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ORIGINAL ARTICLE

Design parameters for coastal dikes

Case study: Meldorf Bight on the German North Sea

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Abstract This paper presents results of the assessment of the design parameters leading to the definition of the crest level of a coastal dike along the German North Sea. Procedures to estimate the design water level have been proposed, distinguishing between comparative and single value procedures. The transformation of the wave characteristics from deep water towards the shallow foreshore was achieved through the application of a spectral wave model. To improve the wave parameter estimations, the existing model was nested to a grid with a higher resolution closer to the coast. The estimation of the wave run-up followed the Dutch procedure with some adjustments to the local wave characteristics and dike geometry. The computed maximum crest level of 8.4 m is below the crest height of the existing dike, which is 8.8 m. However a proposal for a more economical design should be carefully evaluated, paying attention to the uncertainties encountered in this research. The general recommendation is to enhance the reliability of the hindcasted wave parameters through calibration and validation of the wave model and to include in the design process an investigation of the effect of the medium term morphological developments.

Keywords Coastal dikes · Wave modelling · Wave transformation · Design water level · Wave run-up

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List of symbols

Δ	Security surcharge
CVP	Comparative value procedure
d_0	Increase of the tide high water and mean water from the date considered for HHThw up to the calculation date
DWL	Design water level
HAT	Highest tidal level which can be predicted to occur under average meteorological conditions and under any combination of astronomical conditions
HHThw	Highest high tide high water
H_s	Significant wave height
HSpThw	Highest springtide high water
L	Length of the foreshore
MThw	Mean tide high water
NN	German Datum
SVP	Single value procedure
$\tan \alpha$	Equivalent slope
Thw	Tide high water
T_m	Mean wave period
T_s	Significant wave period

1 Introduction

One of the main goals in the design of coastal dikes is to ensure an adequate safety level for those areas that are exposed to high risk of flooding. The land levels in North Germany are relatively close to mean sea level and an extensive system of dikes protects these areas may be affected by these events. The earlier designs of these defences rely on empirical approaches for the definition of their heights. In this paper the procedures nowadays generally applied for the determination of the design parameters, combining results of numerical model simulations with improved empirical equations, are presented and an example application for the dikes along the Meldorf Bight on the German North Sea is described. Emphasis was given to the estimation of design water levels using two different procedures, the single value and the comparative procedure. A nested wave model was applied for the estimation of wave conditions near the dike. The resulting wave characteristics at the toe of the dike were used for the estimation of the wave run-up values along its entire extension [8]. Comparisons between estimated and existing crest levels are presented.

2 Area of investigation

The Meldorf Bight (Fig. 1b), is located within the Dithmarschen Bight between the Eider and Elbe estuaries. The morphology of the bight is dominated by tidal flats and a channel system composed of several tidal channels. On average the maximum water depth in the tidal channel is about 18 m. Depths of about 10 to 15 m are typical for the outer regions. A semidiurnal tide with a range of about 3.2 m controls the domain, causing about 50% of the area to fall dry during low tide. Observed wave heights in the outer region reached up to

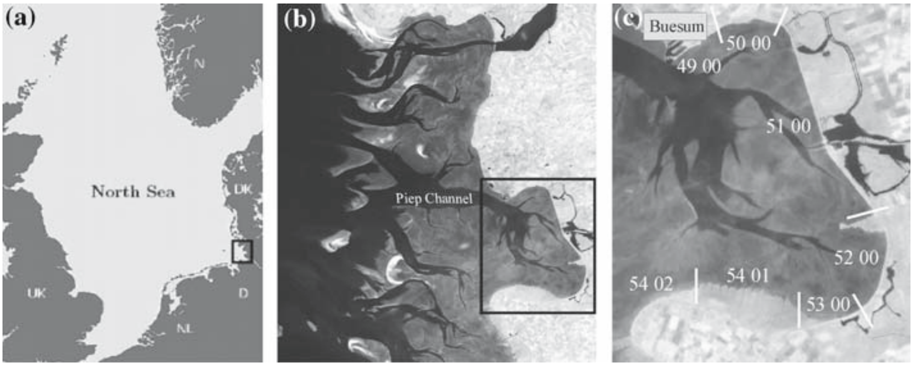


Fig. 1 Area of study. (**a**) North Sea (**b**) Dithmarschen Bight (37 × 54 km) (**c**) Meldorf Bight (9 × 13 km) and considered dike sections

3–4 m although they generally break along the edge of the tidal flats. The hydrodynamics are mainly determined by the conditions along the western boundary through which the tidal wave and swell propagate into the bight. Local wave generation due to wind is found to be the dominant source of energy for wave conditions near the coastline [14]. In general the limited water depths and the complex bathymetry determine strong hydrodynamical-morphological interactions. The entire coastline is protected by dikes (see Fig. 1c).

