SEDIMENT TRANSPORT LOSSES DUE TO WASHOVER ON A BARRIER ISLAND

Final Report

N.A.M. van den Wollenberg
Delft University of Technology
Frederic R. Harris B.V.
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Preface

This report is the result of a nine months study to complete the degree of Civil Engineering at Delft University of Technology. It is titled "Sediment Transport Losses due to Washover on a Barrier Island" and is undertaken in cooperation with the Coastal and Environmental Group of Frederic R. Harris B.V., The Hague. This department provided a perfect atmosphere for a student who wants to complete his/her study. All facilities were available for use and the engineers working at Frederic R. Harris were always willing to assist.

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(Marjolein, bedankt voor je geduld en steun)
Summary

The purpose of this study is to quantify the cross-shore sediment transport, caused by washover flows on barrier islands in general and on the Wai San Ting sandbar in Taiwan in particular. Washover transport on a barrier island is the transport of sediment across the sandbar from its steeper seaward face towards the gentler sloping landward or lee-side, caused by overflowing water from one side to the other. The overwash current is generated by a water level difference across the bar, which is caused by wave set-up and/or a difference in tidal amplitude at both sides of the bar.

To simulate the washover process and to quantify the sediment losses due to washover, hydraulic and morphological models are applied. For the quantification of the washover transport at specific cross sections of sandbars, a one-dimensional model is designed. This stationary wave and flow model is able to compute irregular waves, washover currents and washover transports, driven by tide and wave-induced water level differences across the sandbar. The model links the sediment transport directly to the overwash current, so that the consequences of various overwash events on the cross-shore barrier profile can be examined.

The one-dimensional overwash model is verified by comparing the results with laboratory measurements and by three other mathematical models: MIKE21_HD, which computes the tide and wave-induced overwash currents; MIKE21_NSW, which computes the wave phenomena in near shore areas, and Unibest_LT, a one-dimensional model which is used to compare the wave parameters and wave set-up with the overwash model. To examine the overwash process in more detail a sensitivity study is performed. In the sensitivity study various barrier island geometries, wave and flow conditions are modelled and the effect of the different parameters on the washover process is examined.

In the end of this thesis a case study of a regularly overwashed sandbar in Taiwan is made. The near shore wave model MIKE21_NSW and the hydrodynamic model MIKE21_HD are applied to simulate the two-dimensional wave and flow conditions over and around this sandbar. The results of this 2-D modelling are used for boundary conditions in the 1-D overwash model, which is applied to compute the washover sediment transport on a cross section of the sandbar. With the calculated sediment transport an extrapolation is made to find the yearly sand losses which contributes to the total movement of this barrier island.
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CHAPTER 1

Introduction
1. Introduction

Many barrier beaches around the world are affected by an environmental process that is driven by storm surges, high tides and waves: washover. Washover may be considered as a higher magnitude extension of overtopping, in that the volume of swash that moves over the crest is sufficient to create a unidirectional flow over the back side, eroding and lowering the crest, and depositing material on the landward side.

Washover transport is one of the mechanisms which can affect barrier islands and sandbars. Other transport mechanisms which cause erosion and accretion are:

- **Longshore transport**: breaking waves which approach the bar obliquely transport sediment alongshore.
- **Offshore transport**: storm and typhoon waves transport sediment on and offshore.
- **Channel transport**: tidal channels transport sediment to the back side.
- **Relative sea level rise**: subsidence, sea level rise and compaction of deltas all serve to bring about net losses of material to the active system.
- **Wind transport**: transport of sand by wind.

Except for wind transport, these mechanisms can occur simultaneously with overwash. Wind transport can only occur when the crest of the bar is above the water surface for a longer period of time. All these processes contribute to the total sand loss of sandbars. This study is performed to examine the influence of washovers on the sand losses of barrier islands and to quantify the contribution of the washover transport to the total sand loss.

Washover transport does also play an important rôle on the migration process of the Wai San Ting sandbar in Taiwan. According to estimates based on satellite images and field measurements it was determined¹ that a significant amount of sediment was transported cross-shore from the front to the back side of this sandbar, mainly caused by washovers. This cross-shore transport is for a great deal responsible for the total movement of the sandbar. However, the amount of sediment transported by overwash could not be determined very accurate because not much was known about the washover process itself.

The objective of this study is therefore to quantify the contribution of washover to the total sand loss of barrier islands in general and the Wai San Ting sandbar in Taiwan in particular. In order to achieve this quantification a washover-theory is set-up and a mathematical modelling approach is applied. A scheme of the followed strategy is shown in Figure 1.

Before a theoretical solution of overwash is given, a definition of the problem is described in Chapter 2. In this Chapter overwash is defined and the unknown aspects of the overwash process are specified. Different questions will be treated, such as "what is overwash?", "when

¹: by Frederic R. Harris B.V. and Sinotech Engineering Consultants Inc.
will overwash be generated?", "which parameters need to be sought out that are representative for overwash?", and "what are the difficulties of modelling overwash?".

After the problem is defined, a literature study is performed to obtain a better understanding of the different environmental processes and transport mechanisms affecting barrier islands. In this literature study, which is entitled "Washover Transport on Sandbars", various case studies of overwashed barrier islands are analyzed and some measurements of sand losses due to overwash are examined. In Chapter 3 a summary of this literature study is given.

In order to determine the overwash current and the washover transport, first a theoretical description of the overwash process is given. The theory, described in Chapter 4, is used to set-up a 1-D stationary overwash model, which is described in Chapter 5. This model simulates the overwash current, flowing over the crest of a schematized barrier profile, driven by the tide and (irregular) waves. The results of this 1-D model are verified with published studies, laboratory measurements, a 1-D surf zone model and a 2-D flow and wave model. After the verification of the 1-D model a sensitivity study is performed to determine the influence of the different parameters effecting the overwash process.

To show the limitations of the 1-D overwash model (2-D effects of washover flow) the regularly overwashed Wai San Ting sandbar is modelled using the 2-D wave model MIKE21_NSW and the 2-D flow model MIKE21_HD. The 2-D effects are caused by the real irregular bathymetry, tide-induced ebb and flood currents and varying wave heights and directions. Another reason why these 2-D models are used is to determine the wave and water level conditions at both sides of the sandbar, which are used as boundary conditions for the 1-D overwash model.

The 1-D overwash model is applied to examine the effects of overwash on a particular cross section of the Wai San Ting sandbar. The overwash velocities as calculated in this model are used to compute the sediment transport rates. The changes in the cross-shore profile are determined by computing the gradients in sediment transport. The cross-shore profile changes are determined by using an explicit time stepping technique, assuming stationary conditions during each time step. Using this technique, the morphodynamics of an overwashed barrier island can be simulated. The tide and wave conditions, obtained from the 2-D modelling, are used to determine the yearly washover sediment loss. The computed quantities are compared with the total losses found from satellite images and field measurements. The case study is described in Chapter 7.

In Chapter 8 the study conclusions are described. In the conclusions the effects of washovers on the sediment loss of barrier islands are pointed out and some recommendations are made concerning the modelling of the morphodynamics of barrier islands.
Sediment loss due to washover on a barrier island

Quantification yearly sand losses due to overwash

Problem definition

Literature study

Modelling

1-D modelling
- schematized barrier profiles

Verification
- measurements
- Unibest_LT

Comparison
- flow & wave results MIKE21

1-D modelling
- cross sectional analysis

Case study
- Wei San Ting sandbar
- Taiwan

2-D modelling
- real bathymetry
- 2-D effects washover

Washover sand losses over 1 year:
1-D overwash model

Total sand losses over 1 year:
- sat. images/meas.'s

Figure 1: Scheme of study strategy
CHAPTER 2

Problem Definition
2. Problem definition

2.1 Sand losses Wai San Ting sandbar

The main problem that needs to be sought out is to determine the contribution of washover to the total sand loss of barrier islands in general and the Wai San Ting sandbar in particular. Along many barrier coastlines around the world the effect of overwash on the retreat of the barrier islands and sandbars is significant. In this study one particular sandbar, located at the mid-west coast of Taiwan, is examined: the Wai San Ting sandbar. Before the morphodynamics of this large feature are described, first definitions are given of the terms "sandbars" (or "sand spits") and "barrier islands".

The terms "sandbar" and "sand spit" are used in this report to characterize coastal features which are formed from material moving along the coast. These features are attached at one end to a source of sediment (mainland, river outlet) and extend into open water. Sand spits and sandbars elongate in the general direction of the littoral drift and therefore represent the direct movement of sand along the beach (Leatherman, 1981, in lit. study "Washover Transport on Sandbars").

In contrast to a sand spit or sandbar which is formed from material moving along the coast, barrier islands are built from material moving perpendicular to the coast. The term "barrier" identifies the structure as one that protects other features, such as lagoons and salt marshes, from direct wave attack of the open ocean. Barrier islands can form when there is a sufficient supply of beach material from offshore and the bottom bathymetry is such that the waves break at some distance from the coast, because of a broad shallow foreshore zone (Leatherman, 1981, in lit. study "Washover Transport on Sandbars").

The Wai San Ting sandbar is a sand spit, which is formed on an old relict river delta and is detached from the mainland by a large tidal inlet. The sandbar is moving/rotating towards the coastline at a speed of about 40 m/year at the seaward end. This feature has the function as a sea defence, protecting the coastline in the lee-side of the bar. Estimations show\(^2\) that, about one century, when the Wai San Ting reaches the coastline, the poor protected coast behind the bar will then be exposed directly to the wave and currents, which can cause severe flooding in the overcrowded area behind the coast.

Three main transport mechanisms can be considered to be responsible for the movement of the sandbar: longshore transport, transport through tidal channels and washover transport. From satellite images and field measurements it is estimated that about 4 to 7 m\(^3\) of sand is lost per year over a 10 km stretch of the bar. The contribution of the first two transport mechanisms to the total sand loss can be computed quite accurate but the contribution of the

Sediment loss due to washover on a barrier island

Washover transport is still based on estimates, because not much is known about this process. In this study an attempt is made to quantify the losses caused by washover. In the literature study, which is performed in the early stage of this study, a more detailed description is given of the evolution of the Wai San Ting sandbar caused by various transport mechanisms.

2.2 Definition washover process

Washover transport on an intertidal sandbar or barrier island is the transport of sediment across the bar from its steeper seaward face towards the gentler sloping landward or lee-side (Figure 2). The transport takes place under the influence of washover currents which are generally fast flowing (0.5-2.5 m/s). They occur when the bar becomes submerged at high tide and/or high wind and wave set-up events which raise the water level locally, causing water levels on the bar of several centimetres to several meters. These same events will often combine to cause a higher water level at the front or seaward side of the bar than at the back or lee-side, and thus a higher water level difference across the bar that contributes to the flow.

![Figure 2: Washover transport on an intertidal sandbar](image)

The magnitude and frequency of overwash depends on the following elements:

- barrier exposure and orientation
- shore-parallel variation of the beach crest line.
- frequency of major storms
- wave energy
- tidal range
- beach sediment
- ecological response of vegetation to the overwash process

Normally overwash and overtopping will occur during high-magnitude, low-frequency storms where swash flows create and occupy narrow channels through beach or dune ridges, by
which sediment is transferred into characteristic fan-shaped sinks. This process provides an important mechanism for shoreline and barrier migration. Washover transport often occurs together with other transport mechanisms, such as inlet transport, littoral transport, and aeolian (wind blown) transport. In this study two main hydraulic conditions responsible for washover are examined: the tide and the waves.

In this study schematized barrier profiles are used to control the influence of the barrier's geometry on the process when the effect of the different parameters involving overwash is examined.

**Washover due to the tide**

In order to determine the washover current velocities, the flow is considered to be uniform per metre width of the bar, stationary across the bar, and depth averaged (2-D and 3-D effects neglected, see Figure 3). The overwash process can then be described by two variables:

1. **The total water depth (h + h'),** varying across the barrier island. This total water level is the depth to SWL including the variation of the water surface, relative to MSL.

2. **The flux/discharge Q,** which is the depth averaged velocity multiplied with the total depth [m³/s/m]. This flux is constant across the bar due to the uniform, stationary hydraulic conditions.

To solve these two variables two equations are needed, which must be solved simultaneously. In case of a tide-induced overflow, these equations are the momentum equation and the mass balance equation. To find a solution for these equations, boundary conditions are needed. The boundary conditions can be defined as (a) water level elevations at both sides of the bar (h'front and h'back), and (b) as a water level at one side of the barrier, and as a constant flux Q across the barrier.

In a stationary 2-D situation, the flux Q is constant because of the mass balance (uniform, stationary flow conditions), and every depth is coupled to a specific velocity. When the boundary conditions are specified by the water levels at both sides of the bar, the overwash velocities are determined by one specific Q that corresponds with the water surface slope across the bar and the bottom friction. A difficulty hereby is the position of the downstream boundary water level relative to the crest of the bar (see "?" in Figure 3). The downstream boundary condition is not allowed to influence the overwash process, and must therefore be chosen far enough away from the crest.
In case of an overwash event caused by waves only, some additional phenomena have to be dealt with. One of these is the representation of the wave set-up in combination with a washover flow. The problem is explained first by considering the wave set-up on a normal plane beach (Figure 4).

**Figure 4: Wave set-up on a plane beach**
A lot of research has been done, concentrating on the driving mechanisms in the surf zone, in particular the change in radiation stress and the pressure gradient due to the slope of the mean water level. These studies describe the cross-shore velocity and the wave set-up on a uniform beach with or without a submerged longshore bar in front of the shoreline.

In case of an submerged barrier island with an irregular two-dimensional and (partly) horizontal geometry, the problem rises in what way the wave set-up will develop. Compared with the wave set-up on a normal mainland beach the beach of a barrier "falls away" (Figure 5 and 6), with waves approaching the bar from the right side. The wave set-up will not develop like in the classic situation.

Except for the problem of the development of the wave set-up in front of the bar, also the problem rises which processes determine the water level behind the barrier. Important parameters effecting the downstream water level are the water level in front of the submerged barrier, the bottom roughness of the crest and the geometry of the backside of the bar. The water level in front of the barrier is, in this situation of wave-induced overwash, determined by the wave set-up. The wave set-up is mainly counteracted by the change in radiation stress of the waves. The change in radiation stress is caused by the change in wave energy (dissipation: change in wave height). The change in wave height depends on the depth and the bottom friction, and the depth is related to the mean overwash velocity $U = Q/(h+h')$.

The above described interaction of the different parameters involved with washover due to waves only, points out the complexity of this relative simple, one-dimensional situation. Compared to the tide-induced overwash, now three equations are needed to find a solution for the two variables $Q$ and $h+h'$: the momentum equation for currents and waves, the water mass balance and the wave energy balance. The interaction between velocity, water depth and waves, solved with the three equations, must then be solved iteratively. In section 4.2 is described how these different parameters can be calculated.

Knowing the velocities in combination with the wave and sediment properties, the sediment transport can be determined. With the known gradients in the sediment transport the changes in the cross-shore barrier profile and the sand losses due to the overwash process can be determined, resulting in a movement of the barrier. Considering the Wai San Ting sandbar the sand losses due to washover are determine according to the following three points:

1. Determine the yearly tide and wave conditions  
2. Compute the overwash currents and waves for the representative summer and winter conditions (tide-only and tide+waves)  
3. Compute the sediment losses for both summer and winter conditions  
4. Extrapolate the losses to find the yearly sand losses caused by washover.
Figure 5: Water surface elevation of a wave-induced washover

Figure 6: Development of wave set-up and surface elevation in wave-induced overwash situation
CHAPTER 3

Literature Study
3. Literature study

In the early stage of this study a literature review has been carried out. A considerable amount of references relating to washover studies and case studies of washover events all around the world, described and discussed in various papers and magazines, improved the understanding of the phenomenon "washover". The various washover studies are examined and collected in the report named "Washover Transport on Sandbars; Literature Study". Good descriptions were given concentrating on the consequences of overwash on barrier islands. Also the environmental aspects responsible for overwash, and the locations around the world where overwash occurred frequently, were described in detail. The conclusions found in the literature study are briefly described below.

An often examined environmental mechanism is sea level rise. Barrier beaches all over the world are affected by this phenomenon. This declares the increasing number of studies in the past 15 years which describe the consequences of sea level rise (Orford, Carter and Forbes, 1991; Inman and Dolan, 1981).

Several studies provided measurements of cross-shore profile changes and washover transport rates (Leatherman, 1976; Ritchie and Penland, 1988; Dingier and Reiss, 1989). Despite this, no hydrodynamic and sediment transport computations were carried out to investigate the direct effect of overwash and other coastal processes on the development and migration rate of barrier islands. Also, no information was available of a mathematical approach describing the overwash process. On the other hand, with the increasing knowledge of morphological problems, also the modelling techniques were optimized in the last decade. This will lead to more straightforward transport computations with a more quantified description than a geological one, although for each specific case a geological and morphological study needs to be done.

Having analyzed some quantitative sediment transport rates for each of the mechanisms one can conclude that the contribution of sediment transport due to overwash processes is not negligible (Dingler, Reiss and Plant, 1993; Kochel and Wampfler, 1989; Pilkey, Neal, Monteiro and Dias, 1989). The amount of sediment transported by the overwash current depends on the overwash frequency and the overwash current strength itself. However, one heavy storm can cause sand losses which is equal to the sand loss of a barrier shoreface in a whole year without storms. This indicates that the quantification of overwash transport should be considered over a number of years, when the seasonal changes can be averaged (Leatherman, 1979).

The crestal profile (elevation) of barrier islands is another important factor which determines the transport rate, or, whether or not overwash actually occurs (Schwartz, 1975). Also the weather conditions (storm, wave set-up) have a great deal of influence on the impact of washovers. When a sandbar has experienced a lot of overwash events, it is reshaped completely due to the transport of material from the shoreface to the backside of a bar. Also
the topography (shape) of barrier islands is related to the amount of overwash events: barrier islands are more susceptible for washovers then barrier spits and detached spits, where the longshore current plays a more significant role (Hequette and Ruz, 1991).

In a former study\textsuperscript{3} an attempt was made to reproduce the sediment losses of the Wai San Ting sandbar caused by washover, in order to determine the total movement of that bar. The estimated sediment losses were based on computations using the Chezy formula and the Bijker formula, determining the average sediment transport across the bar. Together with computations of the longshore transport and comparisons of satellite images of the sandbar over a number of years, the total sediment loss and the movement of the sandbar could be estimated. Recommendations were made in that study to examine washover mechanisms in more detail.

\textsuperscript{3}: performed by Frederic R. Harris B.V. and Sinotech Engineering Inc. (1993)
CHAPTER 4

Theoretical background of washover
4. Theoretical background of washover

4.1 Introduction

Barrier islands and intertidal sandbars vary in morphological response to the interaction of the tidal range and wave energy effects. Barrier islands are built from material moving perpendicular to the coast, while barrier spits and intertidal sandbars are also affected by longshore transport mechanisms.

The most important control of the geomorphology of depositional coasts is the type and amount of hydraulic energy expended within an area. Especially for barrier islands their shape and existence depend mainly on the two important energy factors, the wave energy and the tidal current energy, which can be related directly to the tidal range. In areas of average marine wave conditions, coasts with small tidal ranges (microtidal: 0-2m) are usually dominated by wave energy, and coasts with large tidal ranges (macrotidal: >4m) are dominated by tidal currents and tidal-level fluctuations. Coasts with intermediate tides (mesotidal: 2-4m) show influences of both waves and tides and are thus termed "Mixed-energy coasts" (Hayes, 1965). In the literature study "Washover Transport on Sandbars" is declared that in areas with intermediate tides (mesotidal) the barrier islands are affected by washover events caused by both waves and tides (Leatherman, 1976). The length of the barriers in those areas is between 3 and 20 km and they have often a drum stick shape. This study only concentrates on regularly overwashed barrier islands, located in mesotidal areas and thus affected by tides and waves.

The theoretical description of the washover process starts in section 4.2 with a description of the used basic equations. These equations are simplified by assuming that the washover process is stationary (constant in time). The first overwash mechanism theoretically defined is the tide, described in section 4.3. The tidal influences on the overwash process are analyzed, followed by a theoretical formulation of the tide-induced overwash current.

The influence of the waves on the washover process is described in section 4.3. In that section first the theory of wave driven surface elevation (wave set-up) on a normal beach is defined, followed by the development of the wave set-up and the wave-induced current in front and over the crest of a flooded barrier island.

In order to determine the effects of waves on the washover process, wave parameters need to be known. The changing wave parameters, caused by dissipation processes (wave breaking and bottom friction), must be computed from deep sea, over the crest to the lee-side of the barrier. A well known surf zone model, which uses the irregular wave theory, is used to compute these parameters. A description of this surf zone model is given also in section 4.3.

In section 4.4 a theoretical description of the combined tide and wave-induced overwash is given.
Besides the overwash equations also boundary conditions are needed. The boundary conditions to be used and the effects of the chosen boundary conditions on the overwash solution are discussed in section 4.7.

In section 4.8 the used sediment transport formulation is shortly described and for the completion of the subject washover the remaining mechanisms (wind set-up, overtopping, sea level rise, submergence) contributing to overwash are shortly discussed in section 4.9.
4.2 Basic equations

The analysis of waves in the surf zone is restricted to a two-dimensional uniform coast with normal incident waves. The water surface elevation and the wave height can then be determined by simultaneous solutions of the momentum, the mass balance, and the energy balance equation. The wave set-up and the dissipation of energy due to breaking waves and wave bottom friction is described with the model of Battjes and Janssen (1978).

For the description of tide-induced overwash the momentum equation and the mass balance equation need to be solved. Generally, to describe each overwash mechanism (tide-only, waves-only, and tide + waves), modifications of the momentum equations are made. For each overwash condition different driving forces and frictional resistance forces are defined, each representing a specific condition.

The barrier shore and barrier crest are assumed to be infinite long. This means that no water is allowed to flow out of the study area sideways, and the amount of water flowing into the study area is equal to the outflowing water, conform the conservation of mass balance (flux Q is constant).

The used equations are solved for stationary hydraulic conditions. This means that when an overwash current is simulated, actually one moment out of a time varying process is simulated. However, this is acceptable because in reality overwash is a stationary situation for about 3 hours (same wave conditions and water level differences). When also the second order terms are neglected, the following set of equations remains:

Conservation of momentum for wave-induced currents on a beach or sandbar

The equation of momentum conservation is derived from Phillips (1977) and Svendsen et al. (1987), and is written as follows:

\[
\frac{\partial S_{xx}}{\partial x} + \rho \frac{\partial}{\partial x} \left( \frac{Q^2}{h + h'} \right) + \rho g (h + h') \frac{\partial h'}{\partial x} + \frac{\tau_{cw}}{S_{xx}} = 0
\]  

(1)

where  
- \( S_{xx} \): component of radiation stress normal to the shore  
- \( \rho \): density of the water  
- \( Q \): total average flux due to waves and current  
- \( h \): depth below SWL\(^4\), varying with the distance from the shore  
- \( h' \): surface elevation above SWL, averaged over the waves  
- \( g \): acceleration due to gravity

\[^4\): Still Water Level
Sediment loss due to washover on a barrier island

\( \tau_{cw} \) : time averaged bottom shear stress due to waves and current \( [N/m^2] \)

\( x \) : shore-normal coordinate \( [m] \)

**Conservation of momentum for tide-induced currents**

The tide-induced washover current can be calculated with the modified long wave equation:

\[
\rho \frac{\partial}{\partial x} \left( \frac{Q^2}{h + h'} \right) + \rho g (h + h') \frac{\partial h'}{\partial x} + \tau_c = 0
\]  

(2)

where \( h' \) : surface elevation above MSL \( [m] \)

\( \tau_c \) : bottom friction due to current only \( [N/m^2] \)

**Conservation of mass**

The averaged mass conservation equation, when the variation in the water density \( \rho \) is neglected and for a uniform situation (constant hydraulic conditions per unit width of the crest) reads as:

\[
\frac{\partial}{\partial t} (h + h') + \frac{\partial}{\partial x} Q = 0
\]  

(3)

When also a stationary situation is considered (constant hydraulic conditions in time), the flux \( Q \) becomes constant parallel and shore-normal to the crest.

**Conservation of wave action**

For given incident wave parameters and beach profile, the variation of mean wave energy (\( E \)) with distance normal to the shoreline can be calculated from the wave energy balance:

\[
\frac{\partial}{\partial x} \left( E c_g + U \right) \left( \omega_r \right) = -\frac{D}{\omega_r}
\]  

(4)

where \( E \) : short wave energy \( [N/m^3] \)

\( c_g \) : group velocity at which the wave action \( E/\omega_r \) propagates \( [m/s] \)
\[ \omega_t : \text{intrinsic wave angular velocity} \quad [1/\text{s}] \]

\[ D : \text{rate of wave energy dissipation} \quad [\text{Ns/m}^1] \]

\[ U : \text{depth averaged flow velocity} \quad [\text{m/s}] \]

\[ \frac{\partial \omega}{\partial x} = 0 \quad (5) \]

where \( \omega \) : absolute angular wave velocity \[ [1/\text{s}] \]

In the momentum equation for waves, equation 1, the known variables are \( p, g \) and \( h \). The unknown variables \( S_{xx} \) and \( \tau_{wm} \) can be solved with additional equations, defined in Appendix I and section 4.4 respectively. The remaining variables (surface elevation \( h' \) and flux \( Q \)) must be solved simultaneously with the three equations 1, 3 and 4.

To solve the tide-induced overwash current and elevation with equation 2, one extra equation is needed: the bottom friction equation, defined in section 4.3. Together with the equations 2' and 3 a solution can be found for the surface elevation \( h' \) and the flux \( Q \).

There are two ways of solving \( h' \) and \( Q \) from the momentum equations 1 and 2. The first way is to guess the discharge \( Q \) in order to find the surface elevation \( h' \), which can be found by iteration. Another possibility is to define \( h' \) as the downstream boundary and to find a \( Q \) that fits with that boundary condition. With this \( Q \) the surface elevation between the two boundary levels can be solved then by iteration.

A solution for the energy balance (equation 4) can only be found with extra equations for the wave energy \( E \), the group celerity \( c_g \) and the dissipation term \( D \) (Appendix II). The wave height can be determined from the wave energy \( (E \sim H_{rms}^2) \). This means that the energy dissipation over a distance \( dx \) determines the Root-mean-square wave height. The closure relations to determine the wave energy are described in section 4.4 and the short wave parameters which are needed to compute \( E \) and \( c_g \) are defined in Appendix I.
4.3 Tide-induced overwash

Consider a barrier spit with a shallow water area at the lee-side. Due to shallow water effects the tidal amplitude and the tidal current velocity is diminished compared to the amplitude and velocities in deeper water. This results in a water level difference at both sides of the bar (Figure 7). Examples of the shallow water effects are:

- varying water surface width due to flooding and drying of intertidal sandbars
- influences of the bottom friction on the flow field
- variable celerity of the tidal wave, depending on the depth (phase difference).

The induced water level difference across the bar introduces a flow over the bar, when the crest is below the water surface (during flood tide). The flow is a result of a pressure gradient, caused by the water level slope across the bar, and is mainly counteracted by the bottom friction.

Figure 7: Submerged barrier spit with shallow water area at lee (left, B) side
It can be seen in Figure 8 that the overwash intensity and duration depends mainly on the crest height relative to the flood level. When the crest height becomes lower, the overwash duration increases; a higher crest height decreases the overwash duration. The number of washovers caused by the tide is determined by the number of times that the tidal elevation exceeds the crest height. For example, when the height of the bar is several decimeters above MSL, flooding occurs 2 times a day at flood tide (the period of the tide is approximately $12^h 25^m$). Normally washover occurs only during spring tide (= about 4 times per 14-days cycle). A 14-days cycle is the period from spring to neap tide. Figure 9 shows a typical 14-days tide cycle at the west coast of Taiwan (Spring tide March 1992).
Figure 9: Tidal variation from neap to spring tide
Differential equation of tide-induced overwash

The tide-induced overwash process is described with the equation of motion of a tidal wave according to the long wave theory and is given by equation 2 for a stationary situation. For the completeness this equation is presented here again:

\[ \rho \frac{\partial}{\partial x} \left( \frac{\rho g Q^2}{h + h'} \right) + \rho g (h + h') \frac{\partial h'}{\partial x} + \tau_e = 0 \]  

(6)

The first term is the inertia of the water volume flowing over the crest. The second term, the pressure gradient term, is a driving force and the third term is a frictional resistance term. This bottom friction term is given by:

\[ \tau_e = \frac{\rho g Q |Q|}{C^2 (h + h')^2} \]  

(7)

where \( C \) : Chezy friction factor, depending on the total depth \( h + h' \) [m^{1/2}/s]

The momentum equation 6 can be written different when the first term is differentiated and with the water level slope at the left side of the equation (performed in Appendix III). The full "tide-induced-overwash-equation", is now defined by:

\[ \frac{dh'}{dx}_{\text{tide}} = \frac{1}{\rho g (h + h') - \frac{\rho Q^2}{(h + h')^2}} \left( \frac{\rho Q^2}{(h + h')^2} \frac{dh}{dx} - \tau_e \right) \]  

(8)

Since in this ideal situation no waves are present, the friction term is only based on the friction caused by the current (equation 7). Boundary conditions must be specified to find the solution.

Without the inertia term, equation 9 can be written as the Chezy formula, which is a balance between the pressure gradient and bottom friction:

\[ \frac{dh'}{dx} = -\frac{Q |Q|}{C^2 (h + h')^3} \]  

(9)

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To show the influence of the inertia term on the overwash surface elevation and velocity, a comparison is made between the full tide-induced overwash equation 8 and the Chezy equation 9. Figure 10 shows the barrier profile and Figure 11 shows the variation of the velocity and surface elevation across the bar computed with the two formulas and for a tide-induced water level difference of 0.20m, a bottom roughness of 0.02m, and a bottom slope at the front (right) side of the barrier of 1:50. The dotted lines represent the surface elevation and velocity profile calculated with the Chezy formula. It can be seen that, although the Chezy-flux Q is higher than the flux calculated with the full equation, the velocities of the full equation are higher than the velocities computed with the Chezy formula. This difference is only caused by the inertia term.

This results show that the use of the simple Chezy formula does not give the right velocities, and is therefore not applied.

