Hydraulic loading of flood defence structures

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SUMMARY

This report reviews previous investigations on hydraulic loading conditions for different types of flood defence structures such as sea dikes, dunes, beaches and seawalls. The physical processes are discussed in some detail and methods are given to either predict relevant input parameters for failure modes of flood defences or the behaviour of the flood defence system under actions from waves or currents.

With respect to sea dikes this report first reviews the influence of foreshore mobility on hydraulic boundary conditions by performing a sensitivity analysis with the Boussinesq-type model TRITON on a foreshore for which results from field measurements exist. This study provides insight into the influence of variations of the level of the bar, the trough behind the bar, and the low-tide terrace in front of a dike. In addition, estimates have been made of the amount of low-frequency energy, depending of characteristics of the foreshore and the wave conditions.

Furthermore, previous investigations on flow processes on sea dikes induced by wave run-up and wave overtopping have been revisited. Contributions by Schüttrumpf (2001) based on small- and large-scale model tests and by Van Gent (2002) based on small-scale model tests have been reviewed including their prediction methods for flow depths and water velocities at all positions along the dike surface.

The report then summarises investigations performed outside FLOODsite within the European IHP-ARI programme. Within the DIKE-3D project measurements of flow depths and velocities at the inner dike slope have been performed under 3D conditions. Results are compared to the previously mentioned investigations for the reference case (perpendicular wave attack) and angles of wave attack up to 60 degrees.

The analyses on dunes showed that run-up levels and sand dune erosion had great effect on the investigated beach during the last decades. In case of a future sea level rise, the foreshore width and the sand dune volumes are expected to decrease drastically. From existing forecasts, the direct sand dune erosion was calculated to increase by up to 75% by the year 2100. This implies that many houses and other infrastructure located behind the dunes may be subject to more frequent flooding. The study also indicated that a run-up level, with a 100-year return period today, in the future may occur up to 13 times more frequently. It should be pointed out that the presented calculations were, in many cases, made on schematized representative profiles which all are case-specific. For this reason, the presented numbers have to be interpreted with care.

Eventually, the report deals with seawalls. First, a definition of seawalls is given and the most relevant parameters with respect to the hydraulic loading conditions and the structural conditions are introduced. The loading variables are then discussed in some detail where first, the breaker type is discussed and then the various loading conditions leading to potential functional (wave overtopping) or structural failure (toe erosion) is discussed. The last part of this chapter discusses the input variables in much detail with respect to both sea level and waves, and their interaction, and providing examples. Eventually, this leads to recommendations on how to perform a seawall design, with references to most relevant publications.
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1 Introduction

1.1 Motivation
With climatic changes as recently published within the latest IPCC study the estimation and predictability of extreme events becomes increasingly important. Changing environment along the coastlines, increased storminess and more frequent and heavy rainfalls together with increasing temperatures in Europe will change the sources of risk dramatically in the near future. Hence, both statistical tools and the loading of flood defence structures will become increasingly important and good background knowledge of the processes at flood defence structures is required to understand and to be able to predict future effects of changing environments.

This report aims to provide a technical background of some loading processes at flood defence structures. It is mainly focussing on coastal structures with some generic introductory sections on the general concept. Additionally, the report will give details of methods to describe the modifications of wave parameters travelling over the foreshore and will then continue to describe loadings of sea dikes, beaches, dunes and seawalls.

These flood defences are very often complex in shape and it is not easy to describe the loading processes involved. New knowledge in the area of coastal structures is gathered and reviewed here to be able to give guidance on best suitable methods for the prediction of loading of coastal flood defence structures.

1.2 Background
The present study is done within the framework of the European FLOODsite project. The main aim of Task 2 is the assessment of climatic extremes as the main driver for flooding and erosion processes. The emphasis is on the issues contributing most to uncertainty in flood risk management decisions. The analyses are carried out for single climatic variables and for realistic combinations including the “control” exerted by the domain morphodynamic evolution (i.e. marginal, conditional and joint PDFs).

Task 2 focuses on three activities:
   a) Theoretical analysis methods: distribution types and selection, fitting and resampling techniques, temporal sequences and memory effects. This activity will also include SSA (Singular Spectrum Analysis), NN (Neural Networks), POT (Peak-Over-Threshold), GPD (General Pareto Distribution) and spatial/temporal correlations.
   b) Analysis of extreme events: extreme samples (limited) and distributions, 2D distributions and CCA. This activity will also consider the morphodynamic control on extreme PDFs and the coupling between long-term (decadal) climatic trends and coastal processes.
   c) Hydraulic loading of flood defence structures: wave transformation over shallow foreshores; wave induced fluxes and wave-soil-structure interactions. This activity will also deal with the analysis of different flood mechanisms and the corresponding impact on coastal morphology.

The research in Task 2 is targeted at issues contributing most to present uncertainty in flood risk management decisions. It benefits from the combined knowledge and expertise of the atmospheric, riverine and marine research communities.
The objective of the task is to improve our understanding of the primary drivers of flood risk (waves, surges, flow etc.) through research targeted at key issues that contribute most to current uncertainty in flood risk management decisions. These "sources" are defined as the climatic factors inducing flooding, erosion or any other "threat" to the safety/stability of the land-water fringe. These sources therefore include sea waves, storm surges, river water levels and discharges.

The research described in the present report is part of the third activity: *Hydraulic loading of flood defence structures*. The main objectives of this subtask are to predict the wave transformation over shallow foreshores with various complex processes such as percolation, detailed friction and turbulence parameterisations. The emphasis will be on extreme wave evolution over very shallow depths. Furthermore, the hydrodynamic loading of natural and man-made flood defences is studied in much more detail so that appropriate hydrodynamic parameters are available to describe the loading of flood defences at all points of interest, including the associated uncertainties.

### 1.3 Structure of report

The report starts with an overview of the general concept (chapter 2) describing the underlying source-pathway-receptor-consequences (SPRC) model and a distinction in between coastal, fluvial and estuarial risk sources. Details in this first chapter will then be given regarding the sources and the pathways as well as the calculation of the failure probability of coastal defence structures and in particular the problems of dealing with multiple stochastic variables.

Chapter 3 deals with the transformation of waves over shallow foreshore with the help of numerical models. The Boussinesq-type wave model TRITON is described and used here to model the wave transformation and to quantitatively describe the changes of waves travelling over such foreshores.

The loading of sea dikes is dealt with in chapter 4. The principal results described here are going back to large- and small-scale test results on model dikes where detailed measurements were recorded for wave run-up velocities, flow depths, and overtopping waves. The full hydraulic loading of these structures is therefore described by available formulae. Additionally, comparisons are made to 3-dimensional tests which have been performed recently in the 3D wave basin at DHI, Denmark. Simple formulae derived from these tests are also given.

The loading of dunes is described in chapter 5. The chapter describes the physical processes which are involved in dune erosion. These processes are then analytically described in detail where special emphasis is laid upon wave run-up on beaches, overwash and dune erosion. The relevant models are given and analytical results will be discussed.

Finally, chapter 6 describes the wave-induced loading of seawalls. This will essentially comprise methods to assess the overtopping rate of seawalls, the toe erosion and the loading of the seawalls themselves.

The report concludes with a summary and some general recommendation for various coastal defence structures.
2 General Concept

The purpose of this chapter is to discuss the relationship between the risk sources (large waves, high river flow etc) and the risk pathways (probability of flooding), in the context of multiple source variables and multiple flood mechanisms. For example, does one:

- estimate only the probability of the most likely flood mechanism at the location most likely to flood?
- estimate the probabilities of all flood mechanisms at all locations and add them all together?
- estimate the probabilities of all flood mechanisms at all locations, decide which would tend to occur at the same time, and then estimate an overall probability of flooding?
- estimate which parts of the joint probability density of the source variables would cause flooding of one or more types at one or more locations?

All of these approaches have their place, depending on the area concerned, the types of flood defence and the particular problem to be addressed.

2.1 Background

2.1.1 Source, Pathway, Receptors Model

The Source, Pathway, Receptor, Consequence model is often used in flood risk research and is one of the analytical models used in FLOODsite.

- **Source** refers to the environmental variables potentially causing a flood risk, e.g. waves, river flow, sea level, rainfall and wind.
- **Pathway** refers to the action of Source on assets at risk from flooding, e.g. overtopping or breaching of sea walls, overflow of river banks, inundation of land and buildings, overfilling of drains, structural damage and injury.
- **Receptor** refers to assets suffering a loss due to flooding, e.g. flood defences, land, buildings and people.
- **Consequence** refers to the associated nature and value of the loss.

This report considers Source and Pathway, and the potential problems involved in considering multiple sources and multiple pathways. It both provides the general concept of the close relation between these two elements of the model as well as specific details of loading conditions for specific types of structures. The report therefore describes the boundary between Source and Pathway and discusses the problems related to it.

2.1.2 Division into coastal, fluvial and flash floods

In simplified terms ‘coastal flooding’ is caused by high sea levels, large waves, and to a lesser extent wind and current, and combinations thereof. ‘River flooding’ is caused by an elevated water level in the river, in turn caused by high river flow and high downstream sea level. ‘Flash floods’ are caused by rainfall too intense to be removed by sewers or natural drainage.

The three types tend to be thought of separately in design and assessment and so are considered separately in parts of this report. Figure 1 illustrates this separation, as well as the separation of Source and Pathway. It also lists the relevant source variables and some of the flood mechanisms for each type.
2.2 Sources for flood risks

2.2.1 Source types
The source variables relevant to flood risk are listed in Figure 1 (brackets implying possible relevance, depending on the location, and gaps separating the rare and unpredictable geological disturbances).

2.2.2 Source data
As illustrated in Figure 1, information on the magnitude and frequency of high values (or low values of temperature, causing ice) of the source variables can be found in several types of information. Ideally, long-term measurements close to the location of interest would be used, but the available measurements may be of too short duration or too far away from the site. A weather model may provide broader data coverage of some of the variables of interest, but possibly at too coarse a spatial resolution. A site-specific numerical model will usually be used, perhaps calibrated against measurements or wave model data, to predict conditions in terms of the variables and locations required for flood risk estimation.

Design guide values, literature review results and local experience provide additional information which may be used for initial calculations and for checking purposes.
2.2.3 Source analysis
As noted in Figure 1, information on the source variables may need to be processed to a more useful form before use in flood risk calculations. This will often involve tabulation as long-term annual and seasonal distributions, and extrapolation to extreme values. It will often involve the use of a numerical model to infer results for longer periods of time and/or for additional locations. It will sometimes involve joint probability analysis of multiple source variables.

2.3 Pathway analysis

2.3.1 Coastal flood mechanisms
Coastal flooding may be caused by a sustained (for a few hours over high tide) period of heavy overtopping of sea defences, without significant structural damage. The rate of overtopping is predictable, to order of magnitude accuracy, from the wave conditions, sea level, wall crest elevation and other characteristics of the wall and foreshore.

Severe coastal flooding usually involves breaching, i.e. destruction and removal of a length of sea defence, exposing previously protected land at an elevation close to or even below the still water level. Although the process of breaching can be modelled, and the condition of a sea defence can be monitored, prediction of the onset of breaching is very uncertain. At best, a probability of breaching can be estimated for any given sea and defence conditions.

Often, a sand or shingle beach either will be the whole of the sea defence or will provide protection to a sea wall or embankment. A large beach movement, either offshore or alongshore, during storm conditions will alter the standard of service of a sea defence, possibly to an extent regarded as ‘failure’ if it results in enough sea water being allowed onto the land behind.

The three mechanisms outlined above would not necessarily occur in isolation. Beach movement may allow larger waves to reach an embankment behind the beach, causing erosion of the front face of the embankment, causing overtopping, causing erosion of the back face of the embankment, eventually causing breaching.

Failure of sea walls may occur due to gradual damage accumulated over several separate storms, and may be initiated by undermining of the toe of the wall, perhaps over-exposed following beach movement.

2.3.2 Fluvial flood mechanisms
River flooding may begin with overflow of water into a vulnerable area, following a gradual rise in the water level in the river, caused by high river flow, high sea level and/or high rainfall. This can be predicted with more confidence than coastal flooding.

If there is no elevated river defence, the distinction between overflow and breaching may be small, but breaching would involve destruction and removal of a length of flood defence. However, breaching of a levee, protecting low-lying land behind, would allow a much higher flow of water to inundate the land. Although water level can be predicted and the condition of
a flood defence can be monitored, prediction of the onset of breaching is uncertain. At best, a probability of breaching can be estimated for any given river and defence conditions.

**Dam-break** involves structural failure causing sudden release (categorised here as being a high river flow, causing a high flood risk downstream of the broken dam). **Ice blockage**, **landslide blockage** and **seepage** are also mentioned in Figure 1.

### 2.3.3 Flash floods

Flash flooding is categorised an immediate response to a very intense but relatively short spell of rainfall, exceeding the capacity of a river to drain it. Flooding due to **drainage exceeded** is a similar mechanism caused by the drainage capacity of a sewer system being exceeded during a short spell of very intense rainfall. Both of these may be made worse by **ice blockage** or **debris blockage**.

**Groundwater flood**, **ground erosion** and/or **dam-break** may also be triggered by ground wetness and surface water during intense rainfall.

### 2.3.4 Information for modelling of flood mechanisms

As illustrated in Figure 1, the type of information about a defence needed to determine the most likely flood mechanisms relate to its design, crest level and conditions.

**Defence type** indicates sea wall, shingle beach, river wall, flood plain, drainage system etc. **Crest level** is the elevation at which still water would begin to overflow a coastal or river defence. **Defence profile** is intended here to include shape, slope, toe design and material. **Defence condition** and **drain condition** represent current quality relative to designed quality, taking account of deterioration, partial blockage etc. **Ground wetness** is relevant in estimating the proportion of rainfall running into drains, and **drain capacity** refers to the rate of flow that can be drained.

### 2.3.5 Methods for representation of flood mechanisms

The methods are assumed to have available to them information on the magnitudes and frequencies of the source variables and the physical information on the design and condition of the defences. As illustrated in Figure 1, the types of method used include design formulae, empirical models, numerical models, physical models, and literature review of other similar conditions.

### 2.4 Evaluation of flood probability

#### 2.4.1 Case of a single source variable causing a single flood risk

Consider the idealised case of a well-maintained small harbour, accessible from the sea but completely protected from wave conditions, with houses built at ground level on its landward side. Assume that the only relevant source variable is sea level and the only flood mechanism is overflow of the harbour wall reaching the houses. For sea levels high enough to cause flooding, the probability of a given sea level being exceeded equals the probability of the
associated level of flooding being exceeded. For example, the 50 year return period sea level will necessarily cause the 50 year return period level of flooding.

One needs to introduce only a small variation to this idealised situation to disturb that neat relationship between sea level and flooding. In reality, wind and wave effects would permit the same extent of flooding on a slightly lower sea level. Gradual deterioration of the harbour structures and/or gradual climate change would also disturb the relationship. The possibility of damage to the houses by the direct effects of very intense rainfall would also slightly add to the flood risk.

The point of this introductory example is to show that even in a seemingly simple case, the relationship between the probabilities of the source variable(s) and the overall flood probability may not be trivial.

2.4.2 Handling of multiple source variables

Ideally, one would like to have perfect information, not only on the distributions and extremes of each source variable of interest at the location of interest, but also on the dependences between each pairing or grouping of variables. The perfect way of generating such information would be to take simultaneous sequential measurements of all the variables at the location of interest over a very long period of time. This may be a sensible approach where long time series measurements are available, perhaps supplemented, for example, by wave hindcasting from wind measurements or calculations to transform measurements to be representative of a more relevant location.

More commonly, statistical modelling is used to represent the joint probability of occurrence of multiple source variables. This type of approach seeks to combine all available information on the distribution and extremes of each of the source variables, and additional information on the dependence between them. Except in the relatively trivial cases of independence or full dependence, this type of approach can be used for two or three primary source variables, but becomes impractical for more than three variables. Extrapolation of the information available within the source data sets can be done either analytically or by long-term simulation. Handling of multiple source variables is discussed in depth in Use of joint probability methods in flood management: A guide to best practice (Defra / Environment Agency, 2005).

The more variables involved and the more complex the methods used to represent them, the less transparent becomes the relationship between the distributions of the source variables and the probability of any flood risk variables derived from them.

2.4.3 Handling of multiple flood mechanisms

Multiple flood risks might exist where, either different flood mechanisms could occur, e.g. seawall overtopping and seepage through a damaged seawall, and/or different defence sections protect the same area of land.

A common approach is to determine which is the most likely flood mechanism and which is the most vulnerable defence section, for example overtopping of the defence section with the lowest crest height. Then the flood risk associated with that wall section and flood mechanism
is taken to represent the flood risk for the area. Whilst this is usually good enough, it will tend to under-estimate the total flood risk for the area.

An alternative approach might be to estimate the probability of each of the flood mechanisms at each seawall section, and add the resulting probabilities together. However, this approach is not used in practice and would be overly conservative, since the separate risks would tend to occur at the same time, which is during severe weather conditions, and there would be much double-counting of effectively the same flood risks.

A best estimate of the overall flood risk, would be based on the probability of one or more defence sections being affected by one or more flood risks. One approach would be to represent flood risk by defined “failure” whose probability of occurrence can be tested using available information on the probability of the source variables. For coastal flooding, the failure criteria might be that a certain volume of seawater enters a certain land area, that one or more seawall sections breach, or that overtopping rate exceeds an acceptable level on one or more sections. For rainfall flooding it might be that standing water exceeds a certain depth at one or more locations within a town. Handling of multiple flood mechanisms is discussed in *Risk, performance and uncertainty in flood and coastal defence: A review* (Defra / Environment Agency, 2002).

A further complication is that defence performance may depend on pre-existing conditions such as beach level, catchment wetness, whether earlier damage has been repaired and whether earlier blockages have been cleared. Also, structural failure is very difficult to predict and, at best, a probability of breaching can be estimated for given loading conditions. This uncertainty can be accommodated through the use of fragility curves (e.g. Sayers *et al.*, 2005) representing the probability of failure as a function both of load and of defence condition.

### 2.4.4 Handling of multiple source variables and multiple flood mechanisms

In most flood risk studies there will be multiple source variables and multiple flood mechanisms, and potentially all of the complications described both in Section 2.4.2 and in Section 2.4.3 will apply. In seeking to apply greater sophistication to some aspects of the calculations, one should also be aware of other aspects perhaps still treated in a relatively simple way.

Joint probability methods are capable of focusing upon the overall probability of failure of a defence, due to any number of different types of flood risk, perhaps occurring in different types of loading condition. A hierarchy of appropriate methods for handling multiple source variables and multiple flood mechanisms was developed within the UK *Risk Assessment of flood and coastal defences for Strategic Planning* (RASP) research programme. These methods were demonstrated and applied to national flood risk investment planning in the UK *National Flood Risk Assessment* (NaFRA) research programme. This risk-based approach to flood risk in the UK is described in Dawson *et al.* (2004), Defra / Environment Agency (2002, 2005), Hall *et al.* (2003a, 2003b, 2004 and 2005) and Sayers *et al.* (2001, 2002, 2003 and 2005).
3 Wave transformation over foreshore

3.1 Background
In this chapter a study on the sensitivity of hydraulic boundary conditions to foreshore characteristics is described. Processes on shallow foreshores affect the wave boundary conditions at coastal structures. Field measurements at the Petten Sea defence in The Netherlands (see Figure 2) have been used to study these processes. In Van Gent (2001) and Van Gent et al (2001) conditions that occurred during the field measurements were modelled in laboratory circumstances (2D and 3D). The measured wave conditions on the shallow foreshore and wave run-up levels on the dike could rather accurately be reproduced in the laboratory. The tests in a flume with second-order wave generation and active reflection compensation have been used to validate several numerical models including the time-domain Boussinesq-type model TRITON (see Van Gent and Doorn, 2001). It appeared that the wave propagation including several breaker zones with severe wave breaking could be modelled numerically rather accurately. This analysis was based on short waves; it did not address low-frequency energy in particular.

![Figure 2: Picture of the Petten Sea defence](image)

Except for the generation of low-frequency energy on shallow foreshores also the mobility of the foreshore affects the wave conditions that act on the sea defence. Sandy foreshores can have variations within storms, seasonal variations and more long term variations and trends. Variations of the order of magnitude of 1 m have been measured during storms at the Petten Sea defence. The effects of foreshore mobility on the hydraulic boundary conditions are at present not taken into account in the hydraulic boundary conditions for the evaluation of Dutch sea defences. The present study contributes to a better insight into the effects of foreshore variations on the hydraulic boundary conditions.
The time-domain Boussinesq-type model TRITON is used to model the wave transformation over the foreshore. The study will focus on the influence of the foreshore (slope, depth, etc.) on the hydraulic load on the flood defence structures. TRITON is a two-dimensional time-domain Boussinesq-type model with improved linear- and non-linear behaviour (Borsboom et al., 2000). The model has been extended with the implementation of a 2D wave breaking model (Borsboom et al., 2000).

Although the study has been performed on a real existing sea dike, the results can be extended to other types of structures as well. The relevant object of study is the local foreshore and not the type of structure. In addition, in the numerical model simulations the structure itself was replaced by a horizontal flat with equal depth as the local depth at the toe of the structure. Hence, the results can be considered to be generic for different kind of structures.

