Reliability Assessment of the Falster Dike in Denmark

– Final –

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Contents

List of Figures ........................................................................................................................................... iii
List of Tables ............................................................................................................................................... v
Index of Notations and Symbols ............................................................................................................... vi
1 Introduction ............................................................................................................................................. 7
   1.1 Motivation and Objectives .................................................................................................................. 7
   1.2 Investigation Area ............................................................................................................................... 7
2 Data Processing and Boundary Conditions ............................................................................................ 8
   2.1 Topography and Bathymetry ............................................................................................................... 8
      2.1.1 Topography of Dike and Dunes .................................................................................................... 8
      2.1.2 Bathymetry .................................................................................................................................. 8
   2.2 Hydraulic Boundary Conditions ........................................................................................................ 9
      2.2.1 Water Level and Storm Surge Scenarios ...................................................................................... 9
      2.2.2 Wind Parameters ......................................................................................................................... 10
      2.2.3 Sea State .................................................................................................................................... 10
   2.3 Beach and Dune Erosion .................................................................................................................... 11
3 Methodology ........................................................................................................................................... 13
4 Results ....................................................................................................................................................... 14
   4.1 Sea State Simulation with SWAN ...................................................................................................... 14
   4.2 Simulation of Beach and Dune Erosion .............................................................................................. 17
      4.2.1 Numerical Model XBeach .......................................................................................................... 17
      4.2.2 Dune Erosion ............................................................................................................................... 18
      4.2.3 Wave Runup and Overwash ........................................................................................................ 20
      4.2.4 Dune Crossing ............................................................................................................................. 22
      4.2.5 Summary of Beach and Dune Erosion ....................................................................................... 22
   4.3 Deterministic Analysis of Wave Loading ........................................................................................... 23
      4.3.1 Assessment of Dike without Considering Dunes ....................................................................... 23
      4.3.2 Assessment of Dike with Berm ................................................................................................... 24
   4.4 Probabilistic Analysis of Wave Loading ............................................................................................ 27
      4.4.1 Assessment of Dike with Berm ................................................................................................... 27
      4.4.2 Probability Calculation ................................................................................................................ 28
5 Conclusions, Recommendations and Outlook ......................................................................................... 29
List of Figures

Fig. 1.1: Investigation area with topography and bathymetry (Data source: FDB, 2011a and FDB, 2011b) ................................................................................................................................. 7

Fig. 2.1: Detail of topography of Falster Dike with dunes - example of crossing no. 5 in Marielyst (Data source: FDB, 2011b) ........................................................................................................... 8

Fig. 2.2: Bathymetry of Baltic sea at the Falster Island (Data source: FDB, 2011a and Seifert et al., 2001) ................................................................................................................................. 8

Fig. 2.3: Development of water level $h_w$ of four storm surge scenarios ........................................ 9

Fig. 2.4: Maximum wind speed from directions 0° - 180° .................................................................. 10

Fig. 3.1: Classification of coastal protection system ......................................................................... 13

Fig. 4.1: Wave height $H_{\text{sig}}$ for each dike section in a distance of 100 m to the coastline ............................................................................................................................. 15

Fig. 4.2: Wave period $T_{\text{m01}}$ for each dike section in a distance of 100 m to the coastline ............................................................................................................................. 16

Fig. 4.3: Mean wave direction $\theta$ for each dike section in a distance of 100 m to the coastline ............................................................................................................................. 16

Fig. 4.4: $H_{\text{sig}}$ at the model boundary of the cross profile 9+000 .................................................. 17

Fig. 4.5: Initial and erosion profiles of each dune profile .................................................................... 19

Fig. 4.6: Wave runup and dune breach, 4th erosion profile dune section DIII and scenario D ............................................................ 21

Fig. 4.7: Wave runup at crossing Marielyst (left) and Falster south end (without dike behind) (right) ............................................................................................................................ 22

Fig. 4.8: Dune with asphalt cover layer at crossing no. 5 in Marielyst .............................................. 22

Fig. 4.9: Calculated maximum wave overtopping rates ordered by dike sections for each water level scenario with updated dike slopes [$q_{\text{max}} = 6.1 \text{l/(s} \cdot \text{m)}$] ........................................ 24

Fig. 4.10: Determination of effective berm length and berm width (dune section DI) (Scenario D) ............................................................................................................................ 25

Fig. 4.11: Berm factor $\gamma_b$ for each dike section (Scenario A) .......................................................... 25

Fig. 4.12: Berm factor $\gamma_b$ for each dike section (Scenario B) .......................................................... 25

Fig. 4.13: Berm factor $\gamma_b$ for each dike section (Scenario C) .......................................................... 26

Fig. 4.14: Berm factor $\gamma_b$ for each dike section (Scenario D) .......................................................... 26
Fig. 4.15: Berm width $B$ for each dune section.................................................................26

Fig. 4.16: Berm height $h_B$ for each dune section.............................................................26

Fig. 4.17: Calculated maximum wave overtopping rates ordered by dike sections for each water level scenario with consideration of completely eroded dunes [$q_{\text{max}} = 0.5 \text{l/(s·m)}$]........................................................................................................27

Fig. 4.18: Berm factor for each dune section (percentage rate of berm influence) (Szenario D) .........................................................................................................................28
List of Tables

