Breaching of Coastal Dikes

Status Report of Activity 2.4

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Summary:

This report gives a brief summary of the general theory of breaching processes and presents an introduction of available breach models for sea dikes based on a literature review. Moreover, the application of breach models to the pilot site using one example of an estuarine dike in Hamburg is described. Therefore, the methodology and results simulating wave overtopping-induced erosion of grassed inner sea-dike slopes are presented.
ABSTRACT

The report provides an overview about breaching processes and models. At first the boundary conditions for breaching are described and the causes of breach initiation are explained. Moreover, relevant hydrodynamic and morphodynamic processes and the according available models are listed. For more detailed information about the state of the art review of breaching processes and models a listing of references is given.

Moreover, the report introduces existing models for dike breaching that have been developed at Leichtweiß-Institute. Therefore, based on the different hydraulic loading it is distinguished between dike breaching initiated by overflow and wave overtopping and dike breaching initiated by breaking wave impacts.

Furthermore, one example application is presented. Using the example of an estuarine dike in Hamburg breach models for simulating wave overtopping-induced erosion of grassed inner sea-dike slopes are applied. The methods and results are described.
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1 Introduction

1.1 Background and Motivation

In the past, storm surges have frequently led to major damages, also along the German coastline. Due to climate change it may be expected that the risk of flooding will increase in the coming decades. In order to enhance the knowledge related to the predictions of extreme storm surges and the assessment of the associated risk, the joint research project XtremRisK was initiated. The general aim of the project is to quantify the overall flood risk under present and future climate change conditions for an open coast (Island of Sylt, North Sea) and an estuarine urban area (Hamburg, Germany) using an integrated risk analysis approach (Oumeraci (2004)).

One of the major tasks within XtremRisK subproject (SP) 2 is to determine the loading and the stability of all components of flood defence systems. Therefore, a reliability analysis of coastal flood defences is carried out, i.e. the determination of the failure probability (and thus the flooding probability) which is the first component of flood risk (defined as the product of probability $P_f$ and related consequences $E(D)$ here).

Moreover, the total failure of the defence components includes the breach and breach development of flood defence structures such as sea dikes and natural barriers such as dunes. As a result, the breach development can be described in time with specifications on breach initiation, breach duration, and the final breach width and depth. The results (outflow hydrograph and water depth at the breach) will be used as input parameters for the simulation of the flood wave propagation and inundation and the related damages in the study areas.

1.2 Objectives

The objectives of this report are to briefly summarize the general theory of breaching processes and to introduce available breach models for sea dikes based on a literature review. Moreover, it is intended to describe the application of one breach model to the pilot site using one example of an estuarine dike in Hamburg. For this purpose the methodology and preliminary results simulating wave overtopping-induced erosion of grassed inner sea-dike slopes are described.

1.3 Methodology

The main tasks of XtremRisK subproject (SP) 2 in terms of a reliability analysis and the modelling of the breaching of coastal flood defences within SP 2 are summarised in a flow chart (see Fig. 1)
As shown in the flow chart (Fig. 1) the extreme storm surge scenarios developed in SP 1 are used as input parameters in SP 2. Moreover, a description of the flood defence structures was already performed in SP 2 (Naulin et al. (2009)). For the reliability analysis a literature review of failure mechanisms and existing limit state equations was carried out (Naulin et al. (2010)).

The focus of this report is set on the modelling of breaching which will be performed for the flood defence structures such as sea dikes and natural barriers such as dunes. As a result, the breach development can be described in time with specifications on breach initiation, breach duration, and the final breach width and depth. The results (outflow hydrograph and water depth at the breach) are to be used in SP 3 and SP 4 as input parameters for the simulation of the flood wave propagation and inundation and the related damages in the study areas.

Hence, this report is structured as follows: At first in chapter 2 a short introduction on general breaching processes and calculation methods is given. Therefore, fundamental literature sources are given by D’Eliso’s Ph.D. thesis (D’Eliso (2007)) about breaching of sea dikes initiated by wave overtopping, Stanczak’s Ph.D. thesis (Stanczak (2008)) about breaching of sea dikes initiated from the seaside by breaking wave impacts, and Morris et al. (2009) who gives a state of the art review of breaching processes. Since the breaching processes are highly complex it is not indented to give detailed descriptions within this report. Rather, an overview is provided and for further explanations it is referred to the above mentioned references.

Afterwards, in chapter 3 available breaching models are briefly introduced.
In chapter 4 the application of one breach model to an estuarine dike of the pilot site in Hamburg is presented. Therefore, the methodology including the methods to choose the relevant dike breach model and a listing of the chosen input parameters is described. Furthermore, preliminary results of the fixed-bed overtopping simulation and the breach initiation are presented and discussed.

Finally, at the end of the report in chapter 5 a summary with the main results is given. Moreover, the further steps of the breaching modelling are described.
2 Breaching Processes

The process of breaching is the result of the interaction between three elements, i.e. water, soil and structure. Therefore, the breaching process differs according to hydraulic loading, material type and state as well as embankment condition. This chapter provides a brief overview of the breaching process which comprises a complex interaction between hydraulic, geotechnical and structural processes.

2.1 References for State of the Art Reviews

During the last 50 years there have been many initiatives trying to analyse and solve how to predict embankment breach initiation and growth. With the rapid development of personal computers since the 1980’s, many more recent efforts have included the development of various computer models. Many of the models developed have been based upon physical modelling or case study data (Morris et al. (2009)).

There are several documents available as summarised in Tab. 1 which provide a detailed overview of the state of the art with regards to embankment breach initiation, formation processes, and breach development. For example a general overview of breaching processes is given by Morris et al. (2009), breaching of sea dikes initiated by wave overtopping is described and analysed by D'Eliso et al. (2005)/ D'Eliso (2007) and breaching of sea dikes initiated from the seaside by breaking wave impacts is reviewed and analysed by Stanczak et al. (2008b)/ Stanczak (2008). The following sections aim to give a very brief summary of the breaching processes. For detailed information it is referred to the stated references.

