EroGRASS
Failure of Grass Cover Layers at Seaward and Shoreward Dike Slopes

- Design, Construction & Experimental Procedure -
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December 2009
Report

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For additional information on the EroGRASS project please visit: www.kyst.dk/erograss.

Keywords
Grass cover failure, wave impact, run up and run down flows, wave overtopping, grass sods, prototype dike model, Large Wave Chanel.
Summary

A large number of dikes in the North Sea and Baltic Sea regions are covered with grass that is exposed to hydraulic loading from waves and currents during storm surges. During previous storm surges the grass cover layers often showed large strength and remained undamaged. The clear physical understanding of the failure of grass cover layers due to different wave loads is, therefore, indispensable today, especially against the background of enhanced hydraulic impact due to climate change.

The strength of the grass cover layer lies mainly in its ability to withstand three types of wave actions:

- Wave impact due to wave breaking on the seaward slope;
- Wave run-up and run-down flow after wave breaking on the seaward slope;
- Down-slope flow on the landward slope caused by wave overtopping.

The main objectives of this research project were therefore to perform large scale model tests to investigate the failure of grass cover layers due to (i) wave impact, (ii) wave run-up and run-down flow and (iii) wave overtopping.

Wave impact as well as wave run-up and run-down flow may induce grass cover failure on the seaward dike slope. Wave overtopping causes failure of the grass cover at the dike crest and on the shoreward slope. Hence, this research project dealt with the investigation of grass cover failure along the entire dike profile: seaward slope, dike crest and shoreward slope.

To obtain the aforementioned research objectives, large scale model tests at a dike model were performed in the Large Wave Flume of the Coastal Research Centre—a joint centre of the University of Hanover and the Technical University of Braunschweig, Germany. The design, construction and experimental procedure of the large scale model tests were conjointly performed by the FLOODsite Project and the EroGRASS Project. Differences between both projects lie in their scientific research focus: The FLOODsite Project focussed on wave overtopping, the subsequent damage and the breach development initiated from the landward side of the sea dike. As aforementioned, the EroGRASS project focussed on the failure of the grass cover layer at sea dike. The collaboration between the FLOODsite Project and the EroGRASS Project was very useful for the implementation of the most extensive test programme of this kind. Otherwise such a unique enterprise would have been financially and timely hardly affordable.

The dike model represented a typical sea dike. With exception of the seaward slope, it was comparable to typical cross sections of sea dikes built in The Netherlands, Germany and Denmark. This relatively steep seaward slope was chosen to improve the generation of wave impact on the seaward slope. The crest height of the dike model was 5.8m above the bottom of the wave flume and the dike model consisted of a sand core covered by a layer of clay and a grass layer. The 0.2m thick grass cover layer was constructed with grass sods that were excavated at the existing Ribe sea defence in Denmark and transported to the Large Wave Channel in Hannover by trucks.

The test programme was divided into two phases. In the first phase the initiation of grass erosion on the seaward slope due to wave impact and wave run-up and run-down flow was studied. In the second phase the initiation of grass erosion on the landward slope due to wave overtopping was investigated. During testing data concerning wave conditions in front of the dike model, wave overtopping volumes, pressure load due to wave impact and flow velocities
on the seaward slope were recorded. Furthermore, all tests were recorded by two digital video cameras.

This report presents a first reporting of the EroGRASS project including a description of the design and construction of the dike model in the Large Wave Flume, the measuring and observation techniques and the test programme together with examples of records from the performed tests. Focus of this report is put on providing a well-documented description of the aforementioned issues. The data analysis, research results and conclusions concerning the implications for the erosion of the grass cover will be published in a second project report at a later date.

With respect to the scientific research objectives of the FLOODsite Project, the reader is referred to Geisenhainer and Oumeraci (2008). Since both projects cooperated by the design and construction of the dike model as well as the set-up of measurement equipments, both reports comprise of a similar report structure and content.
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1. Introduction

Along the North Sea coasts, protection against coastal flooding is mainly performed by dikes. Grass cover layers on the seaward and shoreward dike slopes are the most prevalent form to protect the dike surface against erosion during storm surges. The grass cover layer is exposed to different forms of wave loading that may provoke the failure of the grass cover. As the failure of the grass cover layer may emerge to an overall dike breach, the clear understanding of the failure processes is essential for estimating the safety of a sea dike. The main objective of the EroGRASS project was therefore to perform large scale model tests in order to investigate the failure of the grass cover layer at sea dikes. The large scale tests at a prototype dike model were performed in the Large Wave Flume (GWK) of the Coastal Research Center (a joint centre of the Universities of Hannover and Braunschweig) in Hannover, Germany.

The report in hand is a first reporting of the EroGRASS project including a description of the design and construction of the dike model, the measuring and observation techniques and the test programme together with examples of records from the performed test runs. Focus of this report is put on providing a well-documented description of the aforementioned issues. The data analysis, research results and conclusions concerning the large scale tests will be published in a second project report at a later date.

The design, construction and experimental procedure of the large scale model tests were conjointly performed by the FLOODsite Project and the EroGRASS Project. Differences between both projects lie in their scientific focusing: The FLOODsite Project focussed on wave overtopping, the subsequent damage and the breach development initiated from the landward side of the sea dike. The EroGRASS project focussed on the failure of the grass cover layer at sea dike due to different forms of wave impact. The collaboration between the FLOODsite Project and the EroGRASS Project was very useful for the implementation of the most extensive test programme of this kind. Otherwise such a unique enterprise would have been financially and timely hardly affordable.

With respect to a more detailed description of the research objectives and tests performed within the FLOODsite Project, the reader is referred to Geisenhainer and Oumeraci (2008). Since both projects cooperated by the design and construction of the dike model as well as the set-up of measurement equipments, both reports comprise of a similar report structure and content.

The FLOODsite Project (http://www.floodsite.net) has been the largest ever EC research project on flood risk management. The project started in 2004 and was completed in February 2009. The project was arranged into seven Themes which were divided into a total of 35 Tasks. With respect to a comprehensive review of failure modes of sea dikes and the large scale model tests on breach development initiated from the landward dike side (see above), attention should especially be drawn to Task 4 and Task 6 of the FLOODsite project.
1.1 State of knowledge

According to Young (2005), the assessment of the grass cover being exposed to these hydraulic loadings involves three distinct scientific and engineering disciplines:

- Hydraulics - wave loading;
- Geotechnics - strength of the soil structure (shear strength and erodibility of the clay layer);
- Botany - composition, management and strength of the grass cover layer.

The structure and division of a grass cover is shown in Figure 1.1. The green leafy part of the grass is the sward. The turf is the root mat which provides the strength and erosion resistance to the clay layer. The roots keep the soil particles together and create a flexible and tough layer that offers significantly higher erosion resistance than a bare clay layer (Young, 2005). Model tests by Möller et al. (2002) showed that as soon as water is flowing over a bare clay surface, gulley formation will start rapidly.

Investigations carried out by Sprangers (1999) show that the performance of grassland depends primarily on its management. The ideal grassland is unfertilised, periodically grazed and rich of species. This form of management provokes a closed turf with fine and coarse roots. This network of fine and coarse roots makes the top soil a strong and flexible layer that can deform without tearing (TAW, 1997). Hence, the root network is important to keep the clay particles together during wave loading. The TAW (1997) states further that the structure of the soil in between the roots is at least as important. Both aspects are the most important characteristics for erosion resistance of the grass cover at sea dikes during wave loading.

At the same time as the grass turf establishes, so the clay properties change. The moisture content changes are leading to changes in the soil structure, i.e. (i) increase in the moisture content leads to the softening of the clay layer, as the soil pore water pressure increases and undrained shear strength decreases; (ii) decrease in the moisture content leads to the clay shrinkage and development of the desiccation cracks within impermeable layer, as well as increase in undrained shear strength and permeability of the soil. The change of the moisture content profile close to the surface is due to the plant extraction of water from the clay. The soil cracking produces a soil that consists of aggregates of various dimensions. The composition of these cracks and aggregates, together with pores and aggregates made by animals, is called the soil structure (TAW, 1996).
According to TAW (1996), the soil structure may be present to a depth of more than 0.8 m. In some cases, mainly in the UK, it can be found that desiccation cracks can vertically penetrate clay structure to a depth of 1m (Zielinski et al., 2008). The soil cracking increases the permeability of the soil and the infiltration of water into the subsoil. Infiltration tests at many locations on dikes resulted in infiltration rates of $10^{-5}$ to $10^{-4}$ m/s (TAW, 1996). The infiltration of water into the clay layer can induce the loosening of individual particles, which may lead quickly to major damage of dike construction. In the case of a sandy clay (> 40%), the erosion of the soil may appear even more faster. Hence, the erosion of clay depends on the water content and the sand content. The erosion resistance may be classified into three categories (TAW, 1996):

<table>
<thead>
<tr>
<th>Category</th>
<th>Water content $w$ [%]</th>
<th>Plasticity Index $I_p$</th>
<th>Sand content [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Erosion-resistant clay</td>
<td>&gt; 45</td>
<td>&gt; 0.73 · (w - 20)</td>
<td>&lt; 40</td>
</tr>
<tr>
<td>Moderately erosion resistant</td>
<td>&lt; 45</td>
<td>&gt; 18</td>
<td>&lt; 40</td>
</tr>
<tr>
<td>clay</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay with little erosion</td>
<td>&lt; 45</td>
<td>&lt; 18</td>
<td>&gt; 40</td>
</tr>
<tr>
<td>resistance</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Several large scale tests have been performed to improve the knowledge about the strength of grass cover layers at sea dikes. Young (2005) reports about two physical tests performed in the Netherlands: (i) the Deltagoot test in 1992 and (ii) the Scheldbak test in 1994. At these tests attention was paid on the vegetation quality and soil quality. The aspect of hydraulic loading received less attention.

