Abstract

Coastal defense systems are composed of different coastal protection elements. The safety of the coastal protection system depends on their configuration and can be quantified by the recurrence interval of wave-overtopping at the main dike, i.e. the final protection element.

The recurrence interval of wave-overtopping is calculated by a probabilistic approach using level-III-analysis. This relates the joint-probability-distribution of the load parameter (i.e. water-level, incident wave characteristics and wind situation) at the seaward end of the coastal protection system to the probability-distribution of wave-characteristics within the coastal protection system and to the probability of wave-overtopping.

Evaluating the safety for different scenarios of joint-probability-functions of load-parameter related to climate change the effect of water-level-rises and increasing winds is obtained. First results show that the probability of wave overtopping will be quadrupled due to a rise of the mean sea level of 0.5 m and increase ten-fold due to a rise of 1.0 m.

Comparing the probability distribution of wave-heights at different locations within the coastal protection system the effectiveness of the coastal defense elements is obtained.

INTRODUCTION

Coastal defense systems along the German North Sea coast are composed of different protection elements, e.g. bars, dune-islands, saltmarshes, forelands with or without brush wood fences, summer-dikes and a main dike as the final protection barrier (see Figure 1).

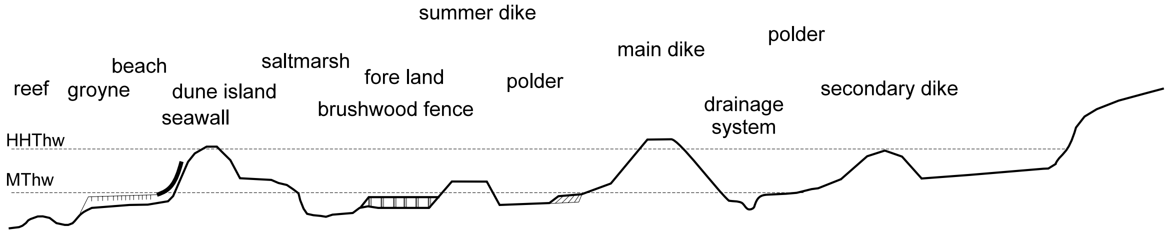

Figure 1: Assembly of a coastal defense system consisting of protection elements.

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Traditional design of coastal defense systems considered so far only the main dike, as the most important defense element, disregarding the offshore protection elements. The important design parameter of the dike is its crest height which is traditionally determined using a deterministic design approach. Within this approach the required crest height is calculated by summation of the mean high water level, the maximum historically recorded deviations of the high water-levels due to spring tides and wind-effects and the maximum possible wave run-up, e.g. German Commitee for Coastal Protection (1993). This does not give the recurrence interval of failure of the coastal defense system.

Probabilistic design concepts enable such limitations of design to be overcome and have been presented by Plate and Duckstein (1988). An adaptation of the probabilistic framework for assessing the safety of flood defences has been worked out by the Technical Advisory Committee on Water Defences (1990).

This paper gives an example of the adaptation of a probabilistic design scheme to the situation at the German coast and a practical application in order to derive the changes in safety of the coastal protection including possible rises in water-level related to climate change.

PROBABILISTIC DESIGN SCHEME FOR COASTAL DEFENSE SYSTEMS

Failure of coastal defense systems occurs mainly due to overflowing and wave overtopping of the dike. Other failure mechanisms, e.g. piping, are neglected within the present paper but can be introduced in the presented probabilistic scheme.

As a standard of safety for the complete coastal protection system the probability or recurrence interval of wave overtopping at the final protection element, i.e. the dike, can be used. The failure mechanism of overtopping can be described mathematically by means of a reliability function

$$Z = h_d - h_{sl} - R_w \tag{1}$$

which depends on the dike height h_d, the water level in front of the dike h_{sl}, and the wave run-up R_w. For $Z < 0$ the protection system fails, i.e. overtopping occurs. The wave run-up can be calculated using Battjes (1971) formula

$$R_w = \frac{1}{n}\overline{T}_d\sqrt{gH_{s,d}} \tag{2}$$

in which $1/n$ is the dike slope, $\overline{T}_d$ is the mean wave period, g is the acceleration due to gravity and $H_{s,d}$ is the wave height in front of the dike. Besides the Battjes formula various others may be found in Tautenhain (1981).