Tab. 1: References for state of the art reviews of breaching of coastal dikes

<table>
<thead>
<tr>
<th>Author</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>D'Eliso et al. (2005),</td>
<td>State of the art review and Ph.D. thesis of breaching of sea dikes initiated</td>
</tr>
<tr>
<td>D'Eliso (2007)</td>
<td>by wave overtopping</td>
</tr>
<tr>
<td>Stanczak et al. (2008b),</td>
<td>State of the art review and Ph.D. thesis of breaching of sea dikes initiated</td>
</tr>
<tr>
<td>Stanczak (2008)</td>
<td>from the seaside by breaking wave impacts</td>
</tr>
<tr>
<td>Morris et al. (2009)</td>
<td>Overview of breaching processes, a review of the current state of the art</td>
</tr>
<tr>
<td></td>
<td>and recommendations as to priorities for ongoing and future research and</td>
</tr>
<tr>
<td></td>
<td>development</td>
</tr>
</tbody>
</table>

2.2 Boundary Conditions

As boundary conditions information about the morphological and hydraulic conditions is needed.

2.2.1 Morphological Boundary Conditions

The morphological conditions can be described by the geometrical parameters and the material characteristics. The most important geometrical parameter that defines the design of a
dike is the construction level of the dike crown. Moreover, specifications of the dike profile with slopes and widths as well as information on the thickness of the cover layers such as clay and grass are needed.

According to the design of the dike, e.g. sandy core with clay layer and grass cover, material characteristics, for all layers the material characteristics are needed. The most important ones for soil are (TAW (1999)): mean density, shear strength: settlement properties, and permeability.

The material type has a significant effect on the breach behaviour. Fig. 2 shows a broad division of breach behaviour by material type where three categories of material typically used to build flood embankments or dams are distinguished: non cohesive fill, cohesive fill and rock fill.

![Fig. 2: Broad division of breach behaviour by material type (Morris et al. (2009))](image)

In the following paragraphs a description of the breach behaviour depending on the material type is given according to Morris et al. (2009):

Non Cohesive Fill, such as sand, will erode relatively quickly. The material is broadly removed through progressive surface erosion.

Cohesive Fill, such as clays, will limit the rate of erosion in comparison to non cohesive fill. In addition, the process of erosion can differ significantly. Cohesive fills tend to erode through a process of head cutting. This process leads to the creation of steps in the eroding face of the embankment which progressively erode upstream. As the head cuts progress, they tend to merge into fewer, but more significant steps until erosion through the crest and upstream face occurs.
Rock fill, can vary in grading from relatively fine material through to large rocks. The finer material can behave as a non cohesive fill material. As larger and larger particles (rocks) are used, a transition occurs whereby the interlocking nature of the rock fill can begin to significantly affect the rate at which material can be eroded during the breaching process.

2.2.2 Hydraulic Boundary Conditions

The main hydraulic boundary conditions are extreme water levels and wave action. Those two conditions are directly related - as the mean water level rises, larger waves occur and the dike is subject to stronger wave impact. Moreover, the duration of a storm surge is important.

2.3 Causes of Breach Initiation

The causes of breach initiation depend on the structure of the dike and on the hydraulic and morphological boundary conditions. In general several causes for the initiation and formation of a breach can be distinguished, e.g. wave overtopping, overflow, wave impact and seepage (Fig. 1). The identification of causes and failure modes that may induce an initial breach can be assessed by a fault tree analysis (Kortenhaus (2003)).

Fig. 1: Causes of breach initiation (D'Eliso (2007))

The main failure mechanisms which may lead to the breaching of sea dikes are (TAW (1999)):

- Erosion and sliding initiated from the landside by wave overtopping and overflow (see D'Eliso (2007));
- Erosion of seaward slope resulting from breaking wave impacts and the flow induced by wave run-up and run-down (see Stanczak (2008))
2.4 Hydrodynamic Processes and Models

Depending on the causes of breach initiation different hydrodynamic processes such as breaking wave impact on the outer dike slope and/ or overflow and wave overtopping on the inner dike slope have to be considered and modelled.

2.4.1 Breaking Wave Impact (Outer Dike Slope)

The loading on the outer dike slope due to breaking wave impact is essentially described by (i) the impact pressure and (ii) the flow velocity field associated with the wave run-up and run-down. A number of possibilities to calculate the impact pressures and flow velocities are available. Tab. 2 summarizes some possibilities to calculate wave impact pressures at the dike. It is distinguished between empirical and semi-analytical formulae and numerical models respectively. Moreover, it is stated if the associated hydrodynamic model is implemented in a breach model, e.g. Stanczak’s preliminary model (PM) or detailed model (DM) respectively. The description of available breach models can be found in chapter 3.

Tab. 2: Excerpt of existing models for wave impact pressures

<table>
<thead>
<tr>
<th>Model type</th>
<th>Process/ Location</th>
<th>Description/ Remarks</th>
<th>References</th>
<th>Implementation in Breach Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave breaking</td>
<td></td>
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</tr>
<tr>
<td>Empirical formulae</td>
<td>Impact Pressures on outer dike slope</td>
<td>Location of wave breaking impact</td>
<td>Schüttrumpf (2001)</td>
<td>Stanczak, PM</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Spatial distribution of impact pressures</td>
<td>Stive (1983)</td>
<td>Stanczak, PM</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Breaker incidence angle</td>
<td>Führböter (1966)</td>
<td>Stanczak, PM</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum impact pressures</td>
<td>Führböter (1966)</td>
<td>Stanczak, PM</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Energy dissipation using SBEACH</td>
<td>Larson &amp; Kraus (1989)</td>
<td>Stanczak, DM</td>
</tr>
</tbody>
</table>