Tab. 2.1: Water level scenarios .......................................................................................................................... 9
Tab. 2.2: SWAN boundary conditions at offshore border .................................................................................... 11
Tab. 2.3: Dune sections with stations marks ....................................................................................................... 12
Tab. 4.1: Characteristics of the SWAN model grids .......................................................................................... 14
Tab. 4.2: Results of probability calculation ....................................................................................................... 28
## Index of Notations and Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>DCA</td>
<td>Danish Coastal Authority (Kystdirektoratet)</td>
</tr>
<tr>
<td>DHI</td>
<td>Danish Hydraulic Institute</td>
</tr>
<tr>
<td>DMI</td>
<td>Danish Meteorological Institute (Danmarks Meteorologiske Institut)</td>
</tr>
<tr>
<td>DVR</td>
<td>Danish Vertical Reference 1990</td>
</tr>
<tr>
<td>FDB</td>
<td>Falster Dike Board (Det falsterske digelag)</td>
</tr>
<tr>
<td>GIS</td>
<td>Geographic information system</td>
</tr>
<tr>
<td>LIDAR</td>
<td>Light detection and ranging</td>
</tr>
<tr>
<td>LWI</td>
<td>Leichtweiss-Institute for Hydraulic Engineering and Water Resources</td>
</tr>
<tr>
<td>B</td>
<td>Berm width [m]</td>
</tr>
<tr>
<td>g</td>
<td>Gravity [m/s²]</td>
</tr>
<tr>
<td>h</td>
<td>Water depth [m]</td>
</tr>
<tr>
<td>h/L</td>
<td>Dispersion parameter [-]</td>
</tr>
<tr>
<td>hₘ</td>
<td>Berm height [m]</td>
</tr>
<tr>
<td>Hᵢ</td>
<td>Incident wave height [m]</td>
</tr>
<tr>
<td>Hₘ₀</td>
<td>Wave height, zeroth moment of wave spectrum [m]</td>
</tr>
<tr>
<td>Hₙ₀</td>
<td>Nominal wave height [m]</td>
</tr>
<tr>
<td>Hᵣ</td>
<td>Reflected wave height [m]</td>
</tr>
<tr>
<td>Hₛ</td>
<td>Significant wave height [m]</td>
</tr>
<tr>
<td>IOW</td>
<td>Leibniz Institute for Baltic Sea Research, Warnemünde</td>
</tr>
<tr>
<td>Kᵣ</td>
<td>Reflection coefficient [-]</td>
</tr>
<tr>
<td>Kₜ</td>
<td>Transmission coefficient [-]</td>
</tr>
<tr>
<td>L</td>
<td>Wave length [m]</td>
</tr>
<tr>
<td>q</td>
<td>Wave overtopping rate [l/(s·m)]</td>
</tr>
<tr>
<td>qₐdm</td>
<td>Admissible wave overtopping rate [l/(s·m)]</td>
</tr>
<tr>
<td>qₘₐₓ</td>
<td>Maximum wave overtopping rate [l/(s·m)]</td>
</tr>
<tr>
<td>SLR</td>
<td>Sea level rise</td>
</tr>
<tr>
<td>SWL</td>
<td>Still Water Level</td>
</tr>
<tr>
<td>T₀₁</td>
<td>Mean wave period, out of zeroth and first moment of wave spectrum [s]</td>
</tr>
<tr>
<td>Tₙ</td>
<td>Mean wave period [s]</td>
</tr>
<tr>
<td>Tₚ</td>
<td>Peak wave period [s]</td>
</tr>
<tr>
<td>u</td>
<td>Velocity [m/s]</td>
</tr>
<tr>
<td>vₘₐₓ</td>
<td>Velocity [m/s]</td>
</tr>
<tr>
<td>γₗₘ</td>
<td>Berm factor [-]</td>
</tr>
<tr>
<td>θ</td>
<td>Wave attack angle [°]</td>
</tr>
</tbody>
</table>

* e.g. | for example (exempli gratia) |
* i.e. | that is (id est) |
1 Introduction

1.1 Motivation and Objectives

The Falster Dike Board (FDB) has commissioned the Leichtweiß-Institute to perform a safety assessment of the Falster coastal defence system, hereafter called “Falster Dike”. The main objective is to assess the reliability of the Falster Dike, which includes (i) the probability of failure of the most critical dike and dune sections and (ii) suggestions of possible countermeasures, based on the results under (i).

The desk study comprises three distinct phases: (i) collation and analysis of data, including generation of missing data, (ii) preliminary analysis of hydraulic boundary conditions and wave loading (runup and overtopping), and (iii) reliability analysis and counter measures. In a preliminary report (no. 001) first results of the safety assessment were shown. The final results of this study under current and future hydraulic conditions are presented in this report.

First, the topography and bathymetry of the Falster coastal protection system are described (section 2.1) and the hydraulic boundary conditions are assessed (section 2.2). As for the coastal protection system consisting of dike and dunes, three cases are considered for the dike in combination with dunes (chapter 3): (a) dunes without dike; (b) dike without dunes; (c) combination of dike and dunes (by means of a berm structure).

The sea state conditions have been simulated by the numerical model SWAN (section 4.1). The behaviour of dunes during storm surges was simulated by the numerical model XBeach (section 4.2). Afterwards, deterministic and probabilistic approaches were performed using the boundary conditions of the Falster Dike (section 4.3 and 4.4). The final results of the reliability assessment of the Falster Dike are summarised together with conclusions and recommendations in chapter 5.

1.2 Investigation Area

The southeast coastline of the island of Falster is shown in Fig. 1.1 together with the topography and bathymetry of the investigation area. The coastal protection system consists of a dike and natural dunes at seaside. The dunes consists of sand material with $d_{50} = 0.2$ mm and partly covered by vegetation. For the safety assessment, the dike with a total length of 17.6 km was investigated. The dike consists of a sand core with a grass cover and the dike crest is used as a
footpath (see Appendix G). Similar sections of the dike were identified using the crest height and the average dike slope (see section 2.1.1 and Appendix A and Appendix I) so that the dike could be split into 24 sections which are all of different lengths but being considered homogeneous in itself (cf. Fig. 1.1).

2 Data Processing and Boundary Conditions

2.1 Topography and Bathymetry

2.1.1 Topography of Dike and Dunes

Topography data from a LIDAR (light detection and ranging) survey was provided by the Falster Dike Board (FDB, 2011b) and were processed with a geographic information system (cf. Fig. 2.1) to examine the following dike and dune characteristics:

- dike orientation (wave attack angle),
- dike height,
- seaward and shoreward dike slope,
- dune sand volume.

As mentioned before, the Falster Dike was divided alongshore into 24 dike sections from DS1 to DS24 (Appendix A). The dunes were categorized in three main dune sections from DI to DIII (Appendix A). A dune profile located south of the southern end of the Falster Dike and the dune crossing no. 5 in Marielyst were additionally analysed since they were believed to be the most critical cross profiles.

2.1.2 Bathymetry

The bathymetry of Falster was determined by echo sounder profiles (FDB, 2011b) and was extended by data from the Leibniz Institute for Baltic Sea Research Warnemünde, IOW (Seifert et al., 2001). These data were used to simulate the development of sea
states, to simulate beach and dune erosion, and to calculate wave runup and mean wave over-topping rates. The bathymetry map of Falster is shown in Fig. 2.2 with an interpolated resolution of 500 m for the coarse grid and 100 m of the nested fine grid.

2.2 Hydraulic Boundary Conditions

2.2.1 Water Level and Storm Surge Scenarios

One of the most important parameters to determine the safety of the coastal protection system is the water level (Tab. 2.1). Four different water level scenarios were considered. Therefore, two different return periods and two different sea level rise (SLR) scenarios were taken into account. The sea level rise was considered to amount 30 cm for a period until 2055-2065 and 100 cm for a period until 2090-2100. The return periods were determined by statistical analysis of the water level of the gauges at Heasnes and Gedser (DCA, 2007).