Figure 18 on page 35 shows the results of the tide induced overwash for a tidal difference at both sides of the bar of 0.2 m, a bar with a crest at -0.3m SWL, a crest length of 300 meter and a bottom roughness of $r = 0.02m$. This washover situation can occur during spring tide, and will last between approximately 1 and 3 hours, depending on the crest height.
Sediment loss due to washover on a barrier island

**Barrier island profile**

\[ m_{\text{front}} = 0.02; \ m_{\text{crest}} = 0.00; \ m_{\text{back}} = -0.007 \]

**Figure 10**

Elevation and Velocity tide-induced overwash

Full overwash equation vs. Chezy equation (\( r = 0.02 \text{m}; \ dz = 0.20 \text{m} \))

**Figure 11**

\[ Q (\text{full eq.}) = 0.260 \text{ m}^3/\text{s/m} \]
\[ Q (\text{Chezy eq.}) = 0.271 \text{ m}^3/\text{s/m} \]
4.4 Waves-induced overwash

The second mechanism that has an effect on the washover transport on a submerged barrier island is the wave-induced overwash current. This current is driven by the momentum of the waves which approach the bar from one side and elevate the water level due to breaking activity.

Wave set-up on a normal beach

When waves approach the shoreline their height will depend on the influence of the bottom. Refraction, diffraction, shoaling and breaking are the different phenomena that waves can undergo. Nearing the breaker line the steepness of the waves increase (H/L), and when the wave height/depth ratio exceeds a critical value (breaker index γ ~ 0.80) the waves break and a change in the radiation stress component normal to the shore will occur. The change of the radiation stress component of interest, $S_{xx}$, will exert a net resultant force on a vertical water element. On a normal beach, this radiation stress resultant is counteracted by a static horizontal pressure gradient from a water surface slope (see Figure 12).

![Figure 12: Equilibrium between surface slope and gradient in radiation stress](image)

This equilibrium between the net resultant force and the water level slope on a normal beach is then given by the following equation:

$$\frac{\partial S_{xx}}{\partial x} + \rho g (h + h') \frac{\partial h'}{\partial x} = 0 \quad (10)$$

Knowing the incoming wave parameters, the surface elevation $h'$ can be calculated. This must be done iteratively; every time $h'$ is changed, the wave parameters will change too.
Wave set-up and overwash on a submerged barrier island

We now consider a submerged barrier island with waves setting up the water level in front of the bar. The direction of the incoming waves is assumed to be perpendicular to the shore. This means that no refraction or diffraction occurs and by using a gentle slope that reflection can be neglected. The decay of the waves is assumed to be the result of two mechanisms: dissipation due to bottom friction and dissipation due to breaking. The effect of the wind is not incorporated here, neither is the effect of wave run-up. The contribution of these mechanisms to washover is shortly described in section 4.9.

Compared to the momentum equation in case of a "normal" wave set-up (equation 10), two extra terms are added to incorporate the influence of the current. These are a discharge (flux) term, and a friction term. The adjusted equation is now identical with the formulation of wave set-up and undertow by Phillips (1977) and Svendsen et al. (1987):

$$\frac{dS_{xx}}{dx} + \rho \frac{d}{dx} \left( \frac{Q^2}{h+h'} \right) + \rho g(h+h') \frac{dh'}{dx} + \tau_{cw} = 0$$

(11)

The first term in this equation is the driving force due to the waves, e.g. the change in radiation stress. This driving force is no longer counteracted by a slope in the water level only, but also by the change in flux relative to the depth (2nd term) and the bottom friction (4th term) (Figure 13). In section 4.2 was explained that the flux is constant for every x, and is equal to the depth averaged velocity times the local water depth: $Q = U(h+h')$. When $Q=0$, no overwash current occurs and the wave set-up can be calculated with equation 10. With a current equation 11 must be used.

Figure 13: Equilibrium of forces in a wave-induced overwash situation
In the occurrence of overwash, the influence of the current on the waves can be significant. Also the waves will have an influence on the current characteristics. In case of an overwash situation with waves propagating in the same direction as the current, the wave height will decrease, the wave celerity will increase and the wave length will increase, according to $L = c_r T$. This results in a smaller wave steepness ($H/L$). Due to the smaller wave height the waves will break later compared to the situation of waves without a current. The contrary is true when the current flows in the opposite direction of the waves. In that situation the wave steepness increase and the wave height increases; the waves break in an earlier stage.

For waves and current combined the wave celerity $c_r$ will change to $c_c$, given by:

$$c_r = c + U_{\parallel} = c + \frac{g T}{2\pi} \tanh \frac{2\pi (h + h')}{L} + \frac{Q}{(h + h')}$$ (12)

where $c$ : wave celerity [m/s]  
$c_c$ : wave celerity in case of waves and current [m/s]  
$U_{\parallel}$ : current velocity component in the direction of the waves [m/s]  
$T$ : wave period [s]  
$L$ : wave length ($c-T$) [m]  
$h' + h$ : total water depth [m]  
$Q$ : flux $[m^3/s/m]$  

Wave energy dissipation; irregular waves

For the simulation of waves in the surf zone in front of the bar, the model developed by Battjes & Janssen (1978) is used. This model describes natural, wind generated irregular waves with stochastically varying wave heights and periods over complex beach topographies with longshore bars. The basis for the statistical description of the wave height is the Rayleigh distribution. The local maximum wave height is determined by the criterium of Miche, which more or less yields that the maximum wave height $H_m$ is a constant times the local water depth. The fraction of the waves that are actually broken ($Q_b$) is given by the number of waves which, according to the Rayleigh distribution, would have been larger than $H_m$. The energy dissipation caused by breaking is described by the bore analogy, using $H_m$ as the height of the bore and taking only the fraction $Q_b$ of the waves that are broken into account. Together with dissipation of energy due to friction, the Rms wave height from deep water to the shore can be determined, using the energy balance.

In Appendix II the two dissipation terms due to breaking ($D_b$) and bottom friction ($D_f$) are defined. Also the theoretical solution of the fraction of breaking or broken waves is explained in that Appendix. Here shortly the probability of breaking and the energy balance are described.
Inside the surf zone, the dissipation of wave energy in the breaking process is dominant. Battjes & Janssen made a simplification of this process by assuming the dissipation rate per unit area $D_b$ equal to the fraction of breaking or broken waves ($Q_b$), multiplied by the dissipation rate of the breaking wave ($D_{bb}$):

$$D_b = Q_b D_{bb}$$  \hspace{1cm} (13)

From the Rayleigh distribution, the fraction of waves which at any point are breaking or broken, can be defined as:

$$Q_b = \exp\left[-\frac{1 - Q_b}{H_{rms}/H_m}^2\right]$$  \hspace{1cm} (14)

where

- $Q_b$: fraction of broken or breaking waves
- $H_{rms}$: root mean square wave height
- $H_m$: maximum breaker height

In very deep water, where $H_{rms}/H_m$ goes to zero, $Q_b$ also goes to zero. If the waves are shoaling, then the ratio $H_{rms}/H_m$ tends to increase, and so does $Q_b$. When the ratio becomes 1, all the waves are broken and the wave height becomes equal to $H_m$ (Figure 14).

![Figure 14: Fraction of breaking or broken waves](image-url)
The energy balance, defined in section 4.2, gives the relationship between the dissipation rate $D ( = D_b + D_f)$ and the Rms wave height. In this section was already explained that the combination of waves and current have an influence on the wave parameters. Considering the energy balance equation 4 in section 4.2, not only the energy $E$ and the wave group celerity $c_g$ change when waves approach the shore, also the wave angular velocity $\omega_r$ varies with the distance from the shore $x$.

The energy balance must then be written as:

$$\frac{\partial}{\partial x} \left( \frac{E(c_g + U)}{\omega_r} \right) + \frac{D_b + D_f}{\omega_r} = 0$$

(15)

where:

- $E$: wave energy [N/m$^1$]
- $c_g$: wave group celerity [m/s]
- $U$: depth averaged current velocity, parallel to wave direction [m/s]
- $\omega_r$: intrinsic wave angular velocity ($= 2\pi / T$) [1/s]
- $D_b$: dissipation rate due to breaking [Ns/m$^1$]
- $D_f$: dissipation due to wave bottom friction [Ns/m$^1$]

For a given depth profile $h(x)$, given incident wave parameters, bottom roughness, and a suitable choice of the model (breaking) parameters $\alpha$ and $\gamma$, equation 15 must be integrated, and solved simultaneously with the momentum equations for waves-induced overwash, to find $H^x$ and the surface elevation $h'(x)$. $H^x$ is defined in the wave energy as $E(x) = l/8\rho g H^x K^2(x)$.

**Bottom friction**

The purpose of this study is to compute the overwash velocity due to currents and waves in order to find the back shore sediment transport. In this light the influence of the combination of waves and currents on the depth averaged velocity, and thus on the sediment transport, must be described. This influence is described by a combined bottom shear stress for currents and waves. This bottom shear stress is used first to compute the overwash velocity and later to determine the sediment transport.

The combination of waves and currents can give a high rate of sediment transport, compared with currents only or waves only. In case of currents only, the average transport velocity is high but the amount of sediment in suspension is low. In case of waves only the contrary applies; the resulting transport velocity is small but the amount of suspended sediment is large. In case of waves and currents both positive factors are combined; the waves cause a large amount of suspended sediment which the current transports.

In this study, the approach of Bijker (1971) is used. Bijker introduced the wave influence via a modification of the bottom shear stress in an existing sediment transport formula for
Sediment loss due to washover on a barrier island currents. His hypothesis implies that the waves contribute primarily to the stirring up of material from the bottom rather than to the transport. It must be noticed that the Bijker formula originally was defined for currents flowing perpendicular to the wave direction. Here the current is in the same/opposite direction as the waves. The sediment transport rates computed later on in this report are therefore conservative. In reality the waves also contribute to the cross-shore sediment transport, resulting in higher transport rates, while in this case the waves only stir up the sediment. Nevertheless the Bijker approach is used because it is a well known and often used method for computing the sediment transport.

In Appendix IV the solution of the combined bottom shear stress is described. Here the general equation of the average total bottom shear stress is given (see also Figure 15):

$$\bar{\tau}_{cw} = \tau_c + \frac{1}{2} \tau_{w,\text{max}}$$

where

- $\tau_{cw}$: combined bottom shear stress for currents + waves (time averaged) [N/m$^2$]
- $\tau_c$: bottom shear stress for currents only [N/m$^2$]
- $\tau_{w,\text{max}}$: bottom shear stress due to the max. orbital wave velocity [N/m$^2$]

Figure 15: Combined bottom shear stress for currents and waves
The final equation, representing the overwash surface elevation and overwash flux caused by the waves, is:

$$\frac{dh'}{dx_{\text{wave}}} = \frac{1}{\rho g (h+h')} \cdot \left( -\frac{\rho Q^2}{(h+h')^2} \cdot \frac{dh}{dx} - \frac{dS_{sx}}{dx} - \tau_{cw} \right)$$

Figure 19 on page 35 shows an overwash situation due to a 2.5m wave coming from the right. The downstream boundary level is held equal to the upstream boundary level in deep water. The barrier profile is identical to the one used in the tide-only case. The only driving force for the current in this case is the wave action. The surface elevation is relative to SWL.
4.5 Combined tide and wave induced overwash

For the combination of tide and waves two situations can be considered:

1) The waves propagate in the direction of the tide-induced overwash current, causing an additional water level set-up due to breaking in front of the bar. This results in an increase in overwash velocity compared to the tide-only situation (Figure 16).

2) The waves approach the bar in the opposite direction of the tide-induced current. This situation is plotted in Figure 17: the waves approach the bar from the right and the tidal elevation at the left side is higher than the elevation at the front side. In this situation the tide-induced water level slope is partly or completely counteracted by the slope generated by the wave set-up from the other (right) side. In which direction the current will flow depends on the combined (tide + wave) water level slope across the bar. In extreme situations, with high waves combined with a small tidal water level difference, the current will flow from the side where the waves approach the bar to the back side (Figure 17). This situation can occur if the water depth at the side where the waves approach is not too shallow. In case of a large intertidal shallow water area in front of the bar with a very gentle bottom slope the waves will break further away from the bar and over a relative large area, resulting in a smaller set-up.

It can be noticed that in case the waves approach the bar in the same direction of the tide, the overwash velocities will be increased. Waves approaching in the opposite direction will decrease the velocities. This directly has an effect on the amount of sediment transported by the overflowing current. The worst case scenario with the largest sand losses will occur when waves and tide approach the bar from the same direction. In this situation the waves contribute to the sediment transport by stirring up material from the bottom and by generating a water level slope, which increases the overwash current velocities.

Figure 16: Water surface elevation and slope of tide- and waves from same direction
Differential equation of tide and wave-induced overwash

For the computation of the combined tide and wave-induced overwash, the equation (17) can be applied again. In this equation the friction term is valid for both waves and currents.

\[
\frac{dh'}{dx_{\text{tide-waves}}} = \frac{1}{\rho g (h + h')} - \frac{\rho Q^2}{(h + h')^2} \cdot \left( \frac{\rho Q^2}{(h + h')^2} \cdot \frac{dh}{dx} - \frac{dS_{xx}}{dx} \right) - \frac{\tau_{cw}}{\rho g (h + h')^2} \quad (18)
\]

The driving force of the waves is given by the radiation stress term \(dS_{xx}/dx\). The combined tide and wave influence is represented by the pressure gradient term and the inertia term. The tide-induced water level difference can be specified by increasing or lowering the upstream or downstream boundary level conditions (\(h'_{\text{front}}\) and \(h'_{\text{back}}\), similar to the tide-only case). At the upstream or downstream boundary condition a deep water wave height and period can then be specified (similar to the waves-only case). After calculating the wave decay and set-up across the bar, the resulting water level elevations and velocities are found.

To illustrate a combined tide and wave-induced washover, in Figure 20 on page 35 the velocity variation and the water surface elevation of such washover is plotted. The upstream boundary level is now equal to the tide-only case and the upstream wave condition is equal to the wave-only situation (\(dz = 0.20\text{m}, H_s = 2.5\text{m}, T_p = 7.8\text{sec}\)). The barrier profile is identical to that of the tide-only and wave-only case.
Sediment loss due to washover on a barrier island

Tide-induced overwash: elevation and velocity
\[ dz = 0.20 \text{m}; r = 0.02 \text{m}; m = 1:25; h_{\text{crest}} = 0.3 \text{m}; Q = 0.28 \text{ m}^3/\text{s} \]

Wave-induced overwash: elevation and velocity
\[ H_s = 2.5 \text{m}; T_p = 7.8 \text{ s}; r = 0.02 \text{m}; m = 1:25; h_{\text{crest}} = 0.3 \text{m}; Q = 0.33 \text{ m}^3/\text{s} \]

Tide & wave-induced overwash: elev. and velocity
\[ H_s = 2.5 \text{m}; T_p = 7.8 \text{ s}; dz = 0.20 \text{m}; r = 0.02 \text{m}; m = 1:25; h_{\text{crest}} = 0.3 \text{m}; Q = 0.62 \text{ m}^3/\text{s} \]
4.6 Boundary conditions

To solve the overwash velocities and elevations with the equations specified in section 4.2 boundary conditions are needed. Besides the given deep water wave parameters $H_s$ and $T_p$, the bathymetry and the bed roughness, two types of boundary conditions can be specified: (a) water levels at both the model boundaries and (b) a constant flux $Q$ across the bar combined with a water level boundary at one side of the model. With a known wave set-up and tidal elevation at the front side, the overwash current is determined by the unknown downstream boundary condition. When the downstream water level is lower than the water level in front of the bar, a current over the bar will occur.

Considering the overwash equation due to tide only (equation 8) it can be seen that $h'$ is present in both the left hand and the right hand side. This means that, for a given $Q$, $h'$ must be solved by iteration (explained in Chapter 5). Besides a constant value for $Q$, also an initial value for the surface elevation in front of the bar and in deep sea ($h'_0$) must be specified. For a tide-only and a waves+tide-situation this initial elevation is equal to the tidal elevation outside the surf zone. For a waves-only-situation the initial water level is SWL ($h'=0$).

The overwash equations due to tide, waves, and tide+waves, must be solved with an upstream and a downstream boundary condition. Two extreme situations can occur:

1. The water level just in front of the crest is equal to water level at the back side. In the ideal 1-D situation, where the water can flow only along the x-axis across the bar, no current will flow over the bar ($Q=0$). In the tide-only case, this situation occurs when no water level difference across the bar is induced by shallow water effects or phase shift of the tidal wave.

In case of a waves-only-situation, no water will flow over the crest when the wave set-up remains constant across and behind the barrier, caused by an emerged beach at the downstream side of the barrier. The water remains in a storage area (back barrier lagoon) and the wave set-up can be calculated with the classic wave set-up equation 10.

This is again an ideal 1-D situation. In reality (2-D situation) the wave-induced level elevation will not be maintained behind the bar due to a varying crest elevation in the length direction of the bar. These locally deeper areas along the crest and in the tidal inlets cause a change in the water level along the crest of the bar: low water levels in the inlets and higher water levels in the "blocked" areas, shown in Figure 21. The wave breaking in front of the bar will induce a wave set-up, causing an increase in water level inshore of the bar ($h'_1$). However, the wave breaking is less intensive in the tidal channels (tidal inlets) due to the larger depth ($h'_2$) and because wave refraction may concentrate the wave energy on the bars at the sides of the channel. The shoreward decrease of the radiation stress and a pressure of a constant water level in the trough inshore of the bar cannot be obtained at all the cross sections along
the coast. An offshore-directed flow is therefore driven out of the channels due to the wave set-up behind the bar. The out-going water flux $\Delta Q$ is compensated by an onshore net flow across the bar.

$h'_1 > h'_2 > h'_3$

Figure 21: Wave set-up behind the bar is not maintained due to circulation flow.

Figure 22 shows the surface elevation of an overwashed barrier island due to tide and waves, which approach the bar from the right, for $Q=0$ to $Q=0.69$ m$^3$/s/m (with $h_{crea} = -0.40m$ SWL, $W_{crea} = 200m$, $r=0.02m$, $m_{fro}=1:50$, $dz=0.10m$, $H_s = 2.5m$, $T_p=7.8s$). The solid line is the surface elevation in the situation with no overwash flow ($Q=0$). In Figure 23, where the overwash current velocities are plotted for the different conditions with varying fluxes, the solid line is equal to the zero-line: $U=0$. 

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Figure 22 and 23 show the surface elevations and velocities computed with the 1-D
overwash model, which is specified later in this report.

2. The water level behind the bar is lower then the water level at the front side. This
situation occurs when the geometry at the lee-side of the barrier allows the water to
flow away (no storage area, no emerged mainland beach). An important aspect when
determining the overwash current is at what location at the back side of the bar the
specified downstream boundary level $h_{\text{back}}$ is equal to the calculated one ($h_{\text{calc}}$). This
location controls the overwash current velocity but is initially unknown. However,
this location can be determined when the surface elevations are computed with the
overwash equations found in section 4.5 (1-D overwash model).

Another important aspect is the influence the back side water level $h_{\text{back}}$ has on the
wave set-up at the front side of the bar. It can be seen in Figure 22 that the
downstream water level does have an effect on the wave set-up at the front side of the
bar. A decreased water level at the back side of the bar causes lower water levels on
the crest and an increase in the overwash current velocities.

Each downstream water level behind the bar is related to one specific $Q$. When the
downstream water level has no influence on the upstream water levels the bar can act
as a weir. When $Q$ increases, the water level drops at the lee-side, which is shown
in Figure 22 ($Q=0.65$ and 0.69 m$^3$/s/m) and the velocities increase (Figure 23,
$Q=0.65$ and 0.69 m$^3$/s/m). In case the sandbar act as a weir the flow at the end of
the crest can become super critical, which means that the maximum current velocity
is determined by $U=\sqrt{gh}$. The water depth in the area of the critical flow is
determined by $(h+h')_{\text{crit}} = (Q^2/g)^{1/3}$.

In case of waves-only the initial water level outside the surf zone does not have to be
higher than the water level at the back side to cause an overwash current. The water
level difference is then caused by the wave set-up just in front of the crest. In a tide-
only situation a water level difference must be present across the sandbar, caused by
a difference in tidal variation at both sides of the bar. A combined tide- and wave
induced overwash is shown in the Figures 22 and 23. Both the wave set-up and the
tidal water level difference contribute to the total water level difference across the
bar.
Surface elevation with increasing flux $Q$

Overwash due to tide and waves: $dz=0.10\text{m}$; $H_s=2.5\text{m}$; $T_p=7.8\text{s}$; $m=1:50$

Figure 22
Overwash current velocity with increasing flux $Q$

Overwash due to tide and waves: $dz=0.10\,\text{m}$; $H_s=2.5\,\text{m}$; $T_p=7.8\,\text{s}$; $m=1:50$

- $Q=0\,\text{m}^3/\text{s/m}$
- $Q=0.50\,\text{m}^3/\text{s/m}$
- $Q=0.58\,\text{m}^3/\text{s/m}$
- $Q=0.65\,\text{m}^3/\text{s/m}$
- $Q=0.69\,\text{m}^3/\text{s/m}$

Figure 23
For given tide and wave conditions, bottom profile, bottom roughness and crest height three different combinations of boundary conditions can be used in order to determine the water surface elevation and the velocity profile across the bar. These combinations are:

- **level-level**: By specifying an upstream and a downstream water level the washover discharge Q can be found by iteration.
- **level-flux**: The upstream boundary is a water level and the downstream boundary a flux. The water surface elevation h' can be calculated by iteration.
- **flux-level**: With a given discharge Q and a fixed water level behind the bar the surface elevation in deep water in front of the bar can be found by trial and error.

The effects of the boundary conditions on the results are examined in the sensitivity study described in section 5.5, as well as the influence of wave parameters, crest height and width, bottom roughness and geometry of the bar.

### 4.7 Used sediment transport formula

The sediment transport caused by the overwash current and waves is calculated with the transport formula of Bijker (1971). As pointed out in section 4.4, the Bijker approach is based on the hypothesis that the waves contribute primarily to the stirring up of material from the bottom, which will be transported by a current. In reality the waves also contribute to the cross-shore sediment transport due to their asymmetry: the wave crest amplitude is higher than the wave trough amplitude and the wave crest is shorter (in duration) than the wave trough. This results in higher transport rates. This effect is neglected and in this case the waves only contribute to the stirring up of the sediment. The sediment transport rates computed later on in this report are therefore a bit conservative. Nevertheless the Bijker approach is used because it is a stable, well known and often used method for computing sediment transports.

In the Bijker formula, the total transport is divided in bed-load transport and transport into suspension. The two formulas are defined in section 5.3: the computation method of the 1-D model.

### 4.8 Other coastal processes and features contributing to overwash

Besides the influence of the tide and the waves (wave set-up), other hydrodynamic mechanisms contribute to overwash, such as wave run-up and overtopping, and wind set-up. The effect of these mechanisms are shortly described in this section. The influence of environmental aspects on washovers as tidal inlets across the bar, sea level rise and settlements are described in the literature study "Washover transport on Sandbars".
Wave run-up & overtopping

Overtopping is the amount of water that is pushed over the top of the bar and is caused by waves which run-up the slope. Overtopping occurs when the bar is emerged. However, overtopping does not take place when the crest height is too far above the SWL line. Whether or not overtopping occurs depends mainly on the run-up height \( R \). This run-up height is determined by the incoming wave characteristics and the beach slope (Irribarren, 1972).

\[
\frac{R}{H} = \left( \frac{\tan \alpha_s}{(H/L_0)^{1/3}} \right) = \xi_0
\]

where:
- \( H \) : incoming wave height [m]
- \( L_0 \) : deep water wave length [m]
- \( \tan \alpha_s \) : bottom slope [-]
- \( \xi_0 \) : Irribarren breaker parameter [-]

The contribution of wave run-up to the cross-shore flow is determined by this run-up height \( R \), the wave height \( H \), the crest height \( h_c \) above MSL, and the width of the crest \( W_c \) (Figure 24). When the run-up height exceeds the crest height, overtopping takes place, pushing water over the crest of the bar. With relative small crest widths (say ± 20m) the amount of water flowing over the crest due to overtopping can be large enough to induce a current that reaches the back side of the barrier. In that case overtopping becomes overwashing.

Figure 24: Definition sketch of wave run-up and overtopping

Overtopping occurs discontinuously as a function of the individual wave that exceeds a certain height. The crest height range on which overtopping can occur can be determined according to the following example:

A barrier island with a steepness of the shore between \( \tan \alpha_s = 0.01 \) and \( \tan \alpha_s = 0.1 \) is approached by waves with a maximum height of 2 m and a period of 6 seconds in
Sediment loss due to washover on a barrier island

A depth of 20 m. The Irribarren breaker parameters are determined at respectively 0.053 and 0.530 (between plunging and spilling). The run-up heights, relative to SWL, are now respectively 0.11m and 1.10m. With a 3m wave the run-up heights become 0.16m and 1.64m respectively.

It can be considered that the steepness of the shore of barrier islands is in that range between 1:10 to 1:100. Knowing the steepness and assuming that the maximum wave height is about 3m, it can be concluded that

\[
\text{overtopping occurs if } \Rightarrow 0.0 \, \text{m} < h_{\text{crest}} < +1.5 \, \text{m (SWL)}.
\]

The crest height on which wave-induced overwash can occur is roughly determined by computing the maximum wave set-up according to the linear wave theory. With the assumption of a maximum breaker height of 3m and a breaker index of $\gamma=0.8$, the maximum wave set-up can be determined by

\[
h_{\text{max}}' \sim \frac{5}{16} \, \gamma \, H_b \tag{20}
\]

The maximum wave set-up in this case is $h_{\text{max}}' \approx 0.60$ m. When it is assumed that no wave set-up will be generated when the crest is 4m below SWL and that the minimum depth of the overwash current on the crest is about 0.20m, it can be concluded that

\[
\text{wave-induced overwash occurs if } \Rightarrow -4.0 \, \text{m} < h_{\text{crest}} < +0.4 \, \text{m (SWL)}.
\]

It can be seen that the crest height on which overtopping can occur must be above SWL with a maximum crest height of about +1.5m while wave-induced overwash occurs mainly on crests which are below SWL. Due to the fact that overtopping occurs on crest heights above the water surface the contribution of overtopping to the sand losses is small relative to the wave-induced washover, which mainly occurs when the bar is submerged. Another important aspect is the amount of water that flows over the top of the bar due to overwash or overtopping. Due to the discontinue character of overtopping, caused by separate waves, much less water is pushed over the top than in case of wave-induced washover, which is a more or less stationary current which can flow over the crest for a few hours.

The width of the crest determines the amount of sand that will be lost. With a very small crest (10-20 meters) the overtopping waves are capable to transport material to the back side of the bar. However when the crest width is several hundreds of meters the overtopping waves cannot create a flow to the back side. They will influence the shape of the crest locally by swashing sand away from the front of the crest to behind laying parts. Actually no sand will be lost, it is just redistributed over the crest.

In general it can be concluded that the contribution of overtopping to the cross-shore transport over the crest of the bar is small compared to washover transport.

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Wind set-up

Wind set-up has the same effect on the washover process as wave set-up. This mechanism can be regarded as an extra water level elevation at the front side of a barrier, added to the varying water levels due to tide and waves. The wind-induced elevation is a result of the wind, blowing over open waters and seas, and pushing up the water against the shore. Abnormal rises in water level due to heavy storms in near shore regions will not only flood low-lying terrain, but provide a base on which high waves can attack the upper part of a beach or barrier and penetrate farther inland.

Compared to wave overtopping the contribution of wind set-up to overwash is about the same as the contribution of wave set-up to overwash. Wind set-up can range from centimetres to several meters, depending on the wind speed and the fetch.

Sea level rise and subsidence

The effect of sea level rise and subsidence on the behaviour of barrier islands is discussed in the literature study "Washover transport on sandbars", and will not be described here.
CHAPTER 5

1-D overwash model
5 1-D Overwash model

5.1 Introduction

In this Chapter a one-dimensional mathematical hydrodynamic/morphodynamic overwash model is described. The reason for developing such a model is that existing surf zone models only concentrate on wave phenomena and cross-shore currents on a beach, where at the lee-side of the bar a mainland beach is located, which maintains the wave and tide induced water level set-up. Most of the models which can simulate overwash currents are not able to compute the wave-influence on the water levels and currents. Computer models which have the ability to compute the wave conditions in the surf zone are not able to compute wave and tide-induced cross-shore overwash currents. The model described in this chapter, however, is relative small and computes the effect of waves and tide on the washover current and on the sediment transport rate directly. This offers the capability to analyze the development of the profile of a barrier island, affected by currents and waves, over a given time period.

In the first section of the described model, overwash velocities, water surface elevations and wave heights are computed. The second part of the model calculates the sediment transport. The bed profile changes, as a result of the calculated accretion and erosion, are used to adjust the initial bottom profile. With the adjusted, new bathymetry the model computes the flow field and wave heights again. This explicit computation method can be repeated many times to simulate the profile development, providing a clear view of the consequences of overwash on barrier islands.

In this Chapter the development and some results of the overwash model will be described. First the lay-out of the model and the model set-up is discussed in section 5.2. Then in section 5.3 the computation method is explained. Different flow diagrams are used to describe the computation process.

In section 5.4 a verification of the model is given. Comparisons are made between the 1-D overwash model results and wave set-up and wave height measurements from laboratory tests performed by others.

The 1-D overwash model must be able to handle with various overwash conditions and barrier profiles. Therefore a sensitivity study of the model is produced. Another reason for producing a sensitivity study is to investigate the influences of several parameters on the washover process and the sediment transport. The important parameters varied in this study are the wave height and period, bottom slope, crest height, crest width and bed roughness.
5.2 Model set-up

The barrier profile is simplified by assuming a relative steep shoreface in front of the bar, an almost horizontal crest and a gentle bottom slope behind the bar, with uniform contour lines. The orientation of the sandbar is such that the waves and the tide approach the bar at the steeper front side perpendicular to the crest. At the lee-side of the bar no mainland beach or backbarrier lagoon is present; the water can flow undisturbed over the crest to the other side of the bar and is not affected by an emerged beach behind the bar.

The cross sectional profile is roughly divided in three parts (Figure 25): the front/shoreface (m1), the crest (m2) and the backside (m3) of the bar. The highest part of the bar (crest), is assumed to be horizontal, or has a small negative slope. The back side of the bar has a steeper slope than the crest, but smaller than the slope at the front-side of the bar, where the effect of breaking waves is significant. This shape is chosen after examination of cross sections of various barrier islands in the literature study “Washover Transport on Sandbars”.