2.4.2 Wave Overtopping and Overflow (Inner Dike Slope)

Wave overtopping and overflow at a dike are normally determined by empirical formulas with the associated mean discharge (EurOtop (2007)). However, a mean value is not sufficient to assess the erosion. Therefore, both flow velocity and flow depth along the dike profile are needed. Tab. 3 summarizes some possibilities to calculate wave overtopping and overflow at the dike. It is distinguished between empirical and semi-analytical formulae and numerical models respectively.
Tab. 3: Excerpt of existing models for wave overtopping and overflow

<table>
<thead>
<tr>
<th>Model type</th>
<th>Process/ Location</th>
<th>Description/ Remarks</th>
<th>References</th>
<th>Implementation in Breach Model</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Wave Overtopping</strong></td>
<td></td>
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</tr>
<tr>
<td>Empirical and semi-analytical formulae</td>
<td>Wave overtopping on inner slope along a plane slope</td>
<td>Vertical velocity, viscous effects neglected</td>
<td>Schüttrumpf &amp; Oumeraci (2005)</td>
<td>D’Eliso, PM</td>
</tr>
<tr>
<td></td>
<td>Wave overtopping on inner slope in the impinging jet region</td>
<td>Flow is still calculated over a plane slope</td>
<td>Schüttrumpf &amp; Oumeraci (2005)</td>
<td>D’Eliso, PM</td>
</tr>
<tr>
<td>Numerical Model</td>
<td>Wave overtopping at outer slope and crest</td>
<td>2D RANS-VOF Model Cobras for regular waves</td>
<td>Liu &amp; Lin (1997)</td>
<td>D’Eliso, DM Stanczak, DM</td>
</tr>
<tr>
<td><strong>Wave Overtopping and Overflow</strong></td>
<td></td>
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<tr>
<td></td>
<td>Flow through the breach channel</td>
<td>Steady non uniform flow</td>
<td>Hassan (2002), Rozov (2003)</td>
<td>D’Eliso, PM Stanczak, PM</td>
</tr>
<tr>
<td>Numerical Model</td>
<td>Flow discharge and velocity during overflow</td>
<td>nonlinear shallow water equations</td>
<td>Tuan (2007), Tuan &amp; Oumeraci (2010)</td>
<td>BREID</td>
</tr>
</tbody>
</table>

2.4.3 Water Infiltration

Water infiltration and seepage flow depends on three main factors: (i) mean water level, (ii) waves (wave set-up; run-up, run-down, and overtopping), (iii) rain. Water infiltration resulting from the high mean water level is the most important factor for dike breaching. Therefore, the influence of waves and rain are normally neglected (D’Eliso (2007), Stanczak (2008)). The available methods for the calculation of water infiltration are shown in Tab. 4.

Tab. 4: Excerpt of existing models for water infiltration

<table>
<thead>
<tr>
<th>Model type</th>
<th>Process/ Location</th>
<th>Description/ Remarks</th>
<th>References</th>
<th>Implementation in Breach Model</th>
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<tbody>
<tr>
<td><strong>Water infiltration</strong></td>
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<tr>
<td></td>
<td>1D seepage flow</td>
<td>Darcy’s flow</td>
<td>Mishra &amp; Singh (2005), Wang et al. (2002), see D’Eliso et al. (2006)</td>
<td>D’Eliso, DM</td>
</tr>
<tr>
<td>Numerical model</td>
<td>water infiltration</td>
<td>Numerical solution of Richard equation combined with Darcy’s law</td>
<td>As described in Dingman &amp; Lawrence (2008)</td>
<td></td>
</tr>
</tbody>
</table>
2.4.4 Breach flow

The flow through the breach channel including the flow velocity and depth along the entire dike profile are calculated using the equations for steady, non-uniform free surface flows (Tab 5).

<table>
<thead>
<tr>
<th>Model type</th>
<th>Process/ Location</th>
<th>Description/ Remarks</th>
<th>References</th>
<th>Implementation in Breach Model</th>
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</table>

2.5 Morphodynamic Processes and Models

2.5.1 Outer Dike Slope

The morphodynamic processes cover the breach formation and growth from the initiation to the final breach, including grass, clay and sand erosion and instability. Erosion and instability processes occurring simultaneously or subsequently may result in a dike breaching. For a dike breaching initiated from the seaside, one may distinguish the following processes (Stanczak (2008)):

- grass erosion
- clay erosion, including the shear failure in water-filled cracks and clay undermining
- erosion and washing-out of the sand core, including the mass instability

These processes with information on available models are summarized in Tab. 6.

<table>
<thead>
<tr>
<th>Model type</th>
<th>Process/ Location</th>
<th>Description/ Remarks</th>
<th>References</th>
<th>Implementation in Breach Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breach Initiation: Grass Erosion</td>
<td>Wave impact theory, Grass erosion</td>
<td>Vertical succession of sod properties</td>
<td>TAW (1997)</td>
<td>Stanczak, PM</td>
</tr>
<tr>
<td></td>
<td>Random distribution of grass parameters</td>
<td>Empirical clay erosion coefficient</td>
<td>TAW (1997)</td>
<td>Stanczak, PM</td>
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<tr>
<td></td>
<td>Empirical grass coefficient</td>
<td>Smith et al. (1994)</td>
<td>Stanczak, PM</td>
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<td></td>
<td>Grass roots properties</td>
<td>Stanczak (2008)</td>
<td>Stanczak, DM</td>
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<tr>
<td></td>
<td>Grass erosion model</td>
<td>Stanczak (2008)</td>
<td>Stanczak, DM</td>
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<tbody>
<tr>
<td></td>
<td>Assumption of the shape of the scour hole</td>
<td>INFRAM (1999)</td>
<td>Stanczak, PM</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Shear failure in cracks</td>
<td>Stanczak (2008)</td>
<td>Stanczak, DM</td>
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</tr>
</tbody>
</table>
Breaching of Coastal Dikes

Chapter 2

Breaching Processes

### Transition phase between clay and sand erosion

|--------------|-------------------------------|-----------------|--------------|