Since the management of flood defence systems recently changes to more reliability and risk-based design concepts, the clear physical understanding of grass cover erosion due to different forms of wave loading has gotten more important. The overall goal is to produce a set of limit state equations (Young, 2005) that predict the failure of the grass cover due to wave loading (wave run-up and run-down flow, wave impact, wave overtopping).

Preliminary theoretical investigations with respect to wave overtopping flow on inner slopes have been undertaken by IHE Delft. Based on a comprehensive literature review, Young (2005) questions the relevance of surface erosion as a failure mode and states that surface erosion may not be the sole mechanism of grass cover failure on inner dike slopes. He suggests a superficial slip model that looks at the shear failure at the interface between the turf and clay layer. The sliding mechanism presumes a condition of full saturation, and seepage parallel to the surface (Young, 2005).

In the case of the top layer being dominated by soil structure, Young (2005) argues further, that the soil (without grass) will not exhibit cohesion. Since turf is a composite material including roots, quantities such as root tensile strength and root area ratio are introduced to quantify the additional strength from roots. Concerning root tensile strength, the literature review by Young (2005) showed that more should be know about the range or distribution of root diameters for dike grassland, before root tensile strength can be quantified. Direct tensile strength tests on the roots of dike grassland have not been performed yet (Young, 2005).

The root area ratio can be back-calculated from root mass, root length and cross section area. Referring to measured profiles by Simon and Collison (2001) and root measurements by Sprangers (1999), Young (2005) states that the root area decreases with depth. The root
measurements by Sprangers (1999) showed that 53-55% of the roots are located in the top 6 cm of the turf, and that 75-80% of the roots is within the top 20 cm. Theoretical investigations for wave impact or wave run-up (seaward slope), comparable to the review for wave overtopping by Young (2005), could not be found in the literature. Concerning wave overtopping and subsequent damage on the landward dike slope, the reader is also referred to the ComCoast project (http://www.comcoast.org). This project developed and demonstrated innovative solutions for flood protection in coastal areas. Amongst others, the strength of a reinforced grass cover layer on the inner slope of a dike was investigated with respect to wave overtopping by carrying out prototype tests. For the prototype tests, a wave overtopping simulator was built. This apparatus was used to test the strength of the grass cover layer with and without reinforcements.

1.2 Motivation and objectives

In recent years, the interest of reliability and risk-based design concepts has clearly grown in the field of coastal engineering. At this, the clear physical understanding of load-resistance processes has become an essential task. At the same time, grass cover layers as revetments for flood defence structures have attracted more interest. A grass cover is now being considered as a constructional component (EAK, 2002) that has to be designed and managed. Design and management of grass cover layers require an improved understanding of the failure development and the interaction between load and resistance. The problem of erosion at dike slopes and crests during wave loading appears here to be critical. Young (2005) indicated the loads which may cause erosion of the grass cover layer at sea dikes, namely:

- Overflow (still water level exceeds the crest level)
- Wave impact due to wave breaking on the seaward slope
- Wave run-up and run-down flow on the seaward slope
- Flow on the shoreward slope due to wave overtopping

The main objectives of this research project was therefore to perform large scale model tests in order to investigate in detail the failure of grass cover layers due to (i) wave impact, (ii) wave run-up and run-down flow and (iii) wave overtopping. The loading by overflow was not considered. Wave impact as well as wave run-up and run-down flow may induce grass cover failure on the seaward dike slope. Wave overtopping may cause failure of the grass cover at the dike crest and on the shoreward slope. Hence, this research project dealt with the investigation of grass cover failure along the entire dike profile: seaward slope, dike crest and shoreward slope.
2. Design of the dike model

The prototype of the selected dike model represents a typical sea dike of the German Bight coast (Figure 2.1). With exception of the relatively steep seaward slope, it is comparable to typical dike cross sections as usually built in The Netherlands, Germany and Denmark. In general, a sea dike consists of a sand core, a clay layer and a grass cover. According to EAK (2002), the landward dike slopes are build 1:3 and seaward dike slopes are recommended not steeper than 1:6. The crest of the sea dike can be 3m wide and the dike height can be 8.4m (Figure 2.1) or more. The seaward and the landward slopes are normally constructed without berms to ease the construction of the sea dike (Oumeraci et al., 2001).

Figure 2.1 Typical cross-section of a grass-covered sea dike at the German Bight coast (EAK, 2002).

2.1 Geometry of the model

The Large Wave Flume (GWK) of the Coastal Research Centre in Hannover (Germany) is 5m wide, 7m deep and 324m long (Figure 2.3). The maximum water depth in the flume is 5.0m. Regular waves can be generated with heights up to $H \approx 2.0m$ and irregular waves with significant heights up to $H_s \approx 1.3m$. The prototype of the dike model was therefore adjusted in its dimensions and geometry to allow for large scale testing in the wave flume.

2.1.1 Cross section

Compared to the prototype, changes were made concerning the dike height and seaward slope. The slope of the seaward side was chosen to 1:4. This relatively steep slope was chosen instead of 1:6 in order to investigate impact loads due to wave breaking on the seaward slope without the damping effect of the previous wave down rush. The landward slope was 1:3. The height of the dike model was 5.8m and the crest was 2.2m wide (Figure 2.2). No berms were constructed on both slopes. The length of the dike model was 5m, like the width of the flume.

Figure 2.2 Cross section of the dike model.
The dike model consisted of a sand core, a clay layer and a grass cover. On both slopes and on the dike crest a clay layer of 0.6m was installed. On top of the clay layer, 0.2m thick grass sods were placed to complete the dike model with a grass cover. By placing the grass sods, the entire clay layer was about 0.8m thick.

In front of the dike model a sloping foreshore was installed to ensure proper conditions for wave transformation (shoaling) in front of the dike model. The slope of the foreshore was 1:40 and the height at the dike toe was 1.0m above the flume bottom. The foreshore length was 40m.

**Figure 2.3** Cross and longitudinal section of the Large Wave Flume (Hannover) showing the positioning of the dike model in the flume.

The combined inlet and outlet of the flume is about 35m from the wave paddle. The model was located about 190m from the wave generator due to a window in the flume wall which allowed the observation of the wave impact on the seaward dike slope (Figure 2.3). The flume area behind the dike model was needed as a reservoir for wave overtopping.

### 2.1.2 Dike toes and transition to flume bottom

The transition between the seaward dike toe and the foreshore is shown in Figure 2.4. The foreshore was built of clay that was connected with the clay layer of the seaward dike slope in order to perform a sealing against infiltrating water (Detail A, Figure 2.6).

**Figure 2.4** Detail A - Transition between dike toe and foreshore (see Figure 2.6).

**Figure 2.5** Detail B - Toe protection of the landward slope (see Figure 2.6)
The toe of the landward slope is stabilised by a concrete corner wall (Detail B, Figure 2.6). In front of the concrete wall the clay is build in down to the flume bottom (Figure 2.5). The concrete wall is required to avoid a head cut erosion of the landward dike toe.

Figure 2.6  Cross section of the dike model showing the construction details A and B.

With the 0.8m thick clay layer and both construction details A and B, the sand core is completely covered by clay and infiltration into the sand core is reduced. To avoid piping as well as pore pressure and seepage, the dike body was drained. The drainage system was located on the flume bottom and was emptied by a pump. The phreatic water was pumped into the reservoir behind the dike model.

2.2  Construction materials

In order to fulfil the objectives of this research project, the dike model had to be covered with a natural grass layer. The challenge was hereby to get the natural grass into the flume. Since it was not feasible to sow grass on the clay layer and wait for a well-established grass cover, grass sods were excavated from an existing sea dike and transported to the wave flume for installation on the dike model.

The flood defence system, where grass sods were excavated, is shown in Figure 2.7. The grass sods originated from the flood defence system near Ribe (Denmark). The grass sods were excavated from the southern wing dike which was reinforced in 1998. The composition of grass community of the southern wing dike is similar to those used in Germany and the Netherlands.

Figure 2.7  Location of the Ribe defence system and its wing dikes (Denmark).
The grass sods were excavated with the underlying clay, as it was important that the interaction between the top soil of the grass layer and the underlying clay was not disturbed. The grass cover was of good quality and adequate for investigation of incipient grass cover erosion.