Tab. 2.1: Water level scenarios

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Water level [m]</th>
<th>Return period [1/years]</th>
<th>Sea level rise (SLR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.50</td>
<td>1/20</td>
<td>hw&lt;sub&gt;20&lt;/sub&gt; return period of 1/20</td>
</tr>
<tr>
<td>B</td>
<td>1.69</td>
<td>1/100</td>
<td>hw&lt;sub&gt;100&lt;/sub&gt; return period of 1/100</td>
</tr>
<tr>
<td>C</td>
<td>1.99</td>
<td>1/100 + SLR&lt;sup&gt;2&lt;/sup&gt;</td>
<td>hw&lt;sub&gt;2065&lt;/sub&gt; 1/100 water level + 30 cm (SLR)</td>
</tr>
<tr>
<td>D</td>
<td>2.69</td>
<td>1/100 + SLR&lt;sup&gt;2&lt;/sup&gt;</td>
<td>hw&lt;sub&gt;2100&lt;/sub&gt; 1/100 water level + 100 cm (SLR)</td>
</tr>
</tbody>
</table>

<sup>1</sup> based on the DVR90 reference level; <sup>2</sup> SLR = sea level rise

The storm surge in 1872 reached a maximum water level in the range of 2.5 m (DVR90) at the eastern coastline of Falster, but seems to include some decimeters of uncertainty due to inaccuracies in reporting and locations where information were collected (DCA, 2012 and DHI, 2006). Therefore, since scenario D considered a very similar water level, it was believed that scenario D also includes calculations for the 1872 storm surge.

In order to consider not only the peak water level during a storm in the Baltic Sea, a time history of water level during a storm was considered. Therefore, a scenario was chosen with a total duration of 12 hours and with a duration of the maximum water level of 3 hours. For this purpose, a linear increase and a linear decrease of the water levels within 4.5 hours were assumed (cf. Fig. 2.3).

![Fig. 2.3: Development of water level \(h_w\) of four storm surge scenarios](image)

The time history of the water level was considered by the numerical dune erosion model to account for the temporal development of the beach and dune erosion. For the simulation of
the sea state and for the calculation of wave runup and overtopping rates, a constant peak water level was assumed, hence assuming a conservative approach.

2.2.2 Wind Parameters

Wind data of the Denmark Meteorology Institute (DMI, 2011) for the stations ‘Gedser Havn’ and ‘Gedser Odde’ were analysed to determine the wind conditions at the island of Falster. Therefore, the wind speed was examined together with the wind direction (Kaste, 2011). The analysis of the wind data resulted in a maximum wind speed of 20.1 m/s in the range of wind direction of 0° (North) to 180° (South). For the calculation of sea states, the maximum wind velocity was set to the direction of 90° (East), again assuming the most conservative approach.

![Fig. 2.4: Maximum wind speed from directions 0° - 180°](image)

2.2.3 Sea State

The sea state was calculated starting from approximately 4 km offshore to the coastline. The following main parameters were computed (using a resolution of 100 m):

- significant wave height $H_{\text{sig}}$,
- peak wave period $T_p$,
- wave attack angle $\theta$.

The results of the sea state computations are given in section 4.1 and Appendix C for each scenario A, B, C and D.
For the analysis of the sea state in front of the coast of Falster, the numerical model ‘SWAN’ (Simulation WAVes Nearshore (SWAN, 2006)) was applied to simulate the specific local conditions. The results of this sea state model were used for the calculation of wave runup, wave overtopping rates, and for the simulation of dune erosion (cf. Section 4.1).

SWAN uses the spectral action balance equation to compute the evolution of wave growth. Terms of sources and sinks denote (SWAN, 2006):

- wave growth by the wind,
- nonlinear transfer of wave energy through three-wave and four-wave interactions,
- wave decay due to whitecapping,
- bottom friction,
- depth-induced wave breaking.

A JONSWAP-Spectrum was implemented as a boundary condition without any a priori restrictions of the spectrum. Therefore the significant wave height, the peak period and the wave direction are needed as an input. Further initial conditions are wind speed and wind direction (SWAN, 2006). For 2D-computations equidistant grids were defined. The optimal cell size was determined to be 50 to 100 m (Wahl, 2007). Therefore, a grid size of 100 m was chosen for the finer grid in front of the coastline. After the calculation of the sea state in a coarse grid of 500 m, the finer grid next to the coastline is nested into the coarser model. The approach and the results of the numerical simulations with SWAN are shown in section 4.1.

The sea state parameters in a water depth of 10 m were calculated by SPM (1984) and EAK (2002) using a fetch length and wind speed, details of which are shown in Kaste, 2011). These results are used as the boundary conditions for the numerical model. In Tab. 2.2 an overview of the SWAN boundary conditions for the coarse grid is shown.

**Tab. 2.2: SWAN boundary conditions at offshore border**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height $H_{\text{sig}}$</td>
<td>m</td>
<td>3.0</td>
</tr>
<tr>
<td>Wave period $T_p$</td>
<td>s</td>
<td>5.5</td>
</tr>
<tr>
<td>Wave angle $\theta$</td>
<td>°</td>
<td>90 (East)</td>
</tr>
<tr>
<td>Wind speed $U$</td>
<td>m/s</td>
<td>20.1</td>
</tr>
<tr>
<td>Wind direction</td>
<td>°</td>
<td>90 (East)</td>
</tr>
</tbody>
</table>

### 2.3 Beach and Dune Erosion

The calculation of dune and beach erosion was performed by the numerical model ‘XBeach’ (eXtreme Beach behaviour). The XBeach model simulates the behaviour of sandy coasts with given hydrodynamic parameters (wave height, wave period, water level, wind, currents, wave-current interaction etc.) and morphodynamic parameters (grain size, sediment transpor-
The numerical model XBeach performs well for dune erosion, overwash and breaching and was therefore selected suitable for the assessment of the Falster Dike reliability (Roelvink et al., 2010).

As one of the first models XBeach can calculate infragravity waves and wave group generated surf and swash motions which are found to be very important when it comes to dune erosion. Furthermore, XBeach provides an avalanching mechanism to simulate the slumping effects at the foredune during storm surge conditions (McCall et al., 2010). The computational simulation takes place in a 2DH environment. As input parameters an initial bathymetry and a grid system are defined. Hydrodynamic, morphodynamic and time parameters are set within the program. The main output is a time-varying bathymetry but also runup levels and temporal change of hydrodynamic and morphodynamic parameters are simulated.

The numerical model XBeach was developed by Unesco IHE, the Delft University of Technology, and Deltares, The Netherlands. XBeach was tested in several case studies as well as in experiments. It has been found that the physics of dune erosion, overwash, breaching, avalanching, swash motion, infragravity waves, wave groups, wave current interaction, as to name a few, during extreme storm conditions are reliably implemented in the model (Roelvink et al., 2010).

The dune profile located south of Falster Dike was calculated first as one of the probably critical cross sections. Therefore, the cross section of the dune and the bathymetry were prepared for the simulation of dune erosion. In addition, the dunes along the coastline were merged to three dune sections. The dune crossing no. 5 in Marielyst was separately assessed because of a very low dune capacity. In Tab. 2.3 five dune profiles are shown with dune sections, station marks, and the corresponding dike sections (see also Appendix A). The initial profiles of five dune sections are shown in Appendix D.