3 Design procedure for coastal dikes

The main design parameters leading to the crest level of sea dikes are: (1) the design water level, (2) the wave run-up and (3) the security surcharge. The design water level is usually defined on the basis of the most severe storms in the region. The wave run-up is estimated on the basis of the design wave conditions at the toe of the dike taking the geometry of the dike into account. The surcharge proposed by the EAK report (2002) [4] is defined considering the global climate changes and the expected sea level rise up to the year 2100. The determination of these parameters is done in several steps. Figure 2 sketches the main phases which are involved in the methodology for the design of sea defences. Each phase may require a different approach in terms of practice. Initially the design water level and wave conditions in deep water are defined. Then the wave transformation towards the shallow water regions in the vicinity of the sea defence is carried out. The resulting wave conditions near the dike are used for the estimation of the wave run-up. The sum of the various contributions leads to the design crest level for the sea defence. The definition of these parameters is elaborated below for both the single value and the comparative/probabilistic procedure, together with a simplified example application for the Meldorf Bight coastline.

3.1 Design water level

The design water level is to ensure security under extreme situations. Despite the fact that the safety standards are quite high they do not guarantee total security. In other words the design water level is a value expressing a certain acceptable risk. In Germany the definition of this parameter is currently established by individual political strategies for each Federal State [4].

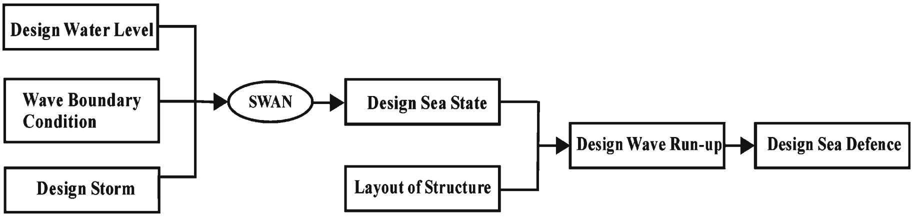


Fig. 2 Conceptual sequence for the design of sea defences

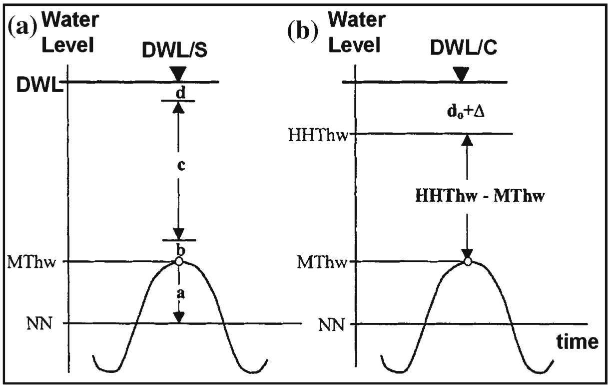


Fig. 3 Procedures to assess the design water level in German North Sea (Modif. After [4](DWL = Design Water Level; NN = German Reference Datum). (**a**) Single value procedure (**b**) Comparative procedure

Two procedures have been used in the assessment of the design water level, i.e. the single value and the comparative which have been applied in the States of Lower Saxony and Schleswig-Holstein respectively (Fig. 3).

3.1.1 Single value procedure

The design water level is made up of the least favourable values not necessarily belonging to the same extreme event (Fig. 3a). The single value approach leads to a design water level of 6.5 m as shown in Table 1. It includes: (a) the Mean High Water of 1.64 m. (b) The highest astronomical tide (HAT) has been assumed as the highest Springtide High Water. The BSH (Federal Maritime and Hydrographic Agency of Germany) provided a HAT value of 2.21 m over a period of 19 years (1984–2003) at the gauge Station Büsum. (c) The maximum wind set-up. As a simplification in this study water level measurements at the Station Büsum during the storm “Anatol” (in December 1999) were considered only. A value of 3.99 m resulted as combination of wind and barometric storm surge. Ideally, this value would be based on a number of extreme events to represent variability in their characteristics. (d) A security surcharge of 0.3 m has been selected on the basis of the recommendations found in the EAK report (2002) [4].