The complete dike surface to be covered with grass sods was calculated to about 190m². An area of about 10m³ of grass sods were estimated for substitution of damaged grass areas during testing. In total, about 200m² of grass sods were therefore excavated. The cross section of an excavated grass sod is shown in Figure 2.8. The excavated grass sods were about 20cm thick and measured 2.35m in length and 1.25m in width. The weight of one grass sod was approximately 1,100 kg.

In order to cover the dike model with a 0.6m thick clay layer, a total clay volume of about 150m³ was needed. Furthermore, a volume of 5m³ was estimated to repair damaged parts of the dike model.

A sand volume of 300m³ was needed for construction of the dike core. The sand was available at the Coastal Research Centre. The medium diameter was determined \( d_{50} = 0.33 \text{mm} \). This grain size is slightly larger than the typical sand material used for sea dikes at prototype. Table 2.1 lists all needed materials for the construction of the dike model.

### Table 2.1 Needed material for construction of the dike model.

<table>
<thead>
<tr>
<th>Type</th>
<th>Area / Volume</th>
<th>Quality / Type</th>
<th>Taken from</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grass</td>
<td>~ 200 m²</td>
<td>Winter grass</td>
<td>Sea defence system at Ribe, Denmark</td>
</tr>
<tr>
<td>Clay</td>
<td>~ 150 m³</td>
<td>Good</td>
<td>Sea defence system at Ribe, Denmark</td>
</tr>
<tr>
<td>Sand</td>
<td>~ 300 m³</td>
<td>( d_{50} = 0.33 \text{mm} )</td>
<td>Available at CRC</td>
</tr>
</tbody>
</table>
2.2.1 Sand for the dike core
The dike core consisted of sand with 70% of sand grains being smaller than 0.25mm (Figure 2.9). The EAK (2002) recommends a percentage of fine particles ($d \leq 0.063\text{mm}$) not higher than 15% and the final compaction product should range within 90-98% of the maximum dry density (Proctor Standard Compaction Test).

![Figure 2.9 Grain distribution curve of the sand used for dike core construction in the Large Wave Flume.](image)

2.2.2 Clay for the dike revetment
The erosion resistance of clay can be categorised by the water content and the sand content. According to TAW (1996), clay is classified into three erosion resistant categories, see Table 2.2. According to EAK (2002), clay that is used for sea dike revetments should also meet the requirements specified in Table 2.3.

<table>
<thead>
<tr>
<th>Clay category</th>
<th>Water content $w$ [%]</th>
<th>Plasticity Index</th>
<th>Sand content [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Erosion resistant</td>
<td>&gt; 45</td>
<td>&gt; 0.73 · (w - 20)</td>
<td>&lt; 40</td>
</tr>
<tr>
<td>Moderate erosion resistance</td>
<td>&lt; 45</td>
<td>&gt; 18</td>
<td>&lt; 40</td>
</tr>
<tr>
<td>Low erosion resistance</td>
<td>&lt; 45</td>
<td>&lt; 18</td>
<td>&lt; 40</td>
</tr>
</tbody>
</table>
Table 2.3  Requirements for clay used as dike revetment (EAK, 2002).

<table>
<thead>
<tr>
<th>Soil property</th>
<th>Threshold</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand content (d &gt; 0.06mm)</td>
<td>&lt; 40%</td>
</tr>
<tr>
<td>Clay content (d &lt; 0.002mm)</td>
<td>&gt; 10%</td>
</tr>
<tr>
<td>Liquidity Limit</td>
<td>$w_L &gt; 25%$</td>
</tr>
<tr>
<td>Plasticity Limit</td>
<td>$w_p &gt; 15%$</td>
</tr>
<tr>
<td>Undrained Shear Strength</td>
<td>&gt; 20 KN/m$^2$</td>
</tr>
<tr>
<td>Undrained cohesion</td>
<td>&gt; kPa</td>
</tr>
<tr>
<td>Dry density</td>
<td>0.85 &lt; $\rho_d$ &lt; 1.45 t/m$^3$</td>
</tr>
<tr>
<td>Water content</td>
<td>&gt; 80% &gt; w &gt; 30%</td>
</tr>
</tbody>
</table>

The soil properties of the clay used for the clay revetment of the dike model (Type B, Table 2.4) were determined by Strathclyde University (Marcin Zielinski) using the geotechnical laboratory at Leichtweiß-Institute. The soil properties of the clay used for the foreland (Type A, Table 2.4) had been determined in a former project by Richwien and Weißmann (2001). The grain size distribution of clay type A and clay type B are shown in Figure 2.10. For both clay types accounts that the soil properties were determined according to the German standards DIN 18121, 18122, 18123 and 18137.

Table 2.4  Soil properties of the clay used for the dike model.

<table>
<thead>
<tr>
<th>Soil property</th>
<th>Clay - Type A Foreshore</th>
<th>Clay - Type B Clay revetment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand content (d &gt; 0.06mm)</td>
<td>12%</td>
<td>20%</td>
</tr>
<tr>
<td>Clay content (d &lt; 0.002mm)</td>
<td>35%</td>
<td>25%</td>
</tr>
<tr>
<td>Liquidity Limit</td>
<td>77%</td>
<td>61%</td>
</tr>
<tr>
<td>Plasticity Limit</td>
<td>45%</td>
<td>28%</td>
</tr>
<tr>
<td>Maximum dry density</td>
<td>1.612 t/m$^3$</td>
<td>1.373 t/m$^3$</td>
</tr>
<tr>
<td>Water content</td>
<td>40% - 50%</td>
<td>26% - 35%</td>
</tr>
</tbody>
</table>
2.2.3 Grass cover

The grass cover consists of grass vegetation which roots in the underlying soil (clay) (Figure 2.11). A good resistant grass cover consists of a high number of species (TAW, 1999).

The soil near the surface of the grass cover layer has a high root density, is elastic in moist conditions and porous. Conversely, the underlying clay is stiff (or plastic when moist or not yet aged) and usually somewhat less permeable. The erosion resistance of the covering layer near the soil surface is (usually) higher than at deeper parts of the layer. The upper, densely rooted part, with an irregular bed structure and a higher erosion resistance, is called sod. A sod with a thick network of roots and grass coverage of more than 70-85% has a good erosion resistance. 65% of the grass roots are located in the upper soil layer (0-6cm depth). Between 6cm and 15cm 20% of the roots can be found. The remaining percentage of roots is located in a depth up to 50cm (Sprangers, 1999).

The degree of prevention of high velocities and stresses at the soil-water interface of vegetal cover is described by the vegetal cover factor \( C_F \), see Temple et al. (1987). The cover factor is dominated by the density and uniformity of density in the immediate vicinity of the soil.
boundary. In Table 2.5 generalised vegetal cover factors are shown. These do not depend on the species of the vegetal cover.

**Table 2.5  Vegetal cover factor by Temple & Hanson (1994).**

<table>
<thead>
<tr>
<th>Cover description</th>
<th>Vegetal cover factor $C_F$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good vegetal cover</td>
<td>0.75</td>
</tr>
<tr>
<td>Fair vegetal cover</td>
<td>0.50</td>
</tr>
<tr>
<td>Poor vegetal cover</td>
<td>0.25</td>
</tr>
</tbody>
</table>

From large-scale tests performed in the Deltagoot (Netherlands) in 1992 (Smith et al., 1994) a conservative classification for the determination of the erosion resistance of grassland against wave impact on the seaward dike slope is given in Table 2.6.

**Table 2.6  Grass erosion coefficient (TAW, 1997).**

<table>
<thead>
<tr>
<th>Erosion-resistant grassland</th>
<th>Expected values for $c_E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good</td>
<td>0.5 to 1.5 $10^{-6}$ (ms)$^{-1}$</td>
</tr>
<tr>
<td>Moderate</td>
<td>1.5 to 2.5 $10^{-6}$ (ms)$^{-1}$</td>
</tr>
<tr>
<td>Poor</td>
<td>2.5 to 3.5 $10^{-6}$ (ms)$^{-1}$</td>
</tr>
</tbody>
</table>

Typical grass species and exemplary fraction found in grass cover layers of sea dikes in the North Sea region are listed in Table 2.7.

**Table 2.7  Typical grass species and fraction on sea dike in the North Sea region.**

<table>
<thead>
<tr>
<th>Grass species</th>
<th>Sort</th>
<th>Portion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Latin</td>
<td>English</td>
<td></td>
</tr>
<tr>
<td>Festuca arundinacea</td>
<td>Tall fescue</td>
<td>Fine Lawn</td>
</tr>
<tr>
<td>Festuca rubra</td>
<td>Red fescue</td>
<td>Suzette S</td>
</tr>
<tr>
<td>Festuca rubra</td>
<td>Red fescue</td>
<td>Echo</td>
</tr>
<tr>
<td>Lolium hybridum</td>
<td>Hybrid ryegrass</td>
<td>Avance</td>
</tr>
<tr>
<td>Agrostis stolonifera</td>
<td>Creeping wheat</td>
<td>Kromi S</td>
</tr>
<tr>
<td>Agrostis capillaris</td>
<td>Browntop</td>
<td>Highland bent.</td>
</tr>
</tbody>
</table>

**Root volume ratio (RVR)**

The influence of the root network on the reinforced soil strength can be investigated by determining the root percentage of a soil sample. The Root Volume Ratio (RVR) of the dike model was determined at the Leichtweiß-Institute, Germany, by using the following procedure:

Soil samples were taken from different grass sods. Each sample block measured 10cm x 10cm x 20cm. The part above the soil surface of each grass block was removed including the lose material on the surface. The block was then cut into 2cm thick slices with a volume of 200cm$^3$. Each slice was carefully washed and the roots were collected. After drying the roots
at a temperature of 105°C for 24 hours, they were weighted. The volume of the entire roots of a slice was determined in relation to the slice volume. In total, six sample blocks were analysed and the root volume ratio (RVR) was determined (Figure 2.12).