Tab. 2.3: Dune sections with stations marks

<table>
<thead>
<tr>
<th>Dune profile</th>
<th>Dune section</th>
<th>Station mark</th>
<th>Dike section</th>
</tr>
</thead>
<tbody>
<tr>
<td>North</td>
<td>D I</td>
<td>0+000 to 3+500</td>
<td>DS02</td>
</tr>
<tr>
<td>Marielyst</td>
<td>D II</td>
<td>3+500 to 9+500</td>
<td>DS13</td>
</tr>
<tr>
<td>Marielyst Crossing No. 5</td>
<td>D II</td>
<td>4+800</td>
<td>DS09</td>
</tr>
<tr>
<td>South</td>
<td>D III</td>
<td>9+500 to 17+600</td>
<td>DS18</td>
</tr>
<tr>
<td>South End</td>
<td>D III</td>
<td>17+600</td>
<td>-</td>
</tr>
</tbody>
</table>
3 Methodology

The coastal protection system at the Falster Dike consists of a dike with natural dunes in front. On this account a theoretical separation of the coastal protection system was performed. The classification of the Falster Dike structure is shown in Fig. 3.1.

In a first step, the hydraulic boundary conditions were determined by water level statistics, wind parameters, topography and bathymetry. These parameters have been used to preliminarily determine the reliability of the Falster Dike by only taking into account the dike (and not the dune) and considering wave overtopping simulations for the four different water level scenarios (Tab. 2.1) (Kaste, 2011).

In the second step, the numerical model SWAN was applied to simulate the sea state in the nearshore area for the four water level scenarios as defined in Tab. 2.1. Deterministic and probabilistic approaches were then applied for the safety assessment of this protection system. Wave runup and wave overtopping rates with regard to the local boundary conditions were determined. The wave runup is measured vertically from the still water level. Wave overtopping describes the mean discharge of waves over the dike crest per meter width in l/(s·m). Two maximum admissible wave overtopping rates were selected as threshold values for the stability of the dike in context of dike erosion. The admissible wave overtopping rates were chosen in regard to the Danish dike regulation with \( q_{adm1} = 0.5 \, l/(s\cdot m) \) since the dike core of the Falster Dike is not protected by a clay layer and in regard to the recommendations of EAK (2002) with \( q_{adm2} = 2.0 \, l/(s\cdot m) \). The EAK (2002) gives estimates for the mean admissible wave overtopping rate for structures at the German coast. The value of 2.0 l/(s·m)) is a safety limit to prevent dike erosion for German dikes with sand core, clay layer and grass cover.

For the analysis of wave runup and wave overtopping rates, three different cases of the combined coastal protection system (Fig. 3.1 a) were determined:

- dunes without considering the dike (Fig. 3.1 b),
- dike without considering the dunes (Fig. 3.1 c),
- dike with a berm (combination of dike and dunes) (Fig. 3.1 d).
**Dunes without considering the dike:** The dunes were separately assessed for the simulation of wave runup, overwash and dune erosion. The beach and dune erosion was simulated by the numerical model XBeach. In order to minimize the simulation efforts, the dunes were divided into three dune sections using the key parameters ‘dune height’, ‘dune capacity’, and ‘distance between dunes and dike crest’.

**Dike without considering the dunes:** The dunes in front of the dike were neglected to determine the wave runup and wave overtopping rates for the dike. For each of the 24 dike sections the dike parameters were determined. Calculations of wave runup and wave overtopping rates were performed according to the EurOtop Manual (EurOtop, 2007).

**Combination of dikes and dunes:** an updated dike geometry with a berm was applied for calculating wave runup and wave overtopping rates (Fig. 3.1 d). The dike and dune geometry was estimated from erosion simulations using XBeach and was simplified to a berm profile. This profile was assumed to no further erode and could therefore be used for wave run-up and overtopping simulations.

In a further step, the combined coastal protection system was assessed by a probabilistic approach. The failure probabilities were calculated by Monte-Carlo simulations with the software tool Palisade @Risk.

### 4 Results

#### 4.1 Sea State Simulation with SWAN

For the calculation of the sea state at the coastline and offshore, the numerical model SWAN was applied. A fine and a coarse grid were interpolated from depth profiles. The characteristics of these grids are shown in Tab. 4.1.

**Tab. 4.1: Characteristics of the SWAN model grids**

<table>
<thead>
<tr>
<th>Description</th>
<th>Origin (UTM 32U)</th>
<th>Cell count</th>
<th>Cell size</th>
<th>Width, Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse grid of the Baltic Sea determined by Seifert et al., 2001</td>
<td>X₀ = 690985  Y₀ = 6033167</td>
<td>25 · 94</td>
<td>500</td>
<td>12000, 47000</td>
</tr>
<tr>
<td>Fine grid interpolated of echo sounder profiles (DCA)</td>
<td>X₀ = 691046  Y₀ = 6051928</td>
<td>46 · 163</td>
<td>100</td>
<td>4600, 16300</td>
</tr>
</tbody>
</table>

1) refers to the upper left corner of the grid with UTM 32U coordinates
For the simulation of the wave conditions in the nearshore area, the fine grid was nested into a coarse grid to consider the offshore wave conditions. The wind (cf. section 2.2.2) and wave boundary conditions (cf. section 2.2.3) were applied to the eastern model border of the coarse grid (about 12 km offshore). In these grids, the wave parameters (e.g. $H_s$, $T_p$) were calculated for each cell, and the output was prepared for a longshore line with 100 m distance to the coastline. The value of 100 m was used as the relevant distance to define the input wave conditions for the dike and corresponds to the horizontal resolution of the grid which the numerical SWAN model uses for wave simulation. These results were used for the calculation of wave runup and wave overtopping rates. The sea state parameters at the offshore boundary of the fine grid were then used for the simulation of dune erosion. In Fig. 4.1, the maximum wave height $H_{\text{sig, max}}$ (determined as $H_{m0}$ for XBeach) is shown for each dike section (for dike sections see Appendix A).

![Maximum wave height $H_{\text{sig, max}}$](image)

**Fig. 4.1:** Wave height $H_{\text{sig}}$ for each dike section in a distance of 100 m to the coastline

It can be seen from Fig. 4.1 that the maximum significant wave height hardly exceeds 2.0 m at the shore which in most cases only occurs for Scenario D which is the highest water level (2.69 m DVR). The low values in dike section 7, 8 and 19, 20 result from a shallower bathymetry.

Fig. 4.2 shows the wave period $T_{m01}$ in a distance of 100 m to the coastline for each dike section (for dike sections see Appendix A). In comparison with the boundary condition of the wave period at a distance of ca. 11 km offshore ($T_{m1,0} = 5.5$ s) a slightly lower wave period was calculated nearshore.
It is surprising to see that Scenario C usually generates the highest wave periods. This is believed to result from the numerical calculation at the shore model boundary. In case of scenario C, there is a numerical issue about the land boundary condition. There are differences due to the interpolation of values between two grid cells and in regard to the water level in combination with the slope of the bathymetry. The water level $h_{w100} = 1.99$ in scenario C m is at the same height as the bathymetry at the model boundary. This is one explanation for differences in the maximum wave period. In sections 8 and 9, wave periods are larger than in the other dike sections due to the SWAN friction model. In general, the SWAN model underestimates the simulated wave period (Wahl, 2007).