Figure 25: Schematized barrier profile and orientation
Initially all the three slopes are held constant. The origin of the x-axis is located where the seaward bottom slope meets the slope of the crest of the bar (m1 = m2), and is positive in the seaward direction. These three sections have parallel contour lines with no irregularities. The crest height is assumed to be equal or smaller than SWL, to be sure that overwash takes places. Also a crest above the water surface can be modeled to compute the wave set-up and wave height. In that case however no overwash current occurs.

**Initial conditions**

The initial conditions that must be specified are:

- wave parameters in deep water: \( H_{rms,0} \) and \( T_p \)
- discharge: \( Q \)
- still water level in front of the bar: \( h_0' = \text{SWL}_{front} \)
- breaking parameters: \( \alpha \) and \( \gamma \)
- barrier profile parameters: \( h_{crest}, \Delta x, m_1, m_2, \) and \( m_3 \)
- roughness parameter: \( \kappa \)

**Barrier profile**

To set-up the model, the barrier profile must be specified first. This does not need to be a trapezium shape but can be every desired shape. The depth at every grid point \( h \) is specified relative to mean sea level (MSL) and is positive downwards. The change of the water surface relative to MSL (surface elevation \( h' \)) is defined positive upwards. The total water depth is the initial depth increased with the surface elevation due to tide and/or waves (\( h+h' \)).

The bottom profile is set-up as follows. For every grid point \( i \) a bottom slope \( m_i \) and the distance to the previous water depth \( \Delta x \) must be specified. The new water depth is then calculated relative to the previous depth (Figure 26), according to the following formula:

\[
  h_i = h_{i-1} - m_{i-1} \cdot \Delta x
\]

where:
- \( h_i \) : water depth at \( x_i \) relative to MSL [m]
- \( h_{i-1} \) : water depth at \( x_{i-1} \) relative to MSL [m]
- \( m_{i-1} \) : bottom slope at \( x_{i-1} \) \( [\Delta h/\Delta x]_{i-1} \) [-]
- \( \Delta x \) : grid spacing [m]
Grid spacing

The grid size $\Delta x$ cannot be chosen too large, otherwise the changes in the bottom profile cannot be followed accurate enough. Also in areas where the wave and flow parameters show large gradients the grid size must be chosen small enough to be able to follow the variations in these parameters. Another reason for choosing a small grid size is to prevent instability problems in the model during the computation. On the other hand, a very small grid size gives large spreadsheets, which can give problems in terms of memory shortage and the computation will become slow.

To investigate the effect of the grid size on the model results, a tide and wave-induced overwash situation is modelled using four different grid sizes: $\Delta x=5m$, $\Delta x=10m$, $\Delta x=20m$ and $\Delta x=40m$. The modelled tide and wave conditions are: $dz=0.10$, $H_s=2m$, $T_p=6s$, $m=1:25$, $r=0.02m$, $W_c=200m$ and $h_{crest}=-0.40m$ MSL. The surface elevations, overwash current velocities and wave height variations across the barrier, computed with the four different grid spacings, are plotted in the Figures 27a, 27b and 27c. Figures 27a and 27b show the surface elevation and the current velocity respectively. It can be observed that the 20m and 40m grid sizes are too large to compute the elevation and velocity variations correct, compared with the 5m and 10m grid computations. Figure 27c shows that for the modelling of the wave parameters in the surf zone the grid size must not be chosen larger than 10m. In the areas where the gradients in water surface elevation, overwash current velocity and wave height are not very large, the 20m and 40m grid sizes are small enough to obtain good model results. Thus, to prevent large spreadsheets and slow computation speeds, the grid size must be varied along the x-axis. In that case small grid sizes should be used in the surf zone and on the crest and larger grid sizes outside these areas.
Sediment loss due to washover on a barrier island

Variable grid spacing: Surface elevation

Variable grid spacing: Current velocity

Variable grid spacing: Wave height
**Bottom friction**

The bottom friction for current only is specified for every grid point by the Chezy formula:

\[ C_i = 18 \log \left( \frac{12(h + h')}{r} \right) \rightarrow \tau_{c,i} = \frac{\rho g Q |Q|}{C_i^2 (h + h')_i} \]  \hspace{1cm} (22)

The bottom friction for waves and current is specified by the Chezy friction and the wave friction factor \( f_w \):

\[ \tau_{cw,i} = \tau_{c,i} + \frac{1}{2} \tau_{w,\text{max},i} \]  \hspace{1cm} (23)

\[ \tau_{w,\text{max},i} = \frac{1}{2} \rho f_w i_0^2 \]  \hspace{1cm} (24)

### 5.3 Computation method

Solving the equations in Chapter 4 in an analytical way is very complicated and therefore not executed. The water level slope, the velocities and the resulting transports are solved numerically by iteration.

The calculation starts at the side from where the waves approach (right side of Figure 26). In that starting point the initial wave parameters and the initial surface elevation are defined. The model calculates per grid point (i) the rms wave height \( H_{\text{rms},i} \), the water surface elevation \( h' \), the depth averaged velocity \( U_i \) and the sediment transport \( S_{\text{tot},i} \) relative to the former grid point (i-1). Figure 28 shows the computation sequence from the computation of the above mentioned parameters from grid point to grid point. Here the most important parameters of this scheme are declared.

### Wave length

After the bottom profile and the wave parameters are specified the wave length is computed with the adjusted dispersion relation for currents and waves (equation 12 in section 4.3). In this equation the velocity \( U_i \) is the unknown variable. The initial value for \( U_i \) is \( U_0 = 0 \), specified at a location far enough from the barrier crest where the overwash current is not influenced by this boundary condition. With the given initial conditions the wave length is found by the iteration steps shown in Table 1.
Table 1: Determination wave length

<table>
<thead>
<tr>
<th>guess $L_{i,\text{guess}}$</th>
<th>Repeat until $L_{i,\text{guess}} = L_{i,\text{calc}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_i = 2\pi/L_{i,\text{guess}}$</td>
<td>$\tanh[k_i (h_i + \gamma)]$</td>
</tr>
<tr>
<td>$L_{i,\text{calc}}$ with equation 12</td>
<td></td>
</tr>
</tbody>
</table>

$\begin{array}{c|c}
\text{YES} & \text{NO} \\
\hline
\text{stop} & L_{i,\text{guess}} = 0.5(L_{i,\text{guess}} + L_{i,\text{calc}}) \\
\end{array}$

4 Energy dissipation

Knowing the wave length $L_i$, the maximum wave height can be calculated with equation 19 in Appendix II, which is based on Miche's criterion for the maximum height of period waves of constant form. After the squared ratio between $H_{rms,i}$ and $H_{m,i}$ is determined, $Q_{b,i}$ can be solved by iteration according to equation 14 in section 4.4. The same iteration method as for the solution of $L_i$ is used here and $Q_{b,calc,i}$ is computed with equation 14. Knowing $Q_{b,i}$ the dissipation of wave energy due to breaking ($D_{b,i}$) can be determined, explained in section 4.4 and Appendix II. In that same Appendix also the dissipation due to bottom friction is defined. The new wave energy is now calculated relative to the wave energy of the previous grid point:

$$E_i = \frac{\omega_{r,i}}{(c_g + U)_i} \left[ \frac{E_{i-1}(c_g + U)_{i-1}}{\omega_{r,i}} - D_{tot,i} \Delta x \right]$$

(25)
Figure 28: Computation sequence for wave-induced and wave+tide-induced overwash
Gradient in radiation stress

With the new calculated wave energy $E_i$, the rms wave height can be determined with:

$$H_{rms,l} = \sqrt{\frac{8E_i}{\rho g}}$$  \hspace{1cm} (26)

and the gradient in radiation stress is defined by:

$$\frac{\Delta S_{xx}}{\Delta x}_l = \frac{E_i(2n_i - \frac{1}{2}) - E_{i-1}(2n_{i-1} - \frac{1}{2})}{\Delta x}$$  \hspace{1cm} (27)

Water surface elevation

The new surface elevation $h'_i$ is calculated relative to the old elevation $h'_{i,1}$, according to the following expressions (for $i=1,2$; see Figure 29):

$$h_2 + h'_2 + m_1 \cdot \Delta x = h_1 + h'_1 + \left(\frac{dh'_1}{dx}\right)_1 \cdot \Delta x$$  \hspace{1cm} (28)

$$h_2 = h_1 - m_1 \cdot \Delta x$$  \hspace{1cm} (29)

The surface elevation is a new grid point $i$ can be found by substituting (29) in (28):

$$h'_2 = h'_1 + \left(\frac{dh'_1}{dx}\right)_1 \cdot \Delta x$$  \hspace{1cm} (30)
Sediment loss due to washover on a barrier island

The gradient in the water surface slope ($\Delta h'/\Delta x$), is calculated with the overwash equations defined in section 4.2. The exact $h'_i$ must be found by iteration per individual grid point. This iteration procedure is described on page 50 in this section.

* Washover current velocity

In Chapter 2 is explained that the flux $Q$ is considered constant for every $x_i$. Once the total depth per grid point is calculated and the flux $Q$ is known (by trail and error), the velocity can be found by dividing the flux by the total depth including the wave set-up:

$$U_i = \frac{Q}{(h_i + h'_i)}$$  \hspace{1cm} (31)

At the end of the crest, where in special circumstances the flow can become critical, the maximum flow velocity is determined by

$$U_{\text{crit},i} = \sqrt{g (h_i + h'_i)}$$  \hspace{1cm} (32)
This critical velocity is related to a specific depth. Knowing the constant flux $Q$, this total depth, or the surface elevation of the critical flow can then be calculated as:

$$h_i' = \left(\frac{Q^2}{g}\right)^{1/3} - h_i$$  \hspace{1cm} (33)

**Sediment transport**

The sediment transport is given by the following formulations:

Bed-load transport:

$$S_{b,i} = 5D_{50} U_i \sqrt{g} \exp\left[-\frac{0.27 (\rho_s - \rho) D_{50} g}{\mu_i \tau_{cw, i}}\right]$$  \hspace{1cm} (34)

where:

- $S_{b,i}$: bed load transport at point $x_i$ [m$^3$/s/m]
- $D_{50}$: mean sediment grain size [m]
- $U_i$: depth mean overwash velocity at $x_i$ [m/s]
- $C_i$: Chezy friction factor at $x_i$ [m$^{1/2}$/s]
- $\mu_i$: ripple factor at $x_i$ [-]
- $g$: acceleration due to gravity [m/s$^2$]
- $-0.27$: experimental coefficient [-]
- $\rho_s$: mass density of sediment [kg/m$^3$]
- $\rho$: mass density of water [kg/m$^3$]
- $\tau_{cw,i}$: combined wave and current bottom shear stress at $x_i$ [N/m$^2$]

The ripple factor is defined by:

$$\mu_i = \frac{\tau_{grain}}{\tau_c} = \left(\frac{C_i}{C_{90,i}}\right)^{3/2}$$  \hspace{1cm} (35)

where:

- $\tau_{grain}$: shear stress on a grain [N/m$^2$]
- $\tau_c$: total bottom shear stress (current only) [N/m$^2$]
Sediment loss due to washover on a barrier island

\[ C_{90,i} = 18 \log \left( \frac{12(h_i + h_i')}{D_{90}} \right) \]  

(36)

where \( D_{90} \) : 90% sediment grain size [m]

Suspended load transport:

\[ S_{s,i} = 1.83 \cdot S_{s,i} \left[ I_{1,i} \ln \left( \frac{3}{r} \right) + I_{2,i} \right] \]  

(37)

where \( S_{s,i} \): amount of sediment in suspension transported at \( x_i \) [m³/s/m]

\( S_{b,j} \): amount of bed-load sediment transported at \( x_j \) [m³/s/m]

\( I_{1,i} \) and \( I_{2,i} \): Einstein integrals, defined at \( x_i \) [-]

The Einstein integrals are numerically solved in Appendix VI.

When on every grid point the sediment transport is calculated, the amount of erosion and sedimentation can be calculated according to the next equations, which describe the change in the water depth (bottom height) due to the gradient in sediment transport:

\[ \frac{\Delta h_i}{\Delta t} = \frac{\Delta S_{ot,i}}{\Delta x} \]  

(38)

\[ h_{new,i} = \left( \frac{\Delta h_i}{\Delta t} \right) \cdot \Delta t + h_{old,i} \]  

(39)

where \( h_{old,i} \): old water depth at point \( i \) [m]

\( h_{new,i} \): new water depth at point \( i \) [m]

\( \Delta t \): time interval [s]

\( S_{ot,x} \): sediment transport rate in x-direction [m³/s/m]

\( \Delta x \): distance over which \( S_{ot,x} \) changes (\( x_i - x_{i+1} \)) [m]

**Computation procedure**

Figure 30 on page 59 shows a scheme of the total computation procedure, with the iteration steps, from the definition of the bottom profile and the initial conditions, via the computation of the overwash current and sediment transport, to the change in the bottom profile due to gradients in sediment transport.
The computation starts at the top of the scheme by specifying initial conditions. These conditions are given at the upstream model boundary. The next step is to put in the bottom profile, defined by a water depth h (below SWL) at point i. Before the computation can be started an initial value for the flux Q must be specified. The best value for Q to start with the computation safely is Q=0 (no overwash, only wave set-up calculated). In the following step i is set to zero again. This step is not necessary when the first time the computation is started, but must be done every time Q is changed and the computation starts again. In the next step i is increased with 1 (next grid point), to determine the new surface elevation h'. However, before the new elevation can be calculated, h' must have a value, otherwise in the first iteration step no comparison between the in- and output elevation h' in and h' out can be made.

To find the surface elevation in the new grid point in case of a wave- or tide-induced overwash situation the wave height H s, the change in radiation stress ΔS, and the water level slope Δh'/Δx, must be calculated. Once a value for h' is computed, a comparison between the input and output value is made. This comparison is necessary because in the overwash equations defined in Chapter 4 the surface elevation h' is present at both sides. Therefore h' can only be solved by iteration. This iteration is stopped when the input elevation and output elevation are equal to each other and i is increased by 1, provided that i is not i max (downstream model boundary). This procedure is repeated until the surface elevation at the last grid point (i max) is calculated.

In this stage of the computation at every grid point the elevation h' is known. Now must be checked whether or not the elevation at the last grid point is equal to the specified downstream boundary elevation h' bound. If this is not the case (downstream boundary level is lower than the upstream (initial) boundary level) then the flux Q must be increased by ΔQ. A flow from the front to the back side is now generated, yielding that Q must have a positive value (Q must be increased by a positive ΔQ). If the reverse situation occurs, ΔQ must have a negative value. The magnitude of ΔQ depends on the difference between the computed h' max and the specified boundary level h' bound and the water depth on the crest. A large water level difference and/or a large crest depth means that ΔQ must be chosen in the order of 1, while a small difference and small crest depth requires a small ΔQ (order of 0.01) to come to a solution.

When Q is changed the model must be run again, beginning at the first grid point (i becomes 0). The runs are continued until the calculated downstream surface elevation is equal to the specified downstream elevation. In this stage of the computation at every grid point the wave parameters, surface elevation and the velocities are known. Now the sediment transport and the new bottom height can be computed. To examine the effects of the changed bottom profile on the overwash process, the new bottom can be put back into the calculation by overwriting the old bottom profile for every grid point. This can be repeated as often as wanted to study the development of the cross-shore barrier profile due various overwash events.
Sediment loss due to washover on a barrier island

Figure 30: Calculation procedure for overwash model ("Tide+Waves" situation)
5.4 Verification of the overwash model

5.4.1 Introduction

The capability of the model for the prediction of washover flow and wave set-up can be checked by comparing the results with measurements. Unfortunately no direct measurements are found of overwash discharges flowing over barrier islands. However, the validity of the model can still be checked by comparing the results with wave set-up and wave height measurements and by comparing the wave results with the results of a 1-D surf zone model. An accurate prediction of these parameters will provide accurate overwash currents, for the overwash current is depending on the wave height, the water level slope and the downstream boundary.

The wave set-up and wave height measurements are obtained from two studies, performed by Battjes & Janssen (1978), and Thorkilsen et al (1991), with measurements of irregular waves and wave set-up on beaches with a submerged longshore bar in front. The 1-D surf zone model Unibest_LT is used to compare the numerically determined wave height and wave set-up of the 1-D overwash model with another numerical model. By comparing these model results the differences in the used computation method (schemes) can be determined. Unibest_LT is not designed to compute cross-shore current velocities. Therefore only the wave parameters are compared with the wave parameters of the overwash model.

To verify the overwash current velocities of the 1-D overwash model a comparison is made with the cross sectional flow results of a 2-D flow model (MIKE21_JHD). This model offers the capability to compute overwash conditions (currents and waves).

5.4.2 Comparison results with measurements Battjes & Janssen

In the experimental study of Battjes & Janssen (1978), wave height and wave set-up measurements were carried out in a flume with an overall length of 45 m, a width of 0.8 m, and a height of 1.0 m (Figure 33). The flume is equipped with a hydraulically driven random-wave generator. At the end of the flume opposite of the wave generator, a beach with an idealized bar-trough profile was built, consisting of two 1:20 plane sections sloping seawards, connected by a 1:40 plane section, sloping shoreward, 4.4 m in length. The height of the bar crest is about 0.50 m above the flume bottom, and the depth of the trough below the bar crest is about 0.27 m.

Because in this experiment no overwash current is simulated, the measurements are compared with the results of the overwash model with a flux \( Q = 0 \).

Figures 31 and 32 show the results of the wave height and the wave set-up, calculated with the overwash model, compared with the measurements of Battjes & Janssen. The incoming wave parameters are defined in Table 2. The values in the figures have been plotted against SWL and the variables are normalized as follows:
Sediment loss due to washover on a barrier island

Wave height comparison
Theory vs Measurement Battjes & Janssen

Wave setup comparison
Theory vs measurements: Battjes & Janssen

Experiment setup: flume

Figure 31

Figure 32

Figure 33
Sediment loss due to washover on a barrier island

\[ H_r = \frac{H_{\text{rms}}}{H_{\text{rms0}}} \quad \text{: relative wave height} \]
\[ h' = \frac{h'}{H_{\text{rms0}}} \quad \text{: relative set-up} \]
\[ d_s = \frac{d}{H_{\text{rms0}}} \quad \text{: relative depth} \]

It can be seen that the calculated wave heights follow the measured values quite good. This trend can also be noticed for the wave set-up, although there is a difference in calculated and measured water levels in the last part of the flume, where the calculated wave set-up starts a little earlier than the measured set-up. Apparently the calculated waves break earlier than the measured waves, which can be confirmed by the wave height comparison. Lowering \( \alpha \) in the breaker dissipation formula induces the waves to break later.

**Table 2: Input parameters B & J**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H )</td>
<td>0.104</td>
<td>[m]</td>
</tr>
<tr>
<td>( T_p )</td>
<td>2.01</td>
<td>[s]</td>
</tr>
<tr>
<td>( m )</td>
<td>variable</td>
<td></td>
</tr>
<tr>
<td>( r )</td>
<td>0.001</td>
<td>[m]</td>
</tr>
<tr>
<td>( \alpha )</td>
<td>1.00</td>
<td>[-]</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>0.80</td>
<td>[-]</td>
</tr>
</tbody>
</table>

**5.4.3 Comparison results with measurements Thorkilsen**

A similar test as described above is performed by Thorkilsen et al. (1991). Near the end of the flume a longshore bar was placed with a crest level of SWL -0.10 m, a crest width of 2m, with a front and a back slope of 1:33. At the end of the flume the waves were dissipated completely by a wave absorber (Figure 36). Figure 34 and 35 show the results of the measurements, compared with the calculated results. In Table 3 the used computation parameters are given. Also in this test the calculated waves break a bit earlier than the measured. Increasing the breaker index \( \gamma \) and/or lowering \( \alpha \) brings the measured and calculated wave set-up levels closer to each other.

**Table 3: Input parameters Thorkilsen**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H_s )</td>
<td>0.140</td>
<td>[m]</td>
</tr>
<tr>
<td>( T_p )</td>
<td>1.80</td>
<td>[s]</td>
</tr>
<tr>
<td>( m )</td>
<td>variable</td>
<td></td>
</tr>
<tr>
<td>( r )</td>
<td>0.001</td>
<td>[m]</td>
</tr>
<tr>
<td>( \alpha )</td>
<td>0.80</td>
<td>[-]</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>0.78</td>
<td>[-]</td>
</tr>
</tbody>
</table>
Sediment loss due to washover on a barrier island

Wave height decay
Theory: \( H_s = 0.14 \text{m}; T_p = 1.8 \text{ sec} \)

Wave setup comparison
Theory vs. measurements; \( H_s = 0.14 \text{m}; T_p = 1.8 \text{ sec} \)

Experiment setup: flume
5.4.4 Comparison washover model with Unibest_LT\(^3\)

Unibest_LT is designed to compute tide and wave-driven longshore currents and sediment transports on an alongshore beach of arbitrary profile.

The surf zone dynamics are derived from a built-in random wave propagation and decay model (ENDEC), which transforms offshore wave data to the coast, taking into account the principle processes of linear refraction and non-linear dissipation by wave breaking and bottom friction. The longshore sediment transports and cross-shore distribution are evaluated according to various formulae, which enables a sensitivity analysis for local conditions.

In this study Unibest_LT is used to compute wave parameters which are transformed to the coast. The results are compared with wave data computed by the 1-D overwash model. No sediment transport rates are compared because Unibest_LT is not able to compute cross-shore sediment transport due to washover currents. The initial conditions, used to compute the wave parameters from the sea side to the shore (for both models), are specified in Table 4:

<table>
<thead>
<tr>
<th>Table 4: Input parameters UNI_LT</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H_s )</td>
</tr>
<tr>
<td>( T_p )</td>
</tr>
<tr>
<td>( m )</td>
</tr>
<tr>
<td>( r )</td>
</tr>
<tr>
<td>( \alpha )</td>
</tr>
<tr>
<td>( \gamma )</td>
</tr>
</tbody>
</table>

The results are shown in Figure 37 (wave height), 38 (wave set-up) and 38a (bottom profile). The wave heights computed with the overwash model follow the UNI_LT wave heights to a quite extent, but near the shoreline the waves computed with the overwash model break sooner than the UNI_LT wave heights. This is the result of the breaking wave fraction \( Q_b \), which in the UNI_LT model reaches its maximum (=1) earlier than the more gradually increasing breaking-wave-fraction-line computed with the overwash model. This difference in breaking wave fraction is probably the result of a different computation scheme used by UNI_LT (in both the models the same dissipation formulae are used). It is not known what computation scheme is used in UNI_LT. The scheme used in the overwash model is a "forward marching" scheme, computing the parameters per grid point relative to the former point. No "weighting" of the grid points just before and just after the specific grid point is taken into account in the overwash model and no central differencing technique is used. Not much time was available in this study to examine the effects of different computation schemes on the model results.

---

\(^3\): UNiform Beach Sediment Transport_Longshore Transport (Delft Hydraulics)
The difference in wave set-up is smaller than the difference in wave height. Thus it can be concluded that the wave set-up, which for a great deal determines the strength of the overwash current, is represented well with the overwash model.

In general, the waves in the overwash model break a bit earlier and the wave set-up is a little bit smaller than the set-up computed with UNI_LT. The overall differences, however, are small.
Sediment loss due to washover on a barrier island

Wave height comparison
Overwash model vs. UNI_LT results: $H_s=2\text{m}; \ T_p=7.8\text{sec}; \ f_w=0.03$

Figure 37

Wave setup comparison
Overwash model vs. UNI_LT results

Figure 38

Bottom profile

Figure 38a
5.4.5 Comparison washover model with MIKE21_HD/MIKE21_NSW

Apart from the verification of the 1-D overwash model wave results, a comparison of overwash surface elevations and current velocities is made with the cross-shore results of a 2-D model, MIKE21_HD\(^6\). The reason for doing this is that this model is capable to simulate currents flowing over a bar due to wave- and tide-induced water level differences and currents. The wave transformation from the sea to the shore is calculated with the near shore wave module MIKE21_NSW\(^7\). This model provides the wave-induced radiation stresses which are put in the hydrodynamic model to compute the wave driven water level elevations and currents.

In this section only the results of the 1-D and 2-D flow models are compared. In Appendix VII the set-up of the used 2-D models is explained and in the Appendices VIII and IX the theoretical background of the 2-D flow and wave model is described.

Due to the fact that the 2-D flow and wave models were set-up in an early stage of this study, before the 1-D wave model was designed, an item used in the 2-D modelling was not taken into account in the 1-D overwash model. This item, the "directional spreading of wave energy", distributes the energy of a single wave in a discrete number of directions (explained in Appendix VIII). This has the effect that the a wave can receive parts of energy from waves which propagate around that specific wave. This results in an earlier breaking of the waves in the 2-D wave model compared to the wave breaking computed in the 1-D overwash model.

Another difference between the 1-D and 2-D model is the effect of the wave-current interaction, which is not taken into account in the 2-D flow model but is taken into account in the 1-D overwash model. The wave-current interaction will have an effect on the bottom shear stress, which is increased compared to the bottom shear stress for currents only.

The modelled six conditions with varying crest height and wave height are shown in Table 5 and the parameters used in both models are given in Table 6.

<table>
<thead>
<tr>
<th>CREST HEIGHT</th>
<th>WAVE CONDITION: H (_s); (T_m) (shore normal)</th>
</tr>
</thead>
<tbody>
<tr>
<td>+0.80 m</td>
<td>HD +082N</td>
</tr>
<tr>
<td>- 0.40 m</td>
<td>HD -042N</td>
</tr>
<tr>
<td>- 0.80 m</td>
<td>HD -082N</td>
</tr>
</tbody>
</table>

\(^6\) MIKE21_HydroDynamic model (Danish Hydraulic Institute)

\(^7\) MIKE21_Near Shore Wave model (Danish Hydraulic Institute)
Table 6: Used model parameters
MIKE21 HD and MIKE21 NSW

<table>
<thead>
<tr>
<th>m</th>
<th>1:50 [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>r</td>
<td>0.02 [m]</td>
</tr>
<tr>
<td>α</td>
<td>1.00 [-]</td>
</tr>
<tr>
<td>γ</td>
<td>0.78 [-]</td>
</tr>
<tr>
<td>DWD(^8)</td>
<td>30.0 [°]</td>
</tr>
<tr>
<td>Δx</td>
<td>20 [m]</td>
</tr>
<tr>
<td>Δt</td>
<td>6 [s]</td>
</tr>
</tbody>
</table>

Wave modeling results

The 2 and 4m wave height decay, taken from a line across the barriers in the middle of the 2-D model, are compared with the 1-D overwash model wave heights in Figures 39a, 39b and 39c for crest heights of +0.80m, -0.40m and -0.80m SWL respectively. Small differences can be expected between the two model results due to the fact that in MIKE21_NSW directional spreading is taken into account, which is not implied in the 1-D overwash model. The wave energy is spread out over a specified angle. This means that waves transfer energy to and receive energy from waves around them. The result of this is that certain waves receive more energy than others and become higher. This means that the waves break earlier (further from the shore) and the shore normal radiation stress component \( S_{xx} \) drops earlier.

The effect of the directional spreading of wave energy can best be observed from the radiation stress variation across the barrier shore, shown in Figures 40a, 40b and 40c. These figures show the radiation stress component \( S_{xx} \) divided by the density of the water \( \rho \). Since \( S_{xx} \) is directly related to the wave energy (and wave height), the radiation stresses computed with MIKE21_NSW drop further from the shore than those computed with the 1-D model.

In both the wave height and radiation stress computations the effect of wave set-up is included. First the radiation stresses computed with the wave model are put in the flow model, which computes the wave set-up. This wave set-up is put back into the wave model so that the effect of the wave set-up on the wave parameters can be determined. This method is explained in more detail in Appendix VII.

\(^8\): maximum deviation from mean wave direction ("directional spreading")
Sediment loss due to washover on a barrier island

Wave height: MIKE21_NSW vs. 1-D overwash model
(Hs=2 m; T=7 sec) and (Hs=4m; T=10 sec); h_crest=+0.8 m

Figure 39a

Wave height: MIKE21_NSW vs. 1-D overwash model
(Hs=2 m; T=7 sec) and (Hs=4m; T=10 sec); h_crest=-0.4 m

Figure 39b

Wave height: MIKE21_NSW vs. 1-D overwash model
(Hs=2 m; T=7 sec) and (Hs=4m; T=10 sec); h_crest=-0.8 m

Figure 39c
Sediment loss due to washover on a barrier island

Radiation stress: MIKE21_NSW vs. 1-D overwash model

(Hs=2 m; T=7 sec) and (Hs=4 m; T=10 sec); h_crest=+0.6 m

Figure 40a

Radiation stress: MIKE21_NSW vs. 1-D overwash model

(Hs=2 m; T=7 sec) and (Hs=4 m; T=10 sec); h_crest=-0.4 m

Figure 40b

Radiation stress: MIKE21_NSW vs. 1-D overwash model

(Hs=2 m; T=7 sec) and (Hs=4 m; T=10 sec); h_crest=-0.8 m

Figure 40c
Flow modelling results

The effect of the waves which break a bit earlier in the 2-D wave model can be examined looking at the differences in wave set-up in Figures 41a, 41b and 41c. Compared to the 1-D overwash model the maximum wave set-down develops further from the barrier shore and the wave set-up starts earlier to rise, caused by the change in radiation stress which drop at an earlier stage. However, the differences in the maximum wave set-up at the beginning of the crest (x=0) are small. This is eminent in the washover process because the wave set-up contributes directly to the overwash current velocities, caused by an increased water level difference across the barrier.