The information about the grass root distribution is of crucial importance for estimating the reinforcement effect of the grass roots on the clay layer. The logarithmic decline of the root volume ratio with increasing depth corresponds with the root volume ratio of ten soil samples analysed by Stanczak (2008).

Figure 2.12 Root volume ratio (RVR) of the grass cover used at the dike model.
3. Construction of the dike model

The construction of the dike model took about six weeks lasting from 14 January 2008 to 7 March 2008. The construction work started with the installation of the foreshore. The sand core including the drainage system was constructed afterwards, followed by the 60cm thick clay layer which was installed on top of the sand core. After compaction of the clay, the 20cm thick grass sods were installed.

3.1 Construction of sand core and foreshore

The construction of the dike model started with installation of the foreshore (Figure 3.1). To indicate the state of the compaction, undisturbed soil samples were taken from the compacted layers using U100 steel tubes. The dry density was calculated for each soil sample extruded from the tube and ranged between 1.52 and 1.612t/m³ with a moisture content of about 22.0% and 22.8%. The applied foreshore clay (Type A) was different to the clay (Type B) which was used for the clay layer at the dike model (cp. Table 2.4). The clay for the foreshore (Type A) was available at the Large Wave Flume. The toe of the foreshore was protected against erosion by three 1.33m long and 40cm high concrete blocks (Figure 3.1). After installation of the foreshore, the lower part of the sand core was heaped up. However, before installing the sand core, the required drainage system was installed. The system consisted of three pipes (diameter of 20cm) wrapped with coco fibres (Figure 3.2). All three pipes ran into a well that was placed on the landward side of the dike model.

The well consisted of three prefabricated concrete rings and was calked against water intrusion from outside (Figure 3.3).

Figure 3.1 Foreshore with toe protection (view from sea side).

Figure 3.2 Drainage system installed at the flume bottom.

Figure 3.3 Well at the landward dike toe.
The completed foreshore is shown in Figure 3.4. The sand of the dike core was transported by a wheel loader (Figure 3.5) from the sand storage into the flume. The sand storage was located on the same yard as the flume. The sand was installed in layers of 50cm thickness (Figure 3.6), and afterwards compacted by a plate compactor. The foreshore as well as the sand core was compacted alongside the flume walls with a pneumatic rammer to assure a good contact between the soil and the concrete walls. Due to only one entrance into the flume, installation of the clay layer on the seaward slope was performed simultaneously during installing the upper sand core layers (Figure 3.7).
3.2 Construction of the clay layer

As aforementioned, clay material for the dike model was dug at a clay pit near the harbour town of Esbjerg (Denmark). The clay was transported by trucks from Denmark to Hannover (Figure 3.8) and stored shortly on the yard of the Coastal Research Centre (Figure 3.9). The clay was transported by the wheel loader into the flume. First the lower part of the clay layer on the seaward slope was installed (Figure 3.10). The 60cm thick clay layer was built in two steps, installing a 30cm thick layer each time with subsequent compaction of the clay material. After completion of the seaward clay layer and the sand core, the clay layer on the landward slope as well as on the crest was built in (Figure 3.11). Clay compaction of the first 30cm thick clay layer was performed by a loader (Figure 3.12). Compaction of the second 30cm thick clay layer was carried out by using a rammer and a small roll (Figure 3.13).

Figure 3.8 Unloading of the clay at the Coastal Research Centre.

Figure 3.9 Temporary stored clay on the yard of the Coastal Research Centre.

Figure 3.10 Lower part of the seaward clay layer.

Figure 3.11 Clay layer on the landward dike slope.
3.3 Excavation of the grass sods

During the last week of January 2008, the grass sods were excavated from the dike crest of the southern wing dike near Ribe (Denmark). The excavation of the 80 grass sods lasted 3 days. The size of one grass sod was 2.35m in length and 1.25m in width. The thickness of the grass sods varied from 17cm to 22cm.

In detail, a tractor was equipped with an attachment composed of one horizontal blade and two smaller vertical blades. The cutting width of the horizontal blade measured 1.25m. The attachment was assigned to cut underneath and at both sides of the grass sod (Figure 3.14). While pulling the attachment, a wooden plate connected to the attachment by two chains was pulled underneath the grass sod at the same time. The wooden plate measured 2.35m in length, 1.25m in width and was 2.5cm thick.

After the wooden plate was pulled under the grass sod, the cutting process was stopped and the fork of a fork lifter was pushed under the wooden plate to lift up the grass sod on the plate (Figure 3.15). Afterwards the grass sod edges were cut straight by hand (Figure 3.16) and the grass sod was transported from the dike crest down to an adjacent field for further action (Figure 3.17). This procedure was applied at all 80 grass sods.
For further handling and transportation, four square timbers were installed under the plate (Figure 3.18) and a wooden frame was built around the grass sod to avoid any damage or the appearances of additional fissures (Figure 3.19). All grass sods were stored on the field close to the wing dike for 1-3 days before they were transported to Hannover.

3.4 Installation of the grass sods

After the grass mats were unloaded from the trucks in Hannover (Figure 3.20) by a fork lifter, the 80 grass sods were temporarily stored outside the Large Wave Flume (Figure 3.21) to allow for continued natural growth with natural light and weather conditions. The disadvantage of doing so was an increase of the moisture content of the soil due to possible precipitation. In order to avoid this, the grass sods were covered with a plastic cover in case of rainfall or snowfall forecast.

The square timbers and the wooden frame were removed before installation of the grass sods. Each grass sod was placed with the fork lifter on a wooden framework (Figure 3.22 & Figure 3.23) which allowed an easier preparation of the sods for installation. Two holes were drilled into the underlying wooden plate which was needed for the later removal of the plate under the grass sod.
To avoid longitudinal joints running all way up both dike slopes, it was decided to install the grass sods in displaced order. The grass sods were installed transverse to the flume and in rows. One row consisted of three grass sods with lengths of 0.9m, 2.35m and 1.45m. These three lengths covered the entire width of the flume (5m). The row positions of the grass sods with lengths of 0.9m and 1.45m were switched in every row. The 2.35m long grass sod always remained as the central grass sod. By that, longitudinal joints running non-stop up both slopes were avoided (Figure 3.24).

In order to get grass sods of 0.9m and 1.45m length, a number of 2.35m long grass sods had to be cut. First the underlying wooden plate was cut using a circle saw and then the grass sod was cut with a steel wire. For transportation of the grass sods from the entrance of the Large Wave Flume to the dike model, a steel beam and a wooden transport frame were used which made it possible to hook the grass sod to the crane runway (Figure 3.25). The wooden transport frame was installed between the steel beam and grass sod to avoid the tilting of the grass sod during transport (Figure 3.26). The steel beam was braced to the grass sod and the wooden transport frame by synthetic ropes (Figure 3.27). Three different steel beams and two different wooden frames were used in order to handle the three different sizes of grass sods. At both holes which before were drilled into the plate, hooks were installed (Figure 3.29). These hooks were designed for pulling the wooden plates underneath the grass sod after being placed on the dike slope.
At the dike model, the grass sod was placed at its installation position (Figure 3.28) and the wooden transport frame, steel beam and synthetic ropes were removed. After one row of grass sods was completed, the steel hooks (Figure 3.29) were connected to a second set of synthetic ropes (blue ropes in Figure 3.30). Next, a wooden beam (5m long) was installed above the row of grass sods (Figure 3.31) by fixing it between the flume walls using wooden wedges. The function of the wooden beam was to provide a bearing during the process of removing the wooden plates under the grass sods.

The crane runway of the Large Wave Flume was used for pulling the wooden plates underneath the grass sods. The synthetic ropes were connected to a steel cable (Figure 3.32) which again was hooked to the crane hook. The steel cable was led over a deflexion pulley to
avoid a lean traction as the crane hook only moves in vertical direction. The deflexion pulley was fixed to a steal beam, which was placed across the flume (Figure 3.33).

First, the wooden plate under the 2.35m long grass sod was removed followed by the two smaller grass sods (Figure 3.34). After installation of the grass sods, all gaps and joints between the sods and the concrete flume walls were closed with clay. The clay, which was filled into the joints, was compacted by hand.
The installation of the grass sods was started at the toe of the seaward slope and continued up to the dike crest. Afterwards the grass sods on the landward slope were installed, again starting at the dike toe. Finally, grass sods were installed at the dike crest whereby the grass cover was closed (Figure 3.35). After installation of the entire grass cover, all gaps and joints were again checked, especially the joints along the flume walls.