3.1.2 Comparative or probabilistic procedure

This method is defined on the basis of the temporal analysis of the worst storm that occurred in the coastal region under investigation over a given time. The design water level is expressed as

Table 1 Components for determining the design water level within the single value procedure

Component		Design water level (m)
a) MThw over NN	Mean High Water over NN: German Reference Datum	1.64
b) HSpThw–MThw	Height difference between the highest springtide high water and mean tide high water	2.21 − 1.64 = 0.57
c) HHThw–Thw	Largest wind set-up (defined as height difference between the highest high tide high water and the associated astronomical tide high water)	3.99
d) Security surcharge		0.30
Design water level	Single value procedure	6.5

return period (100 years) of extreme water level that the dike must be able to withstand [3,7]. This value determined for the year 2000 is derived from time series over the last 50 years (1950–1999) along several locations of the German North Sea and Baltic Sea. Figure 3b displays how, according to the EAK report (2002) [4], the mean tide High Water (MThw) over German Datum, the highest High Water (HHThw) for that area, a security surcharge (Δ) and an increase $d_0$of the tide high water and mean water with respect to the calculation date (year 2000) are combined to result in a design water level of 5.9 m for the Büsum location.

3.2 Design wave conditions

The characteristics of waves in deep water used for design are defined on the basis of semi-empirical relations taking into consideration wind field and fetch length. The empirical nomogram developed by Bretschneider [2] allows to determine the significant wave height and period at a specific location for a given wind speed, duration and fetch. For the coastline in question a westerly wind speed of 30 m/s with duration of 24 h in conjunction with a 610 km fetch length was used. The wind velocity considered for design purposes was defined on the basis of previous studies [11]. The largest possible fetch for westerly wind direction is defined with respect to the UK coast.

The following wave characteristics resulted: (1) significant wave height (H_s) of about 10.4 m; (2) significant wave period (T_s) of 12.8 s; (3) the corresponding mean wave period (T_m) according to Goda (2000) [5] is about 10.7 s. The derived wave conditions for the western open boundary appear to represent possibly unrealistically large waves at this location, due to the local limited water depth. Furthermore, the assumed constant wind conditions over a 24 h period are rather unlikely as storms will gradually migrate across the North Sea. For the sake of simplicity there values have been adopted nevertheless.

3.3 Wave transformation

The sea state near the coastal structure is determined on the basis of the design water level and deep water wave conditions. In this study the wave transformation towards the shallow foreshore was computed using a phase-averaged spectral wave model. The wave model developed for the Dithmarschen Bight is based on the SWAN wave model [1]. The model measures about 37 km by 54 km and covers an area of about 1,640 km^2. The model grid consists of about 43,250 cells with a grid spacing ranging from 80 m to 200 m. The offshore boundary is located about 29 km west from Büsum. Calibration and validation of the model yielded good results [14]. In this study another wave model based on SWAN covering the Meldorf Bight (9 by 13 km) with grid spacing of about 50 m was nested in the larger Dithmarschen Bight Model.

The SWAN computations have been carried out in stationary mode. This approach is valid whenever the time of wave propagation through the model domain is short with respect to time scale of the driving forces (wind and swell). In other words for stationary runs a final state based on the boundary conditions is assumed as constant over the time. According to the literature the stationary assumption holds in case of a domain of about 35 km length [14]. Results of the simulations for the two design water levels (single value and comparative) in conjunction with the design wave conditions in deep water and considering constant and uniform wind field (30 m/s wind speed from the West) are shown in Fig. 4.

It can be seen that the significant wave heights along the open sea western boundary resulted smaller than the imposed values. This is due to the modest water depths (10–15 m) that are insufficient to sustain the imposed wave conditions along the western open boundary. As a result of the depth-induced wave breaking there is a progressive reduction of the wave heights towards the coastline. The resulting wave heights at the toe of the dike in the Meldorf Bight are about 1.5 m. The results obtained are comparable independently of the water level considered in the simulations. In the deeper tidal channels the swell waves can penetrate somewhat further into the domain due to reduced depth-induced dissipation. The distribution of the mean wave period shows a decrease from about 7.5 s at the edge of the tidal flats in deeper water to about 3 s to 4 s in the Meldorf Bight. Although a constant wind field was imposed on the wave simulation, the swell energy has probably dissipated long before reaching the inner Meldorf Bight; therefore their effect on the average wave period has diminished. Figure 4c shows the distributions of the peak wave direction. The higher wave height values in the bight correspond to wave directions from 250° to 270°. Sensitivity tests were carried out to check which factors influence the performance of the model. There are no major differences by testing the numerical parameters, anyway more discrepancies are observed for the physical parameters. The most significant changes occur using distinct bathymetric variations as it can produce an effect on wave characteristics, especially in dynamic shallow water environments. The different scenarios extrapolated from the Beaufort wind-scale show that the output is influenced by the wind speed, differences are more marked with a decrease in the velocities, while crossing a certain level in the magnitude the disagreement is less evident. The fetch, represented by the wind direction, plays as well an important role.