The wave height in front of the dike is a function of the water-level h_{sl}, the incoming wave field on the seaward side of the coastal protection system, which can be described by the significant wave height H_s, the mean period $\overline{T}$ and the angle of propagation α, the wind field with the parameters wind velocity u_w and wind direction α_w. This can be described by

$$
\begin{aligned}
H_{s,d} &= \tilde{f}(h_{sl}, H_s, \overline{T}, \alpha, u_w, \alpha_w) && (3)\\
T_d &= \tilde{g}(h_{sl}, H_s, \overline{T}, \alpha, u_w, \alpha_w) && (4)\\
R_w &= \tilde{h}(h_{sl}, H_s, \overline{T}, \alpha, u_w, \alpha_w) && (5)
\end{aligned}
$$

The transfer functions $\tilde{f}$, $\tilde{g}$ and $\tilde{h}$ depend on the structure of the coastal protection system and its elements and have been determined using numerical wave models.

The parameters h_{sl}, H_s, $\overline{T}$, α, u_w, α_w on the seaward side are probability distributed. Therefore the parameters of $H_{s,d}$, $\overline{T}_d$, R_w and Z at the dike are also probability distributed. The relationship between the probability functions of the seaward parameters and the parameters at the dike are given by

$$
\begin{aligned}
p_{(H_{s,d})} &= \int \cdots \int_{H_{s,d}=\tilde{f}(h_{sl},H_s,\overline{T},\alpha,u_w,\alpha_w)} \cdots \\
&\quad \cdots p_{(h_{sl},H_S,T,\alpha,u_w,\alpha_w)} dh_{sl}\, dH_S\, dT\, d\alpha\, du_w\, d\alpha_w && (6)\\
p_{(\overline{T}_d)} &= \int \cdots \int_{\overline{T}_d=\tilde{g}(h_{sl},H_s,\overline{T},\alpha,u_w,\alpha_w)} \cdots \\
&\quad \cdots p_{(h_{sl},H_S,T,\alpha,u_w,\alpha_w)} dh_{sl}\, dH_S\, dT\, d\alpha\, du_w\, d\alpha_w && (7)\\
p_{(R_w)} &= \int \cdots \int_{R_w=\tilde{h}(h_{sl},H_s,\overline{T},\alpha,u_w,\alpha_w)} \cdots \\
&\quad \cdots p_{(h_{sl},H_S,T,\alpha,u_w,\alpha_w)} dh_{sl}\, dH_S\, dT\, d\alpha\, du_w\, d\alpha_w && (8)\\
p_{(Z)} &= \int \cdots \int_{Z=h_d-h_{sl}-\tilde{h}(h_{sl},H_s,\overline{T},\alpha,u_w,\alpha_w)} \cdots \\
&\quad \cdots p_{(h_{sl},H_S,T,\alpha,u_w,\alpha_w)} dh_{sl}\, dH_S\, dT\, d\alpha\, du_w\, d\alpha_w && (9)
\end{aligned}
$$

with $p_{(h_{sl},H_s,T,\alpha,u_w,\alpha_w)}$ being the joint probability distribution of the parameters on the seaward side of the coastal defense system, $p_{(H_{s,d})}$ the probability density function (pdf) of the significant wave height in front of the dike, $p_{(\overline{T}_d)}$ the pdf of mean wave period at the dike, $p_{(\overline{R}_w)}$ the pdf of the wave run-up, $p_{(Z)}$ the pdf of the reliability function.

By integrating the pdf of the reliability function over a negative range the probability of wave overtopping $p_{Z<0}$, i.e. failure of the coastal protection system, can be calculated:

$$
p_{Z<0} = \int_{-\infty}^{0} p_{(Z)} dZ \qquad (10)
$$

The recurrence interval T_r of wave overtopping equals the inverse of the probability of failure ($T_r = 1/p_{Z<0}$).

Changing hydraulic loads will result in variations of the joint pdf of the incident parameters and therefore alter the reccurence interval of failure. Variations in the form of the coastal protection system will alter the transfer functions $\tilde{f}$, $\tilde{g}$, $\tilde{h}$ and therefore alter the recurrence interval as well.