In order to plane the surface of the grass cover and to strengthen the connection between the grass sods and the underlying clay layer, the grass sods were compacted by using a vibrating plate (Figure 3.36). After a few trials on the seaward slope, local damage of the grass cover occurred (red circles in Figure 3.36 and Figure 3.37) and the compaction of the grass sods was stopped.
3.5 **Artificial lightning and irrigation of the grass cover**

Artificial lighting of the grass cover was needed since the light quantity inside the Large Wave Flume was not enough to support grass growth. In order to establish the best conditions for grass growth, special lamps were used for illumination. Each lamp had a power of 800W.

![Figure 3.38](image1.png) **Illumination of the landward slope.**

![Figure 3.39](image2.png) **Six illumination sets for artificial lightning of the grass cover.**

Six illumination sets at three different lengths (Figure 3.39) were hung up 2.0m above the grass cover parallel to the dike surface (Figure 3.38 and Figure 3.40). The long illumination set consisted of four lamps (Figure 3.39, type a). The medium set comprised three lamps (Figure 3.39, type b) and the short set consisted of two lamps (Figure 3.39, type c). The lamps were rigged to aluminium ladders. The aluminium ladders were chosen in order to have a light construction that allowed to be carried by two persons.

The lamp sets were hung up above the dike surface with sisal ropes. The ropes were fixed to the railing at both sides of the flume. Since the sets had to be removed before the tests and installed again after the tests, flexible fasteners were used. The sets were moved by the crane runway and stored on the flume gangways during testing. Installation and displacement of all six sets lasted about 30 minutes.

![Figure 3.40](image3.png) **Cross view of the positioning of the artificial illumination sets.**

Figure 3.41 shows the positioning of the illumination sets in plan view. On the seaward slope three long sets (type a) were installed. The dike crest was partly illuminated by the most...
upper set (type a) of the seaward slope and by a short set (type c). The toe of the landward slope was illuminated by a medium set (type b) and the main part of the landward slope was illuminated by a long set (type a).

![Diagram of artificial illumination sets](image)

*Figure 3.41 Plan view of the positioning of the artificial illumination sets.*

The grass cover was illuminated during the recovery period, which was a 4-week long period between the end of construction and the start of the test programme. This period was chosen to allow the grass cover to recover after installation. During the recovery period the lamps were switched on for 24 hours. During the test period, the grass cover was only illuminated between the test runs.

At a later point in time during testing, an overtopping container was installed above the landward slope. The installation of the overtopping container caused, however, that the position of the long lamp set (type a) and the medium lamp set (type b) had to be changed.

During the recovery period the grass cover was irrigated four times using a common irrigation system (Figure 3.43). Furthermore, the grass cover was mowed 3 times using a power mower since the slope of the landward side was too steep for a simple lawn mower (Figure 3.42). The stem length was between 4 and 5cm after mowing. The swath was removed to avoid mouldering of the grass.

![Mowing the grass cover](image)

*Figure 3.42 Mowing the grass cover on the seaward slope.*

![Irrigation of landward slope](image)

*Figure 3.43 Irrigation of landward slope.*
3.6 Pumps

Two pumps, being available at the Large Wave Flume, were installed on the landward side of the dike model to empty the reservoir for overtopping water. The overtopping water was pumped back to the seaward side of the dike model. The pumps were connected to the internal pipe system of the Large Wave Flume by using steel pipes (Figure 3.44). The pump wells were protected by a fine wire mesh to avoid suction of grass leaves and swards. The wire mesh was fixed around the pumps by using small sand containers which were placed on the flume bottom (Figure 3.44, red circle). After each test run with wave overtopping, the wire meshes were cleaned to avoid a decrease of the pumping capacity.

Figure 3.44 Two pumps on the landward side.
4. Measuring and observation techniques

The main objective of the test programme was to investigate the failure of the grass layer on the seaward and landward slope due to (i) wave impact, (ii) wave run-up and run-down flow and (iii) wave overtopping. For this purpose measuring and observation devices were installed on both dike slopes, such as:

- wave gauges,
- pressure transducers,
- velocimeters (mini-propellers),
- overtopping container,
- video and photo cameras.

The used measuring and observation devices are described in the following. The description includes, besides technical data, also the type of installation and the positioning at the dike model.

4.1 Wave gauges

Three arrays including four wave gauges each were installed. The positions of the wave gauges are listed in Table 4.1 (see also Figure 4.1).

<table>
<thead>
<tr>
<th>Array</th>
<th>Wave gauge</th>
<th>Distance from the wave paddle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Array 1</td>
<td>1</td>
<td>50,1m</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>52,2m</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>55,9m</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>61,3m</td>
</tr>
<tr>
<td>Array 2</td>
<td>5</td>
<td>79,05m</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>81,15m</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>84,85m</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>90,25m</td>
</tr>
<tr>
<td>Array 3</td>
<td>9</td>
<td>116,0m</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>118,0m</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>120,0m</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>122,0m</td>
</tr>
</tbody>
</table>
4.2 Pressure transducers

To measure the wave impacts on the seaward slope, five pressure transducers (PT) with a capacity of 5bar were installed on the seaward slope. The distance between two pressure transducers was approximately 1.25m corresponding to the width of one grass sod row. The horizontal distance was 1.2m (Figure 4.2). This distance was chosen as it fits with the width of one grass sod and avoids an installation of the transducers inside one grass sod. It was expected that installation inside a grass sod would have negative impact on the stability of the grass sod, since the sod would had to be cut for installation of the transducer. The distance of the pressure transducer from the flume wall was 1.2m.

For installation of the pressure transducers five green plastic pipes were placed in a depth of 10cm in the clay layer before installation of the grass sods (Figure 4.3). A thin pull rope was placed in each cladding tube (Figure 4.4) to allow for pulling the measuring cable that connects the pressure transducer with the receiver. Figure 4.5 and Figure 4.6 show the installed cladding tubes in the grass cover layer.

The pressure transducers were installed at the end of the recovery period of the grass cover. The plug connector of the transducer was pulled through the cladding tube together with a new pull rope, in the case that a pressure transducer got damaged and had to be changed.
during testing. The transducer was mounted to a 1m long and 12mm thick reinforcing steel which was driven into the clay layer. The transducer was flushed with the topsoil surface and the hole around the cladding tube as well as the tube itself was filled with clay to avoid intrusion of water (Figure 4.7 and Figure 4.8).

Figure 4.3 Channel for the cladding tubes.

Figure 4.4 Empty cladding tube including the pull rope inside.

Figure 4.5 Location of the empty cladding tubes.

Figure 4.6 Measuring cables and cladding tubes.

Figure 4.7 Installed pressure transducer.

Figure 4.8 Pressure transducer after a test run.
4.3 Velocimeters (Mini-propellers)

On the seaward slope five “Schiltknecht” mini propellers (head diameter of 22mm) (Figure 4.10) were installed to measure the velocity of run-up and run-down flow. The positioning and orientation of the propellers is shown in Figure 4.9 and Figure 4.12. The “Schiltknecht” mini propellers measure velocities from 0.02 to 5.0m/s. To avoid disturbances of the propeller by the grass cover or grass swards, the propellers were installed at an adequate distance above the grass cover (Figure 4.11). Furthermore, the grass sward was cut in the areas around the propellers.

Figure 4.9 Locations of “Schiltknecht” mini propellers.

Figure 4.10 Schiltknecht” mini propeller.

Figure 4.11 Installed “Schiltknecht” mini propeller.

Figure 4.12 Orientation of mini propeller.

4.4 Overtopping container

To measure the volume of wave overtopping, an overtopping container was installed at the landward slope. The container was located 4.0m from the landward edge of the dike crest and consisted of a steel frame (Figure 4.13), a container, an inlet and a pump. The rigid steel frame was mounted at two steel beams above the landward dike slope. The steel beams
(Figure 4.13, left side) were placed across the flume and bolted to the gangway at both sides of the wave flume with built-in U-profiles. The installed steel frame and its position in the flume are shown in Figure 4.14.

The container was seated on three bearings. Two bearings were rigid and the third bearing consisted of a load cell. This transducer recorded the container weight during a whole test run, i.e. changes by overtopping water were also recorded. The entrance of the inlet (Figure 4.15) was located at the landward edge of the dike crest. To avoid the inflowing water to overflow the container, a rebound wall was fixed to the container wall opposite to the inlet (Figure 4.16).

Due to the limited capacity of the overtopping container, the captured water had to be removed continuously to avoid an overflow of the container. The water was pumped over a hose (Figure 4.16) into the reservoir behind the dike model.
4.5 Observation techniques

Video cameras

All tests were recorded by two digital video cameras. The two digital JVC cameras, including a hard disk of 60GB (GZ-HD3E), produced videos with a resolution of 1440x1080. The locations of the video cameras were changed depending on the test phases.