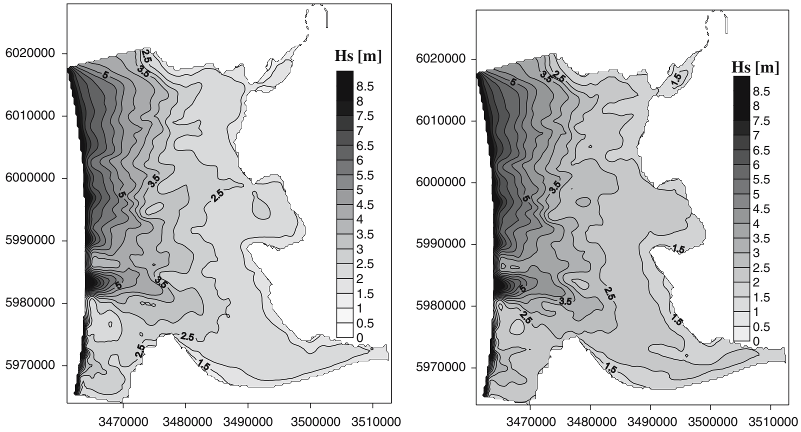
3.4 Design wave run-up and layout of the structure

Wave run-up values were determined on the basis of the Dutch procedure with some adjustments as proposed by Van Der Meer [13]. The equation proposed by Hunt (1959) [6] was adapted to fit to the local wave characteristics and dike geometries with coefficients established through field measurements and physical model test [3,4]:

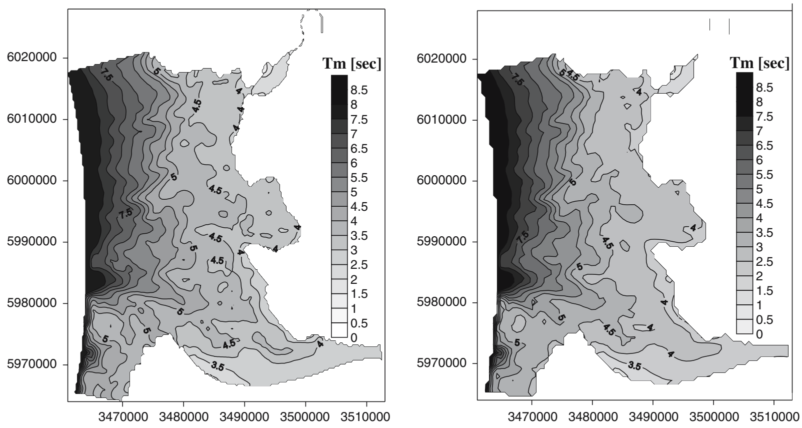
$$z_{98} = 1.6 \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_b \cdot \sqrt{\frac{g}{2\pi}} \cdot \sqrt{H_s} \cdot T_p \cdot \tan\alpha \qquad (1)$$

where:

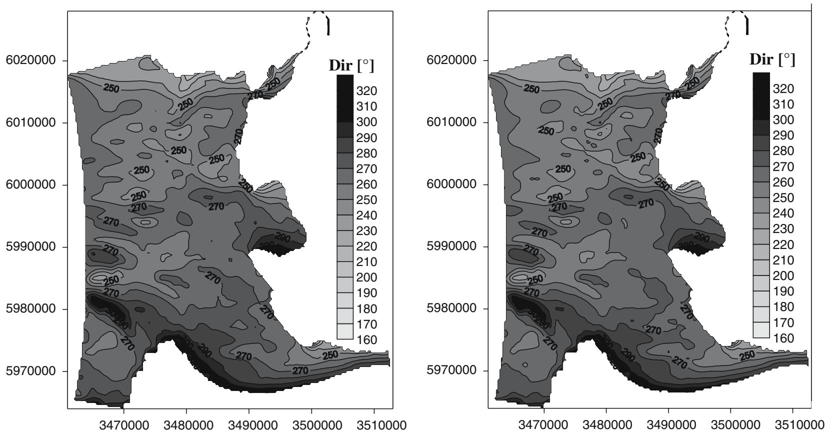
z_{98}	2% wave run-up level above still water line	[m]
γ_{b}	Influence factor for a berm	[–]