The wave attack angle, defined as the wave direction perpendicular to the coastline is given in Fig. 4.3. The dike sections are again given in ascending order from North to South.

In most cases (for dike sections 10 to 24), the wave attack is almost perpendicular to the coast whereas the northern part of the Falster Dike ranging from dike section 1 to 7 is mainly influ-
enced by an oblique wave attack with a higher longshore component. It can be concluded, that in this case there exists a higher potential of sediment transport rates in longshore direction.

In the same way, the sea state parameters were extracted at the offshore boundaries of the echo sound profiles for numerical simulation of dune erosion with the model XBeach. For application of the dune erosion model XBeach, a relation of wave height \( H_{\text{sig}} \) to water depth \( d \) is used to calculate the temporal development of the wave height \( H_{\text{sig}} \) for each time step. In Fig. 4.4, the example of wave heights \( H_{\text{sig}} \) at the offshore boundary of dune section D II (station mark 9+000) are shown for each water level scenario (see also Appendix C).

![Wave height \( H_{\text{sig}} \) at the model boundary of the cross profile 9+000](image)

**Fig. 4.4:** \( H_{\text{sig}} \) at the model boundary of the cross profile 9+000

### 4.2 Simulation of Beach and Dune Erosion

#### 4.2.1 Numerical Model XBeach

The dunes were analysed without consideration of the dike (cf. Fig. 3.1 c). For the calculation of dune and beach erosion, wave runup and overwash, the numerical model ‘XBeach’ (eXtreme Beach behaviour, cf. section 2.3) was applied. The dunes along the coastline were merged to three dune sections with following marks: DI 0+000 to 3+500, DII 3+500 to 9+000, DIII 9+000 to 17+600 (cf. Appendix A). For each dune section, the particular worst case cross section was chosen in respect to dune volume and dune height (Fig. 4.5, initial profile 1, 3, and 4, respectively). Therefore a GIS analysis of high resolution topography was performed to compare the characteristic of dune cross sections (see Fig. 2.1). Additional dune cross sections in DII and DIII were also chosen. Both additional profiles are quite unique because they either consist of an asphalt crossing (crossing no. 5 in Marielyst) through the dike-dune system in section DII (Fig. 4.5, initial profile 2) or they represent the dune at the south end of the Falster dike (in dune section DIII) where the dike line is missing (see Fig. 4.5, initial profile 5; see also Appendix A).

The wave parameters calculated by SWAN are used as hydraulic input parameters for the numerical XBeach simulations. Values between \( H_{\text{int}} = 2.67 \text{ m} - 2.75 \text{ m} \) for the different cross sections located along the coastline were obtained. With respect to the different scenarios a rising water level of \( h_{w,100} = 1.69 \text{ m} \) in scenario B and \( h_{w,2100} = 2.69 \text{ m} \) in scenario D were es-
imated to analyse a high and an extreme storm surge event (cf. section 2.2). This was necessary to assess the safety of the combination of dike and dune. Scenario D was used to determine the maximum erosion profile for an assessment of the dike. A JONSWAP based wave spectrum with a wave period $T_p = 5.5$ and the simulated storm duration $t = 12h$ remains constant for all simulation runs. The morphological time acceleration factor “morfac factor” of 5 was applied to speed up the morphological time scale. It means to save the running time of the numerical simulation by an acceleration of the morphological processes of five times relative to the hydrodynamic timescale. In case of a morfac factor of 5 the model runs for 12 minutes each hour, during which the bottom changes per step are multiplied by a factor of 5. This saves a factor of 5 in computation time. This factor is a calibrated default parameter of XBeach to obtain realistic dune erosion profiles (Roelvink et al., 2010).

A sieve sample analysis (cf. Appendix H) of dune sand material next to crossing no. 5 in Marielyst yields a mean grain diameter $d_{50} = 0.2$ mm with $d_{90} = 0.3$ mm. The uniformity of the dune sand was determined to $C_u = 1.8$. These characteristics were taken into account for the material of beach, dune and dike as an input for XBeach.

4.2.2 Dune Erosion

Five different representative dune cross profiles were determined (section 4.2). For each cross profile, storm surge scenario B and D (section 2.2) were simulated by XBeach. The initial profile (green) and the erosion profiles for scenario B (blue) and D (red) are shown in Fig. 4.5, respectively where the y-axis represents the dune height and the x-axis represents the distance to the coastline as defined in Appendix A. Additionally, the erosion profiles are given in Appendix D in a larger version.
It can be seen from Fig. 4.5, that only small erosion volumes at the dune toe were observed for each cross section profile for storm surge scenario B. This means that the dunes are not eroded under these conditions and therefore do not lose their function.

In Scenario D, the erosion volume increases significantly as compared to scenario B for each cross section profile (Fig. 4.5). In case of the 4th erosion profile (Fig. 4.5, erosion profile 4)
the dune is completely eroded and a berm like structure has been generated. Once the dune is fully eroded only the dike line behind in combination with the created berm structure protects the hinterland from flooding. This is considered the worst case scenario and used for further calculations.

Regarding the two additional cross sections, the crossing in Marielyst (Fig. 4.5, erosion profile 2) and the cross section at the south end without the dike behind (Fig. 4.5, erosion profile 5), no significant changes with respect to the erosion volumes are observed. In the first case, hardly any erosion is visible which is due to the shallow dune front in this area. Therefore, wave runup is most likely the critical factor. In the latter case, approximately half of the dune cross section is eroded, leaving the other half of the dune to protect the hinterland from flooding.

4.2.3 Wave Runup and Overwash

Wave runup is calculated by an internal function of the XBeach program (Roelvink et al., 2010). For each time step, the last wet point on the beach is provided and interpreted as the actual wave runup. The maximum runup and the corresponding time can be found by analyzing the whole time series.

For each cross section this wave runup is calculated for scenario B (blue) and scenario D (red). All figures are given in Appendix E. For further analysis, only the worst case scenario (Fig. 4.5, erosion profile 4) from the erosion analysis and the two additional profiles in Marielyst (Fig. 4.5, erosion profile 2) and the dune profile without dike at the South of the Falster Dike (Fig. 4.5, erosion profile 5) were investigated.

Wave runup, overwash and dune breach for the worst case scenario D (Fig. 4.5, erosion profile 4) are shown in Fig. 4.6 for dune section DIII.
In Fig. 4.6, wave runup can be observed in front of the dune, followed by erosion and overwash of the dune. After a storm surge duration of 4.9 hours the dune breaches and wave runup occurs for the next 7.1 hours on the dike, resulting in erosion on the outer dike slope. The maximum wave runup is about 3.1 m which is still approximately 1.0 m lower than the dike crest. After the storm, when the water level is lowered again to mean water level, the eroded dune material is displaced and the dune is formed to a berm structure in front of the dike.