There exists a strong correlation between wind conditions and wave conditions. It is therefore possible to estimate the wave conditions at the seaward boundary using wind data, e.g. by using $\alpha = \alpha_w$, $H_s = 0.283 \cdot u_w^2/g \tanh\left(0.53 \cdot (gd/u_w^2)^{(3/4)}\right)$ and $T = 7.54 \cdot u_w/g \tanh\left(0.833 \cdot (gd/u_w^2)^{(3/8)}\right)$, in which d is the water depth depending on h_{sl}, described in CERC (1984) or by using locally valid equations from field measurements, e.g. for the East-Frisian coast of Germany the relationship $H_s = 0.35\left(u_w^2/g\right)^{0.66}$ given by Niemeyer (1979). The derivation of wave conditions from wind conditions is preferable because in contrast to measurements of water-levels and wind conditions, long term measurements of wave conditions are very rare. Since H_s, T and α depend on u_w, α_w and h_{sl}, the joint probability function $p_{(h_{sl},H_S,T,\alpha,u_w,\alpha_w)}$ reduces to $p_{(h_{sl},u_w,\alpha_w)}$ and Eqs. 6 to 9 reduce to a triple integration.

NUMERICAL MODELING OF WAVE PROPAGATION

The transfer functions $\tilde{f}$, $\tilde{g}$ and $\tilde{h}$ are determined using the wave-models HISWA (HIndcast Shallow Waves), published by Booij et al. (1993), and SWAN (Simulation WAves Nearshore), published by Ris (1997).
Both wave-models are based on the action conservation equation

$$\frac{\partial}{\partial t}N(\sigma,\theta) + \frac{\partial}{\partial x}c_x N(\sigma,\theta) + \frac{\partial}{\partial y}c_y N(\sigma,\theta) + \frac{\partial}{\partial \sigma}c_\sigma N(\sigma,\theta) + \frac{\partial}{\partial \theta}c_\theta N(\sigma,\theta) = \frac{S(\sigma,\theta)}{\sigma} \quad (11)$$

where $N(\sigma,\theta) = E(\sigma,\theta)/\sigma$ is the action density, which is equal to the energy density related to the relative frequency, c_x, c_y, c_σ and c_θ are the velocities of the progation of action in the spatial (x,y), frequency (σ) and directional (θ) domain. $S(\sigma,\theta) = S_{in}(\sigma,\theta) + S_{ds}(\sigma,\theta) + S_{nl}(\sigma,\theta)$ represents the sources and sinks of wave-energy by wind-generation, dissipation due to white-capping, wave-bottom interactions and depth-induced wave breaking and due to conservative nonlinear wave-wave interactions.

The model SWAN solves Eq. 11 assuming a stationary state, i.e. $\frac{\partial}{\partial t}N(\sigma,\theta) = 0$, with a Finite Difference Method using implicit mixed upwind/central order schemes in the four-dimensional propagation space, e.g. Holthuijsen et al. (1993).

Within HISWA the action balance is also simplified under the assumption of stationarity. HISWA uses only parametric distributions of action in frequency space characterized by the frequency integrated action density $N_0(\theta) = \int_0^\infty N(\sigma,\theta)d\sigma$ and the average frequency $\sigma_0(\theta) = \int_0^\infty \sigma N(\sigma,\theta)d\sigma$ for each spectral direction. For the numerical solution HISWA uses an explicite finite difference scheme in the three-dimensional propagation space as presented by Booij (1985).

Both wave-models were applied to the German coast. The model areas are presented in Fig. 2.

The eastern area next to Norderney (shaded rectangular on the left hand side) relates to a coastal defense system consisting of barrier reefs, forelying dune islands, wide wadden areas and a wide fore-land with a summerdike.
Fig. 3 shows the coastal defense system and an example of the calculated wave-propagation