Significant Wave Height distribution (a, b)



Mean Wave period distribution (c, d)



Peak wave direction distribution (e, f)

a) Single Value Procedure **b)** Comparative Procedure

Fig. 4 SWAN simulation results for the wave parameters distributions in the Dithmarschen Bight

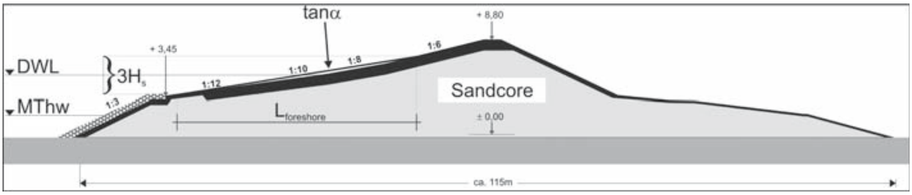


Fig. 5 Design dike geometry (DWL = Design Water Level; MThw = Mean High Water; H_S = significant wave height; $L_{\text{foreshore}}$ = length of the foreshore; $\tan\alpha$ = equivalent slope)

γ_f	Influence factor for a roughness elements on slope	[–]
γ_β	Influence factor for angled wave attack	[–]
g	Acceleration of gravity	[m/s^2]
H_s	Significant wave height at the toe of dike	[m]
T_p	Peak period	[s]
$\tan\alpha$	Equivalent slope	[–]

A subdivision of the Meldorf Bight into sections according to the Master Plan for the State of Schleswig-Holstein [9] was considered (Fig. 1c). Estimations of the wave run-ups were made for the cross-section of the dike shown in Fig. 5, which was considered representative for the dikes around the Meldorf Bight. A roughness element (γ_f) of 0.9 is assigned to dikes covered with grass, as it occurs in this case. An equivalent slope ($\tan\alpha = 0.109$) for determining the wave run-up was defined following the approach proposed by Niemeyer et al. [12]. The influence factor for the angle of wave attack is defined on the basis of an empiric relation presented in the EAK report (2002) [4]:

$$\gamma_\beta = 0.35 + 0.65\cos\beta \tag{2}$$

where the angle of wave attack β is defined as the angle between the direction of the propagation of the waves and the perpendicular to the long axis of the dike.

The presence of a berm is connected to slopes milder than 1:15, which does not occur for the case in question (Fig. 5). According to the available data on the dike's geometry, the absence of berm results in an influence factor $\gamma_b$which does not affect the wave run-up estimation.

The wave parameters (H_s and T_p) were taken from the results of the wave model simulations at observations points located at the toe of the dike. The peak values for the wave run-up levels are mainly found in the dike sections 5200 and 5100 (Figs. 1c and 6), where the maximum influence of wave action coming from the west can be expected. The peak values slightly decrease northward (moving anticlockwise) where the westerly waves less affect the coast in a gradually decreasing manner. The wave run-up is equal to zero in the sections 5000 and 4900, where the bight is relatively sheltered.

3.5 Design sea defence

The design crest level is assessed by summing up the design water level, the maximum level of wave run-up and an additional safety margin of 0.5 m. The actual dike height for the Meldorf Bight domain is 8.8 m (Fig. 5).

Table 2 summarizes the results for the two applied methods and which are the factors determining the required crest level. The model set-up according to the comparative procedure yielded a crest level of 7.3 m, which is 1.5 m lower than the existing dike height of 8.8 m. The single value procedure, with a resulting design water level of 6.5 m suggests a design