### Breach Formation: Core Erosion

| Equilibrium profile approach, sand core erosion | Beach profile model SBEACH | Larson et al. (1990) | Stanczak, PM |

### Breach widening and deepening

| Core wash-out (overflow) | Sediment transport model | Visser (1988), Tuan et al. (2007) | Stanczak, PM, Stanczak, DM |

#### 2.5.2 Inner Dike Slope

Dike breaching initiated from the landside is also the result of a series of physical processes of erosion and mass instability that occur simultaneously or as a sequence of events. For a dike breaching initiated by wave overtopping the following main processes are identified (D'Eliso (2007)):

- grass erosion
- clay erosion, headcut erosion and advance in the clay cover, headcut in sand-clay scour
- sand erosion, breach slopes instability
- grass and clay cover instability

These processes with information on available models are summarized in Tab. 7.
Tab. 7: Excerpt of existing models for morphodynamic processes for dike breaching initiated from the inner dike slope

<table>
<thead>
<tr>
<th>Model type</th>
<th>Process/ Location</th>
<th>Description/ Remarks</th>
<th>References</th>
<th>Implementation in Breach Model</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Breach Initiation: Grass Cover Failure</strong></td>
<td></td>
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</tr>
<tr>
<td>Empirical</td>
<td>Grass erosion and sod stripping</td>
<td>Excess shear stress relation, empirical parameters for grass, uniform flow, grass cover factor</td>
<td>Temple et al. (1987)</td>
<td>D’Eliso, PM</td>
</tr>
<tr>
<td><strong>Breach Formation: Clay Layer Failure</strong></td>
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</tr>
<tr>
<td>Empirical</td>
<td>Local erosion</td>
<td>Excess shear stress relation, simplified empirical approach, uniform flow</td>
<td>Temple &amp; Hanson (1994)</td>
<td>D’Eliso, PM, D’Eliso, DM</td>
</tr>
<tr>
<td>Headcut erosion</td>
<td>Scour erosion, empirical continuous approach</td>
<td></td>
<td>Robinson (1992)</td>
<td>D’Eliso, PM</td>
</tr>
<tr>
<td></td>
<td>Headcut advance, empirical continuous approach</td>
<td></td>
<td>Temple &amp; Moore (1997)</td>
<td>D’Eliso, PM</td>
</tr>
<tr>
<td>Discrete approach</td>
<td>Headcut erosion, (instability with shearing, turning, bending)</td>
<td>Scour erosion</td>
<td>Stein et al. (1993)</td>
<td>D’Eliso, DM</td>
</tr>
<tr>
<td></td>
<td>SITES Model</td>
<td></td>
<td>NRCS (1997)</td>
<td>D’Eliso, PM, D’Eliso, DM</td>
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<tr>
<td><strong>Full Breach Formation/ Breach Development</strong></td>
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<tr>
<td></td>
<td>Continuous erosion, 1D Exner eq. For the eroded breach area</td>
<td></td>
<td>Visser (1998), Hassan et al. (1999)</td>
<td>D’Eliso, PM, D’Eliso, DM</td>
</tr>
<tr>
<td></td>
<td>Mass instability, simple approach with force balance</td>
<td></td>
<td>Hassan (2002)</td>
<td>D’Eliso, PM, D’Eliso, DM</td>
</tr>
<tr>
<td></td>
<td>HR Breach Model</td>
<td></td>
<td>Mohamed (2002)</td>
<td>D’Eliso, PM</td>
</tr>
<tr>
<td></td>
<td>BRES Model</td>
<td></td>
<td>Visser (1998),</td>
<td>D’Eliso, PM, D’Eliso, DM</td>
</tr>
</tbody>
</table>
2.6 Breach Parameters

Dike breaching is a 3D space-time dependent process described by breach parameters and flow. The total time of dike breaching can be given as a sum of the following phases (which differ slightly due to the causes of breach initiation) (Fig. 3/ Fig. 4):

- **Time of grass failure** $t_{gf}$ - time between the incipient erosion and the time of grass failure;
- **Time of cover failure** $t_{cf}$ - time between the incipient erosion and the time when the revetment fails and the sand core becomes unprotected;
- **Time of core failure** $t_{sf}$ - time between the incipient erosion and the time when the erosion reaches both slopes and the erosion progress becomes irreversible;
- **Total breaching time** $t_{tb}$ - time between the incipient erosion and the time when the water level on the landside becomes equal to the one on the seaside;
- **Breach initiation time** $t_{i}$ - time between the incipient erosion and the initiation of the breach;
- **Breach formation time** $t_{f}$ - time between end of the breach initiation and the time when a breach is fully formed (full breach) for breaching initiated by wave overtopping (D'Eliso (2007))
  - time between end of the breach initiation and the end of the breach formation (cover failure) for breaching initiated by wave impact (Stanczak (2008))
- **Breach development time** $t_{d}$ - time between the end of the breach formation and the time when the erosion rate becomes negligible (final breach) for breaching initiated by wave overtopping (D'Eliso (2007));
  - time between the end of the breach formation and the start of the erosion of the inner slope (full breach) for breaching initiated by wave impact (Stanczak (2008))
- **Core wash-out time** $t_{c}$ - time between the end of the breach development and the time when the water level on the landside becomes equal to the one on the seaside (final breach) for breaching initiated by wave impact (Stanczak (2008))
Breaching of Coastal Dikes

Chapter 2

Breaching Processes

Fig. 3: Definition of breaching phases, breach processes and breach parameters for breaching initiated by wave overtopping (D’Eliso (2007))

Fig. 4: Definition of breaching phases, breach processes and breach parameters for breaching initiated by wave impact (Stanczak (2008))
2.7 Breach Outflow Description

The induced breach outflow is usually defined as the discharge with time along the centreline of the breach. A typical breach flood hydrograph that might result from breach through a dike is shown in Fig. 5. In general, the shape and duration of the hydrograph depend on the type of hydraulic loading (e.g. volume of water retained behind the dike; decreasing upstream water level, constant upstream water level, variation in loading (storm, tidal etc.)).