The velocity variation across the barriers is plotted in the Figures 42a, 42b and 42c. In the first plot (42a) no washover current is generated due to the simple fact that the crest remains above the water surface. The velocity variation in Figures 42b and 42c show small differences between the 2m wave and the 4m wave overwash condition. The velocities computed with the 1-D overwash model are a bit smaller in case of \( H_s = 2 \text{m} \) and are a bit higher in case of \( H_s = 4 \text{m} \) compared to the MIKE21_HD velocities. Although the same bed roughness and wave bottom friction factor is used, the velocities between the models differ. The motivation for this difference can be found in:

1) The maximum wave set-up which differs at the beginning of the crest caused by directional spreading of wave energy in MIKE21_NSW
2) A different solution technique used in both models.
3) Wave-current interaction which is not used in the 2-D model

General conclusion

Apart from the difference of the wave parameters caused by the directional spreading of the wave energy in MIKE21_NSW, the differences in overwash properties (\( h' \) and \( U \)) are small. In section 5.4 was already pointed out that the overwash model offers good results and has the capability of simulating wave heights and wave-induced water level elevations quite accurate compared with measurements. The comparison of the 1-D overwash model results with the 2-D hydrodynamic model MIKE21_HD shows that besides the wave simulation also the washover current can be simulated properly.
Sediment loss due to washover on a barrier island

Surface elevation: MIKE21 HD vs. 1-D overwash model

(\(H_s = 2\ m; T = 7\ sec\)) and (\(H_s = 4\ m; T = 10\ sec\)); \(h_{\text{crest}} = +0.8\ m\)

- \(H_s = 2\ m\) (MIKE21)
- \(H_s = 2\ m\) (1-D model)
- \(H_s = 4\ m\) (MIKE21)
- \(H_s = 4\ m\) (1-D model)

Figure 41a

Surface elevation: MIKE21 HD vs. 1-D overwash model

(\(H_s = 2\ m; T = 7\ sec\)) and (\(H_s = 4\ m; T = 10\ sec\)); \(h_{\text{crest}} = -0.4\ m\)

- \(H_s = 2\ m\) (MIKE21)
- \(H_s = 2\ m\) (1-D model)
- \(H_s = 4\ m\) (MIKE21)
- \(H_s = 4\ m\) (1-D model)

Figure 41b

Surface elevation: MIKE21 HD vs. 1-D overwash model

(\(H_s = 2\ m; T = 7\ sec\)) and (\(H_s = 4\ m; T = 10\ sec\)); \(h_{\text{crest}} = -0.6\ m\)

- \(H_s = 2\ m\) (MIKE21)
- \(H_s = 2\ m\) (1-D model)
- \(H_s = 4\ m\) (MIKE21)
- \(H_s = 4\ m\) (1-D model)

Figure 41c
Sediment loss due to washover on a barrier island

Washover current velocity: MIKE21 HD vs. 1-D overwash model

(Hs=2 m; T=7 sec) and (Hs=4 m; T=10 sec); h_crest=+0.6 m

Figure 42a

Washover current velocity: MIKE21 HD vs. 1-D overwash model

(Hs=2 m; T=7 sec) and (Hs=4 m; T=10 sec); h_crest=-0.4 m

Figure 42b

Washover current velocity: MIKE21 HD vs. 1-D overwash model

(Hs=2 m; T=7 sec) and (Hs=4 m; T=10 sec); h_crest=-0.8 m

Figure 42c
5.5 Sensitivity study 1-D overwash model

5.5.1 General

In section 5.4 the theoretical background and the computation method of the overwash model was described; now the model can be applied to make a few test runs. These test runs must demonstrate the influence of the different input parameters on the overwash process. These runs will provide, under controlled conditions, a clear view on the importance of the individual parameters.

The sensitivity study consists of five test runs, each with its own variable parameters and with output in the form of velocities, wave heights and water level elevations over the bar. The effect of the different parameters on the sediment transport is restricted to the examination the washover transport caused by a varying wave height and wave set-up. In section 5.5.8 the barrier profile development caused by tide-induced overwash, wave-induced overwash and tide and wave induced overwash is studied.

The executed test runs to study the overwash process are:
- run 1: Overwash with variable bottom slope
- run 2: Overwash with variable crest height
- run 3: Overwash with variable crest width
- run 4: Overwash with variable wave height
- run 5: Overwash with variable bed roughness
- run 6: Sediment transport with variable wave height.

5.5.2 Run 1: Overwash with variable bottom slope

To examine the effect of the bottom slope on the overwash wave heights and overwash velocities, five runs with different bottom slopes are produced. The steepness of the slopes are respectively 1:50, 1:25, 1:16.7, 1:12.5 and 1:10 (Figure 43). The used constant parameters are shown in Table 5.

<table>
<thead>
<tr>
<th>Table 5: Input parameters Run 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_s$</td>
</tr>
<tr>
<td>$T_p$</td>
</tr>
<tr>
<td>$r$</td>
</tr>
<tr>
<td>$h_{crest}$</td>
</tr>
<tr>
<td>$\Delta z_{tide}$</td>
</tr>
<tr>
<td>$\alpha$</td>
</tr>
<tr>
<td>$\gamma$</td>
</tr>
</tbody>
</table>
Sediment loss due to washover on a barrier island

Examined bottom slopes

\[ H_s = 2.5 \text{m}; \ T_p = 7.8; \ r = 0.02 \text{m} \]

Wave height & variable bottom slope

\[ H_s = 2.5 \text{m}; \ T_p = 7.8; \ r = 0.02 \text{m} \]
Sediment loss due to washover on a barrier island

**Surface elevation & variable bottom slope**

$H_s=2.5m; \ T_p=7.8s; \ r=0.02$

**Velocity & variable bottom slope**

$H_s=2.5m; \ T_p=7.8s; \ r=0.02m$

Figure 45

Figure 46
Figure 44 shows the wave decay due to breaking and bottom friction. The waves approach the bar from the right. It can be noticed that as the slope becomes steeper, the waves break further to the shore and the wave set-up, shown in Figure 45, increases. The increased wave set-up, due to the increased slope steepness, causes a larger water level difference over the bar, and thus higher velocities (Figure 46).

In Figure 46 is shown that just downstream of the crest a sort of a set-down can be observed. This is created by the overwash current which reaches its maximum velocity at that point. There the "snelheidshoogte" ($U^2/2g$) is relatively high and the "energy height" ($E$) drops from the front to the end of the crest, caused by bottom friction, so that the surface elevation becomes under the 0m-line.

**General conclusion run 1:** With increasing bottom steepness and a constant wave which approach the bar shore normal, the overwash velocities increase. This will result in larger transport across the bar.

### 5.5.3 Run 2: Overwash with variable crest height

In this run the height of the crest is varied with the same constant parameters in section 5.5.1, except for the bottom slope, which is constant at 1:25 (Figure 47). In reality the crest height is not varying, but the water level varies due to the tidal variation of the water level and wave set-up's, which causes flooding of the crest of a barrier island. Varying the crest height however has the same effect on the overwash process as varying the initial water level. The examined crest heights are 0.4m, 1m, 2m, 4m, and 8m below SWL.

The wave height variation is presented by Figure 48. It is obvious that the larger the water depth on top of the crest the higher the waves remain (less breaking). The breaking point of the waves is shifted more downstream when the depth on top of the crest increases (point of maximum wave set-down). The maximum wave set-up is reached on the bar with the smallest crest height (-0.4m SWL), caused by the shallower water in front of the crest.

Due to the diminished wave energy dissipation, the change in radiation stress also becomes smaller, which means that the wave set-up becomes smaller. In this test case the water level difference on both sides of the bar is only caused by the wave set-up (Figure 49). When the set-up becomes smaller, the velocities drop and therefore the sediment transport drops until no transport at all occurs ($h_{crest} = 8$m). The velocities (Figure 50), like the discharges, decrease with decreasing crest heights. This is the case in a wave-only situation. However, when a constant water level difference over the bar is ensured by the tide, the velocities and discharges increase with increasing crest height. In reality this will never occur, because when the crest is far below the water surface the tidal current experiences less bottom friction when flowing over the bar and the gradient in water level (water surface slope) decreases.

**General conclusion run 2:** With increasing crest height, the wave-induced overwash current velocities decrease, which will cause smaller sediment transports.
Examined crest heights

$m=1:25$

Figure 47

Wave height & variable crest height

$H_s=2.5 \text{ m}; T_p=7.8 \text{ sec}; m=1:25; r=0.02\text{m}$

Figure 48
Sediment loss due to washover on a barrier island

**Surface elevation & variable crest height**

$H_s=2.5\, m;\, T_p=7.8\, sec;\, m=1:25;\, r=0.02\, m$

![Graph showing surface elevation and variable crest height](image)

**Velocity & variable crest height**

$H_s=2.5\, m;\, T_p=7.8\, sec;\, m=1:25;\, r=0.02\, m$

![Graph showing velocity and variable crest height](image)
5.5.4 Run 3: Overwash with variable crest width

The width of the crest of barrier islands can vary between 50m and 2000m, depending on their origin (built up from on- and offshore transport or from material deposits from alluvial rivers and/or longshore transport mechanisms). The effect of the crest width on washover currents is significant. To show this influence in this test case five different crests are examined with widths of 50m, 100m, 200m, 400m, and 800m. The same input parameters are used here as in run 1, except for the bottom slope, which is this run is 1:25. The barrier profiles with different crest widths are shown in Figure 51. Figure 52 is a graph of the wave height decay across the bar. Together with the variation of the surface elevation across the bar in Figure 53 it can be seen that the longer the crest width, the higher the wave set-up and (therefore) the smaller the wave decay when the waves travel over the crest (caused by larger depths due to the wave set-up).

Looking at the velocities in Figure 54 it can be noticed that the smaller the width the higher the maximum velocity. The reason for this high velocities is that the current is slow down by the bottom friction over only 50m, while with a crest width of 800m the current experiences bottom friction over 800m.

General conclusion run 3: Wave set-up on barrier islands with a wide crest is higher than the wave set-up on barrier islands with small crests. Wave-induced current velocities reach higher velocities on barrier islands with small crest widths than on wider crests.

05.5.5 Run 4: Overwash with variable wave height

An important driving force for washover is the wave energy. This wave energy can be varied by varying the wave height and/or the period. The examined combinations are: $H_s = 1.0m$ with $T_p = 4.44s$, $H_s = 2.5m$ with $T_p = 6.5s$, $H_s = 3.5m$ with $T_p = 7.78s$, $H_s = 5.0m$ with $T_p = 8.89s$. The same input parameters are used here as in run 1, except for the bottom slope (1:25).

Figure 55 displays the wave decay when the waves propagate over the bar. The higher the wave in deep water, the higher the wave is at the beginning of the crest ($x=0$), caused by the wave set-up, which reaches its maximum at that point. The wave set-up is maximum for the highest wave ($H_s=5m$), shown in Figure 56. In Figure 57 the velocities show the same trend: the higher the deep sea wave, the higher the set-up, the higher the velocities. The driving mechanism for the wave set-up is the change in radiation stress, displayed in Figure 58.

General conclusion run 4: High waves which approach the bar shore normal cause high wave set-up’s at the front side of the bar. The water level differences across the bar therefore increase, which results in higher overwash current velocities. High waves and high overwash current velocities will increase the erosion of the shoreface and the washover transport to the lee-side of a bar.
Examined crest widths
Hs=2.5m; Tp=7.8s; r=0.02; m=1:25

Wave height & variable crest width
Hs=2.5m; Tp=7.8; r=0.02m; m=1:25
Sediment loss due to washover on a barrier island

**Surface elevation & variable crest width**

\[ H_s = 2.5 \text{m}; \ T_p = 7.8 \text{s}; \ r = 0.02; \ m = 1:25 \]

![Graph showing surface elevation with varying crest widths](image)

**Figure 53**

**Velocity & variable crest width**

\[ H_s = 2.5 \text{m}; \ T_p = 7.8 \text{s}; \ r = 0.02; \ m = 1:25 \]

![Graph showing velocity with varying crest widths](image)

**Figure 54**
Sediment loss due to washover on a barrier island

Examined wave height

\[ h_{\text{crest}} = 0.4 \text{m}; r = 0.02 \text{m}; m = 1:25 \]

\[ \begin{align*}
-600 & -400 & -200 & 0 & 200 & 400 & 600 \\
\hline
-2 & -1 & 0 & 1 & 2 & 3 & 4 & 5 & 6
\end{align*} \]

\text{distance from the shore } x (\text{m})

--- \( H_s = 1.0 \text{ m} \) --- \( H_s = 2.5 \text{ m} \) --- \( H_s = 3.5 \text{ m} \) --- \( H_s = 5.0 \text{ m} \)

Figure 55

Surface elevation & variable wave height

\[ h_{\text{crest}} = 0.4 \text{m}; r = 0.02 \text{m}; m = 1:25 \]

\[ \begin{align*}
-600 & -400 & -200 & 0 & 200 & 400 & 600 \\
\hline
-0.6 & -0.4 & -0.2 & 0 & 0.2 & 0.4 & 0.6 & 0.8
\end{align*} \]

\text{distance from the shore } x (\text{m})

--- \( H_s = 1.0 \text{ m} \) --- \( H_s = 2.5 \text{ m} \) --- \( H_s = 3.5 \text{ m} \) --- \( H_s = 5.0 \text{ m} \)

Figure 56
Sediment loss due to washover on a barrier island

**Velocity & variable wave height**

\[ h_{\text{crest}} = 0.4 \text{m}; r = 0.02 \text{m}; m = 1:25 \]

![Velocity & variable wave height](image)

**Radiation stress Sxx & variable wave height**

\[ h_{\text{crest}} = 0.4 \text{m}; r = 0.02 \text{m}; m = 1:25 \]

![Radiation stress Sxx & variable wave height](image)

---

Figure 57

Figure 58

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5.5.6 Run 5: Overwash with variable bed roughness

This test case is performed to examine the effect of the increasing bed roughness. The crest is at a depth of -0.4m SWL, shown in Figure 59. It can be expected that when the bed roughness increases, the current velocities decrease. In this run the roughness is varied from 1mm to 100mm. Looking at the wave heights in Figure 60 it can be seen that the rougher the bed, the higher the waves remain on the crest, thus the higher the wave decay upon the crest.

This phenomenon is caused by the surface elevation across the bar, which is higher in case of a rough bottom than in case of a smooth bottom (Figure 61). The velocities decrease when the roughness is increased, as expected (Figure 62).

**General conclusion run 5:** When the bed roughness of a sandbar in increased, the wave heights and the surface elevation will become higher; the current velocities will decrease, resulting in lower transport capacities.

5.5.7 Run 6: Sediment transport with variable wave height

To examine the washover transport caused by wave-induced overwash, the sediment transport rates for the four wave heights from run 4 are calculated. The different parameters used to compute the sediment transport with the Bijker formula, are given in Table 6:

<table>
<thead>
<tr>
<th>Table 6: Input parameters sediment transport wave-induced washover</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_s$</td>
</tr>
<tr>
<td>$T_p$</td>
</tr>
<tr>
<td>$\alpha$</td>
</tr>
<tr>
<td>$\gamma$</td>
</tr>
<tr>
<td>$m_{\text{front}}$</td>
</tr>
<tr>
<td>$h_{\text{crest}}$</td>
</tr>
<tr>
<td>$D_{50}$</td>
</tr>
<tr>
<td>$D_{90}$</td>
</tr>
<tr>
<td>$r$</td>
</tr>
<tr>
<td>$\rho_s$</td>
</tr>
<tr>
<td>$\rho_w$</td>
</tr>
</tbody>
</table>
Sediment loss due to washover on a barrier island

**Figure 59**
Examed bottom slope

![Graph showing depth vs. distance from the shore](image)

**Figure 60**
Wave height & variable bed roughness

![Graph showing wave height vs. distance from the shore](image)

---

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Sediment loss due to washover on a barrier island

Surface elevation & variable bed roughness

\[ H_s = 2.5 \text{m}; \quad T_p = 7.8 \text{s}; \quad m = 1:50 \]

![Graph of surface elevation and variable bed roughness](image)

Velocity & variable bed roughness

\[ H_s = 2.5 \text{m}; \quad T_p = 7.8 \text{s}; \quad m = 1:50 \]

![Graph of velocity and variable bed roughness](image)
Looking at Figure 57 it can be noticed that the overwash current velocity just before the end of the crest reaches its maximum. This is caused by the downstream boundary condition forcing the surface elevation to SWL and creating high velocities. It is obvious that at that point of the bar the sand losses will be maximum, which is confirmed by the transport rates plotted in Figure 63. Large gradients in sediment transport occur where the current flows over the sharp edges of the idealized barrier profile plotted (Figure 64), resulting in erosion. From the sea side the transport rates increase due to wave action and due to the development of the overwash current (from $x=600\text{m}$ to $x=0\text{m}$, see also Figure 65, the bottom profile). In the first 100 metres of the crest deposition of material will occur, caused by the diminished wave action in combination with the overwash current (from $x=0\text{m}$ to $x=-100\text{m}$). At the last section of the crest (from $x=-100\text{m}$ to $x=-220\text{m}$) the overwash current reaches its maximum velocity, which causes relative high erosion rates. Once at the end of the crest the depth increases again, the velocities (Figure 57) drop quickly and so does the sediment transport, which results in accretion.

Two mechanisms causing erosion can be distinguished. The first mechanisms is the erosion caused by the waves. The front side of the barrier island retreats due to the breaking waves, which stir up the sediment. This material is then transported by the overwash current and is partly deposited on the crest, where the wave energy has become very small. In front of the bar the overwash velocity is relative small compared with the velocity at the back side, caused by the diminishing of the water depth. At this part of the crest the second erosion mechanism, the overwash current, plays an important rôle. The sediment which is eroded by this current is deposited and spread out over the backside of the barrier.
Sediment loss due to washover on a barrier island

Sediment transport due to wave-induced washover

\[ h_{\text{crest}} = 0.4\text{m}; r = 0.02\text{m}; m = 1:25 \]

Figure 63

Gradients in sediment transport due to wave-induced washover

\[ h_{\text{crest}} = 0.4\text{m}; r = 0.02\text{m}; m = 1:25 \]

Figure 64

Barrier profile

Figure 65
5.5.8 Discharge Q with variable crest height & bed roughness

In section 5.5.3 the wave-induced overwash conditions are examined with a variable crest height. To examine the variation of the flux Q, which will occur when the crest is lowered under tide-only and tide+wave washover conditions, in this section 8 different crest heights are modelled. The crest width for each of the examined profile is 300m.

Per crest height three different bed roughnesses are used to show the influence of the bed roughness on the overwash discharge Q. The used model parameters are shown in Table 7.

Table 7: Input parameters for runs with varying crest height and bed roughness

<table>
<thead>
<tr>
<th></th>
<th>Tide only</th>
<th>Waves only</th>
<th>Tide+ Waves</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h'_\text{front}$ (m to MSL)</td>
<td>+0.10</td>
<td>0.00</td>
<td>+0.10</td>
</tr>
<tr>
<td>$h'_\text{back}$ (m to MSL)</td>
<td>-0.10</td>
<td>0.00</td>
<td>-0.10</td>
</tr>
<tr>
<td>$H_s$ (m)</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>$T_p$ (s)</td>
<td>7</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>$\alpha$ (-)</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>$\gamma$ (-)</td>
<td>0.8</td>
<td>0.8</td>
<td></td>
</tr>
<tr>
<td>$m_{\text{front}}$ (-)</td>
<td>1:25</td>
<td>1:25</td>
<td>1:25</td>
</tr>
<tr>
<td>$h_{\text{crest}}$ (m to MSL)</td>
<td>-0.25; -0.50; -1.00; -1.50; -2.00; -3.00; -4.00; -5.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$r$ (m)</td>
<td>0.002; 0.02; 0.2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 65a, b and c show the results graphically. Along the horizontal axis the crest heights are specified and along the vertical axis the discharges. The curves marked with a rectangle represent the tide-only case (T) with three different bed roughnesses. Those marked with a triangle are the discharge curves for the waves-only case (W) and the diamond marked curves represent the tide+waves discharges (T+W), both for three different bed roughnesses. The results are discussed below.

**Tide-only**

Due to the constant water level difference across the bar the discharges increase almost linear with the crest height (depth below MSL). Smaller discharges can be observed with an increased in bed roughness (Figure 65a).

**Waves-only**

In this situation the water level difference is not constant when the crest is lowered caused by the diminished wave set-up. With the smoothest bed ($r=0.002m$) Q increases from the -0.25m crest to the -1.50m crest, and Q becomes smaller and drops to zero when the crest becomes deeper under the water surface (wave set-up becomes smaller) (Figure 65b).
Sediment loss due to washover on a barrier island

Overwash discharge $Q$ with varying crest height

**Tide Only**

Overwash discharge $Q$ with varying crest height

**Waves Only**

Overwash discharge $Q$ with varying crest height

**Tide + Waves**

Figure 65a

Figure 65b

Figure 65c
Tide + waves These curves (Figure 65c) show the same trend as the tide-only case, except for the case where r = 0.2m. In this case the discharges are much lower than the discharges computed with a smoother bed. This shows the "slowing down" effect of the waves on the overwash current. It can be observed that when $h_{crest} > -3.0$m MSL the influence of the waves on the overwash discharge has become very small and the increase in discharge relative to the increase in depth is about the same as with a rougher bottom.
5.6 Barrier profile development due to tide-, wave- and tide+wave-induced overwash

The above described erosion and accretion process can be examined in detail by applying the overwash model to compute the development of the profile of the barrier crest in a tide-, wave- and tide+wave-induced overwash situation.

5.6.1 Profile development due to tide-induced washover

To study the influence of a tide-induced overwash current on the changes of the barrier profile a washover current is simulated for a tidal water level difference of 0.15m, a bed roughness of 0.02m and a bottom slope (at the front side of the bar) of 1:25. The water level at the front side of the bar (SWL\_front) is +0.10m MSL and the water level at the back side (SWL\_back) is -0.05m MSL. The crest of the bar has a width of 300m and the crest height is 0.40m below MSL. The same sediment properties are used as in section 5.5.6, except the Bijker constant, which is in this case set to 1. The explicit time stepping technique, used to compute the profile changes, is defined by the following steps:

- At \( t = 0 \) the initial overwash conditions (\( h', U, S_{tot} \)) are computed across the bar
- For an interval \( \Delta t = 1800 \) seconds (1/2 hour) the changes in barrier profile per metre are calculated according to the following formulae:

\[
h_{i,new} = h_{i,old} + \Delta h_i
\]

\[
\Delta h_i = \left( \frac{\Delta S_{tot}}{\Delta x} \right)_i \cdot \Delta t
\]

- For every grid point the profile is adjusted by \( \Delta h \) and the overwash properties (\( h', U \), and \( U_i \)) are recalculated.

Figure 66 shows the development of the cross shore profile caused by a tide-induced washover at an interval of 24 hours. Concentrating on the shoreface it can be observed that it is retreating from the SWL -1m contour line and upwards. In general the profile development shows the trend of a continuous lowering of the crest, caused by an increase in the current velocity nearing the end of the crest. The eroded sand from the front and from the crest is deposited at the lee-side of the bar, which elevates the bottom there locally.

Figure 67 shows the sediment transport caused by the tide-induced overwash current. It can be seen that at the end of the crest the transport rates are higher than the transport rates at the beginning. This is caused by the diminished depth of the overwash current at the end of the crest, which results in higher velocities. After 4 days of continuous overwash (\( t = 96 \) hrs) the crest is lowered due to erosion and the velocities are increased relative to the initial situation (\( t = 0 \) hrs).
Sediment loss due to washover on a barrier island

Barrier profile development: tide-induced washover
\[ dz=0.15\text{m}; \ h_{\text{crest}}= 0.4\text{m}; \ r=0.02\text{m}; \ m=1:25 \]

Sediment transport: tide-induced washover
\[ dz=0.15\text{m}; \ h_{\text{crest}}= 0.4\text{m}; \ r=0.02\text{m}; \ m=1:25 \]
Sediment loss due to washover on a barrier island

Surface elevation & bottom profile: tide-induced washover
\[dz=0.15\text{m}; h_{\text{crest}}=0.4\text{m}; r=0.02\text{m}; m=1:25\]

Velocity & bottom profile: tide-induced washover
\[dz=0.15\text{m}; h_{\text{crest}}=0.4\text{m}; r=0.02\text{m}; m=1:25\]
Sediment loss due to washover on a barrier island

Figure 69 is a plot of the overwash velocities at t = 0 hrs and t = 96 hrs., caused by the altered bottom profile. It can be seen that the lower the crest, the higher the overwash velocities. This is caused by the water level difference across the bar which is held constant during the overwash period, shown in Figure 68. In reality the diminished crest height can have an effect on the water level differences across the bar. However, when the crest is lowered several centimetres the change in water level differences across the bar can be neglected. In general can be concluded that lowering the crest in a tide-induced overwash case increases the velocities (Figure 69), which results in an increase in washover transport.

5.6.2 Profile development due to wave-induced washover

Figure 70 shows the development of a barrier which is overwash by a wave-induced current, caused by the wave set-up of a 2.5m wave approaching the bar from the right. The same sediment transport properties as in the tide-induced washover case are used here to compute the transport rates, except the Bijker constant b, which is in this situation 5. The profile changes are plotted in an interval of 24 hours. The same technique is used as explained in the tide-induced overwash situation to compute the profile changes. Now that waves are involved in the overwash process not only the overwash conditions h', U and S 교수 must be computed every time step but also the wave parameters and wave-induced water level variation must be determined. The cross-shore profile variations in Figure 70 shows that compared to the tide-induced situation sediment is deposited on the crest. The shoreface retreat occurs over a depth of more than 2.5m. A part of the eroded material is deposited at the back side of the bar.

Figure 71, which is a plot of the sediment transport across the bar at a 24 hour interval, shows that erosion occurs at the front of the bar and on the crest. Deposition of material takes place at the lee-side of the bar. Due to the deposition of sand the crest is elevated and the water depth on top of the crest is decreased, which causes lower velocities and therefore a smaller sediment transport rate. It can be noticed that the peaks in sediment transport, at places where the profile changes where sharp (at t=0s, at x=0m and x=220m), are disappeared after 24 hours of overwash.

Due to the retreat of the barriers shoreface the depth at x=0m is increased. An increase in depth means that the waves remain higher and the set-up is increased. This is shown by Figure 72, which is a plot of the surface elevation and bottom profile after 96 hours (4 days) of continuous overwash. The velocities in Figure 73 show that due to the diminished water depth on the crest the velocities across the bar are decreased.

Examination of the shoreface between the -0.4m and the -1.4m contour line in Figure 72 shows that the average retreat for this 4-days overwash event is about 10m! However, 4 days of continuous overwash is exceptional. Generally a combination of tide and waves provides an overwash duration of approximately 3 hours. In the intermediate time the barrier crest can partly recover due to sand supplied by littoral transport, onshore transport or wind blown transport. When there is a shortage of sand supply the sand losses cannot be reclaimed and the profile remains that specific shape.
Barrier profile development: wave-induced washover

$H_s=2.5 \text{m}; T_p=7.8 \text{s}; h_{\text{crest}}=0.4 \text{m}; r=0.02 \text{m}; m=1:25$

Sediment transport: wave-induced washover

$H_s=2.5 \text{m}; T_p=7.8 \text{s}; h_{\text{crest}}=0.4 \text{m}; r=0.02 \text{m}; m=1:25$
Sediment loss due to washover on a barrier island

**Surface elevation & bottom profile: wave-induced washover**

\[ H_s = 2.5 \text{m}; \quad T_p = 7.8 \text{s}; \quad h_{\text{crest}} = 0.4 \text{m}; \quad r = 0.02 \text{m}; \quad m = 1:25 \]

**Velocity & bottom profile: wave-induced washover**

\[ H_s = 2.5 \text{m}; \quad T_p = 7.8 \text{s}; \quad h_{\text{crest}} = 0.4 \text{m}; \quad r = 0.02 \text{m}; \quad m = 1:25 \]
5.6.3 Profile development due to tide & wave-induced washover

During a tide and wave-induced overwash the waves contribute to the tide-induced water level difference across the bar due to the wave set-up. This results in higher overwash velocities compared to the tide-only and waves-only situation. Combined with the effect of the waves on the sediment transport it can be assumed that this situation causes the highest sand losses. This is confirmed by Figure 74, which is a graph of the changes in bottom profile at a 1 day interval. Contrary to the waves-only profile changes the retreat of the barrier shoreface is not decreasing after a few days of continuous overwash, but is more or less constant per time interval. In the waves-only situation the crest is elevated every time step due to the deposition of sand. When tide and waves are causing the overwash current initially the crest is elevated, but after a few days the sand is removed to the back side of the bar and the crest height remains constant. The sediment transport rates shown in Figure 75 show that the washover transport is more or less constant in the first three days. The transport curve for t=96 hrs is quite different compared to the other curves. This is caused by the time step which is chosen too large (2 hours instead of 1/2 hour) during the computation of the profile changes for day 3 and 4.

Figure 76 shows the differences between the surface elevation and bottom profile after 4 days overwash. Compared to the retreat of the shoreface caused by the wave-induced overwash (Figure 72), the retreat due to the tide and wave-induced washover is about three times higher (±20m at h=-1.4m MSL). Due to this retreat the velocity is diminished at x=0 and increased at x=300 (Figure 77), which results in an increased erosion process on top of the crest; the deposited material is moved to the back side of the bar, resulting in a total movement of the sandbar.

5.6.4 Effect of the initial crest slope during tide & wave-induced overwash

In the tide and wave-induced overwash situation it could be noticed that at the end of the crest the bottom is elevated, while the initial bottom profile is horizontal. This is caused by the increased current velocity, which is due to the decreased water depth on the crest. At the position on the crest where the overwash velocity is maximum erosion will take place, while directly after this location the velocities drop relatively quickly and erosion occurs. This erosion and accretion pattern results in a local "depression". The overwash current flows into this depression which causes an even deeper "hole".
Sediment loss due to washover on a barrier island

**Barrier profile development: tide & wave-induced washover**

dz=0.15m; Hs=2.5m; Tp=7.8s; h_crest= 0.4m; r=0.02m; m=1.25

Figure 74

**Sediment transport: tide & wave-induced washover**

dz=0.15m; Hs=2.5m; Tp=7.8s; h_crest= 0.4m; r=0.02m; m=1.25

Figure 75
Sediment loss due to washover on a barrier island.

**Surface elevation & bottom profile: tide & wave-induced washover**

dz=0.15m; Hs=2.5m; Tp=7.8s; h_crest= 0.4m; r=0.02m; m=1:25

**Velocity & bottom profile: tide & wave-induced washover**

dz=0.15m; Hs=2.5m; Tp=7.8s; h_crest= 0.4m; r=0.02m; m=1:25
To examine if an initial crest slope can avoid these sorts effects, three sandbars with different crest slopes are modelled. The barrier dimensions and the wave and tide parameters are identical to those used in the tide and wave-induced barrier profile development run. To avoid high transport peaks at the sharp edges of the initial profile the barrier profiles and sediment transport curves are plotted after 6 hours of overwash. Figure 78a is a plot of the three examined barrier islands. The first one has a gentle negative slope ($m_c = -1:1000$), the second bar has a horizontal crest and the third bar has a slightly positive slope ($m_c = +1:1000$).