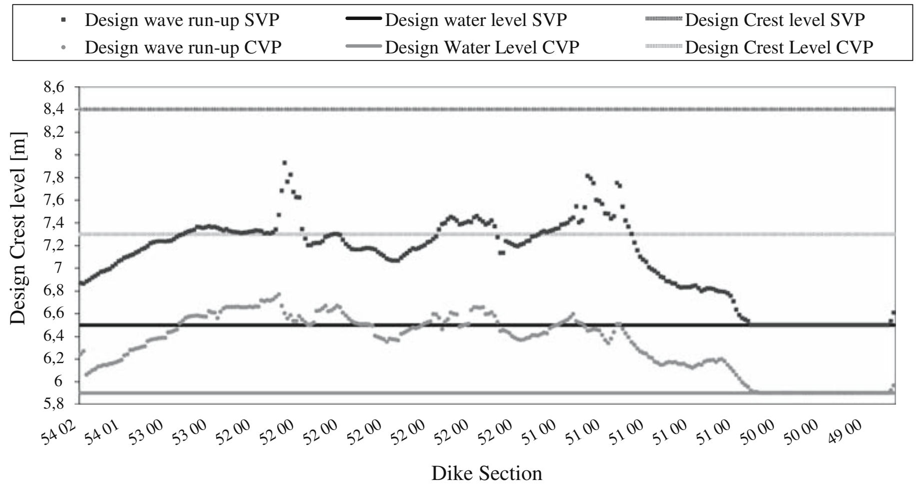


Fig. 6 Results for design crest levels for the Single Value Procedure (SVP) and the Comparative/Probabilistic Value Procedure (CVP) along the Meldorf Bight dikes (see Fig. 1c)

Table 2 Scenarios for the design sea defence

Simulation	Design water level (m)	Max. wave run-up (m)	Additional safety margin (m)	Crest level (m)
Comparative procedure	5.9	0.9	0.5	7.3
Single value procedure	6.5	1.4	0.5	8.4

dike height of 8.4 m, which is 0.4 m lower than the actual dike height. According to these results the dike appears not be endangered.

4 Discussion of the results

Clearly, the two procedures adopted to determine the design water level give different results for the design crest level. Looking at the wave propagation along the domain in both cases a significant portion of the wave energy may have been dissipated due to the limited local depths. Anyway the higher design water level of the single value procedure allows more wave energy to penetrate up to the vicinity of the dikes and therefore results in a higher design dike height. The 1.1 m difference in the design crest level between the single value and the comparative procedure could lead to a rash conclusion about the efficacy and the design risks that each procedure implies. It should be noted that the application of the presented procedures is based on several simplifications and serves solely an exemplary purpose. The attention should focus not on the quantitative difference for the two procedures, but on the initial stage of the study and on its feasibility. It should be taken into account which data were available. It was possible to apply a comparative procedure thanks to the results presented in the EAK report (2002) [4]. In some situation it might not be possible to have all the information necessary to perform a probabilistic analysis. In that case a valid alternative is offered by the single value procedure, where the single factors are more easily determined and the major assumption concerns the estimation of the wind set-up component. In this study a single storm event was considered.

Other issues that may have an impact on the results are identified by the sensitivity analysis which highlighted an influence of the bathymetric input on the wave transformation across the domain. Morphological changes in a tidal channel and flats system may affect the outcome in the medium-long term. Another input parameter that should be carefully evaluated for determining the wave run-up levels is the slope. The equivalent constant slope presented in this study is based on an empirical approach and it involves an extrapolation of the length of the foreshore, which should be checked with other data sets.

5 Conclusions

Independently from the procedure applied for determining the design dike height, the required safety level is largely fulfilled in the entire domain of the Meldorf Bight. The results showed that the existing dike, with a crest level of 8.8 m, is safe. However a proposal for a more economical design should be carefully evaluated, paying attention to the simplifications and assumptions made; the uncertainties encountered in this study that are mainly related to the use of the empirical formulae and to the estimation of the equivalent constant slope. Furthermore the morphological changes that may alter the foreshore during storm events and on the longer term under moderate conditions is another issue that needs further investigation.

The general recommendations for future development is to enhance the reliability of the hindcasted wave parameters at the toe of the dike through calibration and validation of the wave model using measurement data nearer to the coastline and to repeat the presented procedures in a more detailed manner. These investigations should include an accurate representation of the locally varying dike geometry and foreshore bathymetry. It is also recommended that a full scale investigation is carried out to determine the most optimal design dike heights that balance the allowable risks on the one hand and the acceptable costs for construction and maintenance on the other hand. In Germany the concept of achieving the same safety level for all the dikes is still valid. Mai et al. [10] suggested that the design process should include a cost/benefit analysis which should differentiate the risk zones within the hinterland according to the land use as already done in the Netherlands. The relevancy of this suggestion is confirmed by the results of the present study. In this perspective this preliminary study contributes to the estimation of the design parameters with a more systematic approach.

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