The curve of the generic breach hydrograph increases with time. During the time of breach initiation the breach flow discharge is rather low and increases slowly. At the time of the transition to breach formation the critical stage is reached where steady (and relatively slow) erosion cuts through to the upstream face of the embankment initiating relatively rapid and often unstoppable breach growth. Afterwards, during breach formation the outflow discharge increases rapidly due to the rapid vertically erosion of the embankment. The peak discharge is reached when a full breach is formed. Afterwards, during breach development as the breach widens, the discharge through the breach decreases. The decrease is dependent on the hydraulic loading, e.g. for decreasing upstream mean water level it follows the mean water level variations as the breach reaches the equilibrium shape.

![Generic breach flood hydrograph](image)

Fig. 5: Generic breach flood hydrograph (after Morris et al. (2009))
3 Breach Models

Morris et al. (2009) summarizes some of the key points regarding the modelling review of breach models as follows:

- Breaching models vary in detail and approach. Different models may be broadly summarized as (i) non-physically based, empirical models; (ii) semi-physically based, analytical and parametric models and (iii) physically based models.

- It is suggested that the uncertainty in model prediction is inversely related to the model complexity. Hence, whilst non physically based empirical models may be simple, quick and require very little data, their predictions will contain significant uncertainty.

- Uncertainty in breach model prediction is progressively reducing as research advances, but breach prediction still remains a significant source of uncertainty within the overall flood risk analysis process. There is a significant shortage of reliable, large scale data sets against which models may be tested and developed. The complex interactions between soil, flow and structure mean that scaling of breach tests is difficult.

Furthermore, a summary of physically based breach models developed 1965 – 2008 has been provided by Morris et al. (2009). It was recognised that many of these models arose from research projects and are not widely or commercially available.

In the following chapter existing models for dike breaching that have been developed at Leichtweiß-Institute are introduced.

3.1 Existing Models for Dike Breaching initiated by Overflow and Wave Overtopping (Inner Dike Slope)

3.1.1 D’Eliso (2007)

D’Eliso (2007) developed a model for dike breaching due to overtopping that applies to sand-clay dikes with a sand core and a clay cover with grass. The entire model system consists of two parts, i.e. preliminary model and detailed model, each of them divided into two modules: hydrodynamic and morphodynamic. The model is coded in Matlab v.6.0 and consists of 93 and 127 subroutines for preliminary and detailed model, respectively. In the following paragraphs a short summary is given according to D'Eliso et al. (2007). For a detailed description it is referred to D'Eliso (2007).

The preliminary model calculates the flow parameter applying empirical formulae during wave overtopping, while the wave-averaged energy balance is used during the flow through
the breach. The calculations of the grass and clay erosion as well as of the headcut are based on the excess shear stress approach, applying empirical coefficients. During the breach widening and deepening a selected sediment transport formula together with the 1D Exner equation are applied.

The detailed model is essentially based on the preliminary model, but in the hydrodynamic module (i) the application of the Volume of Fluid method can optionally be chosen and (ii) the water infiltration in the dike is calculated. The improvements in the morphodynamic module are as follows: (i) the breach initiation is calculated, not assumed (ii) mass instability of the clay layer with grass is calculated and (iii) the calculation of the headcut erosion both in the cohesive cover and in the sand-clay scour is calculated.

### 3.1.2 BREID Model

The BREID model is a numerical model for simulating BREaching of Inhomogeneous sea Dikes (BREID) developed by Tuan & Oumeraci (2010). It represents the first numerical model developed for the wave overtopping-induced erosion of the inner slope of grassed seadikes. The model consists of three different parts: fixed-bed overtopping, breach initiation and breach development. Moreover, the model is divided into two modules: hydrodynamic and morphodynamic. The model is coded in Delphi (Object Pascal). In the following paragraphs a short summary is given. A detailed description of the hydrodynamic module can be found in Tuan & Oumeraci (2010) and of the morphodynamic module in Tuan & Oumeraci (2012), respectively.

The hydrodynamic module for the numerical simulation of wave overtopping on seadikes is based on the nonlinear shallow water (NLSW) equations. The model can be extended with an additional source term related to the roller energy dissipation in the depth-averaged momentum equation to account for the additional effect from the surface roller motion breaking waves.

The morphodynamic module is based on the approach of excess bed shear for grass erosion modelling. For the determination of the bed shear stress the flow structure of wave overtopping on the inner slope is refined according to turbulent wall jet formulations in order to account for the high turbulence with entrained air bubbles for the conditions of wave overtopping at grass slopes. The critical velocity for grass erosion is determined based on depth-dependent strength concept together with the mobilized shear strength coefficient from equation (1):

\[
\begin{align*}
    u_{\text{gs,c}} &= 0.64 \log \left( \frac{8.8h}{d_a} \right) \sqrt{\frac{\Delta g d_a}{\rho} + \frac{1}{\rho} \left( 0.6 C_r + \mu_r C_r^w \right)} \\
    (1)
\end{align*}
\]

with:

- \( u_{\text{gs,c}} \) = Critical velocity of grass slopes [m/s]
- \( h \) = Flow depth [m]
Breaching of Coastal Dikes

Chapter 3

Breach Models

3.2 Existing Models for Dike Breaching initiated by Breaking Wave Impacts (Outer Dike Slope)

3.2.1 Stanczak (2008)

Stanczak (2008) developed a model for dike breaching due to breaking wave impact on the outer slope that applies to sand-clay dikes with a sand core and a clay cover with grass. The same modelling strategy as used by D'Eliso (2007) is also applied in this case, i.e. the model system consists of two parts: a preliminary model and a detailed model, each of them divided into two modules: hydrodynamic and morphodynamic. The model is coded in Matlab v.7.0. In the following paragraphs a short summary is given according to Stanczak et al. (2008a). For a detailed description it is referred to Stanczak (2008), Stanczak & Oumeraci (2012b), and Stanczak & Oumeraci (2012a).