The effect of the different crest slopes can be observed in Figure 78b, which shows the sediment transport curves after 6 hours of overwash on the three barrier islands. It can be seen that the highest transports occurs on the negative crest slope. The overwash current can flow over this crest very easily compared to the overflowing water on the horizontal and positive crest, which results in high velocities. Compared to the maximum transport rates on the horizontal and positive crest, the maximum transport on the crest with the negative slope is about twice as much! The sediment transport curve drops smoothly to zero at the end of the crest, where the transport curves of the horizontal and positive crest slope show an increase in transport at the end. This increased transport causes the local depression, shown in Figure 76.

From the results it can be concluded that a small negative slope of the crest gives the highest overwash transports and a stable profile development, while on a horizontal and positive crest slope the sediment transport is much lower with an increase in transport at the end of the crest. In reality the crests of regularly overwashed barrier islands show a slightly negative slope, instead of a pure horizontal crest which is assumed in the sensitivity study. It is therefore recommended to use a gentle negative crest slope when the profile development due to overwash must be examined.

The steepness of the crest slope in reality is determined by the overwash conditions, the sediment size and the barrier profile. The crest slope will develop to an equilibrium slope which suites to the overwash conditions.
Sediment loss due to washover on a barrier island

Examined crest slopes (after 6 hrs tide+wave ind. overwash)

\[ m_{\text{front}} = 1:25; W_{\text{cr}} = 300\text{m}; h_{\text{cr}} = 0.4\text{m} \]

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure78a.png}
\caption{Sediment transport (after 6 hrs tide+wave ind. overwash)}
\end{figure}

\[ dz=0.10\text{m}; H_s=2.5\text{m}; T_p=7.8\text{m}; r=0.02\text{m}; m_{\text{fr}}=1:25 \]

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure78b.png}
\caption{Sediment transport (after 6 hrs tide+wave ind. overwash)}
\end{figure}
5.7 Sediment transport curves

The differences in washover transport caused by tide-, wave- and tide+wave-induced washover can be best examined by comparing the sediment transport curves. The transport curves are obtained from the 1-D tide-, wave- and tide+wave conditions discussed in section 5.6. For the completeness the used model parameters, wave and tide conditions are presented in Table 8.

Table 8: Input parameters tide-, wave-, and tide+wave-induced overwash

<table>
<thead>
<tr>
<th></th>
<th>Tide only</th>
<th>Waves only</th>
<th>Tide+ Waves</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h'_{fno}$ (m to MSL)</td>
<td>+0.10</td>
<td>0.00</td>
<td>+0.10</td>
</tr>
<tr>
<td>$h'_{back}$ (m to MSL)</td>
<td>-0.05</td>
<td>0.00</td>
<td>-0.05</td>
</tr>
<tr>
<td>$H_s$ (m)</td>
<td>-</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>$T_p$ (s)</td>
<td>-</td>
<td>7.8</td>
<td>7.8</td>
</tr>
<tr>
<td>$\alpha; \gamma$ (-)</td>
<td>-</td>
<td>1.0; 0.8</td>
<td>1.0; 0.8</td>
</tr>
<tr>
<td>$m_{front}$ (-)</td>
<td>1:25</td>
<td>1:25</td>
<td>1:25</td>
</tr>
<tr>
<td>$h_{crest}$ (m to MSL)</td>
<td>-0.4</td>
<td>-0.4</td>
<td>-0.4</td>
</tr>
<tr>
<td>$W_{crest}$ (m)</td>
<td>300</td>
<td>300</td>
<td>300</td>
</tr>
<tr>
<td>$r$ (m)</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>$D_{50}; D_{90}$ (µm)</td>
<td>200; 300</td>
<td>200; 300</td>
<td>200; 300</td>
</tr>
<tr>
<td>$b$ (-)</td>
<td>1</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>$\rho_w; \rho_s$ (kg/m³)</td>
<td>1021, 2650</td>
<td>1021, 2650</td>
<td>1021, 2650</td>
</tr>
</tbody>
</table>

The three transport curves are plotted in Figure 79a and the bottom profiles of the overwash barrier profiles are presented by Figure 79b. The transport profiles are obtained from the 1-D overwash model after 48 hours of overwash. Then the initial trapezium profile with its sharp edges at the beginning and end of the crest has reshaped itself according to the hydraulic and wave-conditions. The sharp peaks in the transport curves are disappeared and the curves can be compared neater.

It can be noticed directly that the sediment transport caused by tide and waves is about three times higher as the wave-induced washover transport. The sediment transport caused by tide-induced washover is about 10% of the tide+wave transport. This shows again that the contribution of the waves to the overwash transport is significant.
Sediment loss due to washover on a barrier island

**Sediment Transport Curves**

Tide-, Wave- and Tide+Wave-Induced Washover

**Bottom profile**

$\text{m}_{\text{front}} = 1:25; \text{m}_{\text{crest}} = 0; \text{m}_{\text{back}} = 1:250$
Sediment loss due to washover on a barrier island

In the wave-only and tide+wave situation erosion occurs at the front side of the bar and on the crest, from \( x = 300 \text{m} \) to \( x = -100 \text{m} \). Accretion takes place from \( x = -100 \text{m} \) to \( x = -200 \text{m} \) (waves-only) and \( x = -400 \text{m} \) (waves+tide). In the tide-only situation erosion occurs over the whole crest.

The washover transport which contributes to the sediment losses is considered to be at the location on the crest where erosion becomes accretion (\( x = -330 \text{m} \) for tide-only, \( x = -100 \text{m} \) for waves-only and \( x = -150 \text{m} \) for tide+waves). Table 9 shows the found washover transport rates for the three overwash conditions.

### Table 9: Washover transport rates: tide-only, waves-only and tide+waves washover

<table>
<thead>
<tr>
<th>Washover mechanism</th>
<th>location ((x))</th>
<th>sediment transport ((\text{m}^3/\text{s/m}))</th>
<th>relative contribution to total transport</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tide</td>
<td>((-330 \text{m}))</td>
<td>(0.03 \times 10^3)</td>
<td>6%</td>
</tr>
<tr>
<td>Waves</td>
<td>((-100 \text{m}))</td>
<td>(0.13 \times 10^3)</td>
<td>26%</td>
</tr>
<tr>
<td>Tide+waves</td>
<td>((-150 \text{m}))</td>
<td>(0.34 \times 10^3)</td>
<td>68%</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td></td>
<td>(0.50 \times 10^3)</td>
<td>100%</td>
</tr>
</tbody>
</table>

The above transport rates show that, for the specific tide and wave conditions in Table 8, the contribution of the combined tide and wave-induced overwash is about 70%. The waves-only overwash condition contributes about 25% to the total overwash transport caused by the three mechanisms. The contribution of the tide (\(\approx 5\%\)) is very small.

For an indication of the yearly sand losses due to washover caused by *tide and waves*, with conditions specified in Table 8, the following assumptions are made:

-10 time per year overwash: \( H_s = 2.5 \text{m}, \Delta z_{\text{tide}} = 0.15 \text{m}, h_{\text{crest}} = -0.4 \text{m MSL} \)
-1 time per overwash event: 3 hours

The yearly sand loss due to tide-and wave-induced overwash is then:

\[
S_{\text{tot}} = 10 \times 3 \times 3600 \times 0.34 \times 10^3 = 36.7 \text{ m}^3/\text{yr/m}
\]

When overwash occurs on a sandbar with a length of 5km, the yearly sand losses become

\[
S_{\text{tot}} = 36.7 \text{ m}^3/\text{yr/m} \times 5000 \text{m} = 183,600 \text{ m}^3/\text{yr}
\]
CHAPTER 6

Quantification of sand losses due to washover on the Wai San Ting sandbar, Taiwan
6. Quantification of sand losses due to washover on the Wai San Ting sandbar, Taiwan

6.1 Introduction

The huge intertidal sand bar (15*4km), the Wai San Ting, which is located at the west coast of Taiwan, experiences heavy sand losses caused by several erosion processes. One of these processes is the washover current, which transports the eroded sand from the shoreface over the crest of the sandbar to the lee-side, where the sand is deposited.

The reason for examining the washover process on the Wai San Ting sandbar is that this sandbar is retreating towards the shore rapidly caused by longshore and cross-shore sand losses. Due to the fact that the Wai San Ting sandbar protects the south laying coastline against the wave attack from the predominant NE directions, one wants to know what happens in the future when the sandbar is moving to the shore and looses its function as a natural sea defence. Another reason for the understanding of the migration process of this sandbar is that the shallow intertidal areas in front of the dykes of the south laying coast and between the dykes and the sandbar are heavily exploited for oyster cultivation. When these areas become smaller caused by the movement of the sandbar the oyster beds will vanish and the harvesting of the oysters will be disturbed.

In a hydraulic and morphological study the morphodynamics of the Wai San Ting sandbar were examined. From this study it was considered to take a more detailed look at the contribution of washovers to the total sand losses of this sandbar, because very little was known about the processes which are responsible for a washover event.

In Appendix XI-1 the 2-D wave modelling and in Appendix XI-2 the 2-D flow modelling on and around the Wai San Ting sandbar is described. The 2-D flow model MIKE21_HD and the 2-D wave model MIKE21_NSW are applied to get more insight in the 2-D effects of the washover process and to model a real situation of a tide and wave-induced overwash, which can occur two time a day at spring tide. To model the washover flow from the surf zone, over the sandbar to the lee-side, a very fine grid model is used (20m*20m). The results of the 2-D flow and wave modelling are used to specify the boundary conditions for the 1-D overwash model.

This chapter is built up as follows. First in section 6.2 a description of the study area is given. The tide and wave conditions at the site are described in section 6.3. The results of this tide and wave analysis are used to specify the boundary conditions for a 2-D near shore wave and 2-D hydrodynamic model.

In section 6.4 the 1-D overwash model is applied to quantify the yearly sand losses for a specific cross section of the Wai San Ting. Two schematized conditions are modelled: summer spring tide and winter spring tide with waves. An analysis is made of the profile development.
Finally in section 6.5 the contribution of washover transport to the total sand loss is discussed and the relative importance of washover compared with other transport mechanisms for this case study are described.

6.2 Description of study area

The Wai San Ting barrier is located along Yunlin coast at the mid-west coast of Taiwan (Figure 80), between the Choshui River and Kuosheng Kang (Figure 81). This barrier island is formed on an old relict river delta. The Wai San Ting is actually a transgressive barrier spit, which is characterized by a movement caused by a sand deficiency. Across the bar a couple of tidal inlets connect the foreshore with the shallow area at back side of the bar. The barrier is moving (rotating) towards the coastline at a speed of 40 m/year at the seaward end of the barrier, measured from several satellite charts and survey data. This feature is chosen for examination for its function as a sea defence, protecting the coastline in the lee-side of the sandbar. Estimations show that, over one and a half century, when the Wai San Ting reaches the coastline due to the erosion processes at the front side of the bar, the poor protected coast behind the bar will then be exposed directly to the waves and currents, which can cause severe flooding in the overcrowded area behind the coast.

The Wai San Ting sandbar is partly emergent at high tide with the +2m TD level being reached only in a small area close to the seaward end. The seaward slope of the sandbar experiences full exposure to the monsoon waves, particularly near the southern end of the feature where the wave attack is most oblique. Several channels connect the foreshore with the backshore area. Particularly in the northern part, small washover deltas can be seen at the backshore side of these channels. These overwash deltas indicate sediment transport through the channels to the backshore. The lee-side of the sand bar is a gently sloping and wide intertidal area, protected from the waves and currents (Fig. 82). This intertidal sandbar is undeveloped except a small lighthouse near the southern end where the elevation ensures dry land at high water over a small area. Many small boats use the channels which cut across Wai San Ting close to its northern end.

Morphodynamics

The total surface area of the Wai San Ting that is outlined on various map overlays appears to be fairly consistent from year to year and this suggests that the sandbar as a whole may be moving (eg. by washover flows and breakthrough) or there is continual build up of fine material due to weaker transport from south to north in the lee of Wai San Ting.
Sediment loss due to washover on a barrier island

Figure 80: Study area: Taiwan
Figure 81: Study area: Yunlin coast (West coast Taiwan)
Figure 82: Study area: Wai San Ting sandbar
The morphological origin and development of this large barrier island was examined during a hydraulic and morphological study\textsuperscript{10} at the west coast of Taiwan (Yunlin county). In this study historical photographs and satellite imagery were compared and the different mechanisms that cause sediment loss of this barrier were studied. From the analysis of the profile measurements\textsuperscript{11} taken in October 1990, October 1991 and October 1992 and from satellite imagery it could be estimated that the shoreface of the Wai San Ting sandbar experienced an average yearly landward retreat between 40 and 100 m/yr (between the 0 m and the -10 m TD\textsuperscript{12} contour line at the seaward end). Sand balance results, based on cross shore profile data, indicated that for each year ±7.5 million m\textsuperscript{3}/yr of sand was lost for a 10 km stretch between 0 and -10 m LLW (-1 to -11 m TD).

Based on the quantifications of several major mechanisms acting over the 10 km section, it is estimated (in study) that (1) the longshore transport losses through the southern end, (2) the southward losses through the channels which cross the sandbar and (3) the washover losses during high water and storms are each responsible for about one third of the volume loss. Minor mechanisms are offshore transport (undertow and rip currents), subsidence and compaction, wave asymmetry induced onshore transport, wind transport etc.

The above made estimation of the sand losses due to the three main transport mechanisms shows that the contribution of washover to the sediment losses should not be underestimated. In the estimated washover transport rate no extreme events have been considered, neither the effect of wave set-up. Various case studies described in the literature study showed that one extreme event with high surges and high waves can contribute significantly to the yearly washover transport.

6.3 Site conditions: tide and wave analysis

6.3.1 General

The weather in the area is dominated by the two Southeast Asia monsoon conditions. In the winter from October to April, the northeast monsoon persistently causes strong NNE winds and produces a predominantly NNE wave climate. During the summer the southeast monsoon can be characterized as mild and produces moderate winds and a very mild wave climate at the site except for periods when typhoons occur. These typhoons produce very strong winds from random directions and hence high typhoon waves can approach from a wide range of directions, except waves coming from the west because of the sheltering by the Peng Hu islands.

\textsuperscript{10}: Frederic R. Harris B.V. & Sinotech Engineering Inc.

\textsuperscript{11}: taken by the Institute of Harbour and Marine Research and Technology (IHMRT)

\textsuperscript{12}: Taiwan Datum = Mean Sea Level
The Cho Shui River (Figure 81) in the north of the site experiences very large but short duration discharges (2 to 5 days) associated with heavy typhoon rainfall, particularly in the late summer months. The other main river at the site is at its southern boundary, the Peikang River, with a much smaller catchment and discharge flow.

6.3.2 Tides and surges

Tides propagate around the north and south of Taiwan from east to west, causing a convergence in the Taiwan Straights. As a result, the tidal amplitudes are greatest near the convergence on the mid-western side of Taiwan and least at the extremities of the islands. The tidal range across the site can be seen to differ due to those effects with a tidal range (spring tide) of 2.1m in the south and 4.3m in the north of the mid-west coast. The tide type is predominantly semidiurnal, but with a clear daily inequality between consecutive high (and low) waters. This is shown by Figure 83, which is a plot of spring tide in March, at location "A" in Figure 82. At Santiao lun the maximum water level in 8 years was +2.37 m TD. Above this level a surge of +1.0 m (return period of 10 years) must be added to obtain the maximum level which occurs once in 10 years.

The present currents at the site are the result of ocean currents, tides, winds and waves. Due to the fact that washover currents are primarily caused by tides and waves, the ocean and wind driven currents are not discussed here. These currents are reviewed in the Working Paper No. 3 (1993)\textsuperscript{13}. The wave driven currents are discussed in 6.4.1. The tidal currents head alongshore to the NNE in flood and reverse in the ebb. This pattern is being repeated twice a day with peak speeds of about 1 to 1.5 m/s during spring tide.

6.3.3 Waves and winds

Wind and waves measurements where obtained from several stations located in the study area. Figure 80 and 81 show the positions of the gauges in relation to the site. Both the Tong Chi and CBK-11\textsuperscript{14} wave gauges were located offshore in relatively deep water, where as the gauges in Taihsi and Wai San Ting were placed in shallower water close to the shore. Non of the stations reported directional wave measurements. From the above stations, Tong Shi provided the most complete data set and covered a considerably longer time period than the others.

In the Working Paper No. 3 it was concluded that data obtained from stations Tong Chi and CBK-11 are representative of the offshore wave and wind climate for the predominant NNE conditions, since the stations effectively bracket the area being just north and south of the site.

\textsuperscript{13}: Frederic R. Harris B.V. & Sinotech Engineering Inc.

\textsuperscript{14}: China Petroleum Corporation offshore platform, CBK-11
Figure 83: Tidal variation (March '92) from neap to spring tide (location "A" Fig. 82)
Winds

The records at Tong Chi and CBK-11 showed a very strong correlation between deep sea wave heights and wind speeds. Therefore, in the absence of any wave direction recordings, it was decided to use deep sea wind observations. For each wave recording the wave direction was assigned to be that of the wind direction simultaneously measured at the station.

As it is the closer offshore gauge to the site, Tong Chi was selected as the guide to offshore boundary conditions. Table A-2 in Appendix IX shows the wind directional distribution at Tong Chi and CBK-11, recorded over several 12 month periods. The directional distribution is graphically presented by Figure 84. It can be seen that the most time the wind is blowing from northerly directions (66% time/year), and that the predominant wind direction is from NNE (46.5%).

Waves

Table A-3 in Appendix IX presents the yearly wave climate frequency of occurrence of wave height and associated weighted average significant period based on the measurements at Tong Chi (all directions combined). Included in this wave climate are a number of typhoon events each year. It should be noted that the Penghu Islands shelter the wave gauge for wave directions from WSW to NNW, but that the wind does not often blow from this quarter (±9% of the time), the fetch is rather short and from most of this direction range the Penghu Islands shelter the Yunlin Coast.

The monsoon waves are represented by the N, NNE, NE and ENE sectors but predominantly by NNE (shown in Table A-2, Appendix IX). Of all the wave heights larger than 1 meter, about 70% are from NNE+N, these waves having periods (T_{1/3}) of 7 seconds (Table A-3, Appendix IX). Figure 85 shows a graph of the yearly wave climate.

A directional distribution for deep sea waves was derived by assuming the deep sea wave direction for each recording was the same as the wind direction recorded simultaneously at the station.

The schematization of the operational wave conditions (wave height and direction), performed to make them suitable for use as boundary conditions for the 2-D wave modelling, is described in Appendix IX.

Typhoon conditions

Typhoons play an important in the study area. Between 1981 and 1992 an average of 4 typhoons per year was recorded. The significant wave height caused by the typhoons varied from 1 to 7 m and the average wave period was 9s. The direction from where the typhoons strike the coast line of Taiwan varies. The duration of these super storms is about 3 to 5 days.
Sediment loss due to washover on a barrier island

Wind Directional Distribution
Tong Chi; Overall yearly statistics

Yearly Wave Climate
Tong Chi; Measured from 1981 to 1988 (all directions)
6.4 2-D wave and flow modelling

Purpose of the 2-D modelling

The first purpose of the 2-D modelling is to determine the two-dimensional effect which occur during a tide and wave-induced washover and which are mainly caused by the irregular bathymetry. These 2-D effects cannot be modelled with the 1-D overwash model. Also the time varying overwash currents due to the tide must give a clear picture of the different stages of the overwash process, caused by the tidal water level variation.

The second purpose of the 2-D modelling is to obtain wave and tide conditions which are used for the 1-D model boundary conditions. The time varying water levels at both sides of the bar are transferred from one or more points in the 2-D model and are used by the 1-D overwash model to schematize a tide-induced overwash event.

2-D model strategy

With the determined site conditions in section 6.2 and 6.3 (bathymetry, winds, waves, tides) first the wave conditions on and around the Wai San Ting sandbar are modelled. The wave deep sea wave conditions are transferred to the Wai San Ting area by using different model sizes. The largest model covers an area of more than 90km*100km and has a rectangular grid size of 100m*400m, while the smallest model covers a part of the Wai San Ting sandbar and measures 5km*10km with a grid of 10m*40m. Just one wave condition is modelled. This condition occurs during the NE (winter) monsoon and represents waves and winds which will have a significant influence on the erosion process of the Wai San Ting sandbar.

The flow modelling is carried out by using four different model sizes, which are needed to transfer the tidal motion from the Taiwan Straight to the intertidal area around the Wai San Ting sandbar. The smallest model measures 2km*8km with a grid size of 20m*20m. This fine grid is necessary to compute the overwash currents and water surface elevations across the crest of the bar in detail. The radiation stresses computed by the wave model are used in the flow model to determine the wave set-up and wave-induced currents.

The wave and flow conditions during spring tide at both sides of the sandbar are used as boundary conditions for the 1-D overwash model.

The 2-D wave modelling is described in Appendix XI-1 and the description of the 2-D flow modelling is given in Appendix XI-2.

2-D modelling conclusions

The 2-D effects of the tide-induced overwash are restricted to the areas just in front of the bar, and at the lee-side of the bar. At the front side the interaction longshore current/overwash current causes a change in current direction. Also at the end of the crest, where the overwash
current flows into deeper water, the current direction is not shore normal. However, on the crest the current does flow almost perpendicular to the length-axis of the bar, which shows that the 1-D approach, which assumes perpendicular overwash currents, is correct.

The direction of the waves, which is assumed to be shore normal in the 1-D overwash model, is in the 2-D wave model also nearly shore normal, caused by refraction. This is the case for deep sea waves which approach the bar from the NNE and refract in shallow water in front of the bar, until they reach the bar almost shore normal (NNW). It can be concluded that (in the Wai San Ting sandbar situation) waves approaching the bar from northerly directions will refract to a shore normal direction. Waves from westerly directions will refract more than the waves from northerly directions, caused by the orientation of the sandbar (SW). This causes a decrease in wave set-up due to the diminished wave energy, which is spread out over a larger area than in case no refraction occurs. In the 1-D model the wave angle cannot be specified and no refraction can be computed. The influence of refraction on the wave set-up can be included by specifying a smaller wave height at the upstream boundary.

During a wave-induced washover the current direction at the front of the crest varies, caused by the irregularities of the bottom (submerged longshore bars). The wave set-up and the overwash current therefore vary along the crest. This results in a different angle of the overwash current just in front of the bar. On the crest and at the lee-side the wave-induced current flows almost perpendicular to the length-axis of the bar.
6.5 Quantification of sand losses due to washover

In this section an attempt is made to quantify the yearly sediment losses caused by tide and wave-induced overwash. The summer and winter boundary conditions at the front and back side of the Wai San Ting sandbar (H, T, h_{front}, h_{back}) are obtained from the 2-D modelling, both described in Appendix XI. With the known overwash current velocities, wave properties and the bottom profile the barrier profile development can be examined.

6.5.1 1-D modelling of cross section Wai San Ting

Initial bottom profile

Before the sediment transport caused by washover can be determined using the 1-D overwash model a schematization of the bottom profile of the Wai San Ting sandbar is made. Figure 86 shows a 3-D plot of the bathymetry of the section of the Wai San Ting (20m grid size model Appendix XI). From this Figure the typical washover profile can be observed; a relative steep shoreface with breaker bars at the front (north) side and an almost horizontal crest and a very gentle bottom slope at the lee-side of the bar. Figure 87 shows 5 cross sections at an interval of 500m along the x-axis of the 20m flow-model. The thick line represents the schematized barrier profile, which is “averaged” over the 5 profiles. At the front (right side of Figure 87) a longshore breaker bar is located at a depth of approximately -4m TD. This longshore bar is schematized because it may has an effect on the wave height. The waves approach the bar from the right. The highest waves will break on this bar. Between the bar and the crest the bottom slope has a steepness of 1:50. The crest is schematized as flat at an elevation of +0.7m TD. The width of the crest is about 500m. At the lee-side of the crest the profile levels off gently with a slope of 1:2000. The bottom roughness is constant 0.02m.

Schematization yearly tide conditions

The tidal conditions modelled in the 2-D flow modelling are used to specify the boundary conditions. It is considered that washover only occurs at spring tide at HHW. A second schematisation is made concerning the seasonal conditions; one year has only two seasons: a winter and a summer season.

Up till now overwash occurs during summer spring tide and winter spring tide. At this stage of the tidal schematisation the average HHW and LHW in the summer (July) and winter (March) and the number of washovers must be determined. Figure 88 is a plot of the tidal variation from spring to neap tide taken at location “A” in Figure 82. Figure 89 and 90 show the tidal variation of summer and winter spring tide just in front of the sandbar (solid line) and at the back side of the bar (dotted line). These tidal elevations are computed with the 20m grid size flow model at locations T1 and T2, shown in Figure 87. In Figures 89 and 90 the crest height is plotted at +0.7m TD. The number of washovers is determined by counting the number of times the tidal elevation exceeds the height of the crest during spring tide. This
Sediment loss due to washover on a barrier island

Figure 86: 3-D bathymetry plot of section Wai San Ting (20m flow model)
Cross sections Wai San Ting sandbar
interval 25 grid points (=500 m)
Taiwan

Fri Oct 22 1993
nico
name: t1sec5x

Frederic R. Harris
Engineers, Planners,
Economists & Consultants

Sediment loss due to washover on a barrier island
Figure 88: Tide from spring to neap (July '92); tidal variation in front/behind the bar
Figure 89: Tide from spring to neap (March '92); tidal variation in front/behind the bar
Sediment loss due to washover on a barrier island

The duration of an overwash event is specified in the Figures 91 and 92. In that time period the average HHW\textsuperscript{15} and average LHW\textsuperscript{16} is determined for the tide ranges in front and at the lee-side of the bar. The number of overwash events per year is determined as follows:

- per "14-days" tide cycle : 7 * overwash
- per year : 26 "14-days" tide cycles = 182 * overwash
- per summer (1/3 year) : 60* overwash
- per winter (2/3 year) : 120 * overwash

Now the average time per overwash need to be known. Figure 90 shows a close up of a spring tide in July. The overwash duration is determined by the period the crest is submerged, in this case about 4 hours. It is obvious that when the elevation is not as large as in this situation, the overwash duration will decrease. This is shown in Figure 91, where the tidal elevation in front and behind the barrier is plotted during LHW in March. Now the overwash duration is about 3 hours. In general it can be concluded that the higher the tidal elevation relative to the crest height, the longer the overwash duration (and the other way around). This can be repeated for several tidal elevations during LHW and HHW and during summer and winter. The average overwash duration is then estimated at 3 hours. From the Figures 90 and 91 it can be seen that during these 3 a 4 hours the water level difference is more or less constant. Therefore in this study an average water level at the front and back side of the bar is taken during an overwash. These averaged water levels are plotted in Figure 90 and 91.

When for every "overwash peak" in the tidal variation (Figures 88 and 89) the average water levels in the front and at the back side of the back are determined, for winter and summer conditions and for the front and the back side of the bar, the average LHW and HHW can be found. The results are shown in Table 13.

Table 13: Schematisation yearly tidal conditions

<table>
<thead>
<tr>
<th>Tidal elevation</th>
<th>Winter</th>
<th>Summer</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>front side</td>
<td>back side</td>
</tr>
<tr>
<td># HHW's</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td># LHW's</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td># overwash/yr</td>
<td>120</td>
<td></td>
</tr>
<tr>
<td>average HHW</td>
<td>1.07</td>
<td>0.89</td>
</tr>
<tr>
<td>average LHW</td>
<td>0.92</td>
<td>0.75</td>
</tr>
</tbody>
</table>

\textsuperscript{15}: Highest High Water

\textsuperscript{16}: Lowest High Water
Sediment loss due to washover on a barrier island

Figure 90: Highest HW spring tide July '92; tidal variation in T1 and T2
Sediment loss due to washover on a barrier island

Schematisation yearly wave conditions

In the wave and wind analysis in section 6.3 it was determined that during the NE monsoon (winter) the predominant wave direction is NNE, which will occur for about 50% of the year. The waves from the NE and N, with each an yearly occurrence of about 10% are also assumed to contribute to the yearly tide+wave-induced washovers. These NNE, NE and N waves are used as tide+waves overwash boundary conditions in the 1-D overwash model to determine the contribution of the tide and waves to the yearly washover sand loss on the Wai San Ting sandbar. The wave conditions for the NNE, NE and N direction are schematized in Table 14:

Table 14: Schematisation wave conditions (NNE+NE+N)

<table>
<thead>
<tr>
<th>H_s</th>
<th>T_p</th>
<th>% occurrence</th>
<th>% occurrence</th>
<th>% occurrence</th>
<th>% relative distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>NNE</td>
<td>NE</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>calm</td>
<td>-</td>
<td>1.26</td>
<td>0.28</td>
<td>0.22</td>
<td>2.7</td>
</tr>
<tr>
<td>1.0</td>
<td>4.5</td>
<td>24.80</td>
<td>5.54</td>
<td>4.32</td>
<td>53.3</td>
</tr>
<tr>
<td>2.5</td>
<td>7.8</td>
<td>15.40</td>
<td>3.44</td>
<td>2.68</td>
<td>33.1</td>
</tr>
<tr>
<td>3.5</td>
<td>8.5</td>
<td>4.40</td>
<td>0.99</td>
<td>0.77</td>
<td>9.5</td>
</tr>
<tr>
<td>5.0</td>
<td>10</td>
<td>0.64</td>
<td>0.15</td>
<td>0.11</td>
<td>1.4</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td>46.50%</td>
<td>10.40%</td>
<td>8.10%</td>
<td>100%</td>
</tr>
</tbody>
</table>

It is assumed that the waves from the NE (10.4%) and N (8.1%) have the same wave height distribution as the NNE waves (last column in Table 14). The total yearly occurrence for waves coming from these three directions is 66%. This is 2/3 of a year.