Wave runup for the two additional profiles in Marielyst (Fig. 4.5, erosion profile 2) and the dune without dike (Fig. 4.5, erosion profile 5) for scenario D are given in Fig. 4.7. In the case of Marielyst a maximum wave runup of 3.2 m is observed, approximately 1.0 m lower than the dike crest. This is of major importance for the crossing, since there is no dune protection in front and a relatively low slope. In the case of the dune at the Falster south end with no dike behind a maximum wave runup of 3.2 m is observed, which is approximately 1.8 m lower than the dune top. In both cases only wave runup and no overwash is observed, so that there will be no flooding of the hinterland.
Fig. 4.7: Wave runup at crossing Marielyst (left) and Falster south end (without dike behind) (right)

4.2.4 Dune Crossing

As an example of a dune crossing, the biggest crossing, no. 5 in Marielyst, was chosen. The XBeach model was used to determine the wave runup and erosion for this special geometry. Considering an asphalt cover layer at the crossing no. 5 in Marielyst, the erosion profile in Fig. 4.8 is determined. The simulation was run two times for the same profile (a) with a non erodible asphalt layer, and (b) with erodible sand material as bottom layer. The comparison of these two different materials was helpful to analyse dune erosion and wave runup at this dune crossing. The results are shown in Fig. 4.8. In the first case no erosion occurred in the area which was covered by asphalt but at the seaward end of the asphalt layer and in the latter case some erosion over a longer distance was observed.

Fig. 4.8: Dune with asphalt cover layer at crossing no. 5 in Marielyst

4.2.5 Summary of Beach and Dune Erosion

The assessment of dunes was performed by the numerical model XBeach with the storm surge scenarios B and D. No overwash and no wave overtopping was determined for the dunes in
storm surge scenario B and the dunes in all dune sections resisted the load of scenario B (see Fig. 4.5).

Considering the dune crossing in Marielyst and storm surge scenario D it was observed that increased erosion occurs at the seaward end of the asphalt layer. Only wave runup but no overwash (no wave overtopping respectively) was observed.

Considering the dune cross section at the South end of the Falster Dike, without a dike structure behind and storm surge scenario D, it was observed that approximately 50% of the dune was eroded. Only wave runup and erosion but no overwash was observed.

For the worst case cross section profile and scenario D (Fig. 4.5, erosion profile 4) the dune was fully eroded and the outer dike slope was eroded as well. After the storm surge, a berm-like structure in front of the dike was observed. For further analysis the combination of berm and dike as a worst case scenario will be investigated (section 4.3.2 and section 4.4.1).

### 4.3 Deterministic Analysis of Wave Loading

The following three cases discussed in chapter 3

- dunes without considering the dike (Fig. 3.1 b),
- dike without considering the dunes (Fig. 3.1 c),
- dike with a berm (combination of dike and dunes) (Fig. 3.1 d).

will be analysed with respect to wave runup and mean wave overtopping rates in this section. All four scenarios for water levels are considered for each of these three cases and will be discussed in the subsections below.

#### 4.3.1 Assessment of Dike without Considering Dunes

In this section, the dunes in front of the dike were neglected for the calculation of wave runup and wave overtopping rates. For each section, the dike slope on the seaward side is determined using the cross profile from the topography in this section. Calculations were performed according to the EurOtop Manual (EurOtop, 2007). The wave overtopping rates for scenario A, B, C and D are shown in Fig. 4.9 together with the two admissible wave overtopping rates of 0.5 l/(s·m) and 2.0 l/(s·m).
Dike section DS07, DS22 and DS24 exceeded the admissible wave overtopping rate of 2.0 l/(s·m) for the worst case scenario D. Mainly influenced by the outer dike slope, these dike sections are the most critical sections. The outer dike slope for these three sections is steeper compared to the other dike sections. For all other scenarios the maximum wave overtopping rate amounted to 0.6 l/(s·m) and were therefore always below 2.0 l/(s·m).

### 4.3.2 Assessment of Dike with Berm

To consider a combination of dike and dune, the dune erosion model was applied to three dune sections (D1, DII, DIII) as described in section 2.3. In case of erosion profile 4, dune section DIII (Fig. 4.5, erosion profile 4), the dune is completely eroded and a berm like structure has been generated. Therefore, the wave overtopping parameters were revised with respect to a berm in front of the dike for all three dune sections. In Appendix B, examples of the determination of the berm are shown. A berm in front of the dike reduces the wave runup and mean wave overtopping rates. Fig. 4.10 shows an example of determining the berm and dike parameters at dune section D1. The lower the berm factor the higher the efficiency and the lower the wave runup and wave overtopping rate. In case of a berm factor of 1.0 the berm has no influence to the wave runup and wave overtopping rate because of the distance between water level and berm crest.
Fig. 4.10: Determination of effective berm length and berm width (dune section DI) (Scenario D)

In Figures 4.11 to 4.14 the berm factor $\gamma_b$ is shown for scenario A, B, C and D for each dune section. Due to the changing water level for each of these scenarios, the berm factor is changing for each of them. Furthermore, it should be noted that the water level was kept constant for all calculations performed here. This is a conservative approach since the water level will change over time (as indicated in Fig. 2.3) so that the calculated results for wave runup and overtopping will only be valid during the maximum peak water level and are considered to be significantly lower during all other times.

Fig. 4.11: Berm factor $\gamma_b$ for each dune section (Scenario A)

Fig. 4.12: Berm factor $\gamma_b$ for each dune section (Scenario B)
In Fig. 4.15 the berm width is shown. From North to South the berm width is increasing by a larger distance between dune and dike. In addition, the berm height of each dune section is shown in Fig. 4.16.

Considering these berm factors the wave overtopping rates were calculated for each scenario.

In Fig. 4.17 the maximum wave overtopping rates (after completely eroded dunes) are shown. The mean dike slopes with an effective berm length were taken into consideration.
For the case of a completely eroded dune, a berm will stay in front of the dike. This case yields a much lower wave runup and mean wave overtopping rate. A larger distance between dune and dike also decreases the wave runup and wave overtopping rate because of a larger berm width and a shallower dike slope. With respect to the berm height, a value of the highest water level is the most effective berm height for decreasing the wave runup and wave overtopping rates. Dune section DIII has a lower lying theoretical berm height which leads to a higher potential wave runup and wave overtopping rate compared to dune section DI and DII (see Appendix B).

Through the combination of dike and dune, it is concluded, that there is a considerable extra safety of the dike by the dunes in front. Even if the dunes are entirely eroded, the berm in front of the dike reduces the wave overtopping rate. At the crossing no. 5 in Marielyst, the highest wave overtopping rate is calculated, with $q_{\text{max}} = 0.5 \text{l/(s\cdot m)}$. No wave overtopping rate exceeds the admissible values.