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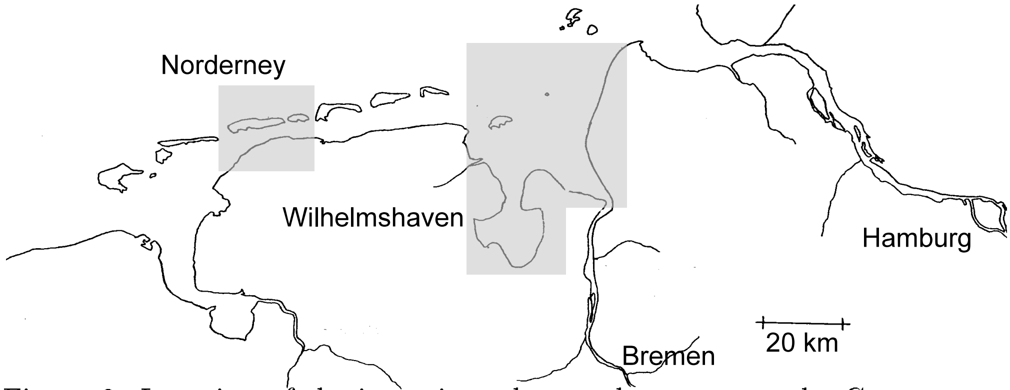

Figure 2: Location of the investigated coastal systems at the German coast

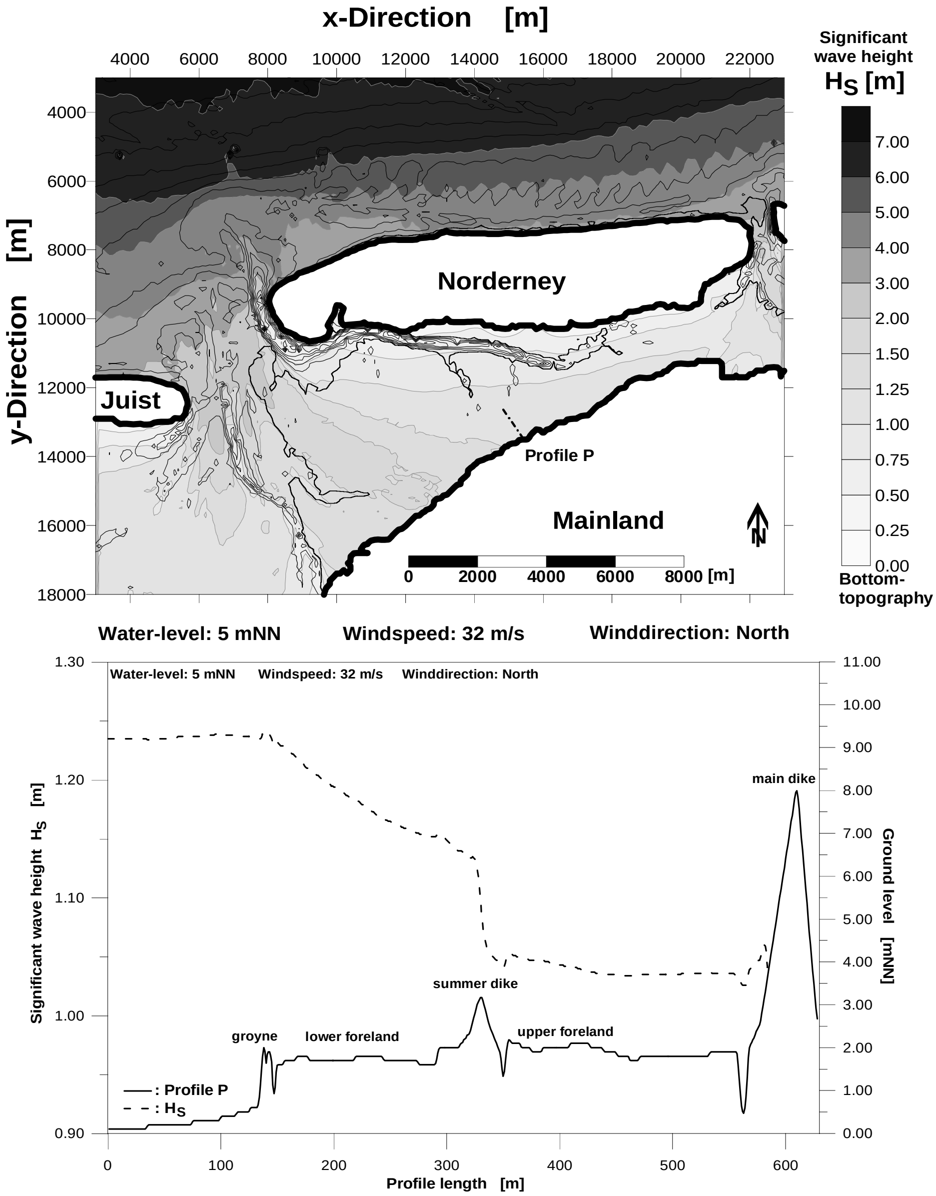

Figure 3: Wave-propagation within the coastal defense system near Norderney calculated with HISWA