The total yearly wave and tide conditions are finally specified as:

\[ \text{2/3 year winter} \quad : \quad \text{waves and tide (March conditions)} \]
\[ \text{1/3 year summer} \quad : \quad \text{tide only (July conditions)} \]

Schematisation yearly Typhoon climate

The contribution of typhoons to the yearly washover transport depends on many factors:

- the number of typhoons per year
- the magnitude of the storm (wave height, wave period, duration)
- the direction of the typhoon relative to the barrier orientation
- the possibility that a typhoon occurs at spring tide
- typhoon at spring or neap tide.
In an overwash situation with high tides the effect of a typhoon, coming from the N or NNE (perpendicular to shore face Wai San Ting) can be significant. The water level will then be elevated due to the tide, wave set-up and wind set-up. The wind set-up can be significant, especially when a typhoon strikes the coastline for 3 or 4 days. An example of a typhoon scenario can be described as follows:

$$\begin{align*}
\Rightarrow h_{\text{crest}} & : +0.8 \text{ MSL} \\
\Rightarrow \text{spring tide}_{\text{front}} & : +1.2 \text{ MSL} \\
\Rightarrow \text{spring tide}_{\text{back}} & : +1.0 \text{ MSL} \\
\Rightarrow \text{wave set-up} & : 0.20 \text{ m} \\
\Rightarrow \text{wind set-up} & : 0.20 \text{ m} \\
\Rightarrow H_s & : 3.50 \text{ m} \\
\Rightarrow T_{p,n} & : 9 \text{ s}
\end{align*}$$

The water level at the front side of the bar in this case is lifted up to $+1.8$ m MSL. The crest of the bar is at $0.8$ m MSL, which means that the crest is 1 m below the water surface. The wave set-up only occurs at the front side of the bar. The water level difference across the bar then becomes $(1.6 - 1.2) = 0.40$ m! This water level difference, which causes high overwash current velocities, in combination with high waves, will erode the front side of the bar and causes large sand losses.

In the Working Paper No. 3 the typhoon wave climate is schematized as follows:

- typhoons per year : 5  
- average duration per typhoon : 72 hours 
- wave period $T_p$ : 10 s  
- directions : N (2 typhoons), NNE (1), SSW (2).

The contribution of typhoons to the total yearly sand losses of the Wai San Ting is represented by the schematized wave conditions (Table 14; $H_s=3.5$ m and $H_s=5.0$ m) and will therefore not be determined separately. In section 6.5.3 the retreat of the Wai San Ting shoreline caused by a typhoon is computed to show the contribution of a typhoon event to the total yearly sand loss due to washovers.
6.5.2 Quantification of yearly sand losses

The yearly wave and tide conditions are schematized in section 6.5.1 as:

- 1/3 year summer: tide only (July conditions)
- 2/3 year winter: waves and tide (March conditions)

It is assumed that in the summer only tide-induced washovers will occur and in the winter only combined tide and wave-induced washovers.

Sand losses due to tide-induced overwash

The sediment losses due to tide-induced overwash, which occurs during the one third of a year (summer), is determined by measuring the sediment transport capacity during a HHW and a LHW. In Figure 92 the washover situations due to the averaged HHW and LHW is shown. It can be seen that the maximum velocities occur at the end of the crest, which implies that the maximum sediment transport will occur there. This is confirmed by Figure 93, which is a plot of the sediment transport curves across the bar during a HHW and LHW, computed after 24 hours of overwash. The initially "sharp" trapezium barrier profile is then a bit more adjusted to the overwash conditions. In the HHW overwash condition erosion occurs from x=100m to x=-500m and accretion from x=-500m to x=-650m.

The sand losses are defined by the sediment transport which occurs on the location of the crest where the gradient in sediment transport is zero (no erosion or accretion). Looking at Figure 93 it can be seen that these locations are x=-490m for the HHW washover and x=-500m for the LHW washover. The found sediment transport rates are given in Table 15.

In section 6.5.1 it was determined that the number of washovers caused by HHW and LHW is 60 per 1/3 year. From these 60 tide-induced overwash event 30 washovers occur during HHW and 30 washovers during LHW. When the average overwash duration is estimated at 3 hours, the following sediment losses for 1/3 year (summer) can be determined (Table 15):

<table>
<thead>
<tr>
<th></th>
<th>$S_{in}$ per washover (m$^3$/s/m)</th>
<th># washover per summer</th>
<th>$S_{ou}$ per summer (m$^3$/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HHW washover</td>
<td>$0.13 \times 10^4$</td>
<td>30</td>
<td>0.04</td>
</tr>
<tr>
<td>LHW washovers</td>
<td>$0.17 \times 10^4$</td>
<td>30</td>
<td>0.06</td>
</tr>
<tr>
<td>TOTAL</td>
<td>$0.30 \times 10^4$</td>
<td>60</td>
<td>0.10</td>
</tr>
</tbody>
</table>

Table 15: Tide-induced sediment losses in a 1/3 year
Surface elevation & velocity tide-induced overwash during HHW and LHW (Wai San Ting sandbar)

![Graph showing surface elevation and velocity overwash](image)

Sediment Transport: Tide-induced washover during HHW & LHW (Wai San Ting sandbar)

![Graph showing sediment transport](image)

**Figure 92**

**Figure 93**
Surface elevation & velocity tide-induced overwash during HHW and LHW (Wai San Ting sandbar)

Figure 92

Sediment Transport: Tide-induced washover during HHW & LHW (Wai San Ting sandbar)

Figure 93
Sand losses due to combined tide + wave-induced overwash

As determined in section 6.5.1 the other 2/3 of the year the tide and the waves coming from the N, NNE and N are together responsible for overwash. The tide conditions at both sides of the bar are given in Table 13 and the wave conditions at the north side of the Wai San Ting sandbar are given in Table 14. In this tide and wave situation not only the tidal water levels are specified at both model boundaries, but also the wave conditions at the upstream boundary. The yearly sand losses are now determined by 120 washovers due to the tide (HHW and LHW) in combination with four different wave conditions, which contribute to the sand losses in relation to their yearly occurrence. Therefore 8 different overwash conditions must be computed to determine the yearly contribution of tide and wave-induced overwash.

It is assumed that the N, NE and NNE waves approach the barrier shore perpendicular. This assumption is allowed when it is considered that the waves from these three directions will refract even before the surf zone is arrived due to the shallower areas around the Wai San Ting sandbar, compared to the deeper sea from where the waves are coming (and where the wave measurements were taken). Waves from these three directions will occur about 65% per year (=2/3 of a year).

In Figure 94 and 95 these 8 conditions are presented as surface elevations and overwash velocities (4 HHW conditions and 4 LHW conditions). The conditions are computed after 3 hours of tide and wave-induced overwash for the "warming-up" of the model (initial profile reshaped a bit). It can be seen that the waves contribute to the total water level difference across the bar due to the wave set-up. The corresponding sediment transport curves are plotted in Figure 96 for HHW conditions and in Figure 97 for LHW conditions. From these figures it can be seen that from x=400m to x=0m erosion occurs, mainly caused by the waves. From x=0m to x=-300m a part of the sediment is deposited on the crest and from x=-300m to x=-500m again sand is eroded from the bar and is deposited at the back side (x=-500m to x=-1200m).

The sediment loss of the bar is here defined as the sediment transport which occurs at the beginning of the horizontal crest (x=0m). From the breaker line to this point only erosion take place; sand is eroded from the shoreface and is deposited on top of the crest and/or is transported by the overwash current to the back side of the bar. Thus erosion from the shoreface is here considered as "sediment loss". The sediment losses per wave and tide condition per winter (=2/3 year) are presented in Table 16. The determined total sand loss in that half year is about 80 cubic meters per meter length of the sandbar. Compared to these losses the tide-induced washover losses can be neglected. The 2.5m wave height has the largest contribution to the sediment losses.

From the above computed sand losses due to tide and wave-induced washovers it can be concluded that

the total yearly sand loss due to washovers on the Wai San Ting sandbar driven by tides and waves is 80 m³/m.
Surface elevation & velocity tide+wave-induced overwash
Winter: during HHW (Wai San Ting sandbar)

Figure 94

Surface elevation & velocity tide+wave-induced overwash
Winter: during LHW (Wai San Ting sandbar)

Figure 95
Sediment Transport: Wave+Tide-induced washover

During HHW

[Graph showing deposition and erosion changes with varying wave heights]

Figure 96

Sediment Transport: Wave+Tide-induced washover

During LHW

[Graph showing deposition and erosion changes with varying wave heights]

Figure 97
Table 16: Tide & wave-induced sediment losses (winter)

<table>
<thead>
<tr>
<th>Condition</th>
<th>$S_{m}$ per washover (m$^3$/s/m)</th>
<th># washover per 2/3 year**</th>
<th>$S_{m}$ per 2/3 year (m$^3$/m)***</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calm; HHW</td>
<td>0.13 * 10$^{-6}$</td>
<td>1.62</td>
<td>0.00</td>
</tr>
<tr>
<td>Calm; LHW</td>
<td>0.17 * 10$^{-6}$</td>
<td>1.62</td>
<td>0.00</td>
</tr>
<tr>
<td>Hs = 1.0m; HHW</td>
<td>0.06 * 10$^{-3}$</td>
<td>31.98</td>
<td>20.72</td>
</tr>
<tr>
<td>Hs = 1.0m; LHW</td>
<td>0.02 * 10$^{-3}$</td>
<td>31.98</td>
<td>6.91</td>
</tr>
<tr>
<td>Hs = 2.5m; HHW</td>
<td>0.11 * 10$^{-3}$</td>
<td>19.86</td>
<td>23.59</td>
</tr>
<tr>
<td>Hs = 2.5m; LHW</td>
<td>0.04 * 10$^{-3}$</td>
<td>19.86</td>
<td>8.58</td>
</tr>
<tr>
<td>Hs = 3.5m; HHW</td>
<td>0.16 * 10$^{-3}$</td>
<td>5.70</td>
<td>9.85</td>
</tr>
<tr>
<td>Hs = 3.5m; LHW</td>
<td>0.08 * 10$^{-3}$</td>
<td>5.70</td>
<td>4.92</td>
</tr>
<tr>
<td>Hs = 5.0m; HHW</td>
<td>0.32 * 10$^{-3}$</td>
<td>0.84</td>
<td>2.90</td>
</tr>
<tr>
<td>Hs = 5.0m; LHW</td>
<td>0.19 * 10$^{-3}$</td>
<td>0.84</td>
<td>1.72</td>
</tr>
<tr>
<td>TOTAL</td>
<td>120</td>
<td>79.46</td>
<td></td>
</tr>
</tbody>
</table>

Tide-only situation, see Table 15

Relative distribution % per 2/3 year, * # HHW's (60) or # LHW's (60); see Table 14

# washovers per 2/3 year * $Stot * 3 * 3600s

When the length of the barrier exposed to washover is estimated at 10km (see Figure 82), the total sediment loss of 0.8 million m$^3$ can be found. Estimates based on satellite pictures and field measurements showed that the total sand loss of the Wai San Ting sandbar is between 4 and 7 million cubic meters. The contribution of the washover losses to this total volume of sand losses is then 11 to 20%. However, the real washover losses can be lower or higher than the found 0.8 million m$^3$. The differences can be caused by the following items:

- Schematization of the tide and wave conditions: the tidal elevations will show more variations in time due to winds and storm events than the 4 used HHW's and LHW's in the summer and winter.

- Schematization of the barrier profile: due to a lack of data of the crest geometry the crest if assumed horizontal, while in reality the crest can have a small positive or negative slope, which will have a significant effect on the washover transport. Another important aspect is the chosen cross-section of the Wai San Ting sandbar for the 1-D model. It is assumed that the sediment losses are equal at all cross sections along the sandbar, while in reality larger losses can occur in the more seaward areas of the bar and less overwash will occur at cross section more to the mainland shore.
Also the exact crest height could not be determined while this is an evident aspect of the overwash process. The chosen crest height of +0.70m TD will be too high or too low at certain parts along the bar. When the real crest is lower, then the effect of washover on the sand losses will increase significantly.

- **1-D approach:** the effect of refraction and diffraction of the waves is not included in the model although the assumption is made that waves approach the crest almost shore normal are acceptable in certain situations.

- Interaction of the longshore current with the overwash current: sand transported alongshore can be taken over by the overwash current and transported to the back side of the barrier, which will cause even higher sand losses at the front.

- Seasonal and yearly changes of the barrier profile: although the bar is migrating towards the shore the general shape and the crest height remained fairly consistent from year to year. This shows that there must be other mechanisms which "feed" the sandbar, such as on-shore transport by waves, wind blown transport and sand supply from the coastline above the Wai San Ting by littoral transport.

- Storm surges where not taken into account during the computations, while during heavy storms and typhoons the water level set-up against the coast can vary between several centimetres to meters. This will have a dramatic effect on the sand losses.

Despite the above mentioned limitations of the 1-D overwash model the most important aspects of washover are taken into account (tidal variations, irregular waves, irregular bathymetry, profile changes) and the real sand losses will be in the range between 0.5 and 2 million m$^3$, around the losses found with the 1-D overwash model.

### 6.5.3 Wai San Ting cross sectional profile development

The 1-D overwash model is applied again in order to determine the yearly development of a specific profile of the Wai San Ting sandbar. Figure 98 shows the results of the profile changes due to a half year tide and wave-induced overwash events. The wave and tide conditions schematized in 6.5.1 are used here to compute the sediment transport rates which cause the bed profile changes. It can be seen that the front side of the bar retreats at an equal rate up to the 0-meter line. This retreat is purely caused by the waves breaking action at the front side of the bar. The material moved up by the waves is deposited on top of the crest. At the back side also erosion takes place due to the increased overwash current velocity near the end of the crest. It should be noticed that this profile is caused only due to the wave and tide combined overwash. In reality between wave and tide-induced overwash events also tide-induced washovers occur during spring tides. This will reshape the profile a bit, lowering the increased crest level again. However, it was already concluded that the effect of the tide-induced current is much less than the effect of tide and waves combined.
Profile development Wai San Ting sandbar
60 wave+tide-induced washovers (1/3 year)

Figure 98

Profile development Wai San Ting sandbar
60 tide-induced washovers (1/3 year)

Figure 99
This is confirmed by Figure 99, which shows the profile development due to tide only for 60 overwash events (1/3 year). The shoreface remains on its position and the only erosion occurs at the back side.

In general it can be concluded that the profile of this barrier island is determined by the combined hydraulic conditions of tide and waves. The influence of the tide-induced currents will increase when the crest width becomes smaller.

**Retreat shoreface**

The shoreface retreat is an important tool by which the effect of washover on a sandbar can be measured. Satellite images of the position of the Wai San Ting sandbar in different years showed that the maximum shoreface retreat is up to 40m near the southward end of this barrier.

From Figure 98 it can be observed that the retreat of the shoreface due to waves and tides is about 20m. The retreat of the shoreface of the Wai San Ting sandbar due to a 3 washovers during a 2 days typhoon, plotted in Figures 100a and 100b, shows that the effect of a typhoon on a sandbar can be significant. The retreat is here about 4 to 5 m, for this single typhoon! When also the contribution of 2 other typhoons are considered, the barrier shoreface retreat can be easily become 15 to 20 meters, which is significant compared to the total yearly retreat of 40m. The tidal water levels at both side of the bar are determined in section 6.5.1: HHW; winter conditions. The wave conditions are: $H_s = 3.5m; T_p = 10s$.

To compare the reshaping of the profile with measured profile changes, in Figure 101 a plot is given of the profile changes of the Assateague island, located along the Virginia Coast in the south east of the United States (Leatherman, 1976). The profile changes are caused by a major washover event during one storm in 1974. This plot shows that due to one single storm the retreat can be about 10m! Unfortunately no hydraulic and wave conditions at the site were specified so that this barrier island could not be modelled with the 1-D overwash model. It can be seen very clearly that the material which is removed from shoreface is deposited at the back of the barrier, a development which can also be noticed in Figure 100a.
Profile development Wai San Ting sandbar
After 3 washovers during a 2 days typhoon (Hs=3.5m; Tp=10s)

Retreat shoreface Wai San Ting sandbar
After 3 washovers during a 2 days typhoon (Hs=3.5m; Tp=10s)
Sediment loss due to washover on a barrier island

Figure 101: Large-scale displacement of sand landward by overwash activity, caused by the northeaster storm of 1 December 1974, Assateage Island, Virginia Coast, US (Leatherman, 1976)
Study conclusions

This study is carried out to improve the understanding of the washover process and to quantify the washover sand losses on the Wai San Ting sandbar in Taiwan.

The washover process was examined during a literature study. After having analyzed all the available literature and papers about washovers it can be concluded that no theoretical and mathematical approaches were undertaken to describe the overwash process in more detail than the field studies do. Even no hydraulic models were applied to investigate the washover effects on the sediment transport, although a lot of barrier islands around the world are affected by this mechanism.

The overwash theory, which is used to set-up a 1-D overwash model, computes waves, water levels, currents and the sediment transport across a barrier island due to wave and/or tide-induced washover. Although the model uses four iteration steps to solve the complex washover theory, the results of the 1-D overwash model, which were compared with laboratory measurements and other flow and wave models, showed that the right basic formulation were used and that the numerical approach was set-up correctly.

By applying the 1-D overwash model to schematized barrier profiles, it appeared that the downstream water level, which controls the overwash process for a great deal, drops directly behind the barrier crest to the downstream water level due to the increase in water depth. It also determined that the profile of the barrier crest is responsible whether or not overwash takes place and what the impact of the overwash current is on the sand losses of the bar.

Other important parameters affecting the overwash process are the tidal variation at both sides of the bar, the wave set-up, the bottom slope, the crest width and crest height and the bed roughness. A sensitivity study, performed with the one-dimensional overwash model, provided the following conclusions:

- With increasing steepness of the shore the wave-induced overwash current velocities increase.

- When the crest height is lowered the overwash velocities drop in case of a wave-induced overwash situation and rise when the tide is the driving force. When only the tidal water level difference across the bar drives the overwash current, the velocities keep increasing when the crest is lowered and the water level difference across the sandbar is held constant.

- Higher waves cause higher wave set-ups and therefore higher overwash current velocities.

- Larger crest widths slow down the overwash current due to the bottom friction. However, overwash currents flowing over small crest widths (< 100m) can become super critical (with velocities up to 2 m/s), which causes a significant erosion of the barrier.
The 1-D overwash model computes washover conditions one dimensionally on a cross section of a sandbar. To determine the 2-D effects of washover caused by a real bathymetry and 2-D varying tide and wave-induced currents, 2-D wave and flow models were applied on the Wai San Ting sandbar.

The currents patterns of the 2-D flow model showed that the current velocities due to a wave-induced overwash vary along the barrier shore, caused by an irregular bottom profile. The wave patterns showed that the wave direction near the crest during an overwash is almost perpendicular to the length-axis of the bar, caused by refraction. Maximum current velocities that occurred during the wave-induced washover are about 0.4 m/s, caused by a 2.5m wave.

Overwash currents caused by tidal water level variations flow perpendicular over the crest and remain uniform, until the depth behind the bar increases again. Maximum overwash velocities could be determined of 0.5 m/s.

The sediment transport rates caused by tide and/or wave-induced washover on different barrier profiles showed that on sandbars with small crests (<200m) the tide-induced washover causes a general lowering of the crest, while in case waves are involved in the washover process material is removed from the shoreface and is deposited on top of the crest and at the lee-side. The crest therefore becomes locally higher and influences the overwash conditions. The slope of the crest also has a significant effect on the washover transport. A small positive slope diminishes the overwash transport while a small negative crest slope can increase the sediment transport rate by a factor 2.

The sand losses on the Wai San Ting sandbar were determined by applying the 1-D overwash model on a specific cross section of the bar and by schematizing the yearly tide and wave conditions. The found sand losses caused by tide-induced washovers are very small (less than 1%) compared to the tide and wave-induced overwash losses. It was computed that the contribution of overwash on Wai San Ting total 0.8 million m$^3$ over a 10km stretch. A total yearly sand loss of 4 to 7 million m$^3$ was determined by estimates based on satellite images and field measurements. The contribution of washovers to the total sand loss is then 11 to 20%. It can also be concluded that the washover losses during typhoon conditions are significant. Computations showed that after three typhoon overwash events the shoreface was retreated about 5m.

Although the overwash process was studied in detail the determination of the washover sand losses can still be improved. More accurate sand losses due to washover can be obtained when more details of the crest are known and when some 2-D effects are taken into account such as refraction and the longshore current. The estimate can be improved in a real case by good calibration data which would need to include annual and seasonal depth profiles across the bar, waves, flow and water level measurements and a more detailed sediment size.
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<th>Symbol</th>
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<tr>
<td>a</td>
<td>amplitude of a short wave</td>
<td>[m]</td>
</tr>
<tr>
<td>(a_0)</td>
<td>max. horizontal water particle just outside the boundary layer</td>
<td>[m]</td>
</tr>
<tr>
<td>A</td>
<td>spectral wave action density</td>
<td>[-]</td>
</tr>
<tr>
<td>b</td>
<td>Bijker's constant</td>
<td>[-]</td>
</tr>
<tr>
<td>c</td>
<td>wave celerity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>(c_g)</td>
<td>wave group celerity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>(c_r)</td>
<td>intrinsic wave celerity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>C</td>
<td>Chezy coefficient</td>
<td>[m^{1/2}/s]</td>
</tr>
<tr>
<td>(C_{90})</td>
<td>Chezy coefficient related to (D_{90})</td>
<td>[m^{1/2}/s]</td>
</tr>
<tr>
<td>(C_r)</td>
<td>Courant number based on wave celerity</td>
<td>[-]</td>
</tr>
<tr>
<td>(C_{rt})</td>
<td>Courant number based on current speed</td>
<td>[-]</td>
</tr>
<tr>
<td>(d_1)</td>
<td>dimensionless depth ((h/H_{on}))</td>
<td>[m]</td>
</tr>
<tr>
<td>(D_{50})</td>
<td>grain size exceeded by 50% (by weight) of the bed material sample</td>
<td>[\mu m]</td>
</tr>
<tr>
<td>(D_{90})</td>
<td>grain size exceeded by 90% (by weight) of the bed material sample</td>
<td>[\mu m]</td>
</tr>
<tr>
<td>(D_{W})</td>
<td>total wave energy dissipation rate</td>
<td>[N/ms]</td>
</tr>
<tr>
<td>(D_b)</td>
<td>wave energy dissipation due to breaking</td>
<td>[N/ms]</td>
</tr>
<tr>
<td>(D_{bh})</td>
<td>wave energy dissipation due to breaking of one wave</td>
<td>[N/ms]</td>
</tr>
<tr>
<td>(D_f)</td>
<td>wave energy dissipation due to bottom friction</td>
<td>[N/ms]</td>
</tr>
<tr>
<td>(DWD)</td>
<td>maximum deviation from mean wave direction ((MIKE21_NSW))</td>
<td>[^]</td>
</tr>
<tr>
<td>(E)</td>
<td>wave energy per unit surface area</td>
<td>[N/m]</td>
</tr>
<tr>
<td>(f)</td>
<td>wave frequency</td>
<td>[1/s]</td>
</tr>
<tr>
<td>(f_p)</td>
<td>peak wave frequency</td>
<td>[1/s]</td>
</tr>
<tr>
<td>(f_w)</td>
<td>friction factor ((Swart, Johnsson))</td>
<td>[-]</td>
</tr>
<tr>
<td>(g)</td>
<td>acceleration due to gravity</td>
<td>[m/s^2]</td>
</tr>
<tr>
<td>(h)</td>
<td>water depth below still water level</td>
<td>[m]</td>
</tr>
<tr>
<td>(h_{crest})</td>
<td>water depth on top of the crest</td>
<td>[m]</td>
</tr>
<tr>
<td>(h_{new})</td>
<td>(new) water depth, after adjusted bottom profile</td>
<td>[m]</td>
</tr>
<tr>
<td>(h_{old})</td>
<td>(old) water depth, before adjusted bottom profile</td>
<td>[m]</td>
</tr>
<tr>
<td>(h_{tot})</td>
<td>total water depth ((surface elevation+depth below still water level))</td>
<td>[m]</td>
</tr>
<tr>
<td>(h')</td>
<td>water surface elevation above still water level</td>
<td>[m]</td>
</tr>
<tr>
<td>(h'_0)</td>
<td>initial water surface elevation ((at\ boundary))</td>
<td>[m]</td>
</tr>
<tr>
<td>(h'_{front})</td>
<td>water surface elevation at front side barrier</td>
<td>[m]</td>
</tr>
<tr>
<td>(h'_{back})</td>
<td>water surface elevation at back side barrier</td>
<td>[m]</td>
</tr>
<tr>
<td>(h'_{min})</td>
<td>minimum water surface elevation ((breaker line))</td>
<td>[m]</td>
</tr>
<tr>
<td>(h'_{max})</td>
<td>maximum water surface elevation ((at\ beginning\ crest))</td>
<td>[m]</td>
</tr>
<tr>
<td>(h''_\text{in})</td>
<td>input value in iteration process 1-D overwash model</td>
<td>[m]</td>
</tr>
<tr>
<td>(h''_\text{out})</td>
<td>output value in iteration process 1-D overwash model</td>
<td>[m]</td>
</tr>
<tr>
<td>(h''_{max})</td>
<td>water surface elevation at last grid point (i) ((downstream\ boundary))</td>
<td>[m]</td>
</tr>
<tr>
<td>(h''_{bound})</td>
<td>input water surface elevation at downstream boundary</td>
<td>[m]</td>
</tr>
<tr>
<td>(h'_\star)</td>
<td>dimensionless water surface elevation ((h'/H_{on}))</td>
<td>[-]</td>
</tr>
<tr>
<td>(H)</td>
<td>wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>(H_0)</td>
<td>deep sea wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>(H_m)</td>
<td>maximum wave height ((Miche's\ criterion))</td>
<td>[m]</td>
</tr>
</tbody>
</table>
List of Symbols

- \( \text{H}_{\text{rms}} \) : root mean square wave height [m]
- \( \text{H}_{\text{rms0}} \) : deep sea root mean square wave height [m]
- \( \text{H}_{\text{sig}} \) : significant wave height (averaged 1/3 of the highest waves) [m]
- \( \text{H}_{\text{sig0}} \) : deep sea significant wave height [m]
- \( \text{H}_\ast \) : dimensionless wave height \((\text{H}/\text{H}_{\text{rms0}})\) [-]
- \( \text{HHW} \) : High High Water (relative to MSL) [m]
- \( i \) : number of discrete point in x-direction [-]
- \( i_{\text{aux}} \) : last grid point of 1-D overwash model [-]
- \( I_1 \) : first Einstein integral [-]
- \( I_2 \) : second Einstein integral [-]
- \( j \) : number of discrete point in y-direction [-]
- \( k \) : wave number \([1/m]\)
- \( L \) : wave length [m]
- \( L_0 \) : deep water wave length [m]
- \( L_{\text{guess}} \) : estimated wave length (in dispersion relation) [m]
- \( L_{\text{calc}} \) : calculated wave length with dispersion relation [m]
- \( \text{LHW} \) : Low High Water (relative to MSL) [m]
- \( m \) : bottom slope [-]
- \( m_n \) : moments of the action spectrum \([m^2]\)
- \( m_{\text{crest}} \) : bottom slope of the crest [-]
- \( m_{\text{front}} \) : bottom slope front side barrier island [-]
- \( m_{\text{back}} \) : bottom slope back side barrier island [-]
- \( M \) : Manning friction coefficient \([m^{1/3}/s]\)
- \( \text{MSL} \) : Mean Sea Level [m]
- \( n \) : ratio between \(c_g\) and \(c\) [-]
- \( \text{ndir} \) : number of discrete directions in directional spreading [-]
- \( P_b \) : probability of breaking according to the Rayleigh distribution [-]
- \( Q \) : flux (discharge per unit time per unit width) \([m^3/s/m]\)
- \( Q_{\text{old}} \) : old flux before computation new water surface el. across bar \([m^3/s/m]\)
- \( Q_{\text{new}} \) : new flux as input for computation new water surface el. across bar \([m^3/s/m]\)
- \( r \) : Nikuradse bed roughness [m]
- \( R \) : wave run-up height [m]
- \( S_b \) : bottom sediment transport (Bijker) \([m^3/s]\)
- \( S_s \) : suspended sediment transport (Bijker) \([m^3/s/m]\)
- \( S_{\text{tot}} \) : total sediment transport \((S_b+S_s)\) \([m^3/s/m]\)
- \( S_w \) : wind speed \([m/s]\)
- \( S_{xx} \) : xx-component of the radiation stress (normal to the shore) \([N/m]\)
- \( S_{xy} \) : shear stress component of the radiation stress \([N/m]\)
- \( S_{yy} \) : yy-component of the radiation stress (parallel to the shore) \([N/m]\)
- \( \text{SWL} \) : Still Water Level (relative to MSL) [m]
- \( \text{SWL}_{\text{fr}} \) : Still Water Level at the front side of the bar (relative to MSL) [m]
- \( \text{SWL}_{\text{bk}} \) : Still Water Level at the back side of the bar (relative to MSL) [m]
- \( t \) : time [s]
- \( t_{\text{max}} \) : maximum period of computation [s]
List of Symbols

$T$ : wave period [s]
$T_m$ : mean wave period [s]
$T_p$ : peak wave period [s]
$T_{sig}$ : significant wave period [s]
$T_{1/3}$ : one-third highest wave period [s]
$u$ : horizontal particle velocity (in a wave) [m/s]
$u_0$ : horz. particle velocity just outside the boundary layer [m/s]
$u_0$ : max. horz. particle velocity just outside the boundary layer [m/s]
$u_s$ : shear stress velocity for waves [m/s]
$U$ : depth averaged overwash current velocity (normal to the shore) [m/s]
$U_0$ : depth averaged overwash current velocity at upstream boundary (i=0) [m/s]
$U_{1/2}$ : depth averaged current velocity component parallel to wave direction [m/s]
$U_{max}$ : maximum depth averaged overwash current velocity [m/s]
$U_s$ : shear stress velocity [m/s]
$V_{tot}$ : total volume of overtopped water $[m^3/s/m]$.
$w$ : fall velocity particle [m/s]
$W_c$ : crest width [m]
$x$ : coordinate normal to the shore [m]
$y$ : coordinate along the coast [m]
$z$ : elevation above the bed level [m]
$z_*$ : Rouse number (w/U.) [-]
$dz_{side}$ : tidal water level difference at both side of a barrier [m]
$\alpha$ : constant of the order one (Battjes and Janssen) [-]
$\alpha_s$ : angle of bottom slope [*]
$\beta$ : normalisation factor [-]
$\gamma$ : wave breaking index $(H_y/h_0)$ [-]
$\gamma_1$ : wave breaking index $(H_y/h_0)=1$ [-]
$\gamma_2$ : wave breaking index $(H_y/h_b)$ [-]
$\Delta$ : relative density bed material [-]
$\Theta$ : direction of wave propagation [*]
$\Theta_d$ : direction step of wave propagation [*]
$\Theta_m$ : mean direction of wave propagation [*]
$\Theta_w$ : mean direction of the wind [*]
$\kappa$ : Von Karman coefficient (=0.4) [-]
$\mu$ : ripple factor $(C/C_{90})^{1/2}$ [-]
$\xi$ : Irribarren parameter [-]
$\rho_w$ : mass density of water $[kg/m^3]$.
$\rho_s$ : mass density of sediment $[kg/m^3]$.
$\tau_c$ : bottom shear stress, currents $[N/m^2]$.
$\tau_w$ : bottom shear stress, waves $[N/m^2]$.
$\tau_{c,max}$ : maximum bottom shear stress, waves $[N/m^2]$.
$\tau_{cw}$ : bottom shear stress, currents and waves $[N/m^2]$.
$\omega$ : wave angular velocity $[m/s]$.
$\omega_r$ : intrinsic wave angular velocity (incl. current influence) $[m/s]$.
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APPENDIX I: Short wave parameters

The parameters $S_{xx}$, $c_g$ and $E$ can be derived from the linear short wave theory for progressive waves. The linear wave theory is applied here because this theory can be modeled quite good. Another reason is that other linear wave models show good results compared with measurements. The restriction to progressive short waves is justified since on gently sloping beaches, applied in this study, most short wave energy is dissipated rather than reflected.