### 4.4 Probabilistic Analysis of Wave Loading

#### 4.4.1 Assessment of Dike with Berm

A probabilistic approach was applied with using the software tool Palisade @Risk by means of a Monte-Carlo-Simulation. In a first approach the dunes were neglected for the probabilistic calculation (cf. Fig. 3.1 c). A maximum failure probability $P_f = 0.43$ results for dike section DS18 when only wave overtopping is considered as failure mode and when applying a scenario with a water level of $h_{w,100} = 2.28 \text{ m}$ (Kaste, 2011). Furthermore, relatively high failure probabilities due to wave overtopping were determined for DS19, DS20, DS22 and DS24.

In the next step, in regard to section 4.3.2, the probabilistic approach was applied to the combination of dike and dunes (cf. Fig. 3.1 d) taking into account the failure mechanisms ‘wave
overtopping’, ‘overflow’ and ‘erosion of outer dike slope’. The fault tree with the main failure mechanisms is shown in Appendix F.

The berm factors $\gamma_b$ for each dune section related to the dike sections were taken into account for the scenario D. Fig. 4.18 shows the berm factors for scenario D for each dune section. The smaller the berm factor $\gamma_b$ the smaller the wave runup and the wave overtopping rate respectively.

![Berm factor $\gamma_b$ (Scenario D)](image)

**Fig. 4.18: Berm factor for each dune section (percentage rate of berm influence) (Scenario D)**

### 4.4.2 Probability Calculation

The case of a dike with completely eroded dunes is applied for the probabilistic analysis (cf. section 4.3.2). Therefore, there are changes in the geometry of the dike as compared to previous probability calculations and which have been already discussed in section 4.3.1. Tab. 4.2 lists results of probability calculations for all dike sections employing the berm factors from the fully eroded dunes. These probability calculations include additional failure modes (cf. Appendix F) than discussed before but also the previously considered ones (‘wave overtopping’, ‘overflow’ and ‘erosion of the outer dike slope’). A critical overtopping rate was defined as 0.5 l/(s∙m). A value of $P_f = 1.0$ is very high and a value of 0.0 is a very low failure probability.

**Tab. 4.2: Results of probability calculation**

<table>
<thead>
<tr>
<th>Probability</th>
<th>DS 1 – DS 24 Pf</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inundation</td>
<td>0.00E+00</td>
</tr>
<tr>
<td>Overflow</td>
<td>0.00E+00</td>
</tr>
<tr>
<td>Wave overtopping</td>
<td>0.00E+00</td>
</tr>
<tr>
<td>Dike breach</td>
<td>0.00E+00</td>
</tr>
<tr>
<td>Erosion seaward slope</td>
<td>0.00E+00</td>
</tr>
<tr>
<td>Erosion landward slope</td>
<td>0.00E+00</td>
</tr>
<tr>
<td>Failure inner dike</td>
<td>0.00E+00</td>
</tr>
<tr>
<td>Failure dike top</td>
<td>0.00E+00</td>
</tr>
</tbody>
</table>

It can be seen that there is no failure probabilities calculated for the aforementioned conditions which means that the failure probabilities are smaller than $P_f = 10^{-10}$. The failure probabilities for the failure modes shown in Tab. 4.2 are very low. This is mainly reasoned due to
the effect of the berm structure in front of the dike and due to the shallower outer slope between dike crest and dike toe.

5 Conclusions, Recommendations and Outlook

The reliability of Falster Dike as a coastal defence system was assessed, which includes the probability of failure of the most critical dike and dune sections. The objective is to determine suggestions of possible counter-measures based on the results of the safety assessment. The desk study comprised three distinct phases: (i) collation and analysis of data, including generation of missing data, (ii) preliminary analysis of hydraulic boundary conditions and wave loading (runup and overtopping), and (iii) reliability analysis and counter-measures.

In chapter 2 the data processing of topography, bathymetry and hydraulic conditions was described together with the used software tools. The safety assessment was performed with deterministic and probabilistic approaches by four storm surge scenarios and three different cases: (i) dunes without considering the dike, (ii) dike without considering the dunes, (iii) combination of dike and dunes by means of a seaside berm structure (cf. chapter 3).

At first it is concluded, there is no proper exceeding of the admissible wave overtopping rate \( q_{adm1} = 0.5 \text{ l/(s·m)} \). No urgent hazard exists in regard to the wave overtopping rates at that time. Under current conditions the safety of the coastal protection system is sufficient. In the context of future conditions with a sea level rise in 2090 to 2100, wave overtopping rates reaching the admissible value are predicted for one profile.

Based on the present state of the analysis of the Falster Dike the following is concluded:

- under the theoretical assumption only having the dike as flood protection, there is no significant wave overtopping for scenario A, B, C while scenario D may lead to wave overtopping rates up to 6.2 \( \text{l/(s·m)} \). However, since the dunes are placed in front of the dike, wave overtopping in scenario D will not occur. This again shows the importance of having dunes in front of the dike.

- the combination of dune and dike adds a significant extra safety so that the safety is sufficient and there is no immediate need for any countermeasures to be installed.

Potential weak spots for the worst case scenario D:

- The dune crossings, especially crossing no. 5 in Marielyst is a potential weak spot because of beach erosion at the upstream end of the asphalt cover layer. In general, crossings are potential weak spots due to the missing extra safety of the dunes in front of the dike. Nevertheless, there is no urgent need of countermeasures until there is no displacement of the beach and dune at crossing no. 5.

- The dune profile in section DIII at the south end of the Falster Dike is a potential weak spot due to a missing dike behind the dune. However, the dune is stable even under an
extreme storm surge event of scenario D. The dunes at the south end of the Falster Dike should be maintained to retain the coastal protection system.

- Dike sections DS1, DS7, DS8, DS16, DS17, DS18, DS20, DS21, DS22 and DS24 are one of the potential most critical dike sections because of the relative steep dike slope compared to the other sections. However, since the dunes are placed in front of the dike, wave overtopping in scenario D will not occur.

Recommendations

The assessment of the coastal protection system of South Falster results in following recommendations: (i) no immediate action in dike reinforcement is required, (ii) constant maintenance of dike and dunes and (iii) consideration of improving the dune at the south end is recommended.
Literature

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DMI (2011): Wind data of Gedser Havn (01.01.06-31.08.11) and Gedser Odden (01.01.94-31.08.11), Danmarks Meteorologiske Institut (DMI).
FDB (2011a): Data of echo sounder measurements at east coast of Falster Island, Danish Coastal Authority, Lemvig, Denmark. unpublished.