3.2 The Boussinesq-type wave model TRITON

3.2.1 Description of the numerical model

An important aspect in the dynamics of near-shore waves is the steepening and breaking of waves on shallow banks and in the surf zone close to the beach. The dissipation and nonlinear wave-wave interaction due to wave breaking have a significant effect on both the wave energy and its distribution over the wave spectrum. Both are important parameters in coastal dynamics studies. It is therefore essential to include an adequate description of wave breaking when modelling near-shore wave dynamics.

Boussinesq-type wave models are an attractive way to model wave dynamics in coastal regions. They describe frequency dispersion as well as nonlinear wave behavior up to high levels of accuracy if necessary, using a fairly cheap 2-D formulation involving only the free-surface elevation and horizontal velocities. This result is obtained by eliminating the vertical velocity and all vertical variations from the full 3-D free-surface flow equations, by approximating them in terms of the other variables.

The 2-D Boussinesq-type model used in this paper is the model that has been developed recently at WL|Delft Hydraulics under the name TRITON. The derivation of the model equations and the underlying motivation can be found in Borsboom et al. (2000). Here the governing equations are given only briefly.

The equations are formulated in the unknowns total water depth $H$ and depth-integrated velocity $\mathbf{q}$. The depth-integrated continuity equation can then be written as:

$$\frac{\partial H}{\partial t} + \nabla \cdot \mathbf{q} = 0$$

(1)

The momentum equation has been derived from the dynamic free-surface boundary condition of the potential flow model by means of a truncated series expansion. This leads typically to an equation in non-conservative form, but the result has been reformulated such that an equation expressing the conservation of depth-integrated momentum was obtained, using the freedom in design that exists in developing a Boussinesq-type model:
\[ \frac{\partial q}{\partial t} + \nabla \cdot (\bar{u}q) + \nabla \left( \frac{1}{2} u \bar{H}^2 \right) = g \left( \frac{3}{2} \bar{H} - \frac{1}{2} H + \frac{1}{4} H (\nabla h \cdot \nabla \zeta) \right) \nabla h \] 

(2)

with depth-averaged velocity \( \bar{u} = \frac{q}{H} \), and \( \zeta \) and \( h \) respectively the water elevation and water depth with respect to some reference level (hence \( H = \zeta + h \)). Auxiliary variable \( \bar{H} \) is a function of total water depth and \( \nabla h \) and given by:

\[
\bar{H} = H - \alpha H^2 \nabla^2 H - \beta H \nabla h \cdot \nabla H \nabla \bar{H} = H - (\alpha - \frac{1}{2}) H \nabla^2 H - (\beta - \frac{1}{2}) H \nabla h \cdot \nabla H - \frac{1}{2} (\nabla h \cdot \nabla h) H - \frac{1}{2} (H \nabla^2 h) H
\]

(3)

The model has been extended with the implementation of a 2D wave breaking model based on a combination of the eddy viscosity concept (Zelt 1991, Kennedy et al. 2000, Chen et al. 2000) and the surface roller concept (Schäffer et al., 1993; Sørensen et al., 1998). An algebraic viscosity term in conservative form is added to the momentum equation to include the effect of turbulent energy losses due to wave breaking. The concept of surface rollers is used for the modelling of the eddy viscosity coefficient. The combination has a number of features that makes it suitable for near-shore applications. Mass and momentum are strictly conserved while the wave breaking model only dissipated energy, which is in agreement with physical laws. The results and the comparison with experiments under very different wave conditions demonstrate the good performance of the model. See Borsboom et al. (2001) for a detailed description of the wave breaker model.

3.2.2 Validation of the numerical model

The sensitivity of hydraulic boundary conditions to foreshore characteristics is studied using the foreshore of the Petten Sea defence. This foreshore has relatively small longshore depth variations. Figure 3 shows the geometry of the foreshore, as measured in the field and as modelled on that basis in a wave flume. Besides reproducing prototype storms additional wave conditions were modelled in the wave flume. The storm conditions vary between mild storms and extreme design storms, ranging from mild to severe wave breaking at one or two shallow parts on the foreshore (i.e. the bar and the low-tide terrace in front of the dike).
Figure 3: Schematised foreshore of the Petten Sea defence.

The sensitivity analysis of hydraulic boundary conditions to foreshore characteristics is carried out using the Boussinesq-type wave model TRITON. An essential aspect of this model is that it can model severe breaking waves and the generation of low-frequency waves due to wave breaking (see e.g. Borsboom et al., 2000). In Van Gent and Doorn (2001) this model was used to validate the computed wave energy spectra and wave parameters for this foreshore, using the 20 conditions tested in the physical model without the dike in position. The conclusion was that the differences between the measured and computed wave heights ($H_{m0}$) and wave periods ($T_{m-1,0}$) of the incident waves at the toe of the sea defence were on average about 5 to 10% (based on the energy in short waves). These rather small differences indicate that the model is capable of computing the wave propagation of short waves (here: $f > 0.04$ Hz) over the foreshore with conditions ranging from mild wave breaking to severe wave breaking.

Figure 4 (Van Gent and Giarrusso, 2003) shows for the 20 conditions mentioned a comparison between measured and computed contribution of the low-frequency energy (here: $0.01 \text{ Hz} < f < 0.04 \text{ Hz}$) to the total amount of wave energy at the toe of the sea defence. This figure shows that the contribution of low-frequency energy is large (on average 20%) and that the Boussinesq-type model TRITON provides reasonable estimates of the ratio between low-frequency energy (LFE) and the total amount of wave energy.
3.3 Numerical model computations

The computations to study the sensitivity to foreshore variations have been carried out for different water levels, wave heights and wave steepnesses. The 12 conditions (many with severe wave breaking before reaching the toe) are given in Table 1.

Table 1: Wave conditions

<table>
<thead>
<tr>
<th>Condition</th>
<th>Water level (m, w.r.t. NAP)</th>
<th>Wave steepness</th>
<th>Wave height</th>
<th>Wave period</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.1</td>
<td>0.015</td>
<td>2</td>
<td>9.7</td>
</tr>
<tr>
<td>2</td>
<td>2.1</td>
<td>0.015</td>
<td>3</td>
<td>11.3</td>
</tr>
<tr>
<td>3</td>
<td>2.1</td>
<td>0.015</td>
<td>4</td>
<td>13.1</td>
</tr>
<tr>
<td>4</td>
<td>2.1</td>
<td>0.015</td>
<td>5</td>
<td>14.6</td>
</tr>
<tr>
<td>5</td>
<td>2.1</td>
<td>0.025</td>
<td>4</td>
<td>10.1</td>
</tr>
<tr>
<td>6</td>
<td>2.1</td>
<td>0.035</td>
<td>4</td>
<td>8.5</td>
</tr>
<tr>
<td>7</td>
<td>4.7</td>
<td>0.015</td>
<td>2</td>
<td>9.7</td>
</tr>
<tr>
<td>8</td>
<td>4.7</td>
<td>0.015</td>
<td>3</td>
<td>11.3</td>
</tr>
<tr>
<td>9</td>
<td>4.7</td>
<td>0.015</td>
<td>4</td>
<td>13.1</td>
</tr>
<tr>
<td>10</td>
<td>4.7</td>
<td>0.015</td>
<td>5</td>
<td>14.6</td>
</tr>
<tr>
<td>11</td>
<td>4.7</td>
<td>0.025</td>
<td>4</td>
<td>10.1</td>
</tr>
<tr>
<td>12</td>
<td>4.7</td>
<td>0.035</td>
<td>4</td>
<td>8.5</td>
</tr>
</tbody>
</table>

* Wave conditions refer to deep-water wave conditions (-20 m NAP), 1000 m seaward of the crest of the sea defence; at the toe (65 m seaward of the crest) the water depth is -0.5 m NAP.

Conditions 3 and 10 correspond more or less to conditions that are expected to occur at this site once in about 10 and 10,000 years respectively. The applied wave spectra were TMA-spectra and the number of computed waves was 500 for each condition. In the numerical model computations, and in tests used from the physical model, the sea defence was not included and replaced by a horizontal part of the foreshore, equal to the depth at the toe (-0.5 m NAP), with an open weakly-reflecting outflow boundary at the end. At this open boundary the waves can leave the computational domain, using the long-wave assumption to assess the phase velocity of the outgoing waves. The spatial step and time step in the computations were
set at $\Delta x = 1.0$ m and $\Delta t = 0.01$ s. All computations were carried out in the 1D-mode of the wave model, thus without directional spreading and variations in the longshore direction.

### Table 2: Foreshore geometries*

<table>
<thead>
<tr>
<th>Foreshore</th>
<th>Level of Bar (m)</th>
<th>Level of Trough (m)</th>
<th>Level of Low-tide terrace (m)</th>
<th>Toe (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BASIC</td>
<td>-3.5</td>
<td>-8</td>
<td>-2.5</td>
<td>-0.5</td>
</tr>
<tr>
<td>BAR +</td>
<td>-2.5</td>
<td>-8</td>
<td>-2.5</td>
<td>-0.5</td>
</tr>
<tr>
<td>BAR -</td>
<td>-4.5</td>
<td>-8</td>
<td>-2.5</td>
<td>-0.5</td>
</tr>
<tr>
<td>TROUGH +</td>
<td>-3.5</td>
<td>-6</td>
<td>-2.5</td>
<td>-0.5</td>
</tr>
<tr>
<td>TROUGH -</td>
<td>-3.5</td>
<td>-10</td>
<td>-2.5</td>
<td>-0.5</td>
</tr>
<tr>
<td>TROUGH +</td>
<td>-3.5</td>
<td>-8</td>
<td>-1.5</td>
<td>-0.5</td>
</tr>
<tr>
<td>TROUGH -</td>
<td>-3.5</td>
<td>-10</td>
<td>-2.5</td>
<td>-1.5</td>
</tr>
<tr>
<td>LT-TERRACE +</td>
<td>-3.5</td>
<td>-8</td>
<td>-3.5</td>
<td>-0.5</td>
</tr>
<tr>
<td>LT-TERRACE -</td>
<td>-3.5</td>
<td>-8</td>
<td>-2.5</td>
<td>-1.5</td>
</tr>
<tr>
<td>TOE -</td>
<td>-3.5</td>
<td>-10</td>
<td>-3.5</td>
<td>-1.5</td>
</tr>
<tr>
<td>DEEPEST</td>
<td>-4.5</td>
<td>-6</td>
<td>-1.5</td>
<td>-0.5</td>
</tr>
<tr>
<td>SHALLOWEST</td>
<td>-2.5</td>
<td>-6</td>
<td>-1.5</td>
<td>-0.5</td>
</tr>
</tbody>
</table>

* Levels are with reference to NAP.

A series of foreshore geometries was applied (see Table 2). In this first series of computations the level of a few characteristic locations was modified compared to the ‘basic’ foreshore (see Figure 2). This ‘basic’ foreshore corresponds to the actual foreshore of the Petten Sea defence. Figure 5 shows a graph with the 10 foreshores. In this figure also slopes of several foreshore sections of the ‘basic’ foreshore (denoted by symbols) are given.

![Applied foreshore geometries](chart.png)

**Figure 5: Applied foreshore geometries.**

From this set of $12 \times 10 = 120$ computations the wave heights and wave periods are calculated at several positions on the foreshore: MP3, BAR, MP5, MP6 and TOE (see also Figure 3).

Based on the analysis described in the next section, additional computations were carried out. In addition to variations of the vertical position of characteristics of the foreshore, also computations were performed with variations of the horizontal position of the low-tide terrace. A series of 10 additional foreshores has been used (10 computations with Condition 3). This has been done for two levels of the horizontal low-tide terrace (NAP -1.5 and NAP -2.5...
m) and 5 different widths of this low-tide terrace (between 0 and 90 m). Figure 6 shows this part of the foreshore between MP5 and the TOE. The low-tide terrace at a level of NAP -2.5 m and a width of 45 m is the one which almost corresponds to the ‘basic’ foreshore from Table 1; in the ‘basic’ foreshore the low-tide terrace is not horizontal but has a 1:100 slope.

Figure 6: Applied foreshore geometries (low-tide terrace).

3.4 Analysis

The computations with different foreshores contain conditions for which the wave propagation from deep water to the toe leads to wave heights $H_{m0}$ that reduce with about a factor 0.5 for some conditions, and wave periods $T_{m-1,0}$ that increase for some conditions up to a factor of about 1.5 (based on energy between 0.01 Hz < $f$ < 0.3 Hz). The ratio of low-frequency energy (0.01 Hz < $f$ < 0.04 Hz) and total wave energy (0.01 Hz < $f$ < 0.3 Hz) increases for some conditions from 0 at deep water to 30% at the toe.

Figure 7 shows results for one of the conditions (Condition 3 from Table 1) for the basic foreshore (dashed line), and the two foreshores with a higher and a lower level of the bar, with a total difference in level of the bar of 2 m. This figure shows that at the bar wave conditions vary but that these differences are much smaller for the wave conditions at the toe; the influence of the level of the bar on the conditions at the toe is very limited. Figure 8 shows for the same condition the two foreshores with a higher and a lower level of the trough, with a total difference in level of the trough of 4 m. It appears that also the level of the trough hardly influences the wave conditions at the toe. Figure 9 shows for the same condition the two foreshores with a higher and a lower level of the low-tide terrace, with a total difference in level of the low-tide terrace of 2 m. In contrast to the influence of the level of the bar and the trough, the level of the low-tide terrace affects the wave conditions at the toe considerably. Figure 10 shows results for the shallowest foreshore (with the bar, trough and low-tide terrace all at the shallowest level) and for the deepest foreshore (with the bar, trough, low-tide terrace and toe all at the deepest level). The comparison between Figure 9 and Figure 10 indicate again that the differences for conditions at the toe are mainly caused by the low-tide terrace; other variations in the foreshore have an influence on the local conditions but hardly affect the boundary conditions at the toe.
Figure 7: Influence of variations in the level of the bar

Figure 8: Influence of variations in the level of the trough.
Figure 9: Influence of variations in the level of the low-tide terrace.

Figure 10: Influence of shallowest and deepest foreshore.
Because the level of the low-tide terrace clearly affects the wave conditions at the toe the most, additional computations were done to study the influence of the width of the low-tide terrace (see also Figure 6). It appeared that there is some influence of the width of the low-tide terrace, although smaller than the influence of the level of the low-tide terrace.

**Low-frequency energy**

The described set of computations provides an opportunity to study the amount of low-frequency at the toe of the sea defence. See also the third panels in Figures 6-9 with the percentages of low-frequency energy (%LFE) as function of the position on the foreshore. The prediction of low-frequency energy is of interest since many other numerical wave models do not provide information on the amount of low-frequency energy but model the amount of energy in the short waves only. If such models that do not predict the generation of low-frequency waves due to wave breaking are applied, it is useful if the amount of low-frequency energy in the incident waves can be predicted based on available wave parameters. This is especially valuable because the amount of low-frequency energy affects processes like wave run-up and wave overtopping at sea defences, thus affecting the required crest level of sea defences. That the distribution of the energy over the frequencies does not only affect structures that are vulnerable to processes like wave run-up and wave overtopping, has been shown by Coeveld et al (2005) and Van Gent et al (2006). In these papers tests are described in which the influence of the wave period on dune erosion has been studied. It appeared that larger wave periods result in significantly larger amounts of dune erosion.

Thus, based on variations in foreshores for this particular foreshore with a bar, trough and low-tide terrace in front of the sea defence, the contribution of low-frequency energy to the total amount of wave energy is estimated.

Based on analysis of the results it appeared that the contribution of low-frequency energy can well be estimated using the following parameters/ratios:

- Wave steepness $s_{m-1,0}$ at deep water ($s_{m-1,0} = \frac{2\pi}{g} \frac{H_{m0}}{T_{m-1,0}^2}$)
- Ratio of the deep-water wave height $H_{m0}$ and the water depth $h$ at a distance of $1/3 L_0$ seaward of the toe
- Ratio of the deep-water wave height $H_{m0}$ and the water depth $h$ at a distance of $1/10 L_0$ seaward of the toe

where the deep-water wave length $L_0$ is calculated using the deep-water wave period $T_{m-1,0}$.

The expression calibrated based on all computations reads:

$$\frac{E_{\text{low frequency energy}}}{E_{\text{total energy}}} = 0.0025 \left( s_{m-1,0} \right)^{-1} \left( \frac{H_{m0\text{-deep}}}{h_{1/10 L_0}} \right)^{0.5} \left( \frac{H_{m0\text{-deep}}}{h_{1/3 L_0}} \right)^{0.5}$$

(4)

Figure 11 shows the low-frequency energy (LFE) contribution as calculated using Equation 4 versus the numerical model results. This figure shows that the computational results are reasonably parameterised by Equation 1. This means that given the deep-water wave parameters (without low-frequency energy) $H_{m0}$ and $T_{m-1,0}$ and the foreshore geometry an estimate can be obtained of the contribution of low-frequency energy (LFE) to the total amount of energy (TE) at the toe.

Important to note is that the measurement data, computations and analysis concern a 1D approach. For the site studied here the foreshore can reasonably well be characterised with
one foreshore geometry (parallel depth contours) but at other sites longshore variations may be much larger. Also the effects of reflected low-frequency waves and directional spreading, which are important for the amount of generated low-frequency energy, are neglected here. Since directional spreading usually reduces the amount of generated low-frequency energy due to wave breaking, the estimates of low-frequency energy can better be interpreted as an upper limit. In reality the amount or low-frequency energy is smaller than obtained from the 1D approach here. It is recommended to study the effects of directional spreading and other 3D effects on the estimates of low-frequency.

![Figure 11: Predicted contribution of low-frequency (LFE) to the total amount of wave energy (TE).](image)

### 3.5 Conclusions

The computations with the validated Boussinesq-type model TRITON provide insight into the effects of variations of the foreshore on wave conditions in front of a dike. For foreshores characterized by a bar, a trough and a low-tide terrace in front of a sea defence it appeared that the wave conditions at the toe of the structure are hardly affected by the level of the bar and the trough, but are strongly affected by the level of the low-tide terrace. The width of the low-tide terrace also has some effect. The conditions studied here are within the range of mild wave breaking up to severe wave breaking. Severe wave breaking occurs on the bar and on the low-tide terrace. In this study the decrease in wave height between deep water and the toe reaches levels of 50%, the increase in wave period reaches levels of 50%, and the contribution of low-frequency energy to the total amount of energy reaches levels of 30%.

A method to obtain predictions of the amount of low-frequency energy has been obtained based on the computational results. Based on deep-water wave parameters, together with foreshore characteristics, the contribution of low-frequency energy can reasonably well be estimated. These estimates should be considered as upper limits since the effects of directional spreading have not been quantified in this method. It is recommended to study the
influence of directional spreading and wave reflection, and to validate the developed procedure for other foreshore geometries.
4 Loading of seadikes

4.1 Investigations by Oumeraci et al. (2001)

In this chapter, hydraulic large-scale model tests on sea dikes are described in which measurements were performed regarding the flow processes on the seaward side of the dike, the dike crest and the landward side of the dike, respectively. These tests were performed in the Large Wave Flume (GWK) in Hannover. The wave flume has a length of 324 m, a width of 5.0 m and a depth of 7.0 m. Regular waves up to a height of 2.0 m and wave spectra up to a significant wave height of 1.4 m can be generated in the flume. The model tests comprised three phases:

- 1st Phase: model tests with an impermeable dike (analysis of hydraulic processes),
- 2nd Phase: model tests with small fields of clay on the crest and on the inner slope of the dike (analysis of infiltration and erosion processes) and
- 3rd Phase: model tests with clay on the crest and inner slope (analysis of infiltration and erosion processes).

For the analysis of the hydraulic processes which is the relevant subject within this report, focus will be on phase 1 in the following. The main parameters describing the flow processes at a sea dike can be stated as follows:

- the significant wave height $H_{1/3}$ [m] and the mean wave period $T_m$ [s] for describing the irregular sea state
- the wave run-up height $R_{u,2\%}$ [m] being exceeded by 2% of the incoming waves
- the water layer thicknesses $h_{2\%}$ and $h_{50\%}$ [m] being exceeded by 2% and 50% of the incoming waves, respectively, and the average layer thickness $h_A$ [m]
- the overtopping velocities $v_{2\%}$ and $v_{50\%}$ [m/s] being exceeded by 2% and 50% of the incoming waves, respectively.
- the overtopping rate $q$ [l/s·m]

In the following the model tests performed will be described with regard to the prediction of the aforementioned parameters.

4.1.1 General set-up

In Figure 12, the cross section of the wave flume and the dike tested within Phase 1 can be seen.
Within the first phase, a dike covered by an impermeable layer on both the outer (1:6) and inner slope (1:3) and on the dike crest (width of crest = 2.0 m) was tested. Wave gauges, pressure cells, velocity propellers, wave run-up gauges, layer thickness gauges and an overtopping measurement system were used to measure the following parameters:

- wave parameters in the flume and on the dike,
- pressures on the dike surface (seaward slope, crest and inner slope),
- velocities on the dike surface (seaward slope, crest and inner slope),
- wave run-up heights,
- flow depths (seaward slope, crest and inner slope) and
- overtopping volumes.