In the formulae described below, $h$ is the total depth and $T$ is the peak period $T_p$.

wave number:

\[ k = \frac{2\pi}{L} \]  

(1)

where

- $k$ : wave number [1/m]
- $L$ : wave length [m]

wavelength with current:

\[ L = \frac{gT^2}{2\pi} \tanh \left( \frac{2\pi h}{L} \right) + \frac{QT}{h} \]  

(2)

where

- $L$ : wave length of wave influenced by current [m]
- $g$ : acceleration due to gravity [m/s$^2$]
- $T$ : wave period [m]
- $h$ : total water depth [m]
- $Q$ : flux of flowing water mass [m$^3$/s/m]

wave celerity with current:

\[ c = \frac{gT}{2\pi} \tanh \frac{2\pi h}{L} + \frac{Q}{h} \]  

(3)

where $c$ : wave celerity [m/s]

wave group celerity with current:

\[ c_g = nc = \frac{1}{2}(1 + \frac{2kh}{\sinh(2kh)})c \]  

(4)

where

- $c_g$ : wave group celerity [m/s]
- $n$ : ratio between wave celerity and wave group celerity [-]
wave frequency with current:
\[ \omega_r = \omega + kU_l = 2\pi\left(\frac{1}{T} + \frac{Q}{hL}\right) \]  
where \( \omega_r \): intrinsic wave angular velocity (including current influence) [1/s]
\( \omega \): wave angular velocity [1/s]
\( U_l \): depth averaged current velocity parallel to wave direction [m/s]

wave orbital velocity just above the boundary layer:
\[ u_0 = \frac{\omega_r H_{rms}}{2\sin h(kh)} \cos(\omega t) = \hat{u}_0 \cos(\omega t) \]  
where \( u_0 \): horiz. orbital velocity component just above boundary layer [m/s]
\( \hat{u}_0 \): max. horiz. orbital velocity comp. just above boundary layer [m/s]
\( H_{rms} \): root mean square wave height [m]
\( t \): time [s]

radiation stress:
\[ S_{xx} = (2n - \frac{1}{2})E \]  
where \( S_{xx} \): radiation stress component perpendicular to the coast [N/m]
\( E \): wave energy [N/m]

wave energy:
\[ E = \frac{1}{8}\rho g H_{rms}^2 \]  
where \( \rho \): density of the water [kg/m³]
APPENDIX II: Wave energy dissipation

For given incident wave parameters and beach profile, the variation of mean energy density (E) with distance to the shoreline can be calculated from the wave energy balance, written as

\[ \frac{\partial P_x}{\partial x} + D = 0 \]  \hspace{1cm} (9)

Here \( P_x \) is the x-component of the time-mean energy flux per unit length, normal to the still-water line, and \( D \) is the time-mean dissipation power per unit area. Knowing the dependence of \( P \) and \( D \) on \( E \) and on known parameters such as local mean depth (h), wave frequency (\( \omega_c \)), mass density (\( \rho \)) and gravity of acceleration (g), it is possible to integrate equation 9, depending on the initial condition of the given incident wave, to find \( E \) as a function of \( x \).

Energy dissipation due to breaking waves

Inside the surf zone, the dissipation of wave energy in the breaking process is dominant. A simplification of this complex process is made by assuming the dissipation rate per unit area \( D \) equal to the probability that a wave is breaking (\( P_b \)) multiplied by the dissipation rate of the breaking wave (\( D_{bb} \)):

\[ D_b = P_b D_{bb} \]  \hspace{1cm} (10)

Probability of breaking

Waves break when locally the wave front becomes too steep. For irregular waves this may be the result of several mechanisms, such as interaction between short waves, interaction between wave and bottom, wave and current or wave and wind. Here the effect of wind and current on wave breaking is not included. The only effect of current on waves that is taken into account is the changed wave length, causing a change in wave height.

For wind waves, the distribution of wave height can be described with the Rayleigh distribution. The assumption that when a wave is equal or higher than a maximum wave height \( H_m \), it is breaking or broken, will be written in terms of the probability distribution of the wave heights, \( F(H) \). The shape of \( F(H) \) for the lower, non-broken wave heights is assumed to be the same as it is in the absence of wave breaking, according to the Rayleigh distribution:

\[ F(H) = Pr\{H \leq H\} = 1 - \exp\left(-\frac{1}{2} \frac{H^2}{H_*^2}\right) \] \hspace{1cm} for \( 0 \leq H \leq H_m \)

\[ = 1 \] \hspace{1cm} for \( H_m \leq H \)  \hspace{1cm} (11)
in which the undercore indicates a random variable and $H_*$ is a local parameter. Since equation (11) is expressed in $H_*$ and $H_m$, all the statistics of the wave heights can be expressed in terms of $(H_*, H_m)$. Among those are the rms value ($H_{rms}$), defined by

$$H_{rms} = \left( \int_0^\infty H^2 dF(H) \right)^{\frac{1}{2}}$$

where $H_{rms}$ is the root-mean-square of the wave heights. $H_{rms}$ is the wave height that represents the wave energy in the wave field, because the wave energy is proportional to $H^2$.

$$H_{rms} = \sqrt{H^2}$$

The probability density function is shown in Figure A-1 and the probability distribution curve is shown in Figure A-2. Normally a wave field is characterized by the significant wave height $H_s$ or $H_{1/3}$, which can be defined from the wave height distribution as the average of the highest one third of the waves. The relation between $H_{1/3}$ and $H_{rms}$ is

$$H_s \sim \sqrt{2 \cdot H_{rms}}$$
The probability that at a given point a height is associated with a breaking or broken wave \((Q_b)\), which can be written as

\[ Q_b = Pr(H = H_m) \]  

(15)

Substitution of (11) into (12) and (15) gives

\[ H_{rms}^2 = 2(1 - Q_b)H_r^2 \]  

(16)

and

\[ Q_b = \exp\left(-\frac{\frac{1}{2}H_m^2}{H_r^2}\right) \]  

(17)

Using \((H_{rms}, H_m)\), which have a clearer physical meaning, can be achieved by eliminating \(H_r\) between (16) and (17), which yields for \(Q_b\)

\[ Q_b = \exp\left[-\frac{1 - Q_b}{(H_{rms}/H_m)^2}\right] \]  

(18)

\(Q_b\) expresses the fraction of waves which at any point are breaking or broken, in terms of the ratio of \(H_{rms}\) actually present, to the maximum wave height which the given wave height can sustain. The local value of \(H_{rms}\) can be found by integrating the differential equation (9). The average local energy dissipation rate \(D\) is proportional to \(Q_b\) and it is mainly through \(Q_b\) that the model reacts to changes in depth. In very deep water, where \(H_{rms}/H_m\Rightarrow0\), \(Q_b\Rightarrow0\). If the waves are shoaling then the ratio \(H_{rms}/H_m\) tends to increase, and so does \(Q_b\). When the ratio becomes 1, all the waves are broken and all the wave heights become equal to \(H_m\).

**Maximum wave height**

The maximum breaker height is based on Miche's criterion for the maximum height of periodic waves of constant form:

\[ H_m \approx \frac{0.88}{k} \tan\left(\frac{\gamma kh}{0.88}\right) \]  

(19)

in application to random waves, (19) will be used with \(f\) in the dispersion relation being given as the mean frequency, here defined as \(f_{mean} \approx 1/(0.9*T_p)\).
Energy dissipation in a breaking wave

In order to model the dissipation is a breaking wave, the analogy between breaking waves and bores is used (Le Méhauté, 1962). This results in the following expression:

\[ D_{bb} = 2\alpha f_p E \] (20)

where \( \alpha \) : constant of the order one
\( f_p \) : peak frequency \((2\pi/\omega_n)\); \( \omega_n \) is the angular velocity
\( E \) : wave energy, defined in (8)

The probability that a wave is breaking \( (P_b) \) is now formulated as the fraction of the waves that is breaking or broken \( (Q_b) \), knowing the rms wave height and the maximum wave height. The mean dissipation due to breaking is now defined by

\[ D_b = 2\alpha E f_p Q_b \] (21)

Dissipation of waves due to bottom friction

Bottom friction is the process by which the wave looses some of its energy due to the effect of friction at the sea bottom. This effect is cumulative and the amount of energy dissipated increases with distance, wave height, wave period and decreasing water depth.

Jonsson (1966) carried out experiments to determine the bed shear stress under waves. He found that this shear stress, \( \tau_w \), could be described in terms of the near bed velocity amplitude and the wave friction factor:

\[ \tau_w = \frac{1}{2} \rho f_w u_0^2 \] (22)

where \( \tau_w \) : bottom shear stress due to the wave orbital motion \([N/m^2]\)
\( f_w \) : wave friction factor \([-\]
\( u_0 \) : horz. velocity component just outside boundary layer \([m/s]\)

The wave friction factor is defined by (Swart) as:

\[ f_w = \exp[-6 + 5.2(a_o/r)^{-0.2}] \quad \text{if} \quad a_o/r > 1.59 \] (23)

\[ f_w = 0.30 \quad \text{if} \quad a_o/r < 1.59 \] (24)
where $a_0$ : max. horz. water particle displacement at the bottom \[ \text{[m]} \]
$r$ : bottom roughness \[ \text{[m]} \]

This wave friction factor $f_w$ shows that the friction depends on the relative roughness between the maximum orbital amplitude of the wave at the bottom ($a_0$) and the Nikuradse bed roughness ($r$). With increasing roughness $r$, $f_w$ increases. Also with decreasing amplitude $a_0$, $f_w$ increases. The maximum wave amplitude $a_0$ at the boundary layer near the bed is formulated by:

$$a_0 = \frac{\theta_0}{\omega_r} \tag{25}$$

The wave orbital velocity at the bottom ($u_0$) is defined by:

$$u_0 = \frac{1}{2} \frac{\omega_r H_{\text{rms}}}{\sinh(kh)} \sin(\omega t) \tag{26}$$

where $\omega$ : angular wave velocity \[ \text{[1/s]} \]
$\omega_r$ : intrinsic angular wave velocity \[ \text{[1/s]} \]
$H_{\text{rms}}$ : root mean square wave height \[ \text{[m]} \]
$k$ : wave number \((2\pi/L)\) \[ \text{1/m} \]
$h$ : total depth \((h+h')\) \[ \text{[m]} \]
$t$ : time \[ \text{[s]} \]

The energy decay due to wave friction is given by:

$$D_f = \frac{u_0}{f_w} \tag{27}$$

With equations 22 and 26 this energy decay can be written as:

$$D_f = \frac{1}{8} \rho f_w \frac{1}{\sqrt{\pi}} \left( \frac{\omega_r H_{\text{rms}}}{\sinh(kh)} \right)^3 \tag{28}$$
APPENDIX III: Derivation of tide-, wave-, and tide+wave-induced overwash equation

Derivation tide-induced overwash equation

The momentum equation of the tide-induced current is:

\[ \rho \frac{\partial}{\partial x} \left( \frac{Q^2}{h + h'} \right) + \rho g (h + h') \frac{\partial h'}{\partial x} + \tau_e = 0 \]  \hspace{1cm} (29)

In this equation the flux \( Q \) is constant and the surface elevation \( h' \), the water depth \( h \) and the bottom shear stress \( \tau_e \) are variable which are related to the distance from the shore \( x \). The first term can be written in a different form by differentiation:

\[ \rho Q^2 \cdot \frac{d}{dx} \left( \frac{1}{h + h'} \right) \]

\[ (h + h') = F(x) \quad \Rightarrow \quad \rho Q^2 \cdot \frac{d}{dx} \left( \frac{1}{F(x)} \right) \]

\[ \frac{d}{dx} \left( \frac{1}{F(x)} \right) = -1 \cdot F(x)^{-2} \cdot \frac{dF(x)}{dx} = -\frac{1}{F(x)^2} \cdot \frac{dF(x)}{dx} \]

\[ \rho Q^2 \cdot \frac{d}{dx} \left( \frac{1}{F(x)} \right) = \frac{\rho Q^2}{(h + h')^2} \frac{d(h + h')}{dx} \]

The full equation can now be written as:

\[ \frac{\rho Q^2}{(h + h')^2} \frac{d(h + h')}{dx} + \rho g (h + h') \frac{d h'}{dx} + \tau_e = 0 \]
\[ \frac{d h'}{dx} = \frac{1}{[\rho g(h+h') - \frac{\rho Q^2}{(h+h')^2}]} \left( \frac{\rho Q^2}{(h+h')^2} \frac{dh}{dx} - \tau_c \right) \]  

(30)

The same derivation can be performed to obtain the wave and the tide+wave-induced overwash equation:

\[ \frac{d h'}{dx} = \frac{1}{[\rho g(h+h') - \frac{\rho Q^2}{(h+h')^2}]} \left( \frac{\rho Q^2}{(h+h')^2} \frac{dh}{dx} - \frac{dS_{sw}}{dx} - \tau_{cw} \right) \]  

(31)
APPENDIX IV: Bottom shear stress due to waves and current

The bottom shear stress for currents only just above the boundary layer is defined by

$$\tau_c = \frac{\rho g U^2}{C^2}$$  \hspace{2cm} (32)

where
\begin{align*}
U & : \text{depth averaged current velocity} \quad \text{[m/s]} \\
C & : \text{Chezy friction factor, depending on the depth } h + h' \quad \text{[m}^{1/2}/\text{s}] \\
C & = 18 \log\left(\frac{12(h + h')}{r}\right) \hspace{2cm} (33)
\end{align*}

where
\begin{align*}
r & : \text{the Nikuradse roughness in meters (here 0.02m)} \quad \text{[m]}
\end{align*}

For waves the bottom shear stress is defined as

$$\tau_w = \frac{1}{2} \rho f_w \dot{u}_0^2$$  \hspace{2cm} (34)

where
\begin{align*}
f_w & : \text{wave friction factor (Swart, 1976)} \quad [-] \\
\dot{u}_0 & : \text{horiz. orbital velocity component just above boundary layer} \quad \text{[m/s]}
\end{align*}

This orbital velocity component is defined by:

$$\dot{u}_0 = \frac{\omega \sqrt{H_{rms}} \sin(\omega t)}{2 \sin h(kh)} \sin(\omega t) = \dot{u}_0 \sin(\omega t) \hspace{2cm} (35)$$

where
\begin{align*}
\omega_c & : \text{wave angular velocity including the influence of the current} \quad [1/\text{s}] \\
\omega & : \text{wave angular velocity} \quad [1/\text{s}] \\
H_{rms} & : \text{root mean square wave height} \quad \text{[m]} \\
t & : \text{time} \quad \text{[s]} \\
\dot{u}_0 & : \text{max. horiz. orbital velocity comp. just above boundary layer} \quad \text{[m/s]}
\end{align*}

The maximum bottom shear stress for waves is

$$\tilde{\tau}_w = \frac{1}{2} \rho f_w \dot{u}_0^2$$  \hspace{2cm} (36)
and $\hat{u}_0$ is

$$\hat{u}_0 = \frac{\omega_r H_{\text{rms}}}{2 \sin h(kh)}$$  \hspace{1cm} (37)

The combined bottom shear stress for currents and waves according to Bijker (see Van der Velden, 1990) is:

$$\overline{\tau_{cw}} = \tau_c + \frac{1}{2} \overline{\tau_w}$$  \hspace{1cm} (42)

where $\tau_{cw}$ : time-averaged total bottom shear stress
APPENDIX V: Influence of wave orbital motion on the overwash current: an example

To investigate the effect of the waves on the overwash current, a special situation is calculated. The results are shown in Figure A-3 & A-4. The overwash surface elevation and velocity due to tide-only and due to waves-only is plotted in those figures, both with an identical water level difference across the bar. The tide-induced overwash is caused by a tide-induced water level difference of \( dz = 0.15 \) m (at the right boundary). To examine the influence of the waves on the overwash current, also a wave-induced overwash situation is calculated. A wave height is chosen that causes a wave set-up identical with the tidal elevation at the front side (0.15m). The resulting wave height which causes this wave set-up is \( H_s = 1.8 \) m with a peak period of 6.7 seconds.

In Figure A-3 the surface elevation across the bar is plotted for the wave- and the tide-induced washover. Almost no difference in elevation occurs; the tide-induced elevation is a bit higher than the combination of tide and waves.

Looking at the velocity profiles plotted in Figure A-4, it can be noticed that the overwash velocity is smaller in case of waves, compared to the tide-only situation. This indicates the extra friction caused by the waves. The waves slow down the current a bit by their orbital motion.
Comparison tide vs. wave-induced washover
SURFACE ELEVATION (dz= 0.15m, Hs=1.8m, Tp=6.7s, r=0.02m, m=1:25)

Figure A3

Comparison tide vs. wave-induced washover
CURRENT VELOCITY (dz=0.15m, Hs=1.8m, Tp=6.7s, r=0.02m, m=1:25)

Figure A4
APPENDIX VI: Numerical solution Einstein integrals:

The two Einstein integrals are solved as follows:

\[ I_1 = R \int_{r_c/h}^{1} \left[ \frac{1-y}{y} \right] dy \]  \hspace{1cm} (39)

can be written with \( A = r_c/h \) as:

\[ I_1 = R \cdot (R_{11} - R_{12} + R_{13}) \]  \hspace{1cm} (40)

where

\[ R_{11} = \left( \frac{1}{-z_+ + 1} \right)(1 - A^{z_+^{1}}) \]  \hspace{1cm} (41)

\[ R_{12} = \left( \frac{z_+}{-z_+ + 2} \right)(1 - A^{z_+^{2}}) \]  \hspace{1cm} (42)

\[ R_{13} = \left( \frac{z_+(z_+-1)}{z_+(-z_+ + 3)(1 - A^{1-z_+^{3}})} \right) \]  \hspace{1cm} (43)

and

\[ I_2 = R \int_{r_c/h}^{1} \left[ \ln y \left( \frac{1-y}{y} \right) \right] dy \]  \hspace{1cm} (44)

as

\[ I_2 = R \cdot (R_{21} - R_{22} + R_{23}) \]  \hspace{1cm} (45)

where
\[ R_1 = \frac{1}{-z_* + 1} \left( \frac{1}{-z_* + 1} - A^{-z_* + 1} \right) (\log(A) - \frac{1}{-z_* + 1}) \tag{46} \]

\[ R_2 = \frac{1}{-z_* + 2} \left( \frac{1}{-z_* + 2} - A^{-z_* + 2} \right) (\log(A) - \frac{1}{-z_* + 2}) \tag{47} \]

\[ R_3 = \frac{z_* (z_* - 1)}{z_* (-z_* + 3)} \left( \frac{1}{-z_* + 3} - A^{-z_* + 3} \right) (\log(A) - \frac{1}{-z_* + 3}) \tag{48} \]

\[ R = \frac{0.216 (r/h)^{z_* - 1}}{(1 - r/h)^{z_*}} \tag{49} \]

\[ z_* = \frac{w}{\kappa V_*} \tag{50} \]

\[ V_* = \sqrt{\frac{\tau_{cw}}{\rho}} \tag{51} \]

w is the fall velocity of a sediment particle and \( \tau_{cw} \) is the bottom shear stress for current and waves, defined in §4.6.2. \( V_* \) is the shear stress velocity at height above the bottom \( z_* \). \( \kappa \) is the Von Karman constant (0.4). The depth h is actual (h+h').
APPENDIX VII: Comparison 2-D model results MIKE21_HD/NSW with 1-D overwash model

VII-1 Introduction

Apart from the verification of the 1-D overwash model with measured wave heights and wave set-ups and with the surf zone model Unibest_LT, executed in section 5.4, a comparison of overwash current velocities is made with a 2-D flow model, MIKE21_HD\(^1\). This model is able to simulate currents flowing over a bar due to wave- and tide-induced water level differences and currents. The wave transformation from the sea to the shore is calculated with the near shore wave model MIKE21_NS\(^2\).

The six different conditions which are modelled are shown in Table A-1. Two crest heights below the water surface (SWL) and one above SWL are chosen for examination. The situation of the bar with a crest above SWL is modeled to study the difference between wave set-up in combination with and without an overwash current. The wave direction is perpendicular to the barrier shore.

Table A-1: Modelled crest heights and wave conditions

<table>
<thead>
<tr>
<th>CREST HEIGHT</th>
<th>WAVE CONDITION: (H_s); (T_m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>+0.80 m</td>
<td>HD +082N</td>
</tr>
<tr>
<td>-0.40 m</td>
<td>HD -042N</td>
</tr>
<tr>
<td>-0.80 m</td>
<td>HD -082N</td>
</tr>
</tbody>
</table>

First the wave modelling is described in section VII-2, followed by the flow modelling in section VII-3. The cross sectional results from this 2-D model are compared with the results of the 1-D model in section 5.4.5, (verification of the 1-D overwash model). The location in the 2-D model where the results are compared with the 1-D model results is explained in section VII-4. Descriptions of the used models are given in Appendices VIII and IX.

VII-2 Wave modelling

MIKE21_NS\(^2\) is a wind-wave module which describes the propagation, growth and decay of short-period and short-crested waves in near shore areas. The model takes into account the effects of refraction and shoaling due to varying depth, local wind generation and energy dissipation due to bottom friction and wave breaking. The model also takes into account the effect of wave-current interaction.

\(^1\): MIKE21_HydroDynamic model (DHI)

\(^2\): MIKE21_Near Shore Wave model
Reference level

The datum (reference level) for all six models is chosen at SWL. The water level elevations and wave heights are defined relative to this level.

Bathymetry

The wave model is used here to compute the six different conditions given in Table A-1. The bathymetry of the barrier (with the overwash current pattern of a 2m wave-induced washover) is plotted in Figure A-5 and the 3-D plot with parallel contour lines is shown in Figure A-6. The width of the model (from north to south) is chosen in such a way that evential negative effects (circulation flow, in- and outflow) along the boundaries of the model at the north and south side are not effecting the flow field in the middle of the model.

Grid spacing

Two types of grid spacing must be specified. The first type is the grid spacing for the bathymetry in x and y-direction. The second type is the grid spacing in the θ-direction (for the discretisation of the mean wave direction). The grid spacings are chosen to provide adequate resolutions of the bathymetry, the wind field and current field in the x,y-plane and the wave field in the x,y,θ-space. However, the grid spacings must also be selected to satisfy the stability criterion for the numerical scheme which is applied in MIKE21_NSW (Appendix VII). DHI recommends a grid spacing ratio of $\Delta x/\Delta y = 1/4$.

The grid spacing in the x-y plan is determined from the specification of the bathymetry. The direction of the waves is specified by a maximum deviation from the mean direction (DWD=30°), separated in 10 discrete directions with an interval of 6°. Figure A-9 in Appendix VIII shows the directional spreading with a grid spacing of 6°.

In this case model, no wind effects are taken into account; only the effects of the waves on the overwash process is examined.

Boundary conditions

MIKE21_NSW solves two coupled partial differential equations (Appendix VII) and like all other differential equations, these need boundary conditions at all the open boundaries. Three types of boundary conditions are applied for the four model boundaries (see Figure A-5):

- essential; the incoming energy is specified at the west boundary of the model, where the outgoing energy is fully absorbed.
- symmetry; the derivatives normal to the north and south boundary of the dependent variables are set to zero.
- absorbing; no energy enters the model area and the energy propagates out of the model area at the right boundary without reflections which can propagate back into the model area.

3. Danisch Hydraulic Institute
Figure A-5: 2-D plot of bathymetry schematized barrier island incl. current velocity vectors
Figure A-6: 3-D profile schematized barrier island
The essential boundary conditions are applied in form of the following wave parameters:

1. Significant wave height, \( H_s \) (m)
2. Mean wave period, \( T_m \) (sec)
3. Mean wave direction, \( \theta_m \) (deg)
4. Maximum deviation from mean direction, DWD (deg)
5. Directional spreading index, \( n \) (Appendix VIII)

In this model the waves approach the bar from the left (west boundary), perpendicular to the crest of the barrier. The west boundary therefore is the essential boundary, where the above described different wave parameters are specified: \( H_s = 2m / 4m; T_m = 7s / 10s; \theta_m = 0^\circ; \) DWD = 30°; \( n = 5 \).

**Other model parameters**

Besides the wave parameters other parameters must be specified. These are the model parameters concerning the bottom friction and wave breaking. The bottom friction factor is specified by a constant wave friction factor \( c_f = f_w / 2 \) of 0.005. This factor is used in the equation of dissipation of energy due to bottom friction, described in Appendix VIII. The bottom friction can also be specified directly using the empirical expression of Jonsson, 1966; Swart, 1974 (Appendix II). For the wave breaking parameters of Battjes & Janssen (1978), the following constants are specified: \( \alpha = 1.0; \gamma_1 = 1.0 \text{ and } \gamma_2 = 0.78 \). The dissipation due to wave breaking of steep waves is controlled by the parameter \( \gamma_1 \). The parameters \( \alpha \) and \( \gamma_2 \) are those specified by Battjes & Janssen.

The results of the different runs are discussed in section 5.4.5, where a comparison is made with the results of the 1-D overwash model.

**VII-3 Flow modelling**

The wave-induced overwash currents are computed with the hydrodynamic program MIKE21_HD. MIKE21_HD simulates water level variations and flows in response to forcing and resistive functions such as tidal level differences, wind, barometric pressure, coriolis and bed resistance. Sources of flux and momentum (such as river discharges) can be applied as well as sinks such as evaporation losses. The water levels and flows are resolved on a two-dimensional rectangular grid covering the area of interest. A more detailed description of this model is given in Appendix IX.

This model is applied here to compute the wave driven currents and overwash currents. Normally time varying water level elevations or fluxes, caused by the tidal range, can be specified at the model boundaries. In this situation, however, only currents and fluxes caused by the change in radiation stress, calculated in the wave model, are specified at the boundaries.

**Bathymetry**

The same reference level is chosen as in the wave model and the bathymetry is exact the size of the bathymetry used in MIKE21_NSW.
Appendix

Grid spacing

Contrary to the grid spacing used in the wave model here the spacing in x- and y-direction are chosen identical: \( \Delta x = \Delta y = 20 \text{m} \). The grid size is fine enough to simulate overwash currents over relative wide barrier crests (±300m).

Time step and simulation period

The time step for the simulation is selected as follows: first the grid spacing \( \Delta x \) must be determined, then the maximum allowed Courant number \( C_{rU} \) must be described and the maximum time step \( \Delta t_{\text{max}} \) can be found from the definition of this Courant number:

\[
C_{rU} = \frac{U_{\text{max}} \cdot \Delta t_{\text{max}}}{\Delta x}
\]

where

\( U_{\text{max}} \): maximum current speed which occurs during the simulation \([\text{m/s}]\)

\( \Delta t_{\text{max}} \): maximum time step for stability criterion \([\text{s}]\)

\( \Delta x \): grid spacing in x direction \([\text{m}]\)

Normally a Courant number up to 5 can be specified. The smaller the Courant number the less stability problems occur. No problems occur if 1 is chosen as the maximum Courant number. To avoid instability in the very shallow areas here \( C_{rU} = 1 \) is chosen.

The maximum current speed is \( U_{\text{max}} = \sqrt{gh} \), where \( g \) is gravity and \( h \) is the water depth. In the situation of very shallow water on the barrier crest (say 0.4m) during overwash, the maximum current speed is 2 m/s. With a grid spacing of \( \Delta x = 20 \text{m} \), the maximum time step then becomes (see equation 52): \( \Delta t_{\text{max}} = 10 \text{s} \). Because during the modelling currents will flow over very shallow areas, a time step of 6 seconds is chosen.

In order to simulate a stationary hydraulic condition (constant boundary conditions), the number of time steps is set to 1200. This means that the total simulated period is \( 1200 \times 6 \text{s} = 2 \text{hours} \). A comparison between the results of \( t = 1000 \times 6 \text{s} \) and \( t = 1200 \times 6 \text{s} \), which showed no difference in surface elevation and velocity, proved that the simulation period is chosen long enough to obtain a stationary condition.

Bed resistance

The Manning bed resistance is described in every grid point according to the following expression:

\[
M = C \cdot h^{-1/6} = 18 \log \left( \frac{12h}{r} \right) \cdot h^{-1/6}
\]

where

\( M \): Manning number \([\text{m}^{1/2}/\text{s}]\)

\( C \): Chezy number \([\text{m}^{1/2}/\text{s}]\)

\( h \): total water depth \([\text{m}]\)

\( r \): bed roughness \([\text{m}]\)

The chosen bed roughness is 0.02m.
Appendix

**Eddy viscosity coefficient**

The effective shear stress terms in the momentum equations contain momentum fluxes due to turbulence, vertical integration and subgrid scale fluctuations. The terms are included using an eddy viscosity formulation to provide damping of short-wave length oscillations and to prevent subgrid scale effects (Madsen et al., 1988). Madsen recommends a coefficient for eddy viscosity as follows:

\[
\text{Eddy viscosity coefficient} = 0.1 \times \text{grid size} \times \text{current velocity} \quad [\text{m}^2/\text{s}]
\]

In the flow simulations a value of 2 m$^2$/s is used.