The preliminary model calculates the flow parameter applying empirical formulae during breach initiation and formation, while the wave-averaged energy balance is used during the flow through the breach. The calculations of the grass and clay erosion as well are based on the wave impact approach and excess shear stress approach. During the breach widening and deepening either a selected sediment transport formula together with the 1D Exner equation or volume-averaged approach are applied.

The detailed model is essentially based on the preliminary model, but in the hydrodynamic module (i) the wave loads are calculated using the Volume of Fluid method and (ii) the water infiltration in the dike is calculated. The improvements in the morphodynamic module are as follows: (i) the breach initiation is not assumed but calculated according to new models developed after laboratory experiments by Stanczak (2008) (ii) equilibrium profile approach is introduced (iii) the possible erosion of the inner slope due to wave overtopping is accounted for.
4 Example Application

The dike breaching is modelled for the dikes of the case study area. In agreement with the partners of the XtremRisK-project the subproject of the Elbe Island Wilhelmsburg in Hamburg is the first subproject to be examined for the performance of a risk analysis. For a description of the case study area it is referred to Naulin et al. (2009). In the following chapter the methodology and results of the breach modelling exemplarily for an estuarine dike in Hamburg under the loading of the storm surge scenario HH-XR-2010A are presented and discussed.

4.1 Methodology

The methods in order to choose the relevant dike breach model as well as the applied input parameters for the chosen dike breach model are described in the following section.

4.1.1 Choice of Dike Breach Model

Depending on the hydraulic loading a relevant dike breach model has to be determined.

According to the first BAW-simulations the computed extreme storm surge scenario HH-XR2010A has a peak surge water level of 8.01 mNN for the harbour in Hamburg. Since the heights of the dike crowns vary from 7.80 mNN to 8.35 mNN in some parts a negative freeboard is reached and overflow will occur.

Moreover, preliminary results of the reliability analysis using a fault tree approach show that the non-structural failure modes as overflow and wave overtopping are the governing failure mechanisms leading to a very high probability of failure. Therefore, overflow and wave overtopping are considered the major forcing.

For this reason the BREID model for dike breaching initiated by overflow and wave overtopping and the model by D'Eliso (2007) as described in section 3.1 are applied to the case study area of Hamburg Wilhelmsburg.

4.1.2 Input Parameters

Main input parameters for breach models include information on hydraulics, dike geometry and material properties, respectively.

4.1.2.1 Hydraulics

As hydraulic input parameters the results of the detailed modelling of water level and wave parameters along the flood defence line in Hamburg carried out by subproject 3 should be used. Since these results are not yet available, as a start the hydraulic parameters are estimated for the preliminary breach modelling.
For the water level the results of the simulated extreme storm surge performed by the BAW are used. At gauge the Hamburg Harbour West was chosen (HafenHH1) that represents a cross section of the Norder Elbe and the Süder Elbe. The storm surge event has a peak water level of 8.0 mNN which is 5.9 m above the mean high tide and 0.7 m above the design water level. The occurrence probability was estimated to be $1.0 \times 10^{-5}$ per year (Wahl et al., 2010).

The computation time of the simulation of the water level is 80 hours. The data of the water level is available for a time step of 10 minutes. Considering three tides for the extreme storm surge scenario the duration time equals 37 hours (Fig. 6 a). The computed extreme storm surge has a peak surge water level of 8.01 mNN. For a time period of 3.16 hours the water level is higher than 7.0 mNN (Fig. 6 b). This period of 3.16 hours is chosen for a first simulation of fixed bed overtopping and breaching, respectively.

The wave action is estimated by a generation of random waves using the JONSWAP spectrum with the wave height of $H_{m0} = 0.55$ m and a peak period $T_p = 4.0$ s at the dike toe.

The used friction coefficient is set as a constant Chezy value of $C = 30 \ m^{0.5}/s$ for the grass slope as applied in Tuan & Oumeraci (2010) and suggested by Van der Meer (2007).

The main preliminary hydraulic parameters are summarized in Tab. 8.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>wave height $H_{m0}$ at toe</td>
<td>m</td>
<td>0.55</td>
</tr>
<tr>
<td>peak period $T_p$</td>
<td>s</td>
<td>4.0</td>
</tr>
<tr>
<td>peak surge level (series above 7 mNN)</td>
<td>mNN</td>
<td>8.01</td>
</tr>
<tr>
<td>storm duration</td>
<td>hrs</td>
<td>3.16</td>
</tr>
<tr>
<td>Chezy coefficient: grass slope</td>
<td>$m^{0.5}/s$</td>
<td>30</td>
</tr>
<tr>
<td>inland water level</td>
<td>m</td>
<td>dry bed</td>
</tr>
</tbody>
</table>
4.1.2.2 Dike Geometry

As model case the Klütjenfelder Hauptdeich with a rather low height of the dike of 7.88 mNN crown is analysed. For the dike profile data of inventory measurements were provided by the Landesbetrieb für Straßen, Brücken und Gewässer (LSBG). The standard profile (Regelquerschnitt 1) for the dike is shown in Fig. 7. The measurements were carried out at Dkm 0,114 and are supposed to be valid for a section of 760 meters starting at Dkm 0,000. The dike consists of a sand core with a clay layer and a grass cover.

The existence of a quay road on the seaward side of the dike and of a dike defence road on the landward side of the dike is neglected for the model set-up. Instead the outer geometry of the dike is considered. A grass layer of 0.20 m is assumed. The clay cover thickness on the inner slope, the top, and the outer slope is set to 1.30 m, 2.00 m and 1.50 m, respectively. These values for the clay cover are taken from the middle part of the standard dike profile. The resultant simplified dike profile is shown in Fig. 8. The main preliminary geometric parameters of the Klütjenfelder Hauptdeich are summarized in Tab. 9.