The position of the measurement devices at the dike tested within phase 1 is shown in Figure 13.
Figure 13: Position of measurement devices at the dike tested within phase 1 after Oumeraci et al. (2001)

The measurement system for the overtopping rate consisted of two devices: a discharge meter recording the overtopping water collected through a channel and a buffer at the crest of the dike and an overtopping container at the toe of the inner slope (see Figure 14).

Figure 14: Measurement system for overtopping rates used within phase 1

In addition to the aforementioned measurement devices, 3 cameras were used to record the wave run-up and overtopping processes at the outer slope, the dike crest and the inner slope which can be seen in Figure 15.
4.1.2 Test programme

Regular waves as well as spectra were used for the model tests. The irregular sea states used were TMA spectra, PM and natural multipeaked sea spectra gained from measurements at the German and Dutch North Sea Coast. Three different water levels were used, namely 3.50 m, 4.25 m and 5.00 m in combination with wave periods from 3.5 s up to 13.50 s and wave heights $H_{m0}$ from $H_{m0}=0.40$ m up to $H_{m0}=1.20$ m. During phase 1, 251 tests were performed in total, which comprised

- 24 tests with regular waves,
- 33 tests with TMA spectra,
- 146 tests with natural wave spectra and
- 48 additional tests (regular waves, PM spectra, JONSWAP spectra, natural wave spectra).

Figure 16 gives an overview over the natural spectra used for the tests.
Figure 16: Overview of natural multiplepeaked spectra used for model tests within phase 1

An example for the wave parameter combination used for the TMA spectra is shown in Table 3.
Table 3: Overview of parameter combinations for TMA spectra used for tests within phase 1

<table>
<thead>
<tr>
<th>$T_p$ [s]</th>
<th>$d$ [m]</th>
<th>$H_{m0}$ [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0.40</td>
</tr>
<tr>
<td>3.50</td>
<td>3.50</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>4.25</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>5.00</td>
<td>X</td>
</tr>
<tr>
<td>4.50</td>
<td>3.50</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>4.25</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>5.00</td>
<td>X</td>
</tr>
<tr>
<td>5.50</td>
<td>3.50</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>4.25</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>5.00</td>
<td>X</td>
</tr>
<tr>
<td>7.50</td>
<td>3.50</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>4.25</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>5.00</td>
<td>X</td>
</tr>
<tr>
<td>9.50</td>
<td>3.50</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>4.25</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>5.00</td>
<td>X</td>
</tr>
<tr>
<td>13.50</td>
<td>3.50</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>4.25</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>5.00</td>
<td>X</td>
</tr>
</tbody>
</table>

The range of the investigated parameters for the 1:6 dike can be seen in Figure 17.

Figure 17: Range of investigated wave parameters for the 1:6 dike

Figure 17 also shows the wave parameters of small scale model tests performed in the wave flume of the LWI which were used for comparison. For more detailed information about the test programme see Oumeraci et al. (2001).
4.1.3 **Data analysis procedure**

The data analysis was performed as a peak value analysis including the following hydraulic parameters (related measuring devices in brackets):

- incident wave parameters (wave gauges),
- layer thickness on the dike (layer thickness gauges),
- wave run-up and run-down on the seaward slope (wave run-up gauge),
- flow velocity on the crest and inner slope (micro propellers) and
- overtopping volume (discharge meter at the crest and weighing system at the toe of the inner slope).

Apart from these directly measured quantities the flow velocity on the dike was additionally determined by three other indirect methods:

- wave run-up and run-down velocity on the seaward slope (measured by pressure cells),
- velocity of the wave front over the profile (calculated by the time lag between the signal of two adjacent layer thickness gauges or pressure cells) and
- velocity of the wave front over the profile on the inner slope (from video records).

4.2 **Investigations by Van Gent (2001)**

Van Gent (2001) performed model tests with three different foreshores and three different dike geometries. In detail the following combinations were investigated:

- foreshore 1:100 with a 1:4 dike
- foreshore 1:100 with a 1:2.5 dike
- foreshore 1:250 with a 1:2.5 dike

The test programme comprised the use of single and double-peaked wave spectra in combination with four water levels. The wave trains consisted of approximately 1000 waves. The variation of wave and spectral parameters comprised:

- water depth: for single-peaked spectra within the range of \( H_{\text{m0}}/d = 0.4 \div 3.0 \), for double-peaked wave energy spectra between \( H_{\text{m0}}/d = 0.4 \div 1.5 \)
- wave steepness: for single peaked wave energy spectra within the range of \( s = 0.018 \div 0.044 \) and for double-peaked spectra between \( s = 0.020 \div 0.051 \)
- spectral shape: JONSWAP spectra were used as single-peaked spectra. The superposition of two JONSWAP spectra was used to create double-peaked spectra with the distance between the two individual peaks varying between \( T_{p2}/T_{p1} = 0.4 \div 1.0 \) and the ratio of energy in each individual spectrum varying between \( m_{0,2}/m_{0,1} = 0.5 \div 2.0 \).

4.3 **Formulae derived from 2D tests**

In the following the flow processes will be regarded separately depending on the location where they occurred, namely the outer slope, the dike crest and the inner slope. The formulae derived in this section summarises results from both investigations

4.3.1 **Wave run-up height**

The run-up height at the outer slope is the initialising process for the flow processes. Since the model tests in the GWK were focused on the overtopping processes, there were only a small number of run-up tests performed. Therefore, Oumeraci et al. (2001) derived a formula for the wave run-up based on small scale model tests in the wave flume of the LWI:
\[
\frac{R_u}{H_s} = c_1 \cdot \tanh \left( c_1^* \cdot \xi_d \right) \tag{5}
\]

with:

- \(R_u\) = wave run-up height [m]
- \(H_s\) = significant wave height at the toe of the dike [m]
- \(\xi_d\) = surf similarity parameter: \(\xi_d = \frac{\tan \alpha}{\sqrt{H_s/L_0}}\) [-]
- \(\alpha\) = seaward slope [-]
- \(L_0\) = wave length in deep water: \(L_0 = \frac{gT_{m,1,0}^2}{2\pi}\) [m]
- \(T_{m,1,0}\) = mean wave period [s]
- \(c_1, c_1^*\) = empirical coefficients [-]

The empirical coefficients were determined as follows:

- \(c_1=2.25\) and \(c_1^* = 0.50\) for regular waves
- \(c_1=3.0\) and \(c_1^* = 0.65\) for wave spectra with one peak and \(R_{u,2\%}\)
- \(c_1=2.25\) and \(c_1^* = 0.50\) for wave spectra with one peak and \(R_{u,50}\%

The tests without wave overtopping performed in the GWK showed good agreement with the wave run-up formula shown above. For the prediction of wave the wave run-up height at a sea dike, Van Gent (2001) derived the following formulae, where a distinction between different breaker types was made:

\[
\frac{R_{u,2\%}}{H_s} = c_0 \cdot \xi_{S,-1} = c_1 \frac{\tan \alpha}{\sqrt{H_s/L_0}} \quad \text{for } (\xi_{S,-1} \leq p) \tag{6}
\]

and

\[
\frac{R_{u,2\%}}{H_s} = c_1 - c_2 \cdot \xi_{S,-1} = c_1 - c_2 \frac{\sqrt{H_s/L_0}}{\tan \alpha} \quad \text{for } (\xi_{S,-1} > p) \tag{7}
\]

where

- \(R_{u,2\%}\) = wave run-up height, exceeded by 2% of the incoming waves [m]
- \(H_s\) = significant wave height at the toe of the dike [m]
- \(\alpha\) = seaward slope [-]
- \(T_{m,1,0}\) = mean wave period [s]
- \(\xi_{S,-1}\) = surf similarity parameter: \(\xi_{S,-1} = \frac{\tan \alpha}{\sqrt{H_s/L_0}}\) [-]
- \(L_0\) = wave length in deep water: \(L_0 = \frac{gT_{m,1,0}^2}{2\pi}\) [m]

and

\[
p = 0.5 \frac{c_1}{c_0}\tag{8}
\]
The use of $T_{m-1,0}$ instead of $T_p$ is recommended for single, double and multi-peaked sea states, see Schüttrumpf & Van Gent, 2003. Van Gent (2001) made a distinction between wave spectra which include short waves only (where $c_0 = 1.55$ and $c_1 = 5.4$) and spectra containing short and long waves (where $c_0 = 1.35$ and $c_1 = 4.7$). In this case, the coefficients $c_0$ and $c_1$ are restricted to the use of the wave parameters $T_{m-1,0}$ and $H_s$.

A wider range of validity of the coefficients for other wave periods ($T_p$, $T_{m-2,0}$, $T_{m-1,0}$, $T_{m0,1}$ and $T_m$) and for the use of the wave height $H_{m0}$ is given by Van Gent (2000) and is shown in Table 4.

**Table 4: Coefficients for prediction of wave run-up height $R_{u,2\%}$ for different wave periods, wave heights and energy spectra according to Van Gent (2000)**

<table>
<thead>
<tr>
<th>Wave period</th>
<th>Wave height</th>
<th>$c_0$</th>
<th>$c_1$</th>
<th>Wave energy spectra</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_p$</td>
<td>$H_s$</td>
<td>1.35</td>
<td>4.3</td>
<td>long and short waves</td>
</tr>
<tr>
<td>$T_{m-2,0}$</td>
<td>$H_s$</td>
<td>1.2</td>
<td>4.4</td>
<td>long and short waves</td>
</tr>
<tr>
<td>$T_{m-1,0}$</td>
<td>$H_s$</td>
<td>1.35</td>
<td>4.7</td>
<td>long and short waves</td>
</tr>
<tr>
<td></td>
<td>$H_{m0}$</td>
<td>1.55</td>
<td>5.4</td>
<td>short waves only</td>
</tr>
<tr>
<td></td>
<td>$H_{m0}$</td>
<td>1.45</td>
<td>3.8</td>
<td>long and short waves</td>
</tr>
<tr>
<td>$T_{m0,1}$</td>
<td>$H_s$</td>
<td>1.8</td>
<td>5.1</td>
<td>long and short waves</td>
</tr>
<tr>
<td>$T_m$</td>
<td>$H_s$</td>
<td>1.95</td>
<td>5.2</td>
<td>long and short waves</td>
</tr>
</tbody>
</table>

Schüttrumpf (2001) determined the parameter $c_0$ as $c_0 = 1.25$ and Van Gent (2002) determined $c_0 = 1.35$ and $c_1 = 4.0$ using $H_s$ and $T_{m-1,0}$.

**4.3.2 Flow depth at the outer slope**

For the determination of the layer thickness $h_A$ (see Figure 18), Oumeraci et al. (2001) found a linear distribution as follows assuming a linear decrease of the layer thickness from SWL to $R_{u,2\%}$:

$$
\frac{h_A(x_A)}{x_Z} = c_2 \left( 1 - \frac{x_A}{x_Z} \right) = c_2 \left( 1 - \frac{x_A}{c_1 \sqrt{H_s \cdot L_0}} \right) \quad [-] 
$$

with:

- $h_A$ = layer thickness [m]
- $c_1$ = empirical coefficient [-] for wave run-up according to Eq. (5)
- $c_2$ = coefficient to be determined by experiments [-]
- $x_A$ = horizontal coordinate [m] with $x_A = 0$ at SWL
- $x_Z$ = horizontal projection of the wave run-up height $R_{u,2\%}$ [m].
Figure 18: Definition sketch of flow depth at the outer slope of a sea dike according to Schüttrumpf & Oumeraci (2005)

For the simplification of Eq. (9) the remaining run-up length $x^* = (x_z - x_A)$ (see Figure 18) is introduced. Thus Eq. (9) can be written as follows:

$$h_A(x^*) = c_2 (x_z - x_A) = c_2 \cdot x^*$$

(10)

For the layer thickness $h_A$, different parameters were introduced as follows:

- $h_A$: mean layer thickness [m] for regular waves
- $\overline{h}_A$: mean water coverage [m] for regular waves and wave spectra
- $h_{A,2\%}$: layer thickness [m], exceeded by 2% of the incoming waves
- $h_{A,10\%}$: layer thickness [m], exceeded by 10% of the incoming waves
- $h_{A,50\%}$: layer thickness [m], exceeded by 50% of the incoming waves

The values for these empirical coefficients are given in Table 5.

Table 5: Coefficients $c_2$ for the layer thickness on the outer slope (1:6)

<table>
<thead>
<tr>
<th></th>
<th>$h = c_2 \cdot x^*$</th>
<th>$c_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regular waves</td>
<td>$h_A$</td>
<td>0.026</td>
</tr>
<tr>
<td></td>
<td>$h_A$</td>
<td>0.057</td>
</tr>
<tr>
<td></td>
<td>$\overline{h}_A$</td>
<td>0.010</td>
</tr>
<tr>
<td>TMA spectra</td>
<td>$h_{A,50%}$</td>
<td>0.028</td>
</tr>
<tr>
<td></td>
<td>$h_{A,10%}$</td>
<td>0.042</td>
</tr>
<tr>
<td></td>
<td>$h_{A,2%}$</td>
<td>0.055</td>
</tr>
<tr>
<td>Natural wave spectra</td>
<td>$\overline{h}_A$</td>
<td>0.010</td>
</tr>
<tr>
<td></td>
<td>$h_{A,50%}$</td>
<td>0.025</td>
</tr>
<tr>
<td></td>
<td>$h_{A,10%}$</td>
<td>0.042</td>
</tr>
<tr>
<td></td>
<td>$h_{A,2%}$</td>
<td>0.056</td>
</tr>
</tbody>
</table>

As the layer thickness $h_A$ showed to be dependent on the outer slope additionally, following formula was derived:

$$h_A = \frac{c_2^*}{n} \cdot x^*$$

(11)

with:

- $x^* = x_z - x_A$ (remaining wave run-up height)
- $c_2^* = c_2/n$
The coefficients $c_2^*$ for wave spectra were determined as follows:

\[
\begin{align*}
    h_{A,2\%} &= 0.34 \\
    h_{A,50\%} &= 0.17 \\
    h_A &= 0.06
\end{align*}
\]

In Schüttrumpf & Van Gent (2003) the following formula is given for the prediction of the layer thickness at the outer slope $h_{A,2\%}$:

\[
\frac{h_{A,2\%}}{H_s} = c_{A,h}^* \left( \frac{R_{u,2\%} - z_A}{H_s} \right)
\]

with

\[
\begin{align*}
    h_{A,2\%} &= \text{layer thickness [m] exceeded by 2\% of the incoming waves} \\
    H_s &= \text{significant wave height [m] at toe of structure} \\
    c_{A,h}^* &= \text{empirical coefficient [-]} \\
    R_{u,2\%} &= \text{wave run-up height [m] being exceeded by 2\% of the incoming waves} \\
    z_A &= \text{vertical coordinate [m] with respect to SWL}
\end{align*}
\]

The empirical coefficient is given as $c_{A,h}^* = 0.33$ Schüttrumpf, 2001 and $c_{A,h}^* = 0.15$ (Van Gent, 2002), respectively.

4.3.3 Run-up velocity

The run-up velocity is the other of the important parameters to describe the flow processes at the outer slope. In Figure 19, a definition of the relevant parameters is given, which will be referred to by the following equations.

\[
\frac{v_A}{\pi \cdot H} = a_0^* \cdot \cot \alpha \cdot \xi_d \cdot \sqrt{\frac{R_u \cdot z_A}{H}}
\]

with:

\[
\begin{align*}
    v_A &= \text{wave run-up velocity on the seaward slope [m/s], replaced by } v_{A,50\%} \text{ for wave spectra} \\
    a_0^* &= \text{empirical coefficient [-], gained by means of model tests} \\
    R_u &= \text{wave run-up height [m], replaced by } R_{u,2\%} \text{ for wave spectra}
\end{align*}
\]
\[ z_A = \text{vertical coordinate with respect to SWL} \ [m] \]
\[ H = \text{wave height} \ [m], \text{replaced by } H_s \text{ for wave spectra} \]
\[ T = \text{wave period} \ [s], \text{replaced by } T_m \text{ for wave spectra} \]

The coefficients \( a_0^* \) for regular waves and wave spectra and different statistical parameters are given in

Table 6.

**Table 6: Coefficients \( a_0^* \) for the velocity on the outer slope for large scale model tests**

<table>
<thead>
<tr>
<th></th>
<th>( a_0^* )</th>
<th>( c_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Regular waves</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( v_A )</td>
<td></td>
<td>1.03</td>
</tr>
<tr>
<td>( v_A,50% )</td>
<td></td>
<td>0.82</td>
</tr>
<tr>
<td>( v_A,10% )</td>
<td></td>
<td>1.09</td>
</tr>
<tr>
<td>( v_A,2% )</td>
<td></td>
<td>1.24</td>
</tr>
<tr>
<td><strong>TMA spectra</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( v_A,50% )</td>
<td></td>
<td>0.76</td>
</tr>
<tr>
<td>( v_A,10% )</td>
<td></td>
<td>0.98</td>
</tr>
<tr>
<td>( v_A,2% )</td>
<td></td>
<td>1.11</td>
</tr>
<tr>
<td><strong>Natural wave spectra</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In Schüttrumpf & Van Gent (2003) the run-up velocity \( v_{A,2\%} \) is defined in a slightly different form:

\[
\frac{v_{A,2\%}}{\sqrt{g \cdot H_s}} = c_{A,v}^* \cdot \left( \frac{R_{u,2\%} - z_A}{H_s} \right)^{\frac{1}{2}}
\]  

(14)

with

- \( v_{A,2\%} \) = wave run-up velocity exceeded by 2% of the incoming waves
- \( c_{A,v}^* \) = empirical coefficient

The empirical coefficient \( c_{A,v}^* \) is given as \( c_{A,v}^* = 1.37 \) after Schüttrumpf, 2001 and as \( c_{A,v}^* = 1.30 \) after Van Gent (2002), respectively. For more information see Schüttrumpf & Van Gent (2003).

### 4.3.4 Layer thickness at dike crest

A definition sketch for the layer thickness and the overtopping velocity at the dike crest is given in Figure 20.

![Figure 20: Definition sketch of layer thickness and overtopping velocity at the dike crest of a sea dike according to Schüttrumpf & Oumeraci (2005)](image_url)
The layer thickness at the dike crest $h_c$ is dependent on the layer thickness at the outer edge of the crest and the distance of the position related to the outer edge. The layer thickness at the crest decreases toward the inner slope because some of the water reaching the outer crest point runs back down to the outer slope while the remaining volume moves along to the inner slope.

The layer thickness $h_c$ at every point $x_c$ on the dike crest can be described by the following equation, which was gained empirically using the model tests in the GWK see Oumeraci et al. (2001):

$$\frac{h_c(x_c)}{h_c(x_c=0)} = \exp\left(-c_{c,h}^* \frac{x_c}{B}\right)$$  \hspace{1cm} (15)

with:

- $h_c(x_c=0)$ = layer thickness at the beginning of the dike crest
- $h_c(x_c)$ = layer thickness on the dike crest at the point $x_c$
- $x_C$ = coordinate on the dike crest with $x_c=0$ at the beginning and $x_c=B$ at the end of the crest [m]
- $B$ = width of the dike crest [m]
- $c_{c,h}^*$ = coefficient [-] to be gained empirically

The empirical coefficient $c_{c,h}^*$ for regular waves, TMA spectra and natural spectra is given in Table 7.

**Table 7: Coefficient $c_{c,h}^*$ for the layer thickness at the crest by Oumeraci et al. (2001)**

<table>
<thead>
<tr>
<th></th>
<th>$c_{c,h}^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regular waves</td>
<td>0.58</td>
</tr>
<tr>
<td>TMA spectra</td>
<td>0.89</td>
</tr>
<tr>
<td>Natural wave spectra</td>
<td>1.11</td>
</tr>
</tbody>
</table>

Van Gent (2002) determined the coefficient as $c_{c,h}^* = 0.40$ for spectra.

### 4.3.5 Overtopping velocity at dike crest

For the **overtopping velocity** $v_c$, a theoretical analysis based on the two-dimensional continuity equation and momentum equation for an incompressible flow performed by Schüttrumpf & Oumeraci (2005) gives the following general formula:

$$v_C = v_0 \exp\left(-\frac{x_c \cdot f}{2h}\right)$$  \hspace{1cm} (16)

with:

- $v_C$ = overtopping velocity [m/s] on the dike crest,
- $x_C$ = coordinate on the dike crest [m] with respect to the beginning of the dike crest,
- $h$ = $h_c$ = layer thickness [m],
- $f$ = bottom friction coefficient [-] to be determined empirically,
- $v_0$ = $v_c(x_C=0)$ = overtopping velocity [m/s] at the beginning of the dike crest.
The analysis of the test results showed that the overtopping velocity at the dike crest decreases from the beginning to the end of the dike crest being dependent on the friction and the layer thickness at the beginning of the crest.