within the coastal defense system using HISWA. The wave-propagation within the seaward part of the coastal protection system (Fig. 3, upper part) was calculated with 50 m grid-spacing and shows the importance of fore-lying dune-islands for the safety of the mainland. For the simulation of wave-propagation directly in front of the main dike (Fig. 3, lower part) a grid-spacing of 1 m is used. From the decrease of wave heights along the foreland and the summer dike an increase of safety of the coastal protection system can already be detected.

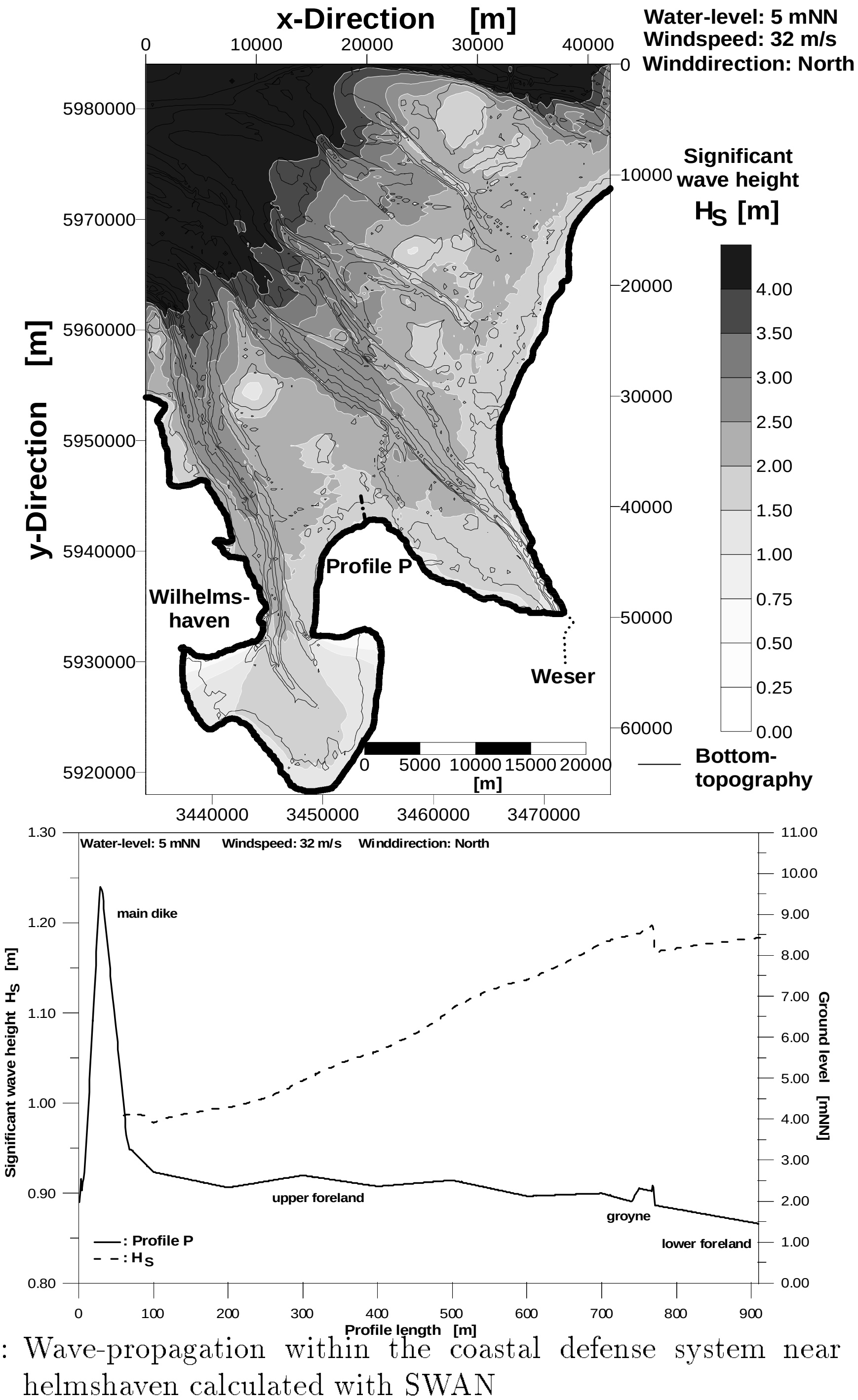

Figure 4: Wave-propagation within the coastal defense system near Wilhelmshaven calculated with SWAN

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In the western area near Wilhelmshaven (Fig. 2) the coastal defense system consists of wide wadden sea areas, foreland and the main dike. For an extreme situation with water-levels of 5 m and northern winds of 32 m/s the significant wave height along this system decreases from 5 m at the seaward end of the protection system to 1.2 m due to the attenuation within the wadden area and to 0.9 m due to the attenuation along the foreland.