Fig. 7: Standard profile of the dike “Klütjenfelder Hauptdeich”

Fig. 8: Simplified cross section of the dike profile “Klütjenfelder Hauptdeich” implemented in the BREID model
4.1.2.3 Material Properties

Data about the material characteristics of sand, clay and grass are rarely or not at all available for the dikes in Hamburg. Therefore, the main parameters of material properties are estimated according to BREID simulation settings by Tuan & Oumeraci (2012) who used experimental data of dikes with Dutch and Danish material characteristics. The main parameters of the material properties are summarised by Tab. 10.

Tab. 10: Main parameters of material properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grass and clay properties</td>
<td></td>
<td></td>
</tr>
<tr>
<td>clay cohesion $c$</td>
<td>kN/m$^2$</td>
<td>24</td>
</tr>
<tr>
<td>mean root tensile strength $t_{r,m}$</td>
<td>$10^3$ kN/m$^2$</td>
<td>20.5</td>
</tr>
<tr>
<td>reference RAR$_0$ (at depth 2.5 cm)</td>
<td>%</td>
<td>0.065</td>
</tr>
<tr>
<td>root decay parameter $\beta$</td>
<td>-</td>
<td>0.12</td>
</tr>
<tr>
<td>mobilized strength coefficient $\mu_r$</td>
<td>-</td>
<td>0.60</td>
</tr>
<tr>
<td>grass erosion coefficient $M_{gs}$</td>
<td>kg/m$^2$/s</td>
<td>5.0E-03</td>
</tr>
<tr>
<td>clay erosion coefficient $M_{gc}$</td>
<td>kg/m$^2$/s</td>
<td>5.0E-03</td>
</tr>
</tbody>
</table>
4.2 Results and Discussion

In the following section results of the simulation of fixed-bed overtopping and breach initiation are presented.

4.2.1 Fixed-bed Overtopping

The results of the simulation of fixed-bed overtopping using the BREID model are computed for every node of the grid considering a space step of 0.15 m. A set of different locations with a distance of \( x = 0.60 \) m was selected for which the output results were written and analysed. The selected locations are shown in Fig. 9.

![Fig. 9: Locations of selected computed results](image)

As results of simulating fixed-bed overtopping the following parameters for a dike profile per one meter width are computed:

- Instantaneous overtopping discharge \( Q_b \) [m³/s]
- Mean overtopping discharge \( Q_{bmean} \) [m³/s]
- Water layer thickness \( h_b \) [m]
- Flow velocity [m/s]

These results of discharges, water layer thickness and flow velocity are examplarly shown for the dike crown with a height of \( z = 7.83 \) mNN at location \( x_b = 168 \) m in Fig. 10 until Fig. 13.
Fig. 10: Instantaneous overtopping discharge at time step \( t \) (\( Q_b \))

Fig. 11: Mean overtopping discharge for time interval \( \Delta t=1s \) (\( Q_{bmean} \))

Fig. 12: Water layer thickness (\( h_b \))

Fig. 13: Velocity (\( u_b \))
As it can be seen a significant overtopping (and later overflow) with discharges higher than 0.5 l/s/m comprises a period of 2.76 hours (from t = 827 s until t = 10768 s). Within this period a total volume of ca. 1152 m³/m overtops the dike. Hence, the computed average discharge equals 0.116 m³/s/m. At the dike crown the computed maximum discharge equals 1.358 m³/s/m. The maximum values of the flow velocity and the water layer thickness are 3.34 m/s/m and 0.23 m/m, respectively.

Considering the results of all selected locations along the inner slope of the dike (Fig. 9) the computed maximum discharge with 1.688 m³/s/m occurred at the middle of the inner dike berm (xb = 178.2 m, z = 5.66 m).

Moreover, the maximum depth-average velocity umax with 5.33 m/s was computed at the end of the inner dike slope (xb = 173.4 m, z = 6.11 m) while the layer thickness equals 0.14 m (t = 6869 s).

The main results of the computed fixed-bed overtopping are summarized in Tab. 11.

### Tab. 11: Main results of computed fixed-bed overtopping

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fixed-bed overtopping computed average discharge q (t = 2.76 hrs)</td>
<td>m³/s/m</td>
<td>0.116</td>
</tr>
<tr>
<td>computed maximum discharge q (xb = 178.2 m, z = 5.66 m)</td>
<td>m³/s/m</td>
<td>1.688</td>
</tr>
<tr>
<td>computed total overtopping volume V (t = 2.76 hrs)</td>
<td>m³/m</td>
<td>1152</td>
</tr>
<tr>
<td>computed maximum depth-average velocity on slope u_max (xb = 173.4 m, z = 6.11 m)</td>
<td>m/s</td>
<td>5.33</td>
</tr>
</tbody>
</table>

#### 4.2.2 Breach Initiation

The breach initiation in terms of wave overtopping-induced erosion of the inner slope of grassed sea-dikes is modelled by the BREID model for moderate grass conditions and under the assumption of weak spots in the grass layer, respectively.

##### 4.2.2.1 Moderate Grass Conditions

To simulate grass erosion, the input parameters as addressed in subsection 4.1.2 were applied. The numerical modelling of wave overtopping-induced erosion of grassed inner sea-dike slopes is based on the approach of excess bed shear for grass erosion as described in subsection 3.1.2. In general, the same parameters as for the simulation of fixed-bed overtopping are computed for a dike profile per one meter width. Besides the discharges, the water layer thickness and the flow velocity, for the mobile bed data also the height of the dike profile considering the reduction due to erosion are available.
The critical velocities for grass $u_{g,c}$ resulting from Eq. (1) with the average overtopping thickness $h_b = 0.04$ m are 4.6 m/s and 3.5 m/s for the top surface and for a depth of 5.0 cm, respectively (Fig. 14). Also from Eq. (1), the critical velocity $u_{s,c}$ for the bare clay is 0.86 m/s, which is significantly smaller than those for the grass reinforced clay.