Introducing an empirical factor based on model tests, Schüttrumpf & Van Gent (2003) used the following equation to describe the overtopping velocity $v_{c,2\%}$:

$$\frac{v_{c,2\%}}{v_{A,2\%}} = \exp \left( -c_{c,v}^* \frac{x_c \cdot f}{h_{c,2\%}} \right)$$

(17)

with

- $v_{c,2\%}$ = wave overtopping velocity [m/s] on the crest exceeded by 2% of the incoming waves
- $v_{A,2\%}$ = run-up velocity at the beginning of the dike crest [m/s] being exceeded by 2% the incoming waves
- $h_{c,2\%}$ = layer thickness on the crest [m], exceeded by 2% of the incoming waves
- $x_c$ = coordinate on the dike crest [m] with respect to the beginning of the dike crest
- $c_{c,v}^*$ = empirical coefficient [-] derived from model tests
- $f$ = friction coefficient [-]

The coefficient $c_{c,v}^*$ was gained from the model tests and is given as $c_{c,v}^*=0.5$ (see Schüttrumpf & Van Gent, 2003).

### 4.3.6 Overtopping velocity and layer thickness at the inner slope

The parameters for the determination of the layer thickness and the flow velocity at the inner slope of sea dikes are shown in Figure 21.

![Figure 21: Definition sketch of overtopping processes at the inner slope of sea dikes according to Schüttrumpf & Oumeraci (2005)](image-url)
A theoretical analysis by Schüttrumpf & Oumeraci (2005) based on the continuity and momentum equations leads to the following equation for the overtopping velocity at the inner slope:

\[
v_B = \frac{v_0B + k_1 \cdot h_B \cdot \tanh \left( \frac{k_1 t}{2} \right)}{1 + \frac{f \cdot v_0B}{h_B \cdot k_i} \cdot \tanh \left( \frac{k_i t}{2} \right)}
\]  
\[(18)\]

where

\[
k_2 = \arctan \left( \frac{f \cdot v_0}{h_B \cdot k_i} \right)
\]  
\[(19)\]

and

\[
k_1 = \sqrt{\frac{2f \cdot g \sin \beta}{h_B}}
\]  
\[(20)\]

with

- \(v_0B\) = velocity at the beginning of the landward slope [m/s]
- \(\beta\) = angle of inner slope [°]
- \(g\) = gravitational constant
- \(h_B\) = layer thickness [m]
- \(f\) = bottom friction coefficient [-]
- \(t\) = time [s]

The layer thickness \(h_B\) in equation (18) is defined as follows:

\[
h_B = \frac{v_0B \cdot h_0B}{v_B}
\]  
\[(21)\]

with

- \(v_0, h_0\) = overtopping velocity, layer thickness at the beginning of the landward slope \((v_0=v_B(s_B =0), h_0=h_B(s_B =0))\).

The velocity \(v_0B\) is time dependent. Schüttrumpf & Oumeraci (2005) declare the time dependency as less relevant compared to the spatial dependency and replace the variable \(t\) in equation (18) by the following term (motion of a mass point on a slope without friction):

\[
t = -\frac{v_0B}{g \sin \beta} + \sqrt{\frac{v_B^2}{g^2 \sin^2 \beta}} + \frac{2s}{g \sin \beta}
\]  
\[(22)\]

Since the layer thickness on the inner slope \(h_B\) is not known, an iteration has to be performed. For this purpose, the layer thickness \((h_B(s_B =0))\) and overtopping velocity \((v_B(s_B =0))\) at the beginning of the landward slope gained from the calculations on the dike crest are used for a first calculation of the velocity by means of equation (21) and (18). The layer thickness can then be determined by
Using this value of the layer thickness, a second approximation is done for the velocity $v_B$ and by means of the latter the layer thickness $h_B$ afterwards.

The analysis of the test results has shown that the overtopping velocities gained by direct measurements by means of velocity propellers and by video analysis fit well with the calculated velocities.

Unlike the formulae for the overtopping velocity at the inner slope developed by Schüttrumpf & Oumeraci (2005) the formulae according to Van Gent (2002) do not require iterations and are given in the following form

$$v_{B,2\%} = \frac{k_2}{k_3} + k_4 \exp\left(-3 \cdot k_2 \cdot k_3 \cdot s_B\right)$$  \hspace{1cm} (24)

where:

$$k_2 = \frac{3}{\sqrt{g \sin \beta}}$$  \hspace{1cm} (25)

$$k_3 = \frac{1}{2} \frac{f}{h_{0,2\%} \cdot v_{0,2\%}}$$  \hspace{1cm} (26)

$$k_4 = v_{0,2\%} - \frac{k_2}{k_3}$$  \hspace{1cm} (27)

with

- $v_{0B}$ = velocity at the beginning of the landward slope [m/s]
- $\beta$ = angle of inner slope [°]
- $g$ = gravitational constant
- $h_B$ = layer thickness [m]
- $f$ = bottom friction coefficient [-]

### 4.3.7 Overtopping rates at the inner slope

For the overtopping rate Oumeraci et al. (2001) developed a formulae generally written as:

$$Q^* = Q_0 \cdot \exp\left(-b \cdot R^*\right)$$  \hspace{1cm} (28)

where

$$Q^* = \frac{q}{\sqrt{2g \cdot H_{m0}}} \frac{1}{\xi_d} \quad \text{and} \quad R^* = \frac{R_c}{R_{u,2\%}}$$  \hspace{1cm} (29)

with

- $\xi_{S-1}$ = surf similarity parameter: $\xi_{S-1} = \frac{\tan \alpha}{\sqrt{H_{m0} / L_0}}$ [-]
- $Q_0$ = overtopping rate at $R_c=0$ [-]
Oumeraci et al. (2001) determined the factor b as $b = 3.67$ and the overtopping rate $Q_0 = 0.038$ for the generic formula. Unlike in many other known overtopping formulae he used the wave period $T_m$ instead of $T_p$ and the deep water length $H_{m0}$ instead of $H_s$ because he found a better agreement to the measured data especially with regard to multi-peaked natural spectra. For these multi-peaked spectra, he determined the factor $b$ as 3.25 (for open coasts) and 4.32 (for wadden seas) based on the natural sea states used for the large scale model tests performed in the GWK.

In Schüttrumpf & Van Gent (2003) an overtopping formula for the average overtopping rate $q$ based on results of 420 model tests is described as follows:

$$\frac{q}{\sqrt{2g \cdot H_s}} = Q_0 \xi \exp \left( -b \frac{R_s}{\xi \cdot H_s} \right)$$  \hspace{1cm} (30)

where

$q = \text{overtopping rate} \ [m^3/(s \cdot m)]$
$H_s = \text{significant wave height} \ [m]$
$Q_0 = \text{empirical coefficient} \ [-]: \ Q_0 = 0.15$
$b = \text{coefficient} \ [-]: \ b = -4.0$
$c_q = \text{coefficient} \ [-]: \ c_q = 0.55$

The model tests used for the determination of the empirical coefficients were 2D and 3D large and small scale model tests in different test facilities (e.g. GWK and LWI wave flume), see Schüttrumpf & Van Gent (2003).

In Schüttrumpf & Van Gent (2003), additional equations are given for the overtopping rate and the overtopping volume exceeded by 2% of the incoming waves gained from small scale model tests by Van Gent (2002). These formulae read as follows:

$$q_{2\%} = c_q \sqrt{\frac{R_{u,2\%} - R_c}{1 + c_q \frac{B}{H_s}}} \left( R_{u,2\%} - R_c \right)^{1.5}$$  \hspace{1cm} (31)

and

$$V_{2\%} = c_v \left( R_{u,2\%} - R_c \right)^2$$  \hspace{1cm} (32)

where

$q_{2\%} = \text{overtopping rate} \ [m^3/(s \cdot m)] \ \text{exceeded by 2\% of the incoming waves}$
$V_{2\%} = \text{overtopping volume} \ [m^3] \ \text{exceeded by 2\% of the incoming waves}$
\[ c'_{q} = \text{empirical coefficient } [-]; \quad c'_{q} = 0.2 \text{ for seaward and } c'_{q} = 0.17 \text{ for landward crest} \]
\[ c''_{q} = \text{empirical coefficient } [-]; \quad c''_{q} = 0 \text{ for seaward and } c''_{q} = 0.1 \text{ for landward crest} \]
\[ c'_{v} = \text{empirical coefficient } [-]; \quad c'_{v} = 1.0 \text{ both for seaward and landward crest.} \]

4.4 3D model tests

4.4.1 Background

In the review of the previous investigations, some factors influencing the flow field at a sea dike have been unattended, like the influence of oblique wave attack on wave run-up and wave overtopping. To investigate the effects of angled wave attack, hydraulic 3-D model tests were performed in the wave basin of the Danish Hydraulic Institute (DHI) comprising the determination of

- the influence of incident wave direction on wave run-up and wave overtopping on smooth dikes
- the influence of directionality of the sea state (long crested waves, short crested waves with various degrees of spreading) on wave run-up and overtopping
- the stochastic behaviour of waves along the structure, incl. the hydrodynamic parameters along the dike cross section, mainly at the inner slope of the dike

In the following, these tests performed within the project “Wave run-up and overtopping of sea dikes under 3D wave attack (DIKE-3D)” will be described in more detail.

4.4.2 Overview of tests

The DHI 3-D shallow water basin is 35 m wide and 25 m long while the maximum operable water depth goes up to 0.9 m. The wave generating system consists of 36 linear paddle segments each of them being 0.5 m wide and 1.20 m high allowing working in water depths from 0.20 to 0.90 m. An active wave absorption system gives the possibility to avoid reflections from the wave maker during the operations.

Two test configurations were tested with two different outer slopes (1:6 and 1:3). The first test configuration, referred to as SM6, consists of a horizontal foreshore, a 1:6 seaward and a 1:4 landward slope, separated by the dike crest with a width of 0.1 m and a height of 0.6 m. The cross section of configuration SM6 is shown in Figure 22.
The overall length of the structure was 14 m. The seaward slope was a combination of concrete and plywood which was chosen to carry the pressure cells, the digital step gauges and the wave run-up gauges. The crest was build of bricks. At the landward slope, two 2.5m wide overtopping tanks were installed consisting of five compartments each whereas the remaining part of the slope was made of concrete. The structure was placed with an angle of 30° toward the wave maker allowing angles of incoming waves from $\theta = 0^\circ \div 60^\circ$ with regard to the structure.

The following measuring components were used to acquire data on the effect of wave obliqueness and wave directionality on wave run-up, wave overtopping and wave impact:

- standard and directional wave gauges in front of the dike
- wave run-up gauges at three different locations on the seaward side of the dike
- wave probes to measure the water level in the overtopping tanks
- further pressure cells to measure the wave impact on the seaward slope

The position of the measuring devices and the plan view of the structure with regard to the wave maker are shown in Figure 23.
Wave maker

2 x 5 wave gauges

3 digital step gauges

Wave run up gauges

1:2 slope

Layer thickness gauges

Dry area

Wave absorption

Overall length of construction: ≈ 14 m

Figure 23: Plan view of structure used for configuration SM6 with location of measurement devices

The second configuration, referred to as SM3, is identical to the one used for configuration SM6 except the seaward slope that was 1:3. The cross section of configuration SM3 and the location of the measuring devices can be seen in Figure 24 and Figure 25, respectively.

Figure 24: Cross section of structure tested as configuration SM3
Overall length of construction: ≈ 14 m

![Diagram of structure](image)

*Figure 25: Plan view of structure used for configuration SM3 with location of measurement devices*

### 4.4.3 Test programme

The purpose of test variations was to analyse the influence of different parameters on wave run-up and wave overtopping. The following hydraulic and other parameters were varied throughout the tests:

- angle of wave attack $\theta$ of the incident waves ($0^\circ$, $10^\circ$, $20^\circ$, $30^\circ$, $40^\circ$, $60^\circ$)
- still water level (to change the freeboard) (0.450 m, 0.475 m, 0.485 m, 0.500 m, 0.510 m, 0.525 m)
- type of waves (regular, random)
- significant wave height $H_s$ (5 cm, 10 cm, 12 cm, 13 cm, 13.5 cm, 15 cm, 17.5 cm)
- peak wave period $T_p$ (1.27 s, 1.46 s, 1.79 s, 2.69 s)
- spreading of the waves $\sigma$ ($0^\circ$, $10^\circ$, $20^\circ$, $30^\circ$)
- different dike configurations (SM3, SM5, SWB3, SWB6)

The angle of wave attack is defined as the angle between the dike and the wave crests. The number of waves for each test depended on the type of waves. Tests with regular waves had a length of 100 waves whereas tests with random waves had a length of 900 waves. The tests with random waves were primarily conducted with a water depth of 0.5 m, some also with a depth of 0.475 m.
4.4.4 Data analysis
During the experiments, tests with regular and random waves were conducted. Generally, it could be observed that different behaviour of these different types of waves occurred at the structure. The reflection of the waves from the seaward side of the dike had much influence on the next approaching waves and either reduced or increased the wave heights in specific places along the dike. For some regular wave tests, this resulted in a pattern of dry and wet areas at the shoreward side of the dike.

4.4.4.1 Wave analysis
Wave parameters were obtained from the wave gauges in form of time series. The characteristics of wave parameters were based on both time domain analysis and frequency domain analysis. For the former, wave parameters were analysed by zero crossing methods. For frequency domain analysis, the time series were transformed into wave spectra which contain a series of wave amplitudes distributed in several numbers of frequencies. From the wave spectrum the wave parameters, which are then referred to as ‘spectral wave parameters’, were obtained.

To separate incident and reflected wave spectra, the measurements were performed with several gauges in an array. This method was proposed by Mansard & Funke (1980), Hashimoto & Kobune (1988) and others by using three gauges parallel to wave propagation in 2D tests. However, in 3D tests, three gauges will not be enough because the reflected waves no longer come parallel to wave propagation. Therefore, in 3D tests, certain formations of gauges have to be developed. To separate incident and reflected wave spectra, BDM (Bayesian Direct Method) was used in the 3D wave analysis.

Not all parts of the time series of the wave gauges can be used in the analysis. Data at the start and the end of the time series were skipped due to ramping up and down (Figure 26).
In the Dike-3D tests, the location of the reflection gauges was not really at the toe of the dike. Since the foreshore was very mild and had almost the same depth over the entire foreshore the incident wave height from the reflection analysis of the wave array position was assumed to be identical to the one at the structure toe.

The outcomes from the FFT are the significant wave height \( H_{m0} \), the wave periods \( T_{m-1,0} \), \( T_{m0,1} \), \( T_{m0,2} \), and peak wave period, \( T_p \) for incident and reflected spectra. The processes of obtaining those parameters are summarised in Figure 27.

From the analysis of the total waves in front of the dike, it became clear that the deviation of the wave periods was negligible along the dike. In most of the cases, the deviation was in the range of the measuring accuracy (range of 5%). This low deviation for the periods was true for both regular and random waves.

For wave heights, a stronger deviation has been noticed. In the case of random waves, this deviation is still limited and is almost always less than 5% along the dike. For regular waves however there is often a stronger deviation (around 10%) along the dike. In Oumeraci et al. (2001) a dike was used which was 10 meters longer on both sides related to the toe wave gauges which apparently resulted in a much lower deviation of the wave heights along the dike.

---

**Figure 26: Typical time series for a wave gauge and gain factor for analysis**

Volt

0

-0.10

+0.10

5 s

5 s

Gain-up

Gain-down

Specific Duration

Analysed data

Gain Factor

Wave Generation

Time

Gain Factor

Analysed data

Gain-up

Gain-down

Specific Duration

Volt

0

-0.10

+0.10

5 s

5 s

Gain-up

Gain-down

Specific Duration

Analysed data

Gain Factor

Wave Generation

Time
4.4.4.2 Wave run-up

In the analysis, there were different wave parameters used for the regular and irregular waves. In the case of regular waves, the wave height and the period were obtained from the time series analysis. For irregular waves, the wave height $H_{m0}$ and the period $T_{m-1.0}$ were used. The difference between the wave period from the time series analysis and from the frequency analysis proved very small most of the time.

Since wave overtopping was observed in a lot of tests, a detailed analysis of $R_{u2\%}$ would not make sense in these cases since it was always higher than the crest of the dike. To know which values can be used, it was decided to look at the overtopping tanks for all tests. When (almost) no water was found in the tanks, the value measured for the wave run-up was considered accurate and was further used for analysis. This method has also been visually checked and found working well. With this background, 65 tests out of approx. 254 tests in the scope of this report were used for wave run-up analysis.

For random waves, it was not possible to draw conclusions from the influence of the angle of wave attack $\theta$ on the wave run-up since reliable wave run-up measurements were only available for two wave steepnesses $s_0$. For one of the wave steepnesses the curve by Wassing (1957) or Van der Meer & De Waal (1990) for long crested waves fitted the data from random waves closely.

From the tests on regular waves it seemed that the reduction factor $\gamma_\theta$ for the angle of wave attack $\theta$ was always around the curve of Wassing (1957) or sometimes around Oumeraci et al.
This was also shown from the regression analysis. The tests where the curve was close to Van der Meer & De Waal (1990) and hence $\gamma_0$ was larger than unity could be generally explained by the fact that the generated wave height was higher for the wave attack angles between 10° and 30°. Only for one wave steepness $s_0$ this was not true and no reason could be found to explain this issue. From these limited amounts of reliable results, it could not be concluded that there was a higher wave run-up at small wave attack angles $\theta$ than for perpendicular wave attack.

4.4.4.3 Wave overtopping over smooth dikes

In this chapter, overtopping equations are derived by applying prediction methods for each configuration. The effects of oblique waves combined with spreading will be compared to various overtopping equations. The differences between these equations will be analysed and a reduction factor for oblique wave attack will be derived.

a) Introduction

Based on the dimensionless equations of Oumeraci et al. (2001), both tests results of SM3 and SM6 are analysed and plotted together with the equation proposed by Van der Meer & Janssen (1995), 5% envelope also included. The graph is shown in Figure 28. It should be noted that $R^*$ and $Q^*$ is different for the two lines given in the graph since the higher curve represents the non-breaking situation and the lower represents breaking waves at the structure.

![Graph showing relative mean overtopping rate for smooth slopes (SM3 & SM6) compared to Oumeraci et al. (2001) vs Van der Meer & Janssen (1995) equations.](image-url)

Figure 28: Relative mean overtopping rate for smooth slopes (SM3 & SM6) compared to Oumeraci et al. (2001)

When comparing the measured results with the curve given by Van der Meer & Janssen (1995), the SM6 results are slightly higher and the SM3 results are slightly lower than the
Van der Meer & Janssen (1995) lines for breaking wave situations. For non-breaking conditions there are only SM3 results available. The results of breaking and non-breaking waves are analysed separately. The division has been made based on the formulae of Oumeraci et al. (2001). Both formulae (breaking waves – non-breaking waves) are calculated and the minimum overtopping is maintained. In this way, the breaking and non-breaking waves were classified. Generally, the border between breaking and non-breaking waves can be determined based on the value of $\xi - 1.0$ where values larger than 1.82 result in non-breaking waves, while values smaller than 1.82 correspond to breaking waves.

b) Breaking waves

In Figure 29 the breaking, long crested waves were investigated. For the calculation of the overtopping value only the tanks, which are not influenced by diffraction, were used. For perpendicular wave attack only the two left tanks were considered, for 60° the two right tanks, and for 10°, 20°, 30°, and 40° all tanks were taken into account. Through the data points a best fit is drawn and the intersection point with the y-axis was fixed to a value of 0.067.

\[ y = 0.067e^{-3.5724x} \quad R^2 = 0.748 \]
\[ y = 0.067e^{-3.7825x} \quad R^2 = 0.7421 \]
\[ y = 0.067e^{-3.9398x} \quad R^2 = 0.7345 \]
\[ y = 0.067e^{-4.8212x} \quad R^2 = 0.7149 \]
\[ y = 0.067e^{-6.2956x} \quad R^2 = 0.5066 \]

\[ y = 0.067e^{-5.714x} \quad R^2 = 0.5688 \]
\[ y = 0.067e^{-3.9396x} \quad R^2 = 0.5398 \]
\[ y = 0.067e^{-4.8212x} \quad R^2 = 0.7149 \]

Figure 29: Relative mean overtopping rate for smooth slopes and breaking, long crested waves (y-intersection value of 0.067)

In Figure 29 the points with a low relative freeboard can generally be found below the trend line. These tests correspond to a dike slope of 1:3. The tests with a higher relative freeboard were performed on a dike with a slope of 1:6. This means that the overtopping discharge seems to be influenced by the slope of the dike (even though this is already incorporated in both the relative freeboard and the relative overtopping discharge). Therefore, the tests with different slopes were displayed separately in Figure 30.
The best fits in Figure 30 only differ from each other by a small factor in the exponential term. Based on these factors, the influence factor of oblique waves can be investigated by the following formula:

$$\gamma_0 = \frac{b(\theta = 0^\circ)}{b(\theta)}$$  \hspace{1cm} (33)

The b-factors of the different trends and their corresponding reduction factors $\gamma_0$ are summarized in Table 8.