Phase 1 - Wave impact

During wave impact tests (phase 1) the digital cameras were installed at the southern gangway of the flume (Figure 4.17), one camera pointing towards the seaward slope (Figure 4.18) and the other camera pointing in opposite direction (Figure 4.19).

Phase 2 - Wave overtopping

During tests comprising wave overtopping (phases 2), the digital video cameras were also installed at the southern gangway of the flume (Figure 4.20). One video camera was installed above the seaward slope pointing towards the dike crest (Figure 4.21). The second camera was installed at the landward side of the dike model pointing towards the landward slope (Figure 4.22).
Figure 4.20 Camera locations during wave overtopping tests (phase 2).

Figure 4.21 View from camera 1.

Figure 4.22 View from camera 2.

Photo cameras
Grass sods, which were damaged during wave impact tests, were photographed after each test run. Degradation of a grass sod, being located at the northern flume wall in the third row (row C), is shown in Figure 5.24. The signboard in Figure 4.23 shows the hydrodynamic parameters of the performed test run. The first number in the first row defines the peak period \( T_p \) (‘50’ = 5.0sec.). The second number represents the significant wave height \( H_s \) (‘08’ = 80cm) and the last number stands for the water depth \( d \) (‘37’ = 3.7m). The abbreviations in the second and third row stand for ‘SS’ = seaward slope and ‘C-L’ is the identification code of the grass sod (‘C’ = row C; ‘L’ = left grass sod of the row form the middle). Identification of the rows starts with ‘A’ at the seaward toe in upward direction. The last row at the landward...
side is stated with code ‘P’. The positions of the grass sods in one row are coded with ‘L’ = left grass sod, ‘M’ = middle grass sod, and ‘R’ = right grass sod. The positions of the grass sods are defined by standing in front of the dike model and looking at the seaward slope. In some cases, the middle grass sod was coded with ‘ML’, ‘MM’ and ‘MR’. ‘ML’ means left side of middle grass sod, ‘MM’ stands for the middle part of the middle grass sod, and ‘MR’ represents the right side of the middle grass sod.

The last row on the signboard informs about the date of the photo.

Figure 4.23 Signboard.

Figure 4.24 Degradation of grass sod L in row C after a test run with wave impact.

Grid

After installation of the grass cover a grid was painted on the northern flume wall. The distance between the vertical lines was 1.0m, whereas the distance between the horizontal lines was 0.5m. The grid was painted following the entire dike surface (seaward slope, crest, landward slope) (Figure 4.21 and Figure 4.22).
5. Test programme

The test programme was divided into two phases. The first phase comprised tests regarding the initiation of grass erosion on the seaward slope due to wave impact and wave run-up and run-down flow. In the second phase the initiation of grass erosion on the landward slope due to wave overtopping was investigated.

In the following, a short overview of the hydraulic parameters of all performed tests is given. Afterwards both test phases are described including photos of the tests and example plots of the measured data.

5.1 Hydraulic parameters

The applied wave spectra based on a TMA spectrum. The water level in the flume was kept constant during test phase 1 and test phase 2. The effect of a tide could not be simulated in the flume.

The following tables list the hydraulic parameters of each test run, i.e. peak period $T_p$, significant wave height $H_s$ and the water depth $d$. The parameters of the wave impact tests (phase 1) are listed in Table 5.1. Table 5.2 shows the hydraulic parameters of the wave overtopping tests (phase 2).

Table 5.1 Hydraulic parameter of wave impact tests (phase 1).

<table>
<thead>
<tr>
<th>Impact</th>
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<td>$T_p = 5.0s$</td>
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<td>$d = 3.7m$</td>
<td>$d = 3.7m$</td>
<td>$d = 3.7m$</td>
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</tbody>
</table>

Table 5.2 Hydraulic parameter of wave overtopping tests (phase 2).

<table>
<thead>
<tr>
<th>Overtopping</th>
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</tr>
</thead>
<tbody>
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<td>$T_p = 5.5s$</td>
<td>$T_p = 6.0s$</td>
<td>$T_p = 6.5s$</td>
</tr>
<tr>
<td>$H_s = 1.0m$</td>
<td>$H_s = 0.75m$</td>
<td>$H_s = 0.85m$</td>
<td>$H_s = 0.9m$</td>
</tr>
<tr>
<td>$d = 4.7m$</td>
<td>$d = 5.0m$</td>
<td>$d = 5.0m$</td>
<td>$d = 5.0m$</td>
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</table>

After each test the Large Wave Flume was drained for water, which lasted about 4 hours. The filling of the flume took about 5 hours. Consequently, testing was normally interrupted by one day with no tests.

5.2 Phase 1: Wave impact

The first test phase included the wave impact tests. After each test run damage of the grass cover was surveyed and documented by photos. If the grass cover was damaged, the grass sod concerned was removed and replaced by a new grass sod.

The wave run-up was recorded by video camera 1 (Figure 5.1), and the shape of the breaking wave was recorded by camera 2 (Figure 5.2).
5.2.1 Example records of wave pressure

Test phase 1 focused on the effect of wave impacts on the grass cover and clay layer underneath. The wave pressure on the seaward slope due to wave breaking around the still water level was measured by five pressure transducers (see section 4.2 and Figure 4.2). In addition, the waves on the seaward slope were measured by wave gauges. Wave run-up and the breaker type were recorded by video cameras (Figure 4.17).

The following plots in Figure 5.3 to Figure 5.8 show the recorded wave height at wave gauge 12 (see Figure 4.1 and Table 4.1) and the recorded induced pressure impacts on the seaward slope in the area of wave breaking (Test 2204080).

Figure 5.3 Recorded wave height at WG 12.
Figure 5.4  Recorded wave pressure on PT 1.

Figure 5.5  Recorded wave pressure on PT 2.

Figure 5.6  Recorded wave pressure on PT 3.
5.2.2 Example of recorded current metres

The following plots in Figure 96 to Figure 99 show the recorded run-up velocities on the seaward slope in the area of wave breaking (Test 2204080) for the hydraulic parameters of $T_p = 5s$, $H_s = 0.9m$ and $h = 3.7m$. 

Figure 5.7 Recorded wave pressure on PT 4.

Figure 5.8 Recorded wave pressure on PT 5.

Figure 5.9 Recorded velocity at Schiltknecht propeller MP1.
5.2.3 Wave run-up

The wave run-up heights are analysed based on the recorded videos.

\[ T_w = 5s \]
\[ H_w = 0.5m \]
\[ h = 3.7m \]

Figure 5.13 Wave run-up height of 5.65m.
5.2.4 Damage of the grass cover due to wave impact

The grass cover was surveyed for damage after each test run with the objective to describe
- the instantaneous damage caused by single breaking wave impact events and
- the damage over the entire duration of the test.

Damage by single wave impact events

The first damage of the grass cover occurred closed to the observation window in the flume wall during the tests on April 11th, 2008. The wave parameters were $T_p = 5.0s$ and $H_s = 0.9m$. The water depth was about 3.7m. The damage was caused by a single impact. As shown in Figure 5.15, only a portion of the 90cm wide grass sod was damaged. The original stage of the grass sod is shown in Figure 5.14. The dimension of the damaged area is illustrated in Figure 5.16. However, the marked area A in Figure 5.16 shows an area of the grass sod that was not eroded directly in front of the observation window. After investigation of the soil surface of the damaged grass sod, it was noticed that the visible clay was not part of the clay layer underneath the grass sod, rather the clay of the grass cover as the hole was just 10cm deep (Figure 5.17).
In order to repair the grass cover layer, the remaining grass sod (area A, Figure 5.16) and the underlying clay were removed. Moreover, the hole was enlarged wherewith the new grass sod piece for repair sized 53cm in width and 70cm in length (Figure 5.18 and Figure 5.19). The repaired grass sod can be seen in Figure 5.20.

![Figure 5.17 Depth of hole.](image)

![Figure 5.18 Width of the prepared hole.](image)

![Figure 5.19 Length of the prepared hole.](image)

![Figure 5.20 Repaired grass sod.](image)

**Damage over entire test duration**

During the entire test duration, different kinds of damage or stages of damage were observed. For example, round balls of different size were observed on the dike surface (Figure 5.21) and within the topsoil. It is, however, important to notice that the development of these balls was not the result of lose clay lumps being moved up and down the slope. These balls were also observed in the soil with a dense root network (Figure 5.22).

In some cases, the clay material which was used to close the joints between the grass sods was removed and had to be replaced. Moreover, small holes (Figure 5.23) were registered after the tests.

The degradation of the grass layer in the surf zone caused by both test periods (phase 1 and phase 2) is shown in Figure 5.24. The grass cover (swords and leafs) in the lower part of the seaward slope remained green and alive longer than the grass within the breaker zone. The
grass cover that was permanently inundated was much less degraded than the grass cover in the breaker zone. After three days without testing, new small grass leafs were noticed.

Figure 5.21  Formation of clay balls lying on the slope surface.

Figure 5.22  Formation of clay balls located in the grass sod.

Figure 5.23  Open joint between two grass sods.

Figure 5.24  Degradation of the grass cover in the breaker zone.