Fig. 14: Theoretical depth-dependent critical velocity of grass and clay for a layer thickness of 0.04 m

The results of the simulation of grass erosion initiated from the computed overtopping (section 4.2.1) are shown in Fig. 15. The maximal height reduction (in $z$ plane) is 7 cm. Hence, the grass erosion is initiated but the erosion of the clay cover has not been started, yet.

Fig. 15: Grass erosion initiated from computed overtopping
The low erosion rate can be explained by looking at the computed velocities (Fig. 13). Only very few waves result in a high velocity which exceed the critical velocity of 4.6 m/s. Moreover, Eq. (1) emphasises that the critical velocity for grass depends on the flow depth. With a decrease of the overtopping thickness the critical velocity decreases and vice versa. To give an example, for \( h_b = 0.01 \) m and \( h_b = 0.23 \) m the critical velocity for grass \( u_{gs,c} \) at the surface is 3.2 m/s and 6.5 m/s, respectively. It seems as if for this simulation high velocities are reached at the same time as the flow depths increase (during overflow conditions) (see Fig. 12 and Fig. 13). Therefore, in these times of flow depth with \( h_b = 0.20 \) m the critical velocity for grass is higher and not exceeded by the actual velocity.

### 4.2.2.2 Weak spots in grass layer

It is pointed out that weak spots in the grass cover, e.g. cracks, wholes, etc. will reduce the resistance of the grass cover of a dike and hence present a danger that a breach might be initiated faster. The BREID model is currently developed further to consider the existence of these weak spots. Therefore, trapezoidal spots which locally reduce the layer thickness of the grass cover will be implemented in the model. Hence, with the local reduction of the grass depth the critical velocity will decrease (see Fig. 14) and the time for a breach initiation will be reduced. Since the function is still in development, the effects of implemented weak spots could not be further analysed within this project.

### 4.2.3 Dike Breach

An overview of the results of the dike breach modeling with the information, whether an initiation and a complete breach in the dike could be modeled, is shown in Tab. 12. The underlying assumptions, such as poor quality grass, are also given in Tab. 12.

To investigate the consequences of a total dike breach, further modelling under the assumption of poor grass conditions were performed. On this basis, possible total dike breaches with the model by D’Eliso (2007) were analyzed. Different scenarios with broken dike breach widths up to 400 m were modeled. The results for one dike breach scenario are exemplarily shown in Tab. 13. The outflow hydrograph at the breach location was used as a boundary condition for the simulation of flood wave propagation in the hinterland. The inundation simulation was performed in subproject 3 of XtremRisK.

### Tab. 12: Results of dike breach modelling

<table>
<thead>
<tr>
<th>Dike Section</th>
<th>Storm Surge Scenario</th>
<th>Breach Initiation (begin of erosion of the landward slope)</th>
<th>Total Dike Breach</th>
</tr>
</thead>
<tbody>
<tr>
<td>Klütjenfelder Hauptdeich (Crown Height 7,80 mNN; Slope:3)</td>
<td>HH_XR2010A</td>
<td>X</td>
<td>X*</td>
</tr>
<tr>
<td></td>
<td>HH_XR2010B</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>HH_XR2010C</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>HH_XR2010A-90</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>HH_XR2100A80</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>HH_XR2100C80</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

Annahmen: *poor grass condition
Tab. 13: Results of dike breach scenario

<table>
<thead>
<tr>
<th>Storm Surge</th>
<th>Dike Section</th>
<th>Breach Width</th>
<th>Volume</th>
<th>Assumption</th>
</tr>
</thead>
<tbody>
<tr>
<td>HH_XR2010A</td>
<td>Klütjenfelder Hauptdeich (Crown Height 7.80 mNN; Slope 1:3)</td>
<td>~ 400</td>
<td>~ 72 Mio.</td>
<td>poor grass condition</td>
</tr>
</tbody>
</table>
5 Summary

The report provides an overview about breaching processes and models. Therefore, at first the boundary conditions for breaching are stated and the causes of breach initiation are described. Moreover, relevant hydrodynamic and morphodynamic processes and the according available models are listed. For more detailed information about the state of the art review of breaching processes and models a listing of references such as D'Eliso (2007), Stanczak (2008), and Morris et al. (2009) is given. Moreover, the report introduces existing models for dike breaching that have been developed at Leichtweiß-Institute.

Furthermore, the methods and the results of an example application of preliminary breach modelling are described in this report. Therefore, the BREID model was applied to simulate the breaching initiated by wave overtopping and overflow. As a first example, the dike section of the Klütjenfelder Hauptdeich as an estuarine dike of the pilot sites in Wilhelmsburg, Hamburg, was chosen. As hydraulic conditions the first storm surge event developed by XtremRisK-subproject 1 was applied. The storm surge has a peak water level of 8.0 mNN and an occurrence probability was estimated to be $1.0 \times 10^{-5}$ per year.

The results of the numerical simulation of wave-overtopping computed an average discharge of 0.116 m$^3$/s/m over duration of 2.76 hours. Hence, a total overtopping volume of 1152 m$^3$/m was estimated. The maximum depth-average velocity $u_{\text{max}}$ on the slope and maximum discharge $q$ was calculated to 5.33 m/s and to 1.688 m$^3$/s/m, respectively.

Moreover, the modelling of breach initiation was carried out. The results for moderate grass conditions show that only erosion of the grass layer could be computed. The maximal height reduction (in z plane) is 7 cm within the total storm surge duration. A total dike breach could be modelled under the assumption of poor grass conditions. An outflow hydrograph was determined at the breach location which will be used for inundation modelling in subproject 3 of XtremRisK.
6 References


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Wang, B. (2003): Oblique Wave Transmission at Low-Crested Structures. MSc Thesis HE 133, also part of the research work undertaken within EU funded project DELOS.