Both applications of the wave models show only one relationship of external load, i.e. waterlevel, wind-velocity and direction, to significant wave parameters at the main dike. Applying the wave models for various combinations of the parameter of external load the transfer functions $\tilde{f}$, $\tilde{g}$ and $\tilde{h}$ are calculated. This calculation requires large computational effort. The required computational time is approximately ten-times higher using SWAN in comparison to HISWA which may still be a reason for prefering HISWA although it uses averaging in the frequency domain.

RESULTS

The transfer-functions (Eqs. 3 to 5) relate the joint pdf of input parameters to the pdf of wave-parameters within the coastal protection system by applying Eqs. 6 to 9.

Fig. 5 shows the pdf of the input parameter water-level and the pdf of the significant wave height at different locations of the coastal defense system near the island Norderney (see Figure 3).

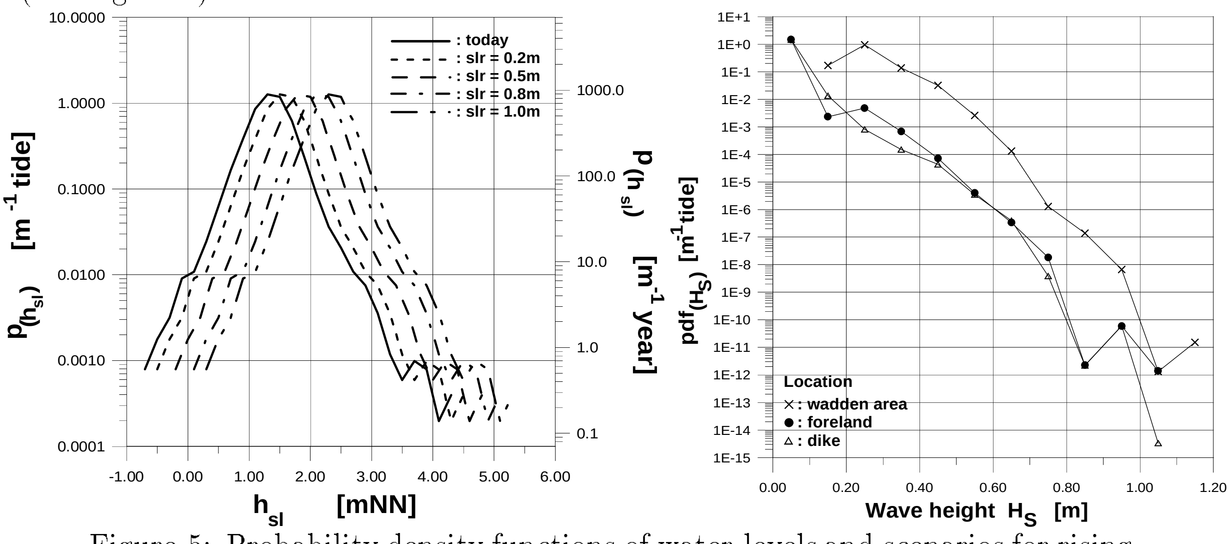

Figure 5: Probability density functions of water-levels and scenarios for rising water levels (left) and of the significant wave heights at different locations within the costal defense system (right)

Fig. 5 also shows scenarios of pdfs for sea-level rises of 0.2 m to 1.0 m. A mean sea-level rise of 0.2 m to 1.0 m, according to the IPCC (1996), may be reached within the next hundred years.