Internet:
World Topographic Map:
http://services.arcgisonline.com/ArcGIS/rest/services/World_Topo_Map/MapServer
Appendix
Appendix A: Overview of Dike and Dune Sections

Overview of dune sections (DI - DIII) and dike sections (DS1 - DS24)
Appendix B: Determination of the Berm Factor

Determination of the berm factor at dune section DI (Szenario D)

Determination of the berm factor at dune section DII (Szenario D)
Determination of the Berm Factor at Dune Section DIII

Berm at profile 10730 (dune section DIII) (Scenario D)

\[ \gamma_{b} = 0.78 \]
\[ L_{Berm} = 124.50 \text{ m} \]
\[ h_{Berm} = 2.25 \text{ m} \]
\[ h_{w2100} = 2.69 \text{ m} \]
\[ H_{s} = 1.95 \text{ m} \]
\[ L_{Slope} = 147.8 \text{ m} \]
\[ \tan a = 0.053 \]

berm width
effective berm length
1.5 \( H_{s} \)
effective slope length

\[ z_{initial} \quad \text{and} \quad z_{erosion} \]
Appendix C: Results of Sea State Simulation (SWAN)

Szenario D: Wave height, wave attack angle and maximum overtopping rate in case of dike without considering dunes

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Details</th>
<th>Wave attack angle</th>
<th>Maximum ovoltopping rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>return period 1/20</td>
<td>- water-level hw_{100} = 1.50 m</td>
<td>0.1 l/(s m)</td>
</tr>
<tr>
<td>B</td>
<td>return period 1/100</td>
<td>- water-level hw_{100} = 1.69 m</td>
<td>0.2 l/(s m)</td>
</tr>
<tr>
<td>C</td>
<td>return period 1/100 (hw_{100} = 1.69)</td>
<td>- including sea-level rise of 0.30 m</td>
<td>0.6 l/(s m)</td>
</tr>
<tr>
<td>D</td>
<td>return period 1/100 (hw_{100} = 1.69)</td>
<td>- including sea-level rise of 1.00 m</td>
<td>6.2 l/(s m)</td>
</tr>
</tbody>
</table>
Appendix D: Simulated Dune Erosion Cross Profiles

1. Erosion Profile (Dune Section D1, Profile 1230)

Numerical Input:
- Scenario B: \( h_{w100} = 1.69 \text{m}, \ H_{m0,\text{max}} = 2.67 \text{m}, \)
- Scenario D: \( h_{w2100} = 2.69 \text{m}, \ H_{m0,\text{max}} = 2.71 \text{m}, \)
  \( T_p = 5.5 \text{s}, \)
  Simulated Time = 12h,
  \( \text{morfac} = 5, \)
  Spectrum: Jonswap

2. Erosion Profile (Dune Section DII, Profile 4803)

Numerical Input:
- Scenario B: \( h_{w100} = 1.69 \text{m}, \ H_{m0,\text{max}} = 2.67 \text{m}, \)
- Scenario D: \( h_{w2100} = 2.69 \text{m}, \ H_{m0,\text{max}} = 2.74 \text{m}, \)
  \( T_p = 5.5 \text{s}, \)
  Simulated Time = 12h,
  \( \text{morfac} = 5, \)
  Spectrum: Jonswap
3. Erosion Profile (Dune Section DII, Profile 8730)

Numerical Input:
Szenario B: \( hw_{100} = 1.69m \), \( H_{max} = 2.70m \),
Szenario D: \( hw_{2100} = 2.69m \), \( H_{max} = 2.75m \),
\( T_p = 5.5s \),
Simulated Time = 12h
morfac = 5
Spectrum: Jonswap

Falster Dike, Section DII, Dune Profile 8730

4. Erosion Profile (Dune Section DIII, Profile 10730)

Numerical Input:
Szenario B: \( hw_{100} = 1.69m \), \( H_{max} = 2.71m \),
Szenario D: \( hw_{2100} = 2.69m \), \( H_{max} = 2.76m \),
\( T_p = 5.5s \),
Simulated Time = 12h
morfac = 5
Spectrum: Jonswap

Falster Dike, Section DIII, Dune Profile 10730
5. Erosion Profile (Dune Section DIII, Profile 17730)

Numerical Input:

Szenario B: \( h_{w_{100}} = 1.69 \text{m}, \)
\( H_{m0,max} = 2.62 \text{m}, \)

Szenario D: \( h_{w_{210}} = 2.69 \text{m}, \)
\( H_{m0,max} = 2.68 \text{m}, \)
\( T_{p} = 5.5 \text{s}, \)
Simulated Time = 12h
\( \text{morfac} = 5 \)
Spectrum: Jonswap
Appendix E: Runup Level for Each Scenario

1. Profile 1230 wave runup

![Wave Runup Scenario B (Dune Section DI, Profile 1230)](image)

**Numerical Input:**
- Scenario B: $h_{w100} = 1.04m$, $H_{m0,max} = 2.671m$, $T_p = 5.5s$
- Simulated Time = 12h
- $\text{morfac} = 5$
- Spectrum: Jonswap
- Max Runup: 4.35m
- Time Max Runup: 6.98h

2. Profile 4802 wave runup

![Wave Runup Scenario D (Dune Section DI, Profile 1230)](image)

**Numerical Input:**
- Scenario D: $h_{w2100} = 2.69m$, $H_{m0,max} = 2.714m$, $T_p = 5.5s$
- Simulated Time = 12h
- $\text{morfac} = 5$
- Spectrum: Jonswap
- Max Runup: 4.99m
- Time Max Runup: 7.91h

3. Profile 4803 with cover layer wave runup

![Wave Runup Scenario B (Dune Section DI, Profile 1230)](image)

**Numerical Input:**
- Scenario B: $h_{w100} = 1.69m$, $H_{m0,max} = 2.672m$, $T_p = 5.5s$
- Simulated Time = 12h
- $\text{morfac} = 5$
- Spectrum: Jonswap
- Max Runup: 2.18m
- Time Max Runup: 5.88h

![Wave Runup Scenario D (Dune Section DI, Profile 1230)](image)

**Numerical Input:**
- Scenario D: $h_{w2100} = 2.69m$, $H_{m0,max} = 2.736m$, $T_p = 5.5s$
- Simulated Time = 12h
- $\text{morfac} = 5$
- Spectrum: Jonswap
- Max Runup: 3.216m
- Time Max Runup: 6.97h
4. Profile 8730 wave runup

5. Profile 10730 wave runup
6. Profile 17730 wave runup
Appendix F: Fault Tree of Probabilistic Approach

Fault tree with main failure mechanisms
Appendix G: Photo of the Falster Dike
Appendix H: Sieve Analysis of Dune Material

Sample of dune sand
Marielyst, Falster
03.02.2012
N 54.690249
E 11.970204

$d_{50} = 0.20$ mm
$d_{60} = 0.22$ mm
$d_{10} = 0.12$ mm

$U = d_{60}/d_{10} = 1.8$
Appendix I: Dike Crest Height with Longitudinal Profile from North to South