5.3  Phase 2: Wave overtopping

The objective of test phase 2 was to investigate the effect of wave overtopping and run-down flow on the landward grass cover. The installed overtopping container (see section 4.4, Figure 5.25) was used to collect a certain part of the overtopping water. The load cell of the overtopping container recorded continuously the changing weight of the container due to the inflow of overtopping water through the inlet as well as due to the lowering of the water level in the container by pumping out the water. The overtopping processes on the landward slope and on the dike crest were recorded by video camera 1 (Figure 4.21) and video camera 2 (Figure 4.22).
5.3.1 Example of wave overtopping records

The overtopping discharge was continuously measured over the entire test duration of test phase 2. Both the individual and average overtopping discharge and their influence on the grass sods are analysed.

Different distinctive points can be seen in Figure 5.27. In Point ‘a’ water flows into the container and its weight increases. In point ‘b’ a constant water level in the overtopping container can be noticed. When the maximum possible water level was reached in the container, the water was pumped out and the pressure decreased. This action can be seen by the vertical line in point ‘c’. At the end of pumping out the water, the pressure increased again immediately (point ‘d’). This effect was caused by water that was flowing back from the hose into the container after the pumps were switched off.
Besides recording the load of the container, the start time and end time of pumping water out of the overtopping container, was written down. The start and end time of pumping actions according to the record in Figure 5.27, are listed in Table 5.3. In the case water had run-over the container, a remark was made in the table.

Table 5.3 Start and end of pumping action (overtopping test 280408).

<table>
<thead>
<tr>
<th></th>
<th>Start</th>
<th>End</th>
<th>Remarks</th>
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<tr>
<td>1.</td>
<td>12:23:07</td>
<td>12:24:04</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>12:24:45</td>
<td>12:25:52</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>12:30:17</td>
<td>12:30:27</td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>12:30:42</td>
<td>12:31:15</td>
<td></td>
</tr>
<tr>
<td>5.</td>
<td>12:35:32</td>
<td>12:36:02</td>
<td></td>
</tr>
<tr>
<td>6.</td>
<td>12:41:27</td>
<td>12:42:00</td>
<td></td>
</tr>
<tr>
<td>7.</td>
<td>12:45:40</td>
<td>12:46:07</td>
<td></td>
</tr>
<tr>
<td>9.</td>
<td>12:54:05</td>
<td>12:54:12</td>
<td></td>
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<tr>
<td>10.</td>
<td>12:55:00</td>
<td>12:55:58</td>
<td></td>
</tr>
<tr>
<td>11.</td>
<td>13:06:32</td>
<td>13:07:10</td>
<td></td>
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5.3.2 Damage of clay and grass cover by wave overtopping

During the wave overtopping tests, severe damage of the landward grass cover due to wave overtopping was not observed.

However, the seaward grass cover was continually damaged. Due to the increased water level, the breaker zone moved upwards whereby the damaged grass sods were found in row K (Figure 5.28). Furthermore, damaged joints between grass sods on the seaward slope were registered (Figure 5.29).

Figure 5.28 Damaged grass cover on the seaward slope during wave overtopping tests.
Figure 5.29 Damaged joint on the seaward slope during wave overtopping tests.
6. Concluding remarks and outcome

The main objectives of the EroGRASS project was to perform large scale model tests in order to investigate in detail the failure of grass cover layers due to (i) wave impact, (ii) wave run-up and run-down flow and (iii) wave overtopping.

Wave impact as well as wave run-up and run-down flow may induce grass cover failure on the seaward dike slope. Wave overtopping may cause failure of the grass cover at the dike crest and on the shoreward slope. Hence, this research project dealt with the investigation of grass cover failure along the entire dike profile: seaward slope, dike crest and shoreward slope.

Valuable measurements and observations have been obtained in breaking wave impact and other processes such as wave up rush and down rush together with subsequent damage of the grass cover layer at the seaward slope. Furthermore, the loading of the grass cover on the shoreward slope was investigated.

6.1 Lessons learned

During the entire project a number of lessons were learned. These experiences are described in the following. First, lessons learned during model set-up and model construction is described, followed by the experiences gained during both test phases.

- The surface of the seaward and shoreward slope was partly uneven and rough. Hence, it is recommended to make the surface more even. This can be achieved if the grass sods have the same thickness. However, it is difficult to go for a constant thickness of each grass sod as the grass layer of a ‘real’ sea dike is a natural product. The method of excavating the grass sods at the Ribe dike showed that cutting the grass sods in horizontal direction is difficult but not impractical. Weather conditions during excavation of the grass sods were very poor. Less precipitation the days before excavation and a more advanced method to control the cutting depth will result in a more constant thickness of the grass sods.

- The approach used for installation of the grass sods has to be improved in the future. The method used implied that every grass sod had to be handled very carefully to avoid additional cracks and fissures. This again was very time-consuming and asked for much strenuous manual work. Tools, such as hydraulic shields, should be developed to reduce the amount of manual work. The disposability of only one crane in the Large Wave Flume turned out to be also time-consuming as the one crane was used for many operations which again resulted in a number of re-settings of the crane.

- Due to the natural structure of the grass sods, the surface of the installed grass cover was uneven. On the seaward side a couple of buckles were observed after installation of the grass cover layer. These buckles justified the attempt of using a compactor to regulate the surface and to improve the contact between the grass sods and the clay layer. However, the application of the compactor was difficult on the seaward slope and implicated local damage of the grass cover. The soil within the topsoil started to liquefy and moved up towards the surface. The grass swards were partly or completely covered by the soil.

- A further problem caused by the uneven seaward slope surface was the influence of the grass sod edges on the stability of the grass sod itself. In some cases, a small step between the lower and the upper grass sod was noticed, which were loaded by wave run-up and run-down flow.
• The lightening, mowing and irrigation of the grass layer was important and enabled the grass layer to grow satisfactorily.

During both phases of testing, the following observations and experiences were made:

• After the first tests (phase 1) earth worms became active and aerated the topsoil and weakened the upper soil layer of the grass cover. Many small mounds of about 5mm in height were observed.

• The seaward slope of the dike model was selected to 1:4 in order to investigate wave impact on the grass layer. At this slope, the run-down water of the foregoing wave was not able to act as a natural damper for the next wave impact. This resulted in a faster degradation of the grass cover during testing. During the second test phase (wave overtopping), the seaward grass cover was already in a decayed condition that implied constant inspection of the entire seaward grass cover layer to avoid its overall failure. This development of the seaward grass cover could have been avoided if the seaward slope would have been 1:6. Furthermore, a simultaneous testing of the different wave loads on both dike slopes would have resulted in a more contemporaneous degradation of both grass covers on the seaward and landward slope.

• The horizontal dike toe on the landward side prevented a complete run-off of the overtopping water. Residual overtopping water remained in this area after each test run. Mouldering of the grass cover was not observed, but the quality of the grass in this area was less compared to other areas on the landward slope.

• No border effects or negative influence of the flume wall on the experimental procedure were observed. Inspection of each grass sod and each joint after every test run was necessary and appropriate. Damage of grass sods and joints were, through this, found immediately.

• The pumping capacity to empty the overtopping container was not large enough. A maximum overtopping rate was limited to about 30l/(s m). Overtopping discharges larger than 30l/(s m) could not be measured without introducing measurement uncertainties as overtopping water swashed out of the container.

• The inlets of the pumps in the reservoir behind the dike model were very easily blocked. The blockage of the inlets decreased the pumping capacity which again caused a rapid increase of the water level in the ‘hinterland’ and a decrease of the water level on the seaside by 10-20cm, since the overtopping water was pumped to the seaward side of the dike model. A simultaneous adjustment of the water level in front of the dike model through the flume inlet was not possible. Re-pumping of the overtopping water has therefore to be improved.

• For future work, it is recommended to have moisture content probes and pressure transducers installed at different depths across and along the dike model to be able to monitor moisture content profile changes and pore water pressure profile changes during wave impact.

6.2 Outcome
The analysis of the obtained data, which has started, will focus on the following aspects:

• Analysis of the hydrodynamic processes associated with failure at the seaward dike slope and their implication for erosion. This includes data and observations
concerning wave run-up and run-down velocities, layer thickness and the implication on erosion and other failure modes. Furthermore, the wave impact pressure caused by surging breakers will be analysed together with an analysis of the impact load time history.

- Investigation of the hydrodynamic processes associated with grass cover erosion at the dike crest and on the shoreward slope in relation to mean wave overtopping rates.
- Analysis of grass cover failure regarding the entire duration of testing and considerations concerning reiterating wave loading events in short chronology.

The analysis of the test data and the implications for the erosion of the grass will be published in a second project report at a later date.
Acknowledgement

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The design, construction and experimental procedure of the large scale model tests were conjointly performed by the FLOODsite Project and the EroGRASS Project. The collaboration between the FLOODsite Project and the EroGRASS Project was very useful for the implementation of this most extensive test programme. Hence, the close teamwork with Mr. Geisenhainer is also gratefully acknowledged.
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Appendix A: Preliminary Data Analysis Report

A1 Data analysis

Introduction

A brief overview of the collected test data and some basic results from the preliminary data analysis are given in this appendix.