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Fig. 5 also shows the joint pdf of the input parameter with weighted influence of the defense elements within the system of Norderney, while Fig. 3 gives the influence for a single situation of load parameters. The probability density of higher waves is reduced from the seaward side (wadden area) of defense system towards the dike. The degree of reduction can be taken as a measure of the effectiveness of the different protection elements.

The effectiveness of defense elements was also analysed by implementing artificial changes within the system, e.g. by removing the summer dike or reducing the height or width of the foreland, and calculating the significant height of waves with different return periods in front of the main dike (see Table 1).

Table 1: Significant height of waves with different return-periods at the main dike after changes in the assembly of the coastal defense system of Norderney

Changes of the coastal system	Recurrence interval (years)				
-	1	10	100	1000	10000
no	0.13 m	0.20 m	0.48 m	0.70 m	0.98 m
without summerdike	0.15 m	0.35 m	0.58 m	0.78 m	0.98 m
reduced foreland height (-1.0 m)	0.28 m	0.32 m	0.60 m	0.85 m	0.98 m
reduced foreland width (-35 %)	0.13 m	0.20 m	0.48 m	0.70 m	0.98 m

Table 1 indicates that the coastal defense system is more sensitive to changes in foreland height than to changes in foreland width. The effectiveness of the coastal protection elements foreland and summer dike decreases with increasing return period.

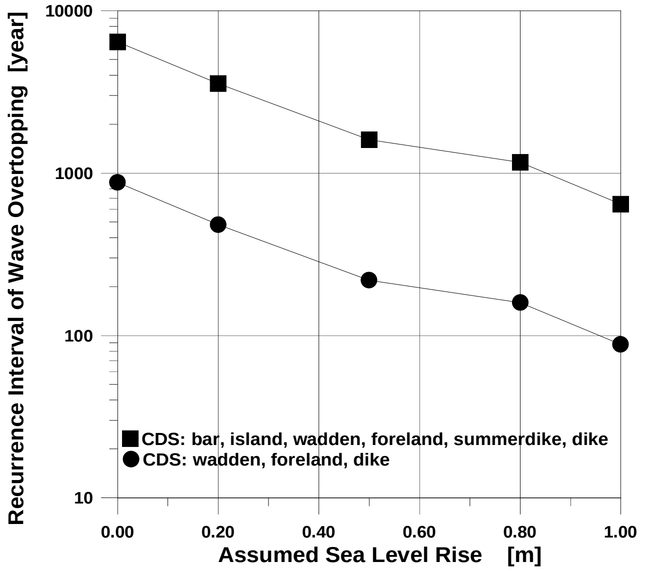

Figure 6: Recurrence interval of wave overtopping as a function of mean sea level rise for different coastal defense systems (CDS).

The variation of safety due to a sea-level rise is calculated applying the probabilistic approach (Eqs. 9 and 10) on varied pdfs of water-level (see Fig. 5). Fig. 6 presents the recurrence interval of wave-overtopping for the coastal defense systems according to Fig. 3 and Fig. 4.

Comparing the recurrence-interval T_r of the two coastal protection systems for today shows the difference between the system near Norderney ($T_r \approx 6500$ years) and the system near Wilhelmshaven ($T_r \approx 800$ years). This is despite the fact, that both coastal defense systems have been designed using the same standard deterministic scheme, described in the introduction. For a 0.25 m sea-level rise the recurrence interval will be reduced for both coastal defense systems by a factor of two, and decreases by a factor of 10 in case of a 1 m water-level rise.

CONCLUSION

The probabilistic design approach shows the shortcomings of traditional deterministic design indicating significant variations of safety along the coast and also the reduction of safety due to sea-level rises.
The increase in computational capacities and the improvements in wave simulations will lead to probabilistic schemes as a standard design procedure for coastal defense systems.

Acknowledgment

This research was supported in part by the Ministry of the Environment of Lower Saxony, Germany. We thank S. Schreeck for her help in executing the wave simulations.

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