**Table 8: b-factors and reduction factors $\gamma_0$ for mean wave overtopping rates**

<table>
<thead>
<tr>
<th>Angle $\theta$</th>
<th>Slope 1/3</th>
<th></th>
<th>Slope 1/6</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$b$</td>
<td>$\gamma_0$</td>
<td>$b$</td>
<td>$\gamma_0$</td>
</tr>
<tr>
<td>$0^\circ$</td>
<td>-4.3429</td>
<td>1.000</td>
<td>-3.3756</td>
<td>1.000</td>
</tr>
<tr>
<td>$10^\circ$</td>
<td>-4.5284</td>
<td>0.959</td>
<td>-3.6304</td>
<td>0.930</td>
</tr>
<tr>
<td>$20^\circ$</td>
<td>-5.0643</td>
<td>0.858</td>
<td>-3.7506</td>
<td>0.900</td>
</tr>
<tr>
<td>$30^\circ$</td>
<td>-5.5505</td>
<td>0.782</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$40^\circ$</td>
<td>-5.8777</td>
<td>0.739</td>
<td>-4.6172</td>
<td>0.731</td>
</tr>
<tr>
<td>$60^\circ$</td>
<td>-8.3739</td>
<td>0.519</td>
<td>-5.9241</td>
<td>0.570</td>
</tr>
</tbody>
</table>

By plotting the numbers in Table 8 as a function of the angle of wave attack $\theta$, the following equation can be derived:
\[ \gamma_0 = 1.0 - 0.0076 \theta \]  \hspace{1cm} (34)

This formula can be compared with available formulae from literature as follows:

- Van der Meer & de Waal (1990) \[ \gamma_0 = \cos^2 (\theta - 10^\circ) \]  \hspace{1cm} (35)
- Banyard & Herbert (1995) \[ \gamma_0 = 1 - 0.000152 \cdot \theta^2 \]  \hspace{1cm} (36)
- Oumeraci et al. (2001) \[ \gamma_0 = 0.35 + 0.65 \cdot \cos(\theta) \]  \hspace{1cm} (37)

These formulae are plotted in Figure 31 together with the results as shown in Table 8:
The formulae from literature do not correspond well with the formula found in this work. Therefore, Eq. (34) is further used here to better compare the data from the model tests.

By using the reduction factor \( \gamma_0 \) the dimensionless x-axis may be changed into \( \frac{R_s}{H_{mi} \cdot \xi_0 \cdot \gamma_0} \).

Plotting the data again with the modified x-axis will then result in a graph shown in Figure 32.

\[ \text{Figure 31: Comparison of reduction factors for angle of wave attack with formulae taken from literature for smooth slopes and breaking, long-crested waves} \]
Figure 32: Relative mean overtopping rate for smooth slopes and breaking, long crested waves, corrected for the angle of wave attack

In Figure 32 the influence of the angle of wave attack is eliminated. However, the influence of the dike slope is still left. This influence can be eliminated in the same way as the elimination of the influence of the angle of wave attack. The formula is based on the test results performed by van der Meer & Janssen (1995). These test results are shown in Figure 33.
Figure 33: Relative mean overtopping rate for smooth slopes, selected test data by TAW (2002)

Also in this graph clear differences for the different slopes can be identified. If the intersection with the y-axis (for $R^* = 0$) is left constant for all slopes different b values for all slopes can be found which then can be related to the slope angle as follows:

$$y_{dope} = 1.1653 - 0.7547 \cdot \tan \alpha$$  \hspace{1cm} (38)

By using this formula in defining the x-values for all data points, Figure 34 will result.

Figure 34: Relative mean overtopping rate for smooth slopes and breaking, long crested waves, corrected for the angle of wave attack and for the slope of the dike
c) Non-breaking waves

During the model tests non-breaking waves were only obtained on a dike with a slope of 1/3. Due to these limited number of tests, an influence of a slope cannot be identified. Nevertheless, the different tests with non-breaking waves are shown in Figure 35. The best fits for the data are also drawn through the intersection point with the y-axis as suggested by Tautenthain et al.(1982).

\[ y = 0.2e^{-2.8615x} \quad R^2 = 0.997 \]
\[ y = 0.2e^{-2.4061x} \quad R^2 = 0.998 \]
\[ y = 0.2e^{-2.8615x} \quad R^2 = 0.997 \]
\[ y = 0.2e^{-3.017x} \quad R^2 = 0.9834 \]
\[ y = 0.2e^{-3.1654x} \quad R^2 = 0.9974 \]

![Figure 35: Relative mean overtopping rate for smooth slopes and non-breaking, long crested waves and different angles of wave attacks](image)

A formula has been developed by Geeraerts et al. (2006) on the basis of a different data set which proposes to eliminate the influence of wave steepness in the case of non-breaking waves. The formula reads:

\[ \gamma_{s0} = 1.14 - 5.65 \cdot s_0 \]  \hspace{1cm} (39)

in which \( s_0 \) is the deep water wave steepness defined as:

\[ s_0 = \frac{2 \cdot \pi \cdot H_{m0}}{g \cdot T_{m-1.0}} \]  \hspace{1cm} (40)

It is proposed here to divide the dimensionless freeboard by the proposed steepness factor \( \gamma_{s0} \). Due to the very limited number of tests there is no evidence at this stage that this correction is necessary. However, for reasons of uniformity with the further paragraphs, the tests are also corrected in this paragraph. The tests, corrected for the steepness, are plotted in Figure 36.
Figure 36: Relative mean overtopping rate for smooth slopes and non-breaking, long crested waves, corrected for wave steepness

The influence of the angle of wave attack is eliminated here in the same way than for breaking waves using a best fit analysis. This yields the following formula for $\gamma_0$:

$$\gamma_0 = 0.9832 - 0.0044 \cdot \theta - 0.00007 \cdot \theta^2$$

(41)

A comparison of this formula has shown that there is hardly any difference between this formula and the formula found for breaking waves. Therefore, it is proposed here to use the formula for breaking waves (Eq. (34)). This leads to a uniform expression for $\gamma_0$ for both breaking and non-breaking waves.
4.4.4.4 Directionality of the sea state

Not only long crested waves were generated, but also short crested waves. The spreading varies between 0° and 30°. The tests are also divided in breaking and non-breaking waves here.

a) Breaking waves

First of all, the influence of the angle of wave attack and the influence of the slope are eliminated. Both influences are liquidated with the Van der Meer & Jansen (1995) formulae found in paragraph 4.4.4.3c). The corrected tests are plotted in Figure 38 for different values of spreading.

By considering Figure 38 it seems that the overtopping is slightly decreasing with increasing spreading. Again a formula can be developed, based on the method described in paragraph 4.4.4.3c). The obtained formula then reads:

\[ \gamma_n = 1.0 - 0.0034 \cdot \sigma \]  \hspace{1cm} (42)

The application of this formula leads to Figure 39.
y = 0.067e^{-3.5529x} \\
R^2 = 0.9041

y = 0.067e^{-3.6531x} \\
R^2 = 0.9181

y = 0.067e^{-3.7603x} \\
R^2 = 0.6393

y = 0.067e^{-3.9622x} \\
R^2 = 0.2628

Figure 38: Relative mean overtopping rate for smooth slopes and breaking, short crested waves, corrected for angle of wave attack and for slope

Figure 39: Relative mean overtopping rate for smooth slopes and breaking, short crested waves, corrected for angle of wave attack, for slope and for spreading

b) Non-breaking waves

In this paragraph the mean overtopping rate for non-breaking waves is discussed. After the elimination of the influence of the angle of wave attack and the influence of the steepness using the previous formulae, Figure 40 can be found.
Figure 40: Relative mean overtopping rate for smooth slopes and non-breaking, short crested waves, corrected for steepness and angle of wave attack

Regarding the directionality of the waves the short crested waves do not follow a trend like the long crested waves, but they form more or less a horizontal line as also found in Van der Meer & Jansen (1995). However, a formula that describes the influence of the directionality could be developed as follows:

$$ \gamma_\sigma = 1.0 - 0.0055 \cdot \sigma $$

This formula is almost equal to the formula found for breaking waves. To obtain uniformity, the formula for breaking waves is also used for non-breaking waves. The elimination of the influence of the spreading leads to Figure 41.
4.4.4.5 Wave pressures on front face

In this section the results from the pressure cells installed in the dike slope are going to be analysed and discussed. First, there will be an overview on how the different shapes of the pressure series look in time. Furthermore, there will be an analysis regarding typical pressure peaks. There will also be a comparison between the statistical results obtained during this project and the results from the research on pressure impacts conducted by Van der Meer & De Waal (1990). Finally, the pressure distribution from the maximum values along the cross section will be investigated (section 4.4.4.5d). The analysis for oblique wave attack has only been performed for wave attack angles $\theta$ up to 40° because for $\theta = 60°$ the pressure cells are outside the wave field.

a) Position of the pressure cells

The pressure cells were installed on fixed positions in the plywood plate of the dike slope. When the dikes were rebuilt from the SM6 to the SM3 dike configuration, the cells stayed at the same place in the plywood plate. The slope or the angle of the plywood plate changes to the still water level and thus the position of the cells changes to the water level. This means that the pressure cells are at different positions for the SM3 and SM6 dike configuration (Figure 42). Note that pressure cell 35 is crossed out in Figure 42 since it did not give any reliable signal throughout all tests.

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*Figure 41: Relative mean overtopping rate for smooth slopes and non-breaking, short crested waves, corrected for the angle of wave attack, the steepness and the directionality*
Figure 42: Location of the pressure cells for the SM6 and SM3 dike configuration

Figure 42 shows that the positions of the cells to the still water level were different for the SM3 and the SM6 configuration. Due to the broken pressure cell 35 the analysis was not performed for the SM6 dike configuration because cell 35 was at a relevant position for the SM6 dike, close to where the top pressure impacts would occur. The results from the pressures cells of the SM6 dike configuration did not show any high pressure peaks, so that the maximum pressures of this dike configuration were not analysed any further.

b) Time series and pressure shapes

In this section a closer view on the different pressure shapes is obtained. The pressures printed were all taken from pressure cell 31. Pressure cell 31 was chosen because at this cell the highest pressure peaks were registered for almost all the tests conducted for the SM3 configuration. In Figure 43 a typical time series of pressure cell 31 is shown for regular waves in test 427. Some high pressure peaks can be observed and these peaks sometimes differ a lot from each other.

Figure 43: Time series of pressure cell 31 for test 427
In Figure 44 a more detailed view is given spanning a few wave periods. First, there is a steep pressure peak at 12.15 s. This peak is short in time and reduces fast. After the reduction of the peak the pressure raises a little again but steeply and then it reduces gradually until the next peak. Despite the regular waves in this test the pressure peaks are very different although the shapes of the peaks are similar.

![Figure 44: Pressure time series with high pressure peaks, pressure cell 31, test 427 (H_{nom} = 0.15 m; T_{nom} = 1.46 s); all measures in model units](image)

The peak pressures usually occur under severe breaking when the breaker tongue hits the dike surface. The second lower and longer lasting peak is related to the maximum run-up of the wave after the impact. This type of breaking waves may be referred as ‘impact waves’.

The pressure shape in Figure 45 is very similar to the above although in this case the pressure peak is much lower. The further reduction of the pressure after the pressure peak seems to be more steeply in this case but that is caused by the scale of the figure. Overall the shape of one peak is very similar to the others.

![Figure 45: Pressure time series with low pressure peaks, pressure peak 31, test 355 (H_{nom} = 0.10 m; T_{nom} = 1.46 s); all measures in model units](image)

Another example is given in Figure 46. In contrast with the previous pressure shapes the different periods do not resemble to each other. There are sometimes pressure peaks followed by a few wave periods where there is no peak. This pressure time series shape is typical for random waves and was observed in all the random wave tests.

![Figure 46](image)
Figure 46: Pressure time series with irregular pressure peaks, pressure cell 31, test 372
($H_{nom} = 0.10 \, m; \, T_{nom} = 1.27 \, s$); all measures in model units.

In Figure 47 another pressure time series is given. The distinctive element of this shape is that
there is a double pressure peak where the first one is smaller than the second. This was true
for all the pressure peaks in the test.

Figure 47: Pressure time series with double pressure peaks, pressure cell 31, test 350,
($H_{nom} = 0.05 \, m; \, T_{nom} = 1.27 \, s$); all measures in model units.

In Figure 48 is the output given from the case where there were no real pressure peaks on the
cell. It is possible to see the pressures corresponding to up and down rushing waves in this
figure.
c) Influence of logging frequency

A pressure peak has been examined more closely to investigate the influence of the logging frequency on the high impact and short duration peaks. The maximum pressure peak observed during all tests has been used for this purpose (Figure 49) which has risen up to more than 6 kPa with impact duration of only 0.08 s.

When looking more closely to the time series in Figure 49 it needs to be checked whether the actual maximum pressure peak could be much larger but just happened to fall between two single measurements. Although this is theoretically possible, it does not seem to result in a big difference. To illustrate this in more detail the peak itself is shown in Figure 50.
Figure 50: Detail from the highest peak, pressure cell 31, test 423 ($H_{nom} = 0.1 \text{ m}, T_{nom} = 1.79 \text{ s}$); all measures in model units

The steep rising part of the pressure impact in Figure 50 lasts for about 0.038 seconds. The pressure cells were logged at 500 Hz so one value every 0.002 seconds was recorded. These values are shown by the big dots on the pressure curve in Figure 50. The three highest dots on the curve are very similar in magnitude. Keeping in mind the gradient before and after the peak it may be concluded that the chance of a higher pressure peak than measured here is extremely low. Therefore, it is assumed here that the logging frequency of 500 Hz was sufficient for all the pressure measurements.

d) Shape of maximum pressure along the cross section of the dike

In this section the shape of the pressures over the cross section of the dike will be analysed using the same statistical values as in the previous section. For random waves these were $p_{99.9\%}$, $p_{99\%}$, $p_{90\%}$ and $p_{50\%}$. For regular waves $p_{99\%}$, $p_{90\%}$ and $p_{50\%}$ were calculated. In the last section it was made clear that there is not much difference between the values of pressure cell 32 and cell 33 which are very close to each other (see Figure 42). Therefore, from these two cells the average of their values is used.

In almost all tests the shape of the maximum pressures along the cross section is very similar and no significant differences between regular and random waves could be observed. Figure 51 till Figure 55 are showing statistical pressures for the SM3 configuration, a water depth $d = 0.475 \text{ m}; H_s = 5 \text{ cm},$ and $T_p = 1.46 \text{ s}$. In Figure 51 an example is given for the values of $p_{99.9\%}$ for random waves and the SM3 configuration. Pressure cell 31 shows a high peak value for all wave attack angles 0. All other pressure cells show more or less the same value.
In Figure 51 the pressures based on the $p_{99\%}$ values is given for all pressure cells. This shape is similar to the shape of the $p_{99.9\%}$ pressure distribution. For wave attack angles $\theta = 0°$ to $30°$ there is a factor of 2.5 to 4.0 difference between the pressures from cell 31 and all other cells. In the case of a wave attack angle $\theta = 40°$ there is no real difference anymore observed in the $p_{99\%}$ values between pressure cell 31 and other cells.

In Figure 52 the pressures based on $p_{99\%}$ values is given for all pressure cells. The resulting distribution is similar to the distribution of Figure 52. The difference between the value at pressure cell 31 and all other cells is not that big anymore (factor of 2.0). From a wave attack angle $\theta = 20°$ and higher no real differences between the statistical pressures from the various pressure cells can be observed.
Figure 52: Pressure $p_{99\%}$ for different wave attack angles $\theta$ and each pressure cell in tests 461-462-463-464-465

Figure 53: Pressures $p_{90\%}$ for different wave attack angles $\theta$ and each pressure cell in tests 461-462-463-464-465

In Figure 54 the pressures based on the $p_{50\%}$ values are given for all pressure cells. There is no real peak anymore at pressure cell 31 and all the pressures at the cells are more or less the same.
Although the pressures in the Figure 51 to Figure 53 were the highest for $\theta = 0^\circ$ and then decreased for the other wave attack angles this was not true in all tests. In a few tests the $p_{99.9\%}$ and $p_{99\%}$ pressures at cell 31 were higher for a wave attack angle $\theta = 10^\circ$ than for a wave attack angle $\theta = 0^\circ$. An example is given in Figure 55.

The differences in between $0^\circ$ and $10^\circ$ wave attack angle is however not believed to be significant, especially because this was observed in a few cases only for the maximum values where strong variations of individual peak pressure may occur anyway.
Banyard & Herbert (1995) has given an overview of impact pressure distributions for 120 consecutive regular waves (Figure 56). The distance between the cells was 5 cm. It is obvious that there is a big difference in the pressure shape along the slope and pressure values for each impact. The distance between the cells in DIKE-3D was around 30 cm, hence a much lower resolution. It may be assumed that variations in pressure maxima resulted from similar measurements as shown in Figure 56. However, the larger distance between pressure cells during the DIKE-3D tests suggested that some spatial resolution was lost from between the pressure cells.

**Figure 56: Pressure distributions for 120 consecutive regular waves from Oumeraci et al. (2000)**

**e) Pressure analysis**

In this section the different relative pressures will be investigated. The pressures $p_{99.9\%}$, $p_{99\%}$, $p_{90\%}$ and $p_{50\%}$ are divided by $\rho \cdot g \cdot H_{m0}$ for random waves. For regular waves $p_{99\%}$, $p_{90\%}$ and $p_{50\%}$ are divided by $\rho \cdot g \cdot H_s$. The wave height $H_{m0}$ and $H_s$ are the incident wave heights and were calculated using the reflection analysis as described under section 4.4.4.1, respectively. There was no pattern found for the influence of the wave attack angle $\theta$ on the pressure. It seems that the relative pressures had very similar values, so no distinction was made between the different wave attack angles $\theta$ in this section. The relative pressures were grouped according to their wave steepness $s_0$ and wave height $H_s$. In Table 9 the results are given for regular waves. In this table the pressures calculated as proposed by TAW (2002) for a 1 in 3 slope are given in the last row. In Table 10 the results are given for random waves.
Table 9: Overview of the relative pressures for regular waves and SM3 configuration

<table>
<thead>
<tr>
<th>d[m]</th>
<th>Hs [m]</th>
<th>s0 [-]</th>
<th>(p_{99%} p.g.Hs[-])</th>
<th>(p_{90%} p.g.Hs[-])</th>
<th>(p_{50%} p.g.Hs[-])</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>0.03</td>
<td>0.95 ± 0.18</td>
<td>0.70 ± 0.23</td>
<td>0.56 ± 0.16</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>0.05</td>
<td>1.87 ± 0.53</td>
<td>1.55 ± 0.50</td>
<td>0.94 ± 0.56</td>
<td></td>
</tr>
<tr>
<td>0.475</td>
<td>0.03</td>
<td>3.47 ± 1.02</td>
<td>2.55 ± 0.43</td>
<td>1.99 ± 0.40</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>0.02</td>
<td>1.02 ± 0.30</td>
<td>0.72 ± 0.23</td>
<td>0.50 ± 0.11</td>
<td></td>
</tr>
<tr>
<td>0.15</td>
<td>0.045</td>
<td>2.85 ± 0.73</td>
<td>1.91 ± 0.52</td>
<td>1.43 ± 0.44</td>
<td></td>
</tr>
<tr>
<td>0.05</td>
<td>0.03</td>
<td>0.85 ± 0.21</td>
<td>0.64 ± 0.04</td>
<td>0.61 ± 0.04</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>0.05</td>
<td>1.03 ± 0.13</td>
<td>0.87 ± 0.07</td>
<td>0.70 ± 0.03</td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>0.03</td>
<td>1.65 ± 0.41</td>
<td>1.16 ± 0.37</td>
<td>0.82 ± 0.43</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>0.02</td>
<td>2.51 ± 0.87</td>
<td>1.84 ± 0.78</td>
<td>1.10 ± 0.42</td>
<td></td>
</tr>
<tr>
<td>0.525</td>
<td>0.02</td>
<td>0.62 ± 0.08</td>
<td>0.59 ± 0.09</td>
<td>0.57 ± 0.07</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>0.05</td>
<td>0.54 ± 0.07</td>
<td>0.51 ± 0.06</td>
<td>0.49 ± 0.06</td>
<td></td>
</tr>
</tbody>
</table>

Führböter (1986) | 6.67 | 5.30 | 4.00 |

For the pressure \(p_{99\%}\), the relative pressures of the DIKE-3D project are around 14% to 52.0% of the value of the relative pressure by Geeraerts et al. (2006). For the pressure \(p_{90\%}\), the relative pressures of DIKE-3D are 9.6% to 48.0% of the value obtained by the corresponding equations from Schüttrumpf & Murphy (2000). When \(p_{50\%}\) pressures are analysed the values of the relative pressure are around 12% to 50% of the values of Banyard & Herbert (1995). It seems that for regular waves the values of the relative pressures are always around 10% to 50% of the values given by Führböter (1986).