In the first section the selected concept of the data analysis is described followed by a description of the methodology for the preliminary analyse of the incident wave parameters and mean wave overtopping discharge. The preliminary results of the data analysis are given in section 2 and, finally, conclusions and recommendations are drawn in third section.

The following data were collected during the performed model tests:

- wave conditions in front of the structure,
- wave overtopping volume per wave,
- wave run-up and run-down flow velocities at seaward slope,
- video records of wave impact on the seaward side and wave overtopping on the landward side,
- pressure measurements on the seaward slope.

A detail description of the applied measuring equipments and their positions are given in chapter 4.

Data analysis tools

The data analysis and visualisation software L-Davis, developed by LWI, is the main tool for the preliminary data analysis. L-Davis performs reflection analysis, time frame setting, statistical analysis of generated waves both in time and frequency domain, signal filtering, event analysis etc.

Most of the tests were recorded with 500Hz frequency. Therefore, before analysing the data, the correct data step had to be set. If the data step for the wave gauge data is set to e.g. 10, the frequency of the wave data set, which is used for the analysis, is 50Hz. Depending on the requirements and the expected accuracy, data steps for all channels had to be carefully selected at the beginning of analysis.

A brief description of L-Davis is given on the website of the Leichtweiß-Institute (LWI, http://www.lwi.tu-bs.de/hyku/english/en_Ldavis-index.html). The latest versions of L-Davis includes help files and can be downloaded from the homepage.

Overview of data

An overview of the available test data is given in Table A1.1. Each measuring device was calibrated before the tests. Data of the first three tests, however, do not have calibration factors due to a problem during data acquisition. Therefore, the calibration factors found for the tests on 2008.04.16 were used for the data analysis of the tests conducted on 2008.04.08, 2008.04.10 and 2008.04.11. Since the test configurations were similar, it was assumed that the calibration factors remained unchanged during these tests. Results from the calibration of the tests, which were conducted afterwards, justify this assumption.
Furthermore, information about the locations of measuring instruments from channel no. 18 to 27 were missing in the “GTX” files (see Figure A1.1) of the data set of test phase 1. Hence, the positions of the measuring devices had to be corrected in all the “GTX” files before data analysis. L–Davis software facilitates this operation.

**Figure A1.1 Typical folder arrangement for data analysis with the L–Davis software.**

**Incident wave parameters**

Three wave gauge arrays were installed in the flume, each consisting of 4 wave gauges. The first array was 50m from the wave maker, the second in a distance of 84m from the wave maker and the third array was installed 120m from the wave maker (Figure A1.2).

**Figure A1.2 Locations of wave gauge arrays**

Measurements from the third wave gauge array are used for the reflection analysis and for the assessment of the wave parameters at the dike toe. The wave parameters $H_{m0}$ and $T_{m-1.0}$ are used as the characteristic parameters. All model tests consist of approximately 1000 waves, which allow for the performance of a statistical analysis on wave parameters.
Wave data were filleted to remove noise using frequency filtering tool provided in the L-Davis software. The frequency of 0.05Hz was used as high pass filter and 0.6Hz as low pass filter. Figure A1.3 shows the frequency spectra using two different filters (top: 0.05Hz as high pass filter and 1.0Hz as low pass filter; bottom: 0.05Hz as high pass filter and 0.6Hz as low pass filter). The wave data analysis showed, that the data contained considerable amount of noise between the frequencies of 0.6Hz and 0.7Hz. It was therefore decided to use 0.05Hz and 0.6Hz as high and low pass filters, respectively.

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Table A1.1 Overview of available test data.

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<th>Date</th>
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<th>MAY</th>
</tr>
</thead>
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<tr>
<td>Wave period Tp [s]</td>
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</table>

Wave overtopping

The overtopping water was collected by a 0.20m and 0.15m wide chute at the landward edge of the dike crest and discharged into an overtopping container. The overtopping container was placed on a load cell which recorded the weight of the container. Figure A1.4 shows a schematic diagram of the wave overtopping measuring arrangement. By recording the weight changes of the container, conclusions on the wave overtopping discharges can be made. When the maximum possible water level in the container was reached, water was pumped out.

By analysing the signal of load cell under the overtopping container results about the mean overtopping discharge and overtopping volume per wave are received. From these results, the mean overtopping discharge per width is calculated.

Figure A1.4 Wave overtopping arrangement

Overtopping data contain a considerable amount of noise and all the overtopping data was filtered with a low pass filter of 2Hz (all signals of more than 2Hz were removed). Figure A2.4 shows a typical plot of a filtered overtopping signal. Due to the limited capacity of the overtopping container, collected water was pumped out during the model test (see Figure A2.4). However, water was only pumped out of the container at moments with no

**Flow velocity on the seaward dike slope**

Wave run-up velocities were measured with velocimeters (mini-propellers). The mini-propellers were frequently disturbed by debris (grass cut), as expected during the planning phase. The analysis of the velocimeter data should therefore be done with cautious.

**Pressure measurement at the seaward dike slope**

A preliminary analysis of pressure data was done under the FLOODsite project and detail analysis is still going on at the LWI.

**A2 Results**

This section describes results from the preliminary data analysis of the model tests. Measured wave parameters were compared with nominal wave parameters, which were given as input to the wave maker. Overtopping results were compared with the formulae given in EurOtop (2007). Only data from the second test phase was used in the preliminary analysis.

**Incident wave parameters at the dike toe**

All calculations were based on the incident wave parameters at the dike toe. Figure A2.1 shows the applied methodology during the preliminary analysis of the wave parameters in front of the dike.

![Figure A2.1 Methodology followed during preliminary data analysis.](image-url)
**Results from the wave analysis**

Incident wave parameters during test phase 2 are plotted in Figure A2.2 and Figure A2.3. As shown in Figure A2.2, incident wave heights are 0% to 10% lower than the nominal values. Also, the wave period $T_{m0,-1}$ is 10% to 20% lower than the nominal $T_p$ value (see Figure A2.3). Data from the second test run on April 30th, 2008 (red circle) show, however, a clear deviation from the general trend.

*Figure A2.2 Measured wave height during test phase 2.*

*Figure A2.3 Measured wave periods during test phase 2.*
Mean overtopping discharge

Mean overtopping discharges were calculated by adding all filling events and dividing the sum by the entire test period. Pumping water out of the container was carried out in periods with no overtopping discharges. As the number of pumping events increased, the event analysis tool of the L-Davis software was used to identify the pumping events in the recording (see Figure A2.4). All the selected events were checked with the manual records of the pumping events. All overtopping events were then added and divided by the total duration of the test in order to find the mean overtopping discharge.

Figure A2.4 Typical overtopping record (Nominal Wave Parameter: H = 0.85m and T = 6.0s).

The mean overtopping discharges were compared with the formulae given in EurOtop (2007). Figure 2.5 shows the preliminary results from the overtopping analysis. The mean overtopping discharge measurements show a reasonable agreement with the guidelines provided by EurOtop (2007).

Figure A2.5 Mean overtopping discharge measurements.
A3 Conclusions and recommendations

The results of the preliminary data analysis show a reasonable agreement between the expected and measured values of the wave parameters in front of the dike and the mean overtopping discharges. The overtopping volume per wave can be found from the time series data of wave overtopping measurements. However, the results should be cross-checked with the manual pumping records as well as videos.

The analysis of the data from the velocimeter should be done with cautious. The video records can be analysed with the objective to find the flow velocities on the dike slopes based on the grid drawn on the flume walls.

Finally, it is recommended to use the L-Davis software for the detail data analysis since it fully supports the format of acquired data and includes a number of possibilities to control the equality of the output.

References


Appendix B: Data storage plan

All data collected during the EroGRASS project, including survey data, video recordings and photograph files are stored on desktop hard drives and on a data server. There will be no storage on CD-ROMs.

Data back-up is ensured by saving all project data on three desktop hard drives with a storage capacity of 500 GB each. One desktop hard drive is placed at a safe-deposit box by the project management. The remaining desktop hard drives are used for analysis work and data exchange between the users. Besides data back-up on two desktop hard drives, a third copy of the project data is saved on the FTP-server of the Large Wave Flume in Hannover.

Project data are available as survey data from wave and water level gauges in the wave flume as well as video recordings for visual documentation of the behaviour of the grass layer. Survey data are saved in a binary format which relates to the recording instruments of the Large Wave Flume in Hannover. The data format is readable by the software L-davis. L-davis is the data analysis and visualization software of the Leichtweiß Institute at the Technical University of Braunschweig.

Video recordings are saved as MOD and TOD files. MOD and TOD are informal names of tapeless video formats used by JVC and Panasonic in some models of digital camcorders. An explanation of these file extensions is not available, however MOD is used exclusively for standard definition video files, while TOD is used for high definition files.

Standard software such as Windows Media Player is not able to read MOD and TOD formats. However, a simple renaming from MOD/TOD to MPEG and the use of the software MpcStar allows playing the videos.
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