Table 10: Overview of the relative pressures for random waves and SM3 configuration

<table>
<thead>
<tr>
<th>d[m]</th>
<th>(H_{m0}) [m]</th>
<th>s0 [-]</th>
<th>(p_{99.9%} p.g.Hm0[-])</th>
<th>(p_{99%} p.g.Hm0[-])</th>
<th>(p_{90%} p.g.Hm0[-])</th>
<th>(p_{50%} p.g.Hm0[-])</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.475</td>
<td>0.015</td>
<td>5.52 ± 1.55</td>
<td>2.55 ± 1.28</td>
<td>0.79 ± 0.28</td>
<td>0.32 ± 0.03</td>
<td></td>
</tr>
<tr>
<td>0.15</td>
<td>0.045</td>
<td>3.85 ± 0.81</td>
<td>2.62 ± 0.29</td>
<td>1.07 ± 0.15</td>
<td>0.24 ± 0.04</td>
<td></td>
</tr>
<tr>
<td>0.05</td>
<td>0.02</td>
<td>1.47 ± 0.33</td>
<td>0.76 ± 0.13</td>
<td>0.51 ± 0.04</td>
<td>0.30 ± 0.03</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>0.04</td>
<td>2.20 ± 0.89</td>
<td>1.13 ± 0.28</td>
<td>0.57 ± 0.11</td>
<td>0.27 ± 0.05</td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>0.03</td>
<td>3.40 ± 1.11</td>
<td>1.91 ± 0.58</td>
<td>0.73 ± 0.23</td>
<td>0.26 ± 0.09</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>0.02</td>
<td>4.20 ± 2.02</td>
<td>2.22 ± 0.74</td>
<td>0.65 ± 0.31</td>
<td>0.42 ± 0.27</td>
<td></td>
</tr>
</tbody>
</table>

Führböter (1986) | 8.00 | 6.67 | 5.33 | 4.00 |

For the pressure \(p_{99.9\%}\), the relative measured pressures are around 18% to 69% of the value of the relative pressure by Führböter (1986). With respect to the \(p_{99\%}\) value the relative pressures are 11.4% to 39% of the values proposed by Führböter (1986). For \(p_{90\%}\) 9.6% to 20% and for \(p_{50\%}\) 6% to 10% of the values by Führböter (1986) were measured. Overall, it seems that for random waves the values of the relative pressures are always significantly lower than those
proposed by Führböter (1986). In a recent study on wave impacts on smooth slopes under 2D conditions Führböter (1986) has confirmed the proposal by Führböter (1986) for regular waves and 1:4 slopes. However, it must be noted that most of these values are extreme values which are more likely to happen under 2D conditions. It is therefore suggested that the reduced impacts on the slope result from oblique wave attack. Furthermore, there was only one pressure cell mounted in the wave impact zone. With the wave impacts occurring very locally one pressure cell was not possible to obtain all the pressure peaks from wave impact. Although the pressures were divided by $\rho \cdot g \cdot H_m$ for random waves and $\rho \cdot g \cdot H_s$ for regular waves, respectively, there was still a significant difference between the different wave heights even when the wave steepnesses were identical. This is the reason why a distinction for the different wave heights was made for Table 9 and Table 10.

f) Summary

The main purpose for the pressure analysis in the DIKE-3D project was to obtain more information about the pressures on the slope and to see if any 3D aspects could be found. From this preliminary analysis some useful suggestions were found for the ongoing research on pressures and their 3D aspects.

When two neighbouring pressure cells are on the same height on the dike slope with 30 cm (two times the maximum wave height in the tests) between them it seems that in general there is not a big difference between the pressures observed from these cells. This was even the case for the extreme pressure peaks $p_{99.9\%}$ and $p_{99\%}$. From this preliminary analysis it seems that over short distances in the range of 30 cm and on the same level on the dike the 3D aspects from the pressures are negligible.

For the shape of the pressures along the cross section of the dike there was always a high peak at pressure cell 31 for $p_{99.9\%}$, $p_{99\%}$ and $p_{90\%}$ but no real distinction any more for $p_{50\%}$. It was not possible to find out whether the angle of wave attack $\theta$ had any influence on these observations.

For the relative pressure heights there was a big difference observed between the results from the DIKE-3D project and the results from Führböter (1986). The possible reason for this is that due to the pressure cell layout not all the high pressure peaks from the wave impacts were obtained during the DIKE-3D tests.

To further investigate the pressure distributions on the dike slope, especially regarding the 3D aspects, a complete array of pressure cells would be needed with more pressure cells in the breaking wave impact zone with a minimum distance to each other. To look into the 3D aspects it would be useful to put three or four columns of these cells on the dike next to each other with a distance of about 30 cm in be

4.4.5 Conclusions

Hydraulic model tests have been performed within the framework of the European Human Mobility Programme IHP ARI (Improving Human Potential – Access to Research Infrastructures). The tests were conducted at the 3D shallow wave basin at DHI (Danish Hydraulic Institute) in Lyngby, Denmark. The tests investigated hydraulic load parameters at a 1:6 and 1:3 sloped sea dike with and without a stilling wave basin. Short- and long-crested waves were investigated with wave directions ranging from 0° to 60°.

The DIKE-3D project aimed at a better understanding of wave run-up, wave overtopping, velocities, and flow depths at the seaside and the landward side of the dike under 3D aspects. In more detail, the objectives were:

- influence of incident wave direction on wave run-up and wave overtopping on smooth dikes
- influence of directionality of the sea state (long crested waves, short crested waves with various degrees of spreading, incl. stochastic behaviour of waves along the structure)

Furthermore, pressures on the seaward side of the dike, flow depths and velocities at the landward side of the dike were also measured and analysed.

The model tests were performed with two different structure configurations as follows:

- SM6: smooth dike with seaward slope of 1:6
- SM3: smooth dike with seaward slope of 1:3

Results have principally confirmed the results of previous tests under 3D conditions for wave run-up and overtopping. For wave run-up (see section 4.4.4.2) emphasis was laid upon finding the reduction factor due to obliqueness of wave attack. For regular waves the reduction factor $\gamma_\theta$ for the angle of wave attack $\theta$ has been found predominantly around the curve of Führböter (1986). Although some of the measurements with slightly oblique wave attack suggested reduction factors $\gamma_\theta$ larger than 1.0 it was believed that those measurements represented some uncertainties in the data rather than a consistent increase in this factor. For random waves only very few measurements could be analysed (note that the key focus of the tests was on wave overtopping rather than wave run-up) where most of the data agreed with the curve of Führböter (1986).

Wave overtopping results have suggested slight modifications for formulae for breaking and non breaking waves at the structure, still using the general overtopping equation as follows:

$$q^* = a \cdot \exp \left( b \cdot \frac{1}{R^* \cdot \gamma_{\text{slope}} \cdot \gamma_\theta \cdot \gamma_{\text{berm}} \cdot \gamma_{\text{SWW}} \cdot \gamma_{\text{spreading}}} \right)$$  (44)

In which $q^*$ is the relative mean overtopping rate [-]; $R^*$ is the relative freeboard [-]; $a$ and $b$ are empirical parameter [-]; and $\gamma_i$ are correction factors for the slope, the angle of wave attack, the wave steepness, the berm and the spreading of the waves, respectively. Those correction factors were all determined from the tests except the berm factor since no tests with berms were conducted. Results for these factors are summarised in Table 11.
Table 11: Overview of parameters for wave overtopping calculations for smooth dikes

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Smooth dike</th>
<th>non breaking</th>
</tr>
</thead>
<tbody>
<tr>
<td>R*</td>
<td>$R_c$ (\frac{\xi_0 \cdot H_{m0}}{c})</td>
<td>$R_c$ (\frac{H_{m0}}{c})</td>
</tr>
<tr>
<td>q*</td>
<td>$q = \sqrt{g \cdot H_{m0}^3 \cdot \tan \alpha \cdot \frac{\xi_0}{c}}$</td>
<td>$q = \sqrt{g \cdot H_{m0}^3}$</td>
</tr>
<tr>
<td>a</td>
<td>0.067</td>
<td>0.20</td>
</tr>
<tr>
<td>b</td>
<td>-3.5452</td>
<td>-2.454</td>
</tr>
<tr>
<td>$\gamma_{\text{slope}}$</td>
<td>$\gamma_{\text{slope}} = 1.1653 - 0.7547 \cdot \tan \alpha$</td>
<td>1.00</td>
</tr>
<tr>
<td>$\gamma_0$</td>
<td>$\gamma_0 = 1.0 - 0.0076 \cdot \theta$</td>
<td>$\gamma_0 = 1.0 - 0.0076 \cdot \theta$</td>
</tr>
<tr>
<td>$\gamma_{a0}$</td>
<td>1.00</td>
<td>$\gamma_{a0} = 1.14 - 5.65 \cdot s_0$</td>
</tr>
<tr>
<td>$\gamma_{\sigma}$</td>
<td>$\gamma_{\sigma} = 1.0 - 0.0034 \cdot \sigma$</td>
<td>$\gamma_{\sigma} = 1.0 - 0.0055 \cdot \sigma$</td>
</tr>
<tr>
<td>$\gamma_b$</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Pressure measurements at the seaward side of the dike were installed to principally show the spatial distribution of pressures along the dike. The results have shown that locations of impact pressures are always located at the same pressure cell slightly above the mean water level. No further details could be obtained from the data since the spatial distribution of the pressure transducers was too low. The lateral distribution however did not show much variation along the dike. The lateral distance of the two pressure transducers was about two times the maximum wave height. The relative pressure heights were in the range of two times lower than the results from Führböter (1986). The possible reason for this is that due to the pressure cell layout not all the high pressure peaks from the wave impacts were obtained during the DIKE-3D tests. More detailed testing would be needed here to finally conclude on this issue.

Overall, the tests have proved to be very useful in obtaining more reliable 3D data on wave run-up and overtopping of sea dikes.
5 Loading of dunes

5.1 Introduction

5.1.1 Background
A coastal dune is a ridge or mound of loose wind-blown material, usually sand, located on the landward side of the beach. Such morphological features are common along the coast in many parts of the world and they constitute an important component in the coastal sediment transport system. Although the morphology of coastal dunes and the formation process by wind have been studied extensively, little is known about their response to extreme waves and water levels during storms. When normal wave and water level conditions prevail, the dune is not exposed to any hydrodynamic impact, but sediment transport and morphological change only take place in the swash zone, which is the region where the water runs up and down the foreshore as the incoming waves dissipate their final energy.

Coastal dunes often constitute the final defence line against high waves and water levels during severe storms. If they are overtopped or breached, serious damage due to flooding and direct wave attack could occur resulting in loss of life and property. Thus, it is of significant value to be able to predict the impact of a storm on a dune in terms of recession distance, eroded volume, and probability of breaching. Furthermore, storms often imply a significant increase in the mean water level (surge) that may induce overtopping by run-up waves or direct overflow (Donnelly et al. 2004). This shoreward transport of water and sediment is denoted as overwash, and it is an important component in the sediment budget for barrier islands and, possibly, their shoreward migration (Fisher and Stauble 1977, Leatherman 1979).

Two classical approaches exist to calculate the effect of high waves and water levels during a storm on coastal dunes, namely the equilibrium profile approach (Vellinga 1986, Kriebel and Dean 1993) and the wave impact approach (Overton et al. 1994, Larson et al. 2004). Equilibrium profile theory assumes that the beach profile strives towards an equilibrium state defined by the wave and water level conditions, which geometrically determines the response of the dune. The wave impact approach estimates the sediment transport from the dune and associated profile change by the waves directly hitting it. Existing analytical and numerical models to predict the response of dunes tend to be based on the equilibrium theory with limited description of the physical processes (Kriebel and Dean 1985, Larson and Kraus 1989), however, some applications exist were the wave impact approach was used (Overton et al. 1994, Nishi and Kraus 1996, Larson et al. 2004).

5.1.2 Objectives
The overall objective of the study presented in this chapter was to investigate the impact on dunes of the combined loading from waves and water level in terms of eroded volume, overtopping, and breaching. A case study was undertaken for the south coast of Sweden to illustrate how a methodology for such an investigation could be developed and executed.
5.1.3 Procedure
An area located to the west of the city of Ystad on the south Swedish coast was investigated to establish the magnitude and associated probability of dune erosion, overtopping, and breaching. This area has extensive dune complexes of various characteristics and it has suffered erosion for a considerable period of time. Long time series of measured (wind and water level) and derived (incident waves) data were employed to compute the run-up level and dune erosion. The run-up level was defined as the sum of the run-up height, obtained from the Hunt formula, and the water level at a specific time with respect to the long-term mean sea level. Eroded dune volume was estimated using the formula developed by Larson et al. (2004), which is based on the dune impact approach. Empirical distribution functions were derived for run-up level and eroded dune volume and compared with the theoretical Gumbel distributions. Estimates were developed for statistical measures of the studied quantities with different return periods. The probability of overtopping for different beach profiles along the study area was also determined. Finally, based on established climate change scenarios, the expected future dune erosion, run-up level, and overtopping were estimated for these scenarios and compared with the present situation.

5.2 Study area and data employed

5.2.1 Overview
The coasts of northern Sweden are largely characterized by rocky shores with a marked glacial rebound (Lambeck 2004). In the south of Sweden, however, the landscape is very low-lying and the coastline is typically made up of sandy beaches. In addition, a relative sea-level rise makes the beaches here generally subject to erosion. The municipality of Ystad, which is located along the south coast of the Skåne province, has been subject to coastal erosion for at least 150 years. Winds over the Baltic Sea generate waves which cause run-up and direct erosion of the sand dunes on the beach. One of the most exposed areas is Ystad Sandskog, located to the east of Ystad city. Here, the beach profiles are characterized by a relatively, narrow dry beach (foreshore) with sand dunes protecting low-lying areas behind the dunes. In case of possible future climate change, with increasing wave heights and water levels, the exposed coastline will suffer from greater impact in terms of erosion and flooding. This implies that popular areas, for tourism and recreation, are in risk of, partly or entirely, disappearing. The main objective of this study was to jointly analyze waves and water levels, based on existing climate data, to make conclusions about how the beach, particularly the sand dunes, at Ystad Sandskog have been influenced by the coastal climate up to today and what the future evolution might be. From wind and water level data at different stations in the south Baltic Sea, representative conditions for the area were constructed. Based on available climate-change forecasts, scenarios were developed for this area, making it possible to identify critical sections on the beach with respect to run-up, overtopping, and erosion of the dunes.

Ystads Sandskog is located in the east part of the municipality (Figure 57). In the west part of the area there is a large number of houses, mainly used for recreational purposes, many of which have a cultural-historical value. The east part consists of a nature reserve area without any buildings. The sand dunes run along the coast, marking the border between the forest and the open beach, which is very popular for recreation in the summer. On top of the dunes there is a lane for biking and walking. The coastline is, since the 1950's, stabilized with a few groins (Figure 58).
5.2.2 Profile measurements

In order to obtain a detailed picture of the nearshore bathymetry in the area, profile measurements were performed along 25 lines (Figure 58). For each line, the approximate volume, dune foot and crest elevation, and the foreshore slope were estimated. These quantities were used as a basis for calculating run-up levels and dune erosion. Along a limited number of lines surveys were carried out at several occasions making it possible to estimate the erosion rate of the dunes along those lines. This rate was employed to validate the dune erosion model employed in the calculations.

5.2.3 Wind data

For the analysis of the Ystad wind climate, data from the Swedish Meteorological and Hydrological Institute (SMHI) stations at Falsterbo, Ystad, and Simrishamn (Figure 57) were used. Extensive analysis of these data indicated to what extent the different stations may be used to represent the particular conditions in Ystad (Dahlerus and Egermayer 2005), however, the details of this analysis will not be discussed here. In total, the measured time series encompassed a period of 44 years (1961-2004). From 1961 to 1972 measurements were made every three hours starting at 3.00 and ending at 18.00, and from 1973 to 2004 measurements were also made at 21.00 and 24.00. The wind speed resolution is 1 m/s and the resolution in direction is 10 deg. The recorded average wind speed for the entire period was 6.8 m/s and the maximum wind speed was 28 m/s.

For a long-term analysis of wind speed, the number of days per year with gale winds was calculated as presented in Figure 59. There was a distinct increase of such events in the period 1965-1985, but there is no evidence of any long-term trends. An extensive analysis of the
temporal variation with respect to wind directions was also performed (although not presented here) and this analysis showed no temporal trend either. The winds were used for calculating wave conditions off the study site using a model developed by Larson and Hanson (1992), where the modification introduced by Dahlerus and Egermayer (2005) to take into account wave growth and wave decay was employed.

![Annual Frequency (%)](image)

Figure 59: Annual frequency of gale winds in Falsterbo 1961-2004.

5.2.4 Water level data

Two time series of data were used in the analysis of water levels in Ystad. The longest time series measured in Ystad extends from 1887 to 1986 and holds daily maximum, minimum, and mean water levels, whereas another shorter series from Ystad (1996-2004) consists of hourly values. For analysis of run-up and dune erosion, a detailed series of water levels were required with duration as long as possible. For this reason the short time series from Ystad was supplemented by one series from Skanör (1992-2004) and another one from Simrishamn (1982-2004), both with hourly observations, to generate a time series consisting of 23 years of hourly water level observations (1982-2004). Correlation analysis between the three stations gave a basis for extending the time series of hourly water levels from Ystad.

The long time series from Ystad are shown in Figure 60 and shows how the annual mean water level varies over the period 1887-1986. A linear trend line was fitted to the data and shows an increase of 0.55 mm/yr.
5.3 Description of physical processes and models used

5.3.1 Incident waves

The wave conditions were computed from the wind observations in about 20 m water depth outside the study area. For most of the calculated waves this depth corresponds to deep water conditions, although for the longer period waves some effects from the bottom are expected. The equations for wave prediction recommended in SPM (1984) were employed with a constant representative depth along each fetch length (see also Larson and Hanson (1992) for a summary of the equations used). Fetch lengths were estimated at a resolution corresponding to the measurements of the wind direction.

A difficulty when employing the SPM method is to take into account wave growth and decay in a proper way. In the present study, the modification developed by Dahlerus and Egertmayer (2005) was employed, where it is assumed that the waves evolve from the existing wave conditions based on a simple exponential response function. For the wave height the evolution equation is written,

\[ \frac{dH}{dt} = k(H_{eq} - H) \]  \hspace{1cm} (45)

where \( H \) is the wave height generated by the wind, \( H_{eq} \) the wave height at equilibrium conditions (determined by the fetch length or wind speed from the SPM method), \( t \) time, and \( k \) a response coefficient. The solution to Eq. 45 is,
\[ H = H_{eq} - (H_{eq} - H_{in}) \exp\left(-\frac{t}{t_{eq}}\right) \]  \hspace{1cm} (46)

where \( H_{in} \) is the wave height at \( t=0 \) when a new set of wind conditions start to apply and \( \lambda = kt_{eq} \) was introduced in which \( t_{eq} \) is the time needed for fetch-limited (equilibrium) conditions to be attained according to SPM (1984). The wind conditions are assumed to be constant for the time period during which Eq. 46 is employed. Also, the direction between consecutive wind measurements should change relatively smoothly so that the simple approach of combining the old and new wave conditions is applicable.

Equation 46 is valid both for \( H_{in} > H_{eq} \) and \( H_{in} < H_{eq} \), which would correspond to decay and growth of the waves, respectively, in response to changes in wind conditions. In the present study, when calculating wave heights from a time series of wind data, Eq. 46 was employed taking into account the initial wave height at each time step. The empirical response coefficient was determined by fitting Eq. 46 towards the wave growth obtained from SPM (1984) corresponding to \( H_{in}=0 \). For a specific fetch length, the SPM equations yield a wave growth of \( \frac{3}{4} \left(\frac{H}{H_{eq}}\right)^{3/4} \) up to \( t_{d}=t_{eq} \), where \( t_{d} \) is the duration of the wind, after which \( H=H_{eq} \). Fitting Eq. 46 to this function gives an optimum value of \( \lambda=2.17 \) for a least-squares fit. A similar evolution equation as Eq. 46 was developed for the wave period.

For the calculation of wave conditions at Ystad, wind data from Falsterbo (Figure 58) were used, because these were considered most representative for the south Baltic Sea. This data series covers the period 1961-2004 with values every three hours, including in total more than 106,000 data points. The calculated mean significant wave height (based only on wave-generating wind directions) was 0.87 m. Large waves are generated by winds from S to WSW and the maximum calculated significant wave height was 4.8 m. Gringorten’s plotting position formula was used to calculate return periods for the wave height. Figure 61 shows the 100-year wave height to be 5.5 m with an \( R^2 \)-value for the fitted line of 0.95.

### 5.3.2 Wave run-up

Run-up height is a critical quantity to estimate in storm impact assessment since it defines the highest elevation where the waves might affect the beach. It is normally referenced to the still water level and includes the wave setup. The earliest formula for wave run-up height \( (R) \) was developed by Hunt (1959),

\[
R = \frac{\tan \beta}{\sqrt{H_o / L_o}} \hspace{1cm} (47)
\]

where \( \beta \) is the beach slope, \( L \) wavelength, and subscript \( o \) denotes deepwater conditions. Subsequent run-up formulas have mainly been variations of Eq. 47, where the selections of representative input wave parameters and slope are the major issues (Mayer and Kriebel 1994, Stockdon et al. 2006). After evaluating several different formulas (e.g., Mase 1989, Mayer and Kriebel 1994), Equation 47 was employed in the present study to compute the run-up height. A constant, average slope based on the foreshore and surf-zone slopes obtained from measured beach profiles was used in Eq. 47.
In order to take into account large angles of incidence with respect to the shoreline, a simple correction was applied to the input wave height according to $H_o = H_w \cos \theta_o$, where $H_w$ is the modified wave height to be used in Eq. 47 and $\theta_o$ the incident wave angle in deep water with respect to the shoreline. A correction factor like the one in Eq. 48 results if run-up height is assumed to be related to the onshore component of the wave energy flux. The run-up height was computed for every value in the calculated wave time series and added to the applicable still-water level given by the water level time series. These generated data constituted a base for determining the risk of dune overtopping and overwash at various alongshore locations in the study area.

![Wave Height (m) vs Return Period (year)](image)

**Figure 61:** Annual maximum wave height in Ystad (period 1983-2004) plotted against the reduced value from the Gumbel distribution using the Gringorten plotting position formula together with a linear fit.

### 5.3.3 Dune erosion and overtopping

The dune erosion model developed by Larson *et al.* (2004) was employed to calculate the eroded volume in connection with wave attack on the dunes. In the case of a dune eroding with negligible changes in the dune foot elevation, the change in the dune volume $V$ is governed by the following equation,

$$\frac{dV}{dt} = -4C_s \left( \frac{R_d - z_o}{T} \right)^2$$

(48)

where $C_s$ is an empirical coefficient, $R_d$ the representative run-up height for dune impact, $z_o$ the dune foot elevation (with respect to the still-water level), and $T$ the wave period. Equation 48 results from combining the sediment volume conservation equation with a transport equation that relates eroded volume to wave impact force on the dunes. The run-up height needed is computed from $R_d = 0.158 \sqrt{H_w T_o}$, where the root-mean-square wave height should be employed for random waves. For constant wave and water level input conditions, Equation 48 is easily integrated to yield the eroded volume ($\Delta V$),
\[ \Delta V = 4C_s \left( R_d - z_o \right)^2 \frac{\Delta t}{T} \]  

(49)

where \( \Delta t \) is the time step during which the forcing conditions are constant. In order for dune erosion to occur, \( R_d > z_o \), otherwise the waves will have no effect on the dune. Equation 49 was used to calculate the eroded volume for each condition in the time series of waves and water levels.

In order to compute the eroded volume, an appropriate value must be selected on the transport coefficient \( C_s \). Time series of profile data along selected lines were available for estimating \( C_s \) in a general sense. Annual values on eroded dune volumes derived from profile measurements were compared to calculated volumes and the value of \( C_s \) was set to match the two volumes. This was done for several years of measurements and an average value of \( C_s = 3.3 \times 10^{-4} \) was obtained, which is in agreement with the values found by Larson et al. (2004) for field conditions. It should be noted that Eq. 49 only predicts the amount eroded from the dune face and the process of sand removal from the foreshore after erosion is not described. This implies an assumption of instantaneous removal of eroded sand in front of the dune, ignoring that this sand in reality offers some protection of the dunes. Thus, the calculated eroded dune volumes are conservative in terms of assessing the risk of severe dune retreat and possible breaching.

### 5.4 Analysis of results

#### 5.4.1 Joint probability of extreme events

The analysis of the joint distribution of wind and water level utilized wind data from Falsterbo in combination with water levels from Simrishamn. The composite time series covers the period 1983 to 2004, with values every three hours, encompassing in total 66,481 values. In the Baltic Sea most of the water level variations are associated with the wind, although some smaller effects may be associated with differences in air pressure over the sea area. Analyses of water levels versus wind direction clearly showed that low water levels are generated for winds from SW to W, whereas high water levels occur for winds from N to NE. This behavior is explained by the shifting water mass in the Baltic Sea, which basically is an enclosed basin. Winds from the northern sector “push” the water to the south part of the Baltic, whereas winds from the southern sector move the water to the north part.

A special analysis was made regarding gale and storm winds with focus on the water levels they typically are associated with winds arriving from the E to W have a potential for generating large waves, so it is of interest to see how often high water levels are associated with strong winds from these directions. The number of data points with wave-generating winds was in total 42,469. The analysis showed that the strongest winds are not necessarily associated with the highest water levels (due to their directional properties). For gale winds the water level varies between -110 cm and +80 cm. For storm winds the fluctuations are typically between -40 cm and +40 cm.

Due to the orientation of the coastline at the study site, wave-generating winds only arrive from the lower part of the compass, i.e., within the sector E-S-W. The largest calculated deep water significant wave height was 4.8 m at a measured water level of +20 cm. At a water level
of about +100 cm the highest calculated waves were about 0.8 m. This shows that high waves do not appear in combination with high water levels.

5.4.2 Existing conditions
A schematic of a profile in the middle of the study site was used for calculation of representative run-up levels (i.e., run-up height added to the current water level). The swash zone was represented by a straight line with a slope \( m = 0.20 \), which is typical for the site (Hanson 2004). The underwater profile was represented by an equilibrium profile with \( A = 0.16 \) (Dean 1977), corresponding to a grain size of 0.40 to 0.45 mm. As previously discussed, Eq. 3 was used to determine run-up heights for the period 1983-2004, from which the run-up levels were derived by taking into account the water levels. The calculated results show a maximum run-up level of 4.6 m and a minimum level of -1.0 m relative to mean sea level (MSL). In addition, the calculations showed that high run-up levels are typically caused by high waves in combination with modest water levels. Figure 62 shows the number of hours/yr the run-up level exceeds a certain level. Based on these results, it is possible to make estimates on dune erosion and the risk of dune overtopping. Gringorten's plotting position formula was used to calculate the return periods for various run-up levels. Figure 63 shows that the 100-year run-up level is 4.9 m with a \( R^2 \)-value of 0.97.

![Figure 62: Hours per year of exceedance of run-up levels simulated for Ystad 1983-2004.](image)

If the run-up level exceeds the dune crest level, overtopping will occur. Based on calculated run-up levels from 1983 to 2004, the number of occasions with overtopping was registered for each of the 25 profile lines. This gives an indication of where the risk of overtopping is the greatest. The calculation showed the largest probability of overtopping is for profile line 0, which is characterized by a narrow beach in front of a steep dune with a crest level of 3.2 m. Also, profiles 10, 11, 15, 21, and 22 showed a high tendency for overtopping. These profiles are characterized by steep dunes but also somewhat lower crest levels. Interestingly, the profiles with low crest elevation (profiles 5, 18, and 24) in combination with a mild foreshore slope do not show any significant overtopping. Thus, the probability for overtopping is more sensitive to foreshore slope than crest elevation for this site. A qualitative validation of the
calculated results was made by comparisons with visual observations and other
documentation on overtopping events and associated flooding.

If the run-up levels reach the dune foot elevation, the dune will experience wave impact and
suffer erosion. In the case that the wave impact has sufficient duration, the erosion will
remove the entire dune (breaching) and water will be able to propagate behind the dunes and
flood these areas. Calculations were made to quantify the dune erosion and to determine the
duration of different combinations of waves and water levels required to completely erode
away the dunes. Analysis of measured profile change in the area (Hanson, 2004) indicated
that situations with high run-up levels have larger impact if they occur at different occasion
than if they occur in combination. This seems reasonable as the eroded sand from the dunes
will deposit on the beach, offering protection from continued erosion. After some period of
time when the sand eroded from the dunes has been removed, however, this effect will be lost.
The increased erosion that have been documented at Ystad in recent years is most likely due
to a reduction in the dune foot elevation $z_0$, which in turn allows the waves to hit the dune
with a larger impact.

As mentioned earlier, a $C_T$-value of $3.3 \times 10^{-4}$ was used based on profile surveys from the
period 2000 to 2004. Employing this coefficient value dune erosion was calculated for the
period 1983-2000. Because detailed information on $z_0$ is not available for the selected period,
the simulations were made for a schematized profile. Thus, the calculation do not give the
actual erosion, but gives an indication how the erosion depends on the profile shape and also
provides a foundation for comparing historic erosion with future dune erosion under different
scenarios. Figure 64 shows the erosion for a section with $z_0 = 1.6$ m. The calculated erosion is
particularly severe in 1983, 1984, and 1990. This is probably caused by events including
strong winds and long duration. The simulations also showed that the erosion process is quite
sensitive to the dune foot elevation $z_0$. A reduction of this elevation by 0.2 m may result in an
increased erosion rate of several m$^3$ per year.
In addition, the dune erosion model was used to calculate the time it takes to erode away the dune (Time Required for Breaching = TRB) based on a specific event for the selected profile lines 13, 15, 17, and 21. Profile 13 was selected because it has a small $z_0$, Profile 17 because it has a small dune volume, and Profiles 15 and 21 represent intermediate conditions. Based on different events, encompassing wave heights and water levels with a duration of three hours, TRB-values were calculated as shown in Table 12 together with the return period for each event. This analysis also indicates that the $z_0$-value is more important than the dune volume for the TRB-value. Profile 13 have the largest dune volume along the study site but is still the profile with the highest breaching risk, due to its low $z_0$-value.

Table 12: Number of events necessary for dune breaching (TRB), for different profiles exposed to combinations of $H_0$ and WL. The return period for the respective combination is shown (“-“ means that there is no risk of dune breaching).

<table>
<thead>
<tr>
<th>$H_0$ / WL</th>
<th>Return period</th>
<th>TRB for respective profile</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Profile 13</td>
</tr>
<tr>
<td>4.0 / 0.0</td>
<td>7 yr</td>
<td>6</td>
</tr>
<tr>
<td>3.5 / 0.0</td>
<td>3 yr</td>
<td>9</td>
</tr>
<tr>
<td>3.0 / 0.0</td>
<td>7 months</td>
<td>11</td>
</tr>
<tr>
<td>2.5 / 0.0</td>
<td>2 months</td>
<td>16</td>
</tr>
<tr>
<td>2.5 / 0.5</td>
<td>2 yr</td>
<td>9</td>
</tr>
</tbody>
</table>

5.4.3 Future climate conditions
The future climate scenarios used in this study, are derived partly from global climate models (Meier et al. 2004, Karoly et al. 2003, Hudson and Jones 2002) with coarse spatial resolution, partly from regional models (Meier et al. 2004) that have been used for making more detailed predictions for the Scandinavian region. The global models were developed within the IPCC framework (IPCC 2001) and the regional models were developed within SWECLIM, a Swedish research project sponsored by SMHI and the Foundation for Strategic Environmental Research (MISTRA), 1997-2003. The scenarios for conditions up to 2100 used in this study, are the so-called A2 and B2 scenarios, where A2 assumes a larger increase of greenhouse gas
concentrations resulting in a mean temperature increase of 3.7°C relative to conditions in 2000 (Bernes, 2003), whereas B2 is more optimistic with lower concentrations (Nakićenović et al., 2000) with a temperature increase of 2.6°C. The regional SWECLIM simulations are built on results from one British (scenarios HA2 and HB2) and one German (scenarios EA2 and EB2) model.

According to the SWECLIM simulations, the wind speed will increase mainly during the winter period from December through February. Wind speeds during these months will increase 5-10% according to scenario EB2, but twice as much according to EA2. Results from scenarios HB2 and HA2 show a more limited increase at around 1-5%. Wind speeds may, thus, in the worst case increase as much as 20% (EA2) but may be limited to only 1% (HB2). A mean change is estimated to 5%. Calculations in the following section will be based on these three scenarios EA2, HB2 and the mean value from the two.

Meier et al. (2004) presents results from SWECLIM simulations using the different scenarios with respect to annual mean water levels. As for wind speeds, the greatest changes are expected during the winter months. Three components are contributing to the changes: glacial rebound (-0.05 mm/yr in Ystad), changes in the wind regime, and the global sea level change, which depends on model and scenario. Simulations for 2100 show a best case (HB2) with a water level reduction by 0.05 m, whereas the worst case scenario (EA2) gives an increase by 0.85 m. The mean scenario results in a water level increase of +0.38 m.

The influence of water level on profile evolution is an important aspect. Bruun (1962) presented a very simple model for profile response to sea level change, based on the equilibrium concept and conservation of mass. In principle, as the sea level increases, sediment will be transported from the inner to the outer section of the profile. This model has later been tested, analysed, and modified in several studies (Silenzio et al., 2002). In this study, a simple model, based on Bruun’s model, is defined. As the sea level increases (S) it is assumed that the profile is receding while the effective slope (β) and the dune foot elevation (z₀) are kept constant (Figure 65), and the dune crest level is also assumed constant. This means that, for a sea level rise, the dune volume as well as the beach width will decrease while the foreshore slope increases. Thus, the profile will not just translate, as in the original Bruun model, but also change its shape. Figure 66 present expected changes in dune volumes for the different scenarios.

![Figure 65: Overview of schematized profile recession following a sea level rise (K1 indicates dune face at level MVY1 and K2 is the dune face at MVY2).](image-url)
As seen from the Figure 66, the change is drastically different between profiles within the study area. Dune volume in Profile 13 is reduced by 50% in the worst scenario (EA2). This is mainly due to the low elevation of the dry beach. Profiles least affected are characterized by a plane-sloping profile shape from the water line to the dune crest. The relationship between sea level rise and dune retreat is about 1:20. In addition, a continued sea level rise will reduce the beach width in front of the dunes. Figure 67 shows the calculated beach width evolution for the different scenarios. The average reduction is about 7.5 m per meter of sea level rise. For profiles 0, 1 and 19 the changes are more dramatic. The reason is that these profiles are protected by seawalls that will not allow the profile to adjust to the sea level rise.

**Figure 66: Variation in dune volumes in the profiles of Ystad Sandskog due to change in mean sea level. Profiles 0, 1, and 19 show no change as they are protected by seawall.**

**Figure 67: Change in beach width for the profiles of Ystad Sandskog due to change in mean sea level.**
Based on the schematized representative profile, the number of hours per year a specific elevation is reached was calculated for the different scenarios, as shown in Figure 68. The calculation included changes in water level, wind, and wave climate. The levels discussed refer to the mean water level in 2004. In connection with the run-up level analysis, the number of overtopping situations over a 20-year period in 2100 was calculated for each of the profiles, and the result is displayed in Table 13. For the worst case scenario (EA2), the probability of overtopping increases significantly for most profiles. Many of the profiles that are not overtopped today are likely to be overtopped within the 100-year perspective.

Table 13: Number of hours with possible overtopping during a 20-year period for the different scenarios compared to today. Profiles with values in parenthesis are protected by seawall. Shaded cells display significant increase in overtopping.
A comparison with respect to return periods between present conditions and the future scenarios is shown in Figure 69. For present conditions, a 100-year return period yields a runup level of 4.9 m. The corresponding value for the EA2 scenario is 6.2 m. The future return period that corresponds to the present 100-year value is for the EA2 scenario only about 8 year, i.e., overtopping and associated flooding will be about thirteen times more frequent in 2100 than what it is today.

The largest uncertainty in simulating future dune erosion is the difficulty to estimate how the dune foot elevation ($z_o$) will vary. In order to proceed with the simulations, the above-mentioned assumptions were made on how the profile is adjusting to a sea level rise (see Figure 69). The calculations indicated a relative erosion rate of HB2 = -15 %, Mean = +22 %, EA2= +75 %, as compared to the present conditions.

![Figure 69: Comparison of return periods of run-up levels for the different scenarios.](image)

5.5 **Conclusions**

The analyses carried in this study showed that run-up levels and sand dune erosion have had great effect on the investigated beach during the last decades. The most sensitive sections are characterized by a mild foreshore slope, a low-lying dune foot, and a steep dune face. In case of a future sea level rise, the foreshore width and the sand dune volumes are expected to decrease drastically. From existing forecasts, the direct sand dune erosion was calculated to increase by up to 75% by the year 2100. This implies that many houses and other infrastructure located behind the dunes may be subject to more frequent flooding. The study also indicated that a run-up level, with a 100-year return period today, in the future may occur up to 13 times more frequently. It should be pointed out that the presented calculations were, in many cases, made on schematized representative profiles. For this reason, the presented numbers have to be interpreted with care. Nevertheless, it is believed that these calculations have given clear indication on the nature, approximate magnitude, and location of problems that can be expected within the next 100 years.
6 Loading of seawalls

6.1 Introduction

6.1.1 Definition, key parameters and condition of a seawall
The draft version of the new ISO standard on 'Actions from waves and currents on coastal structures' provides a definition.

Seawalls are onshore or foreshore structures generally parallel to the shoreline. They are built as vertical face structures such as gravity concrete walls, steel or concrete sheet pile walls, and stone filled cribworks or as sloping structures with revetment typically made of concrete slabs, concrete armour units, or rock armour. The principal function of seawalls is to reinforce a part of the coastal profile and to protect land and infrastructures from the action of waves and flooding.

This definition does therefore allow the inclusion of sloping embankment or ‘dyke’ seawalls or rubble mound embankments or slopes. It is noted that there is no clearly defined transition between (vertical and battered ) seawalls and (mildly sloping) embankment seawalls or dikes, the term “seawall” has been taken in this note as only covering hard sea defence structures, with a seaward face steeper than 1:1.5. Although it may be constructed of large blocks or fronted with sheet piles, as an alternative to solid concrete, it is assumed to act as a long fixed inflexible structure.

The narrow definition above therefore excludes clay embankment seawalls with side slopes typically 1:2-2.5; sand core dykes with slopes often of 1:4 – 6; or rubble mounds with side slopes typically 1:1.5-1:4.

Key parameters of the seawall are the elevations of the wall crest and toe, and information on the wall profile which will include shape, slope, roughness and possibly permeability / porosity. The seabed profile immediately seaward of the wall, and its mobility, may also be relevant in determining the size of waves which reach the structure. The purpose of this chapter is not to dwell on the characteristics of the seawall or to calculate its response to loading, but to use those points to explain which metocean variables and parameters are most relevant for design and assessment of seawalls.

The condition of a seawall, based on age, structural integrity and any previous storm damage, will also affect its ability to withstand further storm wave attack.

6.1.2 Sources: Metocean variables
In Chapter 2, the main metocean variables relevant to sea defence design and performance were identified as waves, tide, surge, wind and current. In the context of loading of seawalls, this list can be reduced. Currents are irrelevant, except possibly for their influence on foreshore level.

Wind has historically not been used directly in seawall design and assessment, although it is, of course, the driving force for generation of high waves and surges, and may affect the
impact of overtopping rate on building and people behind sea defences. Recent work on the spatial distribution of overtopping, and on scale effects, have both suggested that wind effects should be more explicitly considered in future.

The separate tide and surge components of sea level are not usually used directly in design and assessment of seawalls, and it is easier to work in terms of overall sea level (although the separate components may have been considered in deriving design sea level information).

The main metocean variables for use in seawall design and assessment at present are therefore:

- **Waves**, just before breaking through to the toe of the structure: wave height, wave period or steepness, approximate wave direction, extreme values, and appropriate climate change and/or uncertainty allowances
- **Sea level**: extreme values, joint probability with large waves, and appropriate climate change and/or uncertainty allowances

### 6.1.3 Pathways: structure variables

Some design variables relate to the structural stability and integrity of the seawall, to prevent fracture or overturning. The cross-section, depth and material strength depend on the fixed static forces associated with its own mass and the landward conditions, coupled with the slowly-varying force associated with sea level and the rapidly-varying forces associated with waves. Wave loads are resisted primarily by gravity and friction forces, and by the bearing capacity of the seawall’s foundation.

Other design variables relate to the purpose of the seawall. The crest level and possibly the seaward profile will be selected to keep overtopping and flood risk to an acceptable minimum.

Structure variables are discussed further in Section 6.2, to provide a background to the main part of this chapter, i.e. Section 6.3, which discusses derivation of the metocean variables and parameters most relevant to design and assessment of seawalls.

### 6.2 Loading variables most relevant to seawall design

The following books are particularly relevant:

- **Handbook of coastal engineering**, including Chapter 4, *Wave forces on vertical and composite walls*
- **Probabilistic design tools for vertical breakwaters**, including Chapter 2, *Hydraulic aspects*
- **Seawall design (Thomas & Hall)**

Recent research guidance is given by Müller et al. (2007), Datz et al.(2007) and Bruce et al. (2007).
6.2.1 Force and potential for structural failure

Apparently, similar wave conditions may give rise to dramatically different wave pressures or forces depending on the form of wave breaking at or close to a seawall. In this context, the Handbook of coastal engineering identifies three classes of waves:

- **Nonbreaking or pulsating**: the simplest to calculate analytically, and causing the lowest peak forces and pressures of the three
- **Impulsive breaking or impact**: termed plunging, breaking, impulsive or impact; loading generally higher than under the other two types of waves, and hard to estimate using theoretical methods. These loads are characterised by high pressures/forces, but short durations.
- **Broken waves**: turbulent aerated water generally causing lower forces than impulsive breaking or impact waves, and again difficult to derive using purely theoretical methods

Which one or more of these three types is relevant in any particular design depends on wave height, wave period, toe elevation, sea level, wall slope, and to a lesser extent foreshore slope and wave direction.

Estimation of force on a seawall requires information on:

- **Sea level**, as it may limit the wave height at the toe of the seawall, and for its influence on the loading type; as well as a (small) contribution to overall wave force.
- **Wave height and period** in a form permitting estimation of the significant wave height, mean and peak wave period, extreme individual wave height, and corresponding wave length

6.2.2 Overtopping rate and volume, and potential for flooding

Wave overtopping occurs when the still water level is below the seawall crest level but wave action carries water over the crest. Overtopping rate is sensitive to both wave height and wave period, and to (high) sea level. This response is usually presented as a mean discharge, averaged over (say) 1000 waves, and presented as m³/s per m run of seawall, or l/s/m.

6.2.3 Toe erosion and potential for structural failure

Erosion or other damage at the toe can occur when the wave induced water motion at the toe is high enough to cause significant movement of the sediment and/or structure. Scour is sensitive to both wave height and wave period, and may be sensitive to (high) sea level allowing larger waves to reach the toe, and/or (low) sea level causing less attenuation of wave-induced motion with depth.

6.3 Metocean variables and parameters most relevant to seawall design

6.3.1 Sea level

Sea level refers to the average still (i.e. disregarding wave effects) sea surface elevation over an area at a particular time. It is made up of a long term mean sea level, an astronomical tidal component and a meteorological surge component. A surge is produced by the direct effect of
pressure or wind on the water surface, and may be amplified in propagation through shallow water, or by constriction between land masses.

Appropriate design sea levels are often available from national authorities, without the need for new research into the topic. If more information is needed, many years of tide gauge records are available at many locations.

For calculation of overtopping, high and extreme sea levels are of most interest. For other aspects of seawall design and construction, extreme low sea levels may also be important.

Although mean sea level varies only a little over time and over an area, high and low sea levels can vary significantly over tens of kilometres, and it may be necessary to modify or interpolate between published or measured levels to convert them from one location to another. On an open coast, it is often good enough to convert sea levels (relative to mean sea level) from one location to another, just by multiplying the established extreme levels by the ratio of the spring tidal ranges (Mean High minus Mean Low Water Springs) at the two sites. In complex areas, or where surge and tide vary in different ways, it may be helpful to carry out tidal flow modelling, to assist in determining the relationship between the two locations.

Table 14 illustrates a typical desk study approach to the assessment of high and extreme sea levels at hypothetical Location 2. Location 1 is assumed to have an established tide gauge record from which reliable extreme values (Cells B4 B7 in Table 14) are available in the literature. Spring tidal levels are available for both locations (Cells B1, B2, C1 and C2) from which the spring ranges can be determined (Cells B3 and C3). The spring tide range at Location 2 is 1.097 times that at nearby Location 1. Extreme high tide levels for Location 2 (Cells C4 C7) are estimated by multiplying the differences between the extreme levels and MHWS for Location 1 (0.60, 1.03, 1.46 and 1.88m, above MHWS = 2.22m) by 1.097 (to 0.66, 1.13, 1.60 and 2.06m above MHWS = 2.44m).

Information on extreme low tide sea levels is usually less readily available but it may be necessary to make an estimate for use in seawall toe stability calculations. Typically, negative surges are of lower magnitude than positive surges. If site specific analysis is impractical, a reasonable estimate of extreme low sea levels can be made by assuming that an extreme low level is 75% as far below MLWS as the extreme high sea level is above MHWS. This approach was used to estimate the values in Cells C8 C11 of Table 14.

Finally, it is common practice to make some allowance for the possibility of future mean sea level rise. Although a more sophisticated projection of future change might be used, this allowance often takes the form of a simple addition of a constant amount to all sea levels, as noted in Row 12 of Table 14.
### Table 14: Example desk study estimation of high and extreme sea level, based on published extreme levels

<table>
<thead>
<tr>
<th></th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Name of high or extreme sea level</td>
<td>Location 1 (well established levels)</td>
<td>Location 2 (only spring tide range known)</td>
</tr>
<tr>
<td>1</td>
<td>Mean High Water Springs</td>
<td>2.22</td>
<td>2.44</td>
</tr>
<tr>
<td>2</td>
<td>Mean Low water Springs</td>
<td>-2.12</td>
<td>-2.32</td>
</tr>
<tr>
<td>3</td>
<td>Mean Spring Range</td>
<td>4.34</td>
<td>4.76</td>
</tr>
<tr>
<td>4</td>
<td>1 year high tide</td>
<td>2.82</td>
<td>3.10</td>
</tr>
<tr>
<td>5</td>
<td>10 year high tide</td>
<td>3.25</td>
<td>3.57</td>
</tr>
<tr>
<td>6</td>
<td>100 year high tide</td>
<td>3.68</td>
<td>4.04</td>
</tr>
<tr>
<td>7</td>
<td>1000 year high tide</td>
<td>4.10</td>
<td>4.50</td>
</tr>
<tr>
<td>8</td>
<td>1 year low tide</td>
<td></td>
<td>-2.82</td>
</tr>
<tr>
<td>9</td>
<td>10 year low tide</td>
<td></td>
<td>-3.17</td>
</tr>
<tr>
<td>10</td>
<td>100 year low tide</td>
<td></td>
<td>-3.52</td>
</tr>
<tr>
<td>11</td>
<td>1000 year low tide</td>
<td></td>
<td>-3.87</td>
</tr>
<tr>
<td>12</td>
<td>If necessary to make an appropriate allowance for possible future mean sea level rise, add (say) 0.40m to all levels listed above.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 6.3.2 Waves

Waves tend to be more variable than sea level along the coastline, being dependent upon the shape of the coastline and the seabed bathymetry. A site specific wave prediction study is usually necessary for design or assessment of a seawall. It will usually include the following elements:

**Offshore wave conditions.** Typically, these will be deep water open ocean conditions, governed by the shape and size of the area in which the waves are generated and the temporally and spatially varying wind field over that area. Appropriate offshore waves can usually be obtained from standard sources, e.g. forecasting models run by meteorological agencies, previous studies or design guidelines.

**Nearshore wave conditions.** These are predicted through transformation of the offshore waves to a position not far off the wall but seaward of the surf zone. This is usually done with a site specific numerical wave model, taking account, where necessary, of continued wave growth, local bathymetry, refraction, shoaling, headlands, seabed friction, and possibly diffraction.
Swell waves. Swell refers to the longer period wave conditions sometimes found at coastal sites exposed to waves generated over oceanic distances. Swell arises from the decay of distant storms, originally generated over many hours and hundreds of kilometres of fetch distance. The wave heights are usually lower than those of more locally generated storms, but the wave periods are higher. Swell tends to be unrelated to locally occurring weather conditions, and swell waves are often found on Atlantic coasts, even when the weather has been calm for several days previously. A standard analysis of extreme wave conditions will concentrate on the largest wave heights (usually associated with storm wave conditions). Structural response to swell wave conditions can be quite different, and it may be appropriate to consider swell as a separate design loading case (from a separate population of source data) when estimating force or overtopping at a seawall. Derivation of swell design conditions is not trivial, as site-specific wave models or design guidelines are likely to focus on storm waves only. Wave measurements and large area wave models are capable of providing information on swell conditions.

Wave conditions at the toe. These are predicted through site specific (and scenario specific) transformation of the nearshore waves through the surf zone, to the position and water depth of the structure toe. Strictly, this may be stepped slightly seaward of the structure on the basis that any remaining wave transformation takes some distance to complete, see diagram above from Allsop et al (2001).

Depth-limiting calculations are usually done using a relatively simple numerical model, in which wave breaking is the main process considered. Wave set up (and sometimes wave breaking) may be implicit in the later structure variable prediction method, but if not it is estimated explicitly at this stage.

Most organisations involved in seawall design will have the use of a range of wave models, and will select the most appropriate for each particular project. Generic advice on selection and use of wave models is given in the CIRIA/CUR (1991) Manual on the use of rock in hydraulic engineering.

Methods for prediction of marginal (single variable) and joint (multiple variables) extremes are discussed in Sanchez-Arcilla & Gonzalez-Marco (2007), and are not repeated here. Some
special issues associated with extreme values appropriate for design of seawalls are discussed in Sections 6.3.3 to 6.3.6.

6.3.3 Where to evaluate extreme wave conditions

Extreme waves could be estimated offshore (say 10-20km offshore), nearshore (say 300-1000m offshore), immediately seaward of the seawall (50-100m from the toe of the wall) and/or at the toe of the seawall. All are potentially valid approaches, and more than one may be needed to be certain of covering the “worst case” for design of a seawall. For example, transformed offshore extremes may be inappropriate if the seawall is sheltered by a headland or if the wave height at the seawall is severely depth limited. For another example, previously depth limited extremes at the toe of the seawall would be unsafe under scenarios of foreshore erosion or future sea level rise.

Table 15 illustrates typical wave predictions for a hypothetical seawall exposed to large open ocean wave conditions, but with a limited depth of water at its toe. The larger significant wave heights (Hs) are reduced in height more than the smaller waves in the transformation nearshore, particularly after breaking. Peak wave periods seldom change through such transformations, although mean wave periods (Tm) may tend to move slightly. The wave steepness, and hence the type of wave breaking on a slope, may however change noticeably.

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>Offshore</th>
<th>Nearshore</th>
<th>Water depth =6m at toe</th>
<th>Water depth =4m at toe</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hs (m)</td>
<td>Tm (s)</td>
<td>Hs (m)</td>
<td>Tm (s)</td>
</tr>
<tr>
<td>1</td>
<td>4.7</td>
<td>8.0</td>
<td>3.5</td>
<td>8.0</td>
</tr>
<tr>
<td>10</td>
<td>5.5</td>
<td>8.6</td>
<td>4.0</td>
<td>8.5</td>
</tr>
<tr>
<td>100</td>
<td>6.3</td>
<td>9.2</td>
<td>4.5</td>
<td>9.2</td>
</tr>
<tr>
<td>1000</td>
<td>7.1</td>
<td>9.8</td>
<td>4.8</td>
<td>9.7</td>
</tr>
</tbody>
</table>

6.3.4 Waves severely depth limited at the toe of a seawall

If wave heights are severely limited by water depth, it may be best to concentrate on extreme sea level, and to estimate what wave height could exist in that depth of water, irrespective of higher extreme waves further seaward of the wall. Also, in that circumstance it may be more useful to think in terms of extreme wave period (e.g. from a modest swell condition rather than from a severe storm condition) rather than extreme wave height. These arguments must however take account of the sensitivity of the particular response being considered to wave period.

In the hypothetical case illustrated in Table 15, if the water depth at the toe is assumed to be limited to 4m, then Hs at the toe is limited to Hsb = 2.8 m, whether it was originally a 10 year return period or a 1000 year return period condition offshore. There might, therefore be more value in refining the assumptions about sea level and toe level (and hence water depth) than in any refinement of the offshore or nearshore wave predictions. Note however that the wave overtopping or wave loading responses may be quite strongly geared to the wave period, so Hs on its own will not be sufficient.
6.3.5 Wave impact on a seawall dependent upon sea level

Some structure variables are dependent upon sea level, in which case a joint probability analysis of large waves and high (or low) sea levels may be appropriate. Wave loads on a seawall are not usually strongly dependent on water level per se (except through wave breaking) but it is sensible to test the responses at a number of commonly occurring sea levels. Conversely, wave overtopping is commonly assumed to be highest at high sea levels, and probability of scour / damage at the toe to be highest at low sea levels. Recent research has however indicated that these simplifications may be untrue, with some responses reaching maxima at particular mid-range combinations.

Analysis of the joint probability and dependence of large waves and high sea levels is discussed in Sanchez-Arcilla & Gonzalez-Marco (2007). Results are often expressed in terms of joint exceedance extremes, i.e. return periods for sea conditions in which a given sea level is exceeded at the same time as a given wave height. Example results, consistent with the present day sea levels for Location 2 listed in Table 14 and the offshore and nearshore wave conditions listed in Table 15, are given in Table 16, for two alternative typical levels of dependence between waves and sea levels.

Table 16: Example joint exceedance extremes of high sea levels with large offshore or nearshore waves

<table>
<thead>
<tr>
<th>Joint return period (years)</th>
<th>Sea level (m, above mean level)</th>
<th>Offshore waves</th>
<th>Nearshore waves</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>H_s (m) T_m (s)</td>
<td>H_s (m) T_m (s)</td>
</tr>
<tr>
<td>100, assuming low dependence</td>
<td>4.04</td>
<td>3.5 7.2</td>
<td>2.8 7.2</td>
</tr>
<tr>
<td></td>
<td>3.47</td>
<td>4.5 7.8</td>
<td>3.4 7.8</td>
</tr>
<tr>
<td></td>
<td>2.60</td>
<td>5.3 8.4</td>
<td>3.9 8.4</td>
</tr>
<tr>
<td>100, assuming high dependence</td>
<td>4.04</td>
<td>4.7 8.0</td>
<td>3.5 8.0</td>
</tr>
<tr>
<td></td>
<td>3.71</td>
<td>5.2 8.4</td>
<td>3.8 8.4</td>
</tr>
<tr>
<td></td>
<td>3.43</td>
<td>5.8 8.8</td>
<td>4.2 8.7</td>
</tr>
<tr>
<td></td>
<td>3.10</td>
<td>6.3 9.2</td>
<td>4.5 9.2</td>
</tr>
<tr>
<td>1000, assuming low dependence</td>
<td>4.50</td>
<td>3.7 7.3</td>
<td>2.8 7.3</td>
</tr>
<tr>
<td></td>
<td>3.99</td>
<td>4.5 7.8</td>
<td>3.4 7.8</td>
</tr>
<tr>
<td></td>
<td>3.50</td>
<td>5.3 8.4</td>
<td>3.9 8.4</td>
</tr>
<tr>
<td></td>
<td>3.10</td>
<td>6.2 9.1</td>
<td>4.4 9.1</td>
</tr>
<tr>
<td></td>
<td>2.70</td>
<td>7.1 9.8</td>
<td>4.8 9.7</td>
</tr>
<tr>
<td>1000, assuming high dependence</td>
<td>4.50</td>
<td>5.1 8.3</td>
<td>3.8 8.3</td>
</tr>
<tr>
<td></td>
<td>4.12</td>
<td>5.8 8.8</td>
<td>4.2 8.8</td>
</tr>
<tr>
<td></td>
<td>3.75</td>
<td>6.4 9.3</td>
<td>4.5 9.2</td>
</tr>
<tr>
<td></td>
<td>3.36</td>
<td>7.1 9.8</td>
<td>4.8 9.7</td>
</tr>
</tbody>
</table>

Some points to note about potential use of this form of results in design calculations:
- For any particular return period, the results are multi-valued (and not limited to those noted in the table) all of which should be tested as potential worst cases
- Results are quite sensitive to the assumed level of dependence
• This form of results is useful only for design considerations sensitive to both waves and sea level

6.3.6 The key wave parameters

Wave conditions are usually calculated in terms of the parameters of a sea state, averaged over a period of, say, one or three hours. The parameters will include significant wave height, mean and/or peak wave period and mean wave direction. The wave spectrum may also be available. The highest individual wave height and corresponding wave length are unlikely to be available analytically, and so empirical relationships may be needed to derive these parameters for use in some force calculations.

• Maximum individual wave height. A commonly used approximation to the ratio between maximum \(H_{\text{max}}\) and significant \(H_s\) wave height in a sequence of \(N\) waves in deep water is \(H_{\text{max}}/H_s = \sqrt{\ln(N/2)}\). Another is that is \(H_{\text{max}}/H_s = 1.8\). Either of these approaches should be used with some caution for breaking waves. Chapter 2.2 of the PROVERBS final report by Oumeraci et al (1999) discusses the relationship between key wave height parameters in the context of seawall design.

• Corresponding wave length. Where the wavelength is needed close to the seawall or breakwater, a reasonable estimate would be to take the linear wave theory wave length \(L_p\) corresponding to the peak period \(T_p \approx 1.25T_m\) of the spectrum and the depth \(d\) at the toe of the seawall, in which \(L_p = (gT_p^2/2\pi)\tanh(2\pi d/L_p)\). It should be noted, however, that some parameters do use the notional deep-water wavelength, even when apparently determining other parameters close to the structure.

Example calculations of \(H_{\text{max}}\) and \(L_p\), corresponding to the offshore and 6m-depth wave conditions listed in Table 14, are given in Table 17.

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>Offshore</th>
<th>Water depth at toe 6m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(H_s) (m)</td>
<td>(T_m) (s)</td>
</tr>
<tr>
<td>1</td>
<td>4.7</td>
<td>8.0</td>
</tr>
<tr>
<td>10</td>
<td>5.5</td>
<td>8.6</td>
</tr>
<tr>
<td>100</td>
<td>6.3</td>
<td>9.2</td>
</tr>
<tr>
<td>1000</td>
<td>7.1</td>
<td>9.8</td>
</tr>
</tbody>
</table>

6.4 Flood risk

Flood risk is a function of both the probability (chance) and consequence (assets lost, people injured) of flooding. It will include lower probability (rarer) higher consequence (death and destruction) events, and higher probability (commoner) lower consequence (minor damage to buildings) events.

Flood risk assessment is helpful in estimating the value and justifying the cost of seawalls, and may be a necessary part of the decision to build or upgrade defences and/or the design of defences.
Estimates of the probabilities of different types of flood event are best developed from the distributions of the Source variables. Estimates of the consequences require prediction of the Pathway variables (overtopping, breaching, flood inundation) and identification of the Receptors (people, roads, buildings etc) and the extent to which they would be damaged by a flood event.
7 Conclusions and recommendations

The present study is done within the framework of the European FLOODsite project. The main aim of Task 2 is the assessment of climatic extremes as the main driver for flooding and erosion processes. The emphasis is on the issues contributing most to uncertainty in flood risk management decisions. The analyses are carried out for single climatic variables and for realistic combinations including the “control” exerted by the domain morphodynamic evolution (i.e. marginal, conditional, and joint PDFs).

Task 2 focuses on three activities:

a) Theoretical analysis methods: distribution types and selection, fitting and resampling techniques, temporal sequences and memory effects. This activity will also include SSA (Singular Spectrum Analysis), NN (Neural Networks), POT (Peak-Over-Threshold), GPD (General Pareto Distribution) and spatial/temporal correlations.

b) Analysis of extreme events: extreme samples (limited) and distributions, 2D distributions and CCA. This activity will also consider the morphodynamic control on extreme PDFs and the coupling between long-term (decadal) climatic trends and coastal processes.

c) Hydraulic loading of flood defence structures: wave transformation over shallow foreshores; wave induced fluxes and wave-soil-structure interactions. This activity will also deal with the analysis of different flood mechanisms and the corresponding impact on coastal morphology.

The research in Task 2 is targeted at issues contributing most to present uncertainty in flood risk management decisions. It benefits from the combined knowledge and expertise of the atmospheric, riverine, and marine research communities.

The objective of the task is to improve our understanding of the primary drivers of flood risk (waves, surges, flow etc.) through research targeted at key issues that contribute most to current uncertainty in flood risk management decisions. These "sources" are defined as the climatic factors inducing flooding, erosion, or any other "threat" to the safety/stability of the land-water fringe. These sources therefore include sea waves, storm surges, river water levels, and discharges.

The research described in the present report is part of the third activity: Hydraulic loading of flood defence structures. The main objectives of this subtask are to predict the wave transformation over shallow foreshores with various complex processes such as percolation, detailed friction, and turbulence parameterisations. The emphasis will be on extreme wave evolution over very shallow depths. Furthermore, the hydrodynamic loading of natural and man-made flood defences is studied in much more detail so that appropriate hydrodynamic parameters are available to describe the loading of flood defences at all points of interest.

With respect to sea dikes this report first reviews the influence of foreshore mobility on hydraulic boundary conditions by performing a sensitivity analysis with the Boussinesq-type model TRITON on a foreshore for which results from field measurements exist. This study provides insight into the influence of variations of the level of the bar, the trough behind the bar, and the low-tide terrace in front of a dike. In addition, estimates have been made of the amount of low-frequency energy, depending of characteristics of the foreshore and the wave conditions.
Furthermore, previous investigations on flow processes on sea dikes induced by wave run-up and wave overtopping have been revisited. Contributions by Schüttrumpf (2001) based on small- and large-scale model tests and by Van Gent (2002) based on small-scale model tests have been reviewed including their prediction methods for flow depths and water velocities at all positions along the dike surface.

The report then summarises investigations performed outside FLOODsite within the European IHP-ARI programme. Within the DIKE-3D project, measurements of flow depths and velocities at the inner dike slope have been performed under 3D conditions. Results are compared to the previously mentioned investigations for the reference case (perpendicular wave attack) and angles of wave attack up to 60 degrees.

The analyses on dunes showed that run-up levels and sand dune erosion had great effect on the investigated beach during the last decades. In case of a future sea level rise, the foreshore width and the sand dune volumes are expected to decrease drastically. From existing forecasts, the direct sand dune erosion was calculated to increase by up to 75% by the year 2100. This implies that many houses and other infrastructure located behind the dunes may be subject to more frequent flooding. The study also indicated that a run-up level, with a 100-year return period today, in the future may occur up to 13 times more frequently. It should be pointed out that the presented calculations were, in many cases, made on schematized representative profiles which all are case-specific. For this reason, the presented numbers have to be interpreted with care.

Eventually, the report deals with seawalls. First, a definition of seawalls is given and the most relevant parameters with respect to the hydraulic loading conditions and the structural conditions are introduced. The loading variables are then discussed in some detail where first, the breaker type is discussed and then the various loading conditions leading to potential functional (wave overtopping) or structural failure (toe erosion) is discussed. The last part of this chapter discusses the input variables in much detail with respect to both sea level and waves, and their interaction, and providing examples. Eventually, this leads to recommendations on how to perform a seawall design, with references to most relevant publications.
8 References


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