**Boundary conditions**

A simulation model can only be run when boundary conditions are specified. In the 1-D overwash model, boundary conditions are needed for the upstream and the downstream boundary. In the 2-D model boundary conditions, water levels or flow conditions must be specified at the open boundaries. For the simulation of an overwash situation all four boundaries are open. Only wave-driven washovers are simulated, therefore the boundary conditions are obtained from the wave model by selecting the radiation stress components along the wave model boundaries. The inclusion of wave radiation stresses in MIKE21_HD is also specified by a 2-D map similar to that of bed resistance, except that it contains three items ($S_{xx}$, $S_{xy}$, and $S_{yy}$) and a prefix item (bathymetry). The stresses are kept constant in time (steady state wave situation).

Transfer boundary data (only contribution from wave radiation stresses) is generated for the four open boundaries in the model area, using the program WAVCUR, which transforms the radiation stresses at the boundaries to wave-driven currents and water level elevations.

To blow up the constant boundary conditions in time, the program WAVTRN is used. Now the boundary conditions can be specified as fluxes or water levels, which are defined as constants for a given period. This period is equal or longer than the simulation period. The models WAVCUR and WAVTRN are described in Appendix VIII.

The boundary conditions are specified as follows:

- model north and south boundary : water levels
- model east and west boundary : flux

The flux boundaries are chosen at the east and west boundary because no water level variations along those boundaries will occur (symmetrical bathymetry and wave conditions) and only wave driven flows will go in and out the model, perpendicular to the east and west boundaries. Water levels are specified at the north and south boundaries because no flow will be generated perpendicular to those boundaries, where only water level variations are generated by the change in radiation stresses.

**Effect wave set-up**

The wave parameters in MIKE21_NSW are computed relative to a horizontal water level. This means that the effect of the wave set-up on these wave parameters is not taken into
account. To determine the effect of the wave set-up on the waves, the wave set-up, determined by MIKE21_HD, must be put back into the wave model.

The radiation stresses, obtained from MIKE21_NSW, are used in the hydrodynamic model to compute the wave driven currents and water level elevations. The wave set-up is incorporated in the wave model by changing the bathymetry of the wave model corresponding to the changed water level. The deviations from still water level are subtracted from the bathymetry and the waves are calculated again. The new calculated radiation stresses are used in the HD-model to compute the new surface elevation and velocities.

This procedure is carried out once, because it is a time consuming process, especially when a couple of wave conditions and crest heights are examined. In the second iteration step, however, the changes in wave height and radiation stresses (wave model), and the deviation of the water level elevation and velocities (hydrodynamic model) are small compared to the changes after the first iteration.

To illustrate the differences in velocity before and after the wave set-up is included in the wave model, in Figure A-7 the velocity variation over the bar with and without the influence of the wave set-up is plotted. A difference in speed can be noticed on top of the crest, although it is very small.

VII-4 Comparison 2-D results with 1-D overwash model

Three different crest elevations with 2 wave conditions are modelled to study the overwash characteristics on a schematized barrier island. The results can be presented by two-dimensional vector plots of the mean wave direction, the wave height, the radiation stresses, or velocities. However, to have the ability of comparing the results with the 1-D overwash model results, specific cross sections are taken from the schematized barrier half way the model area (line A-A in Figure A-5), from grid point (x,y) = (0,160) to (130,160). This mid-section is chosen to be sure that no side effects of the north and south boundary disturbs the stationary hydraulic flow pattern.

Figure A-5 is a 2-D vector plot of the velocities, obtained from run HD -042N. In the model area the velocities are very small to zero, except the velocities at the crest of the barrier, where the washover takes place.
Figure A-7: Wave-induced overwash current velocity with & without wave set-up included
$h_{crest} = -0.8m; H_s = 4m; T_p = 10s; r = 1:50$
APPENDIX VIII : Theoretical background MIKE21_NSW

MIKE21_NSW is a wind-wave module which describes the propagation, growth and decay of short-period and short-crested waves in nearshore areas. The model takes into account the effects of refraction and shoaling due to varying depth, local wind generation and energy dissipation due to bottom friction and wave breaking. The model also takes into account the effect of wave-current interaction.

MIKE21_NSW is a stationary, directionally decoupled parametric model. To cater for the effect of current the basic equations in the model are derived from the conservation equation for the spectral wave action density. The conservation equation in the frequency domain is solved by introducing the zeroth and first moment of the action spectrum as dependent variables.

The basic equations are solved using the Eulerian finite difference technique. The zeroth and the first moment of the action spectrum are calculated on a rectangular grid for a number of discrete directions. A once-through marching procedure is applied in the predominant direction of the wave propagation.

The basic output from the model is integral wave parameters such as the significant wave height, the mean wave period, the mean wave direction and the directional standard deviation. Output from the model can also be obtained in form of radiation stresses.

MIKE21_NSW can be applied to studies of wave disturbances in coastal areas, which is essential for the estimation of the wave forces at a shoreline. Another important problem in coastal engineering is the simulation of the sediment transport, which for a large part is determined by the wave-induced littoral current. The wave-induced current can be generated by strong gradient in radiation stresses which occur in the surf zone. MIKE21_NSW also offers the facility to calculate the radiation stresses.

Setting up and running the model will normally consist of the six tasks listed below:

- defining and limiting the wave problem
- collecting data
- setting up the model, e.g. defining the model size, the grid size and the bathymetry and defining the boundary conditions
- calibrating and verifying the model
- running the production simulations
- presenting the results

When defining the wave problem the following wave phenomena are important:
Shoaling, refraction, diffraction, bottom dissipation, wave breaking, wind generation, frequency spreading, directional spreading and wave-current interaction.

Wave equations

The basic equations of the near shore wave model MIKE21_NSW are based on the approach proposed by Holthuijzen et al. (1989). These equations are derived from the conservation equation for the spectral wave action density.
A parameterization of this equation in the frequency domain is performed introducing the zeroth and the first moment of the action spectrum as dependent variables. This leads to the following coupled partial differential equations:

\[
\frac{\partial}{\partial x} (c_{gx} m_0) + \frac{\partial}{\partial y} (c_{gy} m_0) + \frac{\partial}{\partial \theta} (c_0 m_0) = T_0
\]

(54)

\[
\frac{\partial}{\partial x} (c_{gx} m_1) + \frac{\partial}{\partial y} (c_{gy} m_1) + \frac{\partial}{\partial \theta} (c_0 m_1) = T_1
\]

(55)

where \( m_0(x,y,\theta) \) : Zeroth moment of the action spectrum [m²]

\( m_1(x,y,\theta) \) : First moment of the action spectrum [m²]

\( c_{gx} \) and \( c_{gy} \) : Components in x- and y-direction of group velocity [m/s]

\( c_0 \) : Propagation speed: change of action in \( \theta \)-direction [m/s]

\( x \) and \( y \) : Cartesian coordinates [m]

\( \theta \) : Direction of wave propagation [°]

\( T_0 \) and \( T_1 \) : Source terms

The moments \( m_0(\theta) \) are defined by

\[
m_n(\theta) = \int_0^\infty \omega^n A(\omega,\theta) d\omega
\]

(56)

where \( \omega \) is the absolute frequency and \( A \) is the spectral wave action density. The propagation speeds \( c_{gx}, c_{gy} \) and \( c_0 \) are obtained using linear wave theory.

The left hand side of the basic equations takes into account the effect of refraction and shoaling. The source terms \( T_0 \) and \( T_1 \) take into account the effect of local wind generation and energy dissipation due to bottom friction and wave breaking. The effects of current on these phenomena are included. The source terms for the local wind generation are derived directly from the Shore Protection Manual formulation (1984) for the wave growth for the fetch-limited wave growth in deep water.

Dissipation due to breaking

Wave breaking is the process by which waves lose (dissipate) energy when the waves have grown too steep (i.e. reach a limiting steepness) and hence become unstable, or when the waves are too high to be supported by the water depth (i.e. reach a limiting H/d).

The formulation in MIKE21 NSW of wave breaking due to large wave steepness and limiting water depth is based on the formulation of Battjes & Janssen (1978), explained in Appendix II.
Appendix

Bottom dissipation

Bottom friction is the process by which the wave loses some of its energy due to the effect of friction at the sea bottom. This effect is cumulative and the amount of energy dissipated increases with distance, wave height, wave period and decreasing water depth.

The formulation of dissipation of energy due to bottom friction is based on the quadratic friction law:

\[
\frac{dE}{dt} = -\frac{1}{12\pi} \frac{f_w}{g} \omega H^3 \sinh(kh) \tag{57}
\]

where

- \( E \) : wave energy = \( H^2/8 \) [N/m]
- \( \omega \) : wave angular velocity [1/s]
- \( H \) : wave height [m]
- \( k \) : wave number [1/m]
- \( h \) : water depth [m]
- \( f_w \) : wave friction factor.

The effect of bottom dissipation on the mean wave period can also be included. The bottom dissipation has the effect that the mean wave period is reduced.

Nonlinear iteration

The source terms due to the bottom friction and the wave breaking are introduced implicitly. The terms are nonlinear and therefore a nonlinear iteration is performed in each grid point in the x-y plane. The simple successive substitution method is applied. The nonlinear iteration is stopped when the second norm of the residual vector becomes less than a specified value and when the number of iterations becomes larger than a specified value.

Numerical scheme

Four different types of discretization of the convective terms in the y- and \( \theta \)-direction can be applied:

1) Upwind Differencing (UD)
2) Central Differencing (CD)
3) Linear Upwinded Differencing (LUD)
4) Quadratic Upwinded Differencing (QUD)

The first-order upwind differencing is only a first-order scheme with inherent numerical diffusion. The quadratic upwind differencing and the central differencing are higher order schemes. However, these schemes may introduce a significant oscillation in the solution at locations with large gradients in the wave field.

In general, the best results are obtained using upwind differencing in both the y- and \( \theta \)-direction. Especially, if the purpose of the wave modelling is to obtain radiation stresses for longshore currents, the upwind differencing should be applied.
Output

The basic results from MIKE21_NSW consist of two-dimensional arrays containing the following integral wave parameters:

1) The significant wave height $H_s$
2) The mean wave period $T_m$
3) The mean wave direction $\theta_m$
4) The directional standard deviation $\sigma$

The significant wave height $H_s$ is defined by

$$H_s = 4\sqrt{E_1}$$  \hspace{1cm} (58)

where the total wave energy $E_1$ is

$$E_1 = \int_0^{2\pi} E(\theta) d\theta$$  \hspace{1cm} (59)

The mean wave period $T_m$ is defined by

$$T_m = \frac{2\pi}{\omega_1}$$  \hspace{1cm} (60)

where $\omega_1$ is the wave angular velocity

Results from MIKE21_NSW can also be obtained in form of two-dimensional arrays containing the x- and y-components of a vector $U = (u,v)$ defined by

$u = H_s \cos (\theta_m)$

$v = H_s \sin (\theta_m)$

A vector plot of $U$ can be used to show the mean wave direction in the model area.

Discrete directional distribution of wave energy

The directional distribution of the wave energy at the boundary is given by

$$E(\theta_i) = E_i D(\theta_i) \hspace{1cm} i = 1, ndir$$

Here $ndir$ is the number of discrete directions, $E_i = H_s^2/16$ is the total energy of the discrete energy spectrum and the directional distribution function $D$ is defined by
where $\beta$ is a normalisation factor. $\theta_d$ must be less or equal to 90 degrees. The directional distribution function $D(\theta)$ for selected values of the directional spreading index is shown in Figure A-8.

An example of a discrete directional distribution of the wave energy is shown in Figure A-9.

- # grid points in $\theta$-direction : 10
- Grid spacing in $\theta$-direction : 6 degr.
- Significant wave height : 1.0 m
- Mean wave direction : 27.5 degr.
- Max. deviation from mean wave direction : 60 degr.
- Directional spreading Index : 2

Figure A-8: Directional spreading function $D(\theta)$ for $n = 2, 4, 6, 8$ and 64
Finally, MIKE21_NSW can provide resulting maps of radiation stresses $S_{xx}$, $S_{xy}$ and $S_{yy}$ which are defined by

$$S_{xx} = \left[ n - \frac{1}{2} + n \cos^2(\theta) \right] E$$  \hspace{1cm} (64)$$

$$S_{xy} = \left[ n \cos(\theta) \sin(\theta) \right] E$$  \hspace{1cm} (65)$$

$$S_{yy} = \left[ n - \frac{1}{2} + n \sin^2(\theta) \right] E$$  \hspace{1cm} (66)$$

**Wind generation**

Wind generation is the process by which the wind transfers energy into the water body for generating waves. The formulation of the wave generation by wind is based on empirical expressions derived from the Shore Protection Manual (SPM) formulation for the wave growth for fetch-limited sea states in deep water.

The wind-field (wind speed $S_w$ and direction $\theta_w$) can be specified either as constants for the entire model area or as a two-dimensional map read in from a data file.
APPENDIX IX : Theoretical background MIKE21_HD

The MIKE21_HD model simulates the water level variations and flows in response to forcing and resistive functions such as tidal level differences, wind, barometric pressure, coriolis and bed resistance. Sources of flux and momentum (such as river discharges) can be applied as well as sinks such as evaporation losses. The water levels and flows are resolved on a 2-dimensional rectangular grid covering the area of interest.

The computational model solves the depth average equation of continuity and conservation of momentum in two horizontal directions and provides water depth, x-flux and y-flux for each grid point and at each time step as possible output. From these data and the bathymetry, the surface elevation, x-velocity, y-velocity, current speed (scalar) and current direction are derived by the model and also offered as output. Flexible output facilities enables a wide variety of plotting options. The flow model calibration is achieved by adjusting the bed roughness coefficient (pattern and magnitude) and changing the viscosity.

The main equations and the numerical algorithm applied in the model are described followed by the physical, mathematical and numerical background for the most important terms in the main equations.

Main equations

The hydrodynamic model in MIKE21 is a general numerical modelling system for the simulation of water levels and flows in estuaries, bays and coastal areas. It simulates unsteady two-dimensional flows in one layer (vertically homogeneous) fluids. The following depth integrated equations describe the flow and water level variations:

conservation of mass:

\[
\frac{\partial \zeta}{\partial t} + \frac{\partial p}{\partial x} + \frac{\partial q}{\partial y} = 0 \quad (67)
\]

conservation of momentum in the x-direction (= y-direction when "x" is substituted by "y"):

\[
\frac{\partial p}{\partial t} + \frac{\partial}{\partial x} \left( \frac{p^2}{h} \right) + \frac{\partial}{\partial y} \left( \frac{g \zeta}{h} \rho_w \frac{p_{w}}{\rho} \frac{\sqrt{p^2 + q^2}}{C^2 \cdot h^2} \right) + \frac{\partial}{\partial x} \left[ \frac{q}{\rho_w} \frac{\sqrt{p^2 + q^2}}{C^2 \cdot h^2} \right] - \frac{1}{\rho_w} \left[ \frac{\partial}{\partial x} (h \tau_{xx}) + \frac{\partial}{\partial y} (h \tau_{xy}) \right] - \Omega q - f V_x = \frac{h}{\rho_w} \frac{\partial}{\partial x} (p_w) = 0 \quad (69)
\]
Appendix

The following symbols are used in the equations:

- \( h(x,y,t) \) : water depth [m]
- \( \zeta(x,y,t) \) : surface elevation [m]
- \( p,q(x,y,z) \) : flux densities in x- and y-directions = \((uh,vh)\) [m³/s/m]
- \((u,v)\) = depth average velocities in x- and y-directions
- \( C(x,y) \) : Chezy resistance [m⁵/s]
- \( g \) : acceleration due to gravity [m/s²]
- \( f(V) \) : wind friction factor [-]
- \( \Omega(x,y) \) : Coriolis parameter, latitude dependent [1/s]
- \( \rho_v \) : atmospheric pressure [N/m²]
- \( \rho_w \) : density of water [kg/m³]
- \( x,y \) : space coordinates [m]
- \( t \) : time [s]
- \( \tau_{xx}, \tau_{xy}, \tau_{yy} \) : components of effective shear stress [N/m]

Numerical formulation

MIKE21_HD makes use of a special technique to integrate the equations for mass and momentum conservation in the space-time domain: The Alternating Direction Implicit (ADI) method. This method calculates the equation matrices for each individual grid line in two steps, using a Double Sweep (DS) algorithm. The equations are solved by this algorithm in one-dimensional sweeps: one in the x-direction and the second in the y-direction.

The difference terms are expressed on a staggered grid in x,y-space as shown in Figure A-10.

In the x-sweep the continuity and x-momentum equations are solved, taking \( \zeta \) from \( n \) to \( n+\frac{1}{2} \) and \( p \) from \( n \) to \( n+1 \). For the terms involving \( q \), the two levels of old, known values are used, i.e. \( n-\frac{1}{2} \) and \( n+\frac{1}{2} \).
In the y-sweep the continuity and y-momentum equations are solved, taking $\zeta$ from $n + \frac{1}{2}$ to $n + 1$ and $q$ from $n + \frac{1}{2}$ to $n + \frac{3}{2}$, while the terms in $p$ use the values just calculated in the x-sweep at $n$ and $n + 1$ (see Figure A-11).

![Figure A-11: Time centering](image)

Adding the two sweeps together gives "perfect" time centering at $n + \frac{1}{2}$. Actually perfect time centering is not possible. The best approximation, without resorting to iteration (which has its own problems), is to use a "side-feeding" technique. At one time step the x-sweep solutions are performed in the order of decreasing y-direction, hereafter called a "down" sweep, and in the next time step in the order of increasing y-direction, the "up" sweep.

During a "down" sweep, the cross derivative $\partial p/\partial y$ can be expressed in terms of $p_{n+1,j,k+1}$ on the "up" side and $p_{n,j,k}$ on the "down" side, and vice versa during an "up" sweep. In this way an approximate time centering of $\partial p/\partial y$ at $n + \frac{1}{2}$ can be achieved, albeit with the possibility of developing some oscillations (zig-zagging).

**Difference approximations**

The different difference approximations used for the discretisation of the mass and momentum equations in time and space are described in the manual of MIKE21_HD, page A-7 to A-44. In these sections the difference approximations of points away from the coast and near the coast are discussed. For the momentum equation for each term the used difference method will be described. These terms are specified on the next page:
The time-derivation term:

\[ \frac{\partial p}{\partial t} \]

The gravity term:

\[ gh \frac{\partial \xi}{\partial x} \]

The convective/cross-momentum correction term:

\[ \frac{\partial}{\partial x} \left( \frac{PP}{h} \right) + \frac{\partial}{\partial y} \left( \frac{gP}{h} \right) \]

The wind friction term:

\[ f(V) \cdot V \cdot V_x \]

The resistance term:

\[ \frac{gP \sqrt{p^2 + q^2}}{C^2 h^2} \]

The Coriolis term:

\[ \Omega \cdot q \]

Calculation of Wave Generated Current and Set-up

With the MIKE21 service program WAVCUR the wave generated current and wave set-up can be calculated along the open boundaries of the model area. The results can be used as boundary data to a HD simulation including wave radiation stresses, after the results have been expanded in time to cover the simulation period, using the service program WAVTRN.
WAVCUR estimates the wave set-up and the wave generated current by solving two reduced stationary one-dimensional momentum equations:

\[ \rho g h \frac{\partial h'}{\partial x} = - \left( \frac{\partial S_{xx}}{\partial x} + \frac{\partial S_{xy}}{\partial y} \right), \text{ assumes } V_x = 0 \]  \hspace{1cm} (76)

which is a balance between the wave set-up and the wave radiation stresses, and

\[ \frac{\rho g V_y^2}{M^2 h^{1/3}} = - \left( \frac{\partial S_{yy}}{\partial x} + \frac{\partial S_{xy}}{\partial y} \right), \text{ assumes } \frac{\partial h'}{\partial y} = 0 \]  \hspace{1cm} (77)

which is a balance between the bed resistance (Manning formulation) and the wave radiation stresses. \( x \) is the orientation normal to the shore and \( y \) parallel to the shore.
APPENDIX X: Wave & Winds Analysis Case Study

In this Appendix the yearly wind directional distribution and the wave climate measured from 1981 to 1988 is presented (Table A-2 and A-3 respectively). By assuming the wind direction equal to the mean wave direction the wave climate can be coupled to the wind directional distribution. The wave height and direction which has the most effect on the study site is determined by "weighting" of the contribution of the waves from the various directions to the total yearly wave energy potential. After all, the wave energy determines for a great deal the influence on the sediment transport and wave-induced currents and surface elevations. This method is described more detailed later in this Appendix.

Table A-2: Wind direction distribution: Tong Chi & CBM-11 (overall yearly statistics)

<table>
<thead>
<tr>
<th>Direction</th>
<th>% time at Tong Chi</th>
<th>% time from N+NNE+NE+ENE at Tong Chi</th>
<th>% time at CBK-11</th>
<th>% time from N+NNE+NE+ENE at CBK-11</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>8.1</td>
<td>66.0</td>
<td>5.7</td>
<td>63.1</td>
</tr>
<tr>
<td>NNE</td>
<td>46.5</td>
<td></td>
<td>16.7</td>
<td></td>
</tr>
<tr>
<td>NE</td>
<td>10.4</td>
<td></td>
<td>28.5</td>
<td></td>
</tr>
<tr>
<td>ENE</td>
<td>1.0</td>
<td></td>
<td>12.2</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>0.9</td>
<td></td>
<td>4.3</td>
<td></td>
</tr>
<tr>
<td>ESE</td>
<td>0.3</td>
<td></td>
<td>1.6</td>
<td></td>
</tr>
<tr>
<td>SE</td>
<td>1.7</td>
<td></td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td>SSE</td>
<td>2.8</td>
<td></td>
<td>0.9</td>
<td></td>
</tr>
<tr>
<td>S</td>
<td>6.2</td>
<td></td>
<td>1.8</td>
<td></td>
</tr>
<tr>
<td>SSW</td>
<td>4.0</td>
<td>0.0</td>
<td>11.8</td>
<td>0.0</td>
</tr>
<tr>
<td>SW</td>
<td>7.1</td>
<td>0.0</td>
<td>7.5</td>
<td>0.0</td>
</tr>
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<td>WSW</td>
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<td></td>
</tr>
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<td>W</td>
<td>2.4</td>
<td></td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>WNW</td>
<td>0.8</td>
<td></td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>NW</td>
<td>2.5</td>
<td></td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td>NNW</td>
<td>1.5</td>
<td></td>
<td>2.7</td>
<td></td>
</tr>
<tr>
<td>Calm</td>
<td>1.8</td>
<td></td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td>100</td>
<td>66.0</td>
<td>100</td>
<td>63.1</td>
</tr>
</tbody>
</table>
Table A-3: Yearly wave climate from 1981 to 1988, Tong Chi (all directions)

<table>
<thead>
<tr>
<th>Wave heights Hs (m)</th>
<th>Weighted average significant wave period (s)</th>
<th>Yearly percent occurrence (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 0.5</td>
<td>5.7</td>
<td>22.46</td>
</tr>
<tr>
<td>0.51 to 1.0</td>
<td>6.2</td>
<td>29.14</td>
</tr>
<tr>
<td>1.01 to 1.5</td>
<td>6.6</td>
<td>16.62</td>
</tr>
<tr>
<td>1.51 to 2.0</td>
<td>6.9</td>
<td>10.48</td>
</tr>
<tr>
<td>2.01 to 2.5</td>
<td>7.2</td>
<td>8.37</td>
</tr>
<tr>
<td>2.51 to 3.0</td>
<td>7.5</td>
<td>5.77</td>
</tr>
<tr>
<td>3.01 to 3.5</td>
<td>7.9</td>
<td>3.71</td>
</tr>
<tr>
<td>3.51 to 4.0</td>
<td>8.4</td>
<td>2.02</td>
</tr>
<tr>
<td>4.01 to 4.5</td>
<td>8.7</td>
<td>1.00</td>
</tr>
<tr>
<td>4.51 to 5.0</td>
<td>9.1</td>
<td>0.23</td>
</tr>
<tr>
<td>5.01 to 5.5</td>
<td>10.5</td>
<td>0.09</td>
</tr>
<tr>
<td>5.51 to 6.0</td>
<td>10.7</td>
<td>0.04</td>
</tr>
<tr>
<td>6.01 to 6.5</td>
<td>10.3</td>
<td>0.04</td>
</tr>
<tr>
<td>6.51 to 7.0</td>
<td>9.3</td>
<td>0.01</td>
</tr>
<tr>
<td>&gt;7.0</td>
<td>9.3</td>
<td>0.02</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td>100.00</td>
</tr>
</tbody>
</table>

Determination operational waves

In order to "weight" the relative contribution of the various directional components and heights the yearly wave energy per wave height and direction is expressed as:

\[
\text{Yearly Wave Energy} = P_w \cdot E \cdot \# \text{ hours}
\]

where \( P_w \) : % occurrence per year
\( E \) : wave energy

Two wave climate seasons are described in the following text: The North East monsoon period and the South West monsoon period, in which most of the typhoons occur.
North East monsoon period

The main wave direction during the NE monsoon which will affect the site is NNE, but also waves from the N and NNW are shown to occur according to the wind records. For the assessment of the yearly wave energy contribution for the direction NNE the Tong Chi wave data were used directly. The yearly percent occurrence of waves coming from the NNE is 46.50%.

The calculated yearly wave energy contributions for the NNE, N and the NNW directions are:

<table>
<thead>
<tr>
<th>Direction</th>
<th>Energy (kN/m/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NNE</td>
<td>21,000</td>
</tr>
<tr>
<td>N</td>
<td>808</td>
</tr>
<tr>
<td>NNW</td>
<td>27</td>
</tr>
</tbody>
</table>

The above indicates that the contribution of the directions N and NNW were small compared to NNE. Therefore the wave modelling is restricted to use waves from the NNE. When the total yearly sand losses of the Wai San Ting sandbar are determined, besides the NNE also the N and the NE directions are taken into account. Here only the wave climate for the direction NNE is schematized in Table A-4.

The volumes shown in Table A-4 are calculated for the overall yearly wave climate, including waves during typhoons. It can be noticed that the contribution of the typhoon waves ($H_s = 5.0$ m) produced 1,800 kN/m/yr, which is on the same order of the 2.5 m waves. However, due to the fact that typhoons primarily occur during the South West monsoon period, the 5.0 m wave condition is omitted in the NNE schematisation.

From Table A-4 it can be observed that the wave height which has the largest effect on the site is $H_s = 2.5$ m. Figure A-12 and A-13 shows that, although the 1m wave has the largest occurrence percentage per year, the 2.5m wave has the largest yearly contribution of wave energy.

<table>
<thead>
<tr>
<th>$H_s$ (m)</th>
<th>$T_m$</th>
<th>% occurrence</th>
<th>Yearly wave energy (kN/m/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>calm</td>
<td>-</td>
<td>1.26</td>
<td>-</td>
</tr>
<tr>
<td>1.0 m</td>
<td>5.2</td>
<td>24.80</td>
<td>2,700</td>
</tr>
<tr>
<td>2.5 m</td>
<td>7.5</td>
<td>15.40</td>
<td>10,500</td>
</tr>
<tr>
<td>3.5 m</td>
<td>8.8</td>
<td>4.40</td>
<td>5,900</td>
</tr>
<tr>
<td>5.0 m</td>
<td>10.0</td>
<td>0.64</td>
<td>1,800</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td>46.50%</td>
<td>20,900</td>
</tr>
</tbody>
</table>
Appendix

Wave Height Occurrence / year
NNE Direction

Figure A-12: NNE wave height occurrence per year

Total Yearly Wave Energy Wave Height
NNE Direction

Figure A-13: Total yearly contribution of wave energy
South West monsoon (typhoon) period

On basis of the study of the typhoon climate for the period 1981 to 1992 in the above mentioned Working Paper No. 3, the following assumptions were made:

- Number of typhoons per year : 5
- Average duration per typhoon : 72 hrs
- Average wave period $T_{1/3}$ : 9 s
- Average typhoon wave height $H_s$ : 2.4 m

For the 5 typhoons per year the following directions were adopted:

N : 2 typhoons  
NNE : 1 typhoon  
SSW : 2 typhoons
APPENDIX XI: 2-D wave and flow modelling: overwash on the Wai San Ting sandbar

Section XI-1 describes the 2-D wave modelling and section XI-2 gives an description of the 2-D flow on and around the Wai San Ting sandbar. The conclusions of the 2-D modelling are given in section XI-3.

XI-1 2-D wave modelling: contribution of wave action to washover on the Wai San Ting sandbar

XI-1.1 Introduction

The purpose of the wave modelling is to study the wave conditions around the Wai San Ting. The modelled waves propagate through a wave field from deep water to the shallow intertidal Wai San Ting area where they experience shoaling, refraction, diffraction and energy losses (breaking and bottom friction). The second purpose of the wave modelling is to provide data for the two-dimensional hydrodynamic model, which simulates the water level set-ups and wave driven alongshore and cross-shore currents caused by the change in radiation stresses in the surf zone and around the studied sandbar. The combined use of the wave and hydrodynamic model provides an indication of the contribution of wave action to the overwash magnitude on the Wai San Ting.

The wave modelling is performed with MIKE21_NSW, a near shore wave model which describes the propagation, growth and decay of short period and short crested waves in nearshore areas. A description of the model is given in Appendix VIII.

XI-1.2 Operational wave conditions

The results of the wind and wave analysis from section 6.3.3 are used here for the specification of the operational wave climate. A "weighting" of the contribution of the waves from various directions on the total yearly wave energy is carried out in Appendix X. This assessment and "weighting" of the waves was carried out for the SW monsoon period, during which typhoons play a major role, and the NE monsoon, during which strong winds consistently blow from the NNE. In Appendix IX is the choice for the operational waves used in the wave model explained. Here only the resulting wave climate is specified. In Table A-5 the results are shown of the wave analysis. The 5 most contributing directions to the transport capacity are listed in that table (next page).

Typhoons are not included in the 2-D wave modelling. In section 6.5, where the yearly sand losses due to washovers on the Wai San Ting sandbar is quantified with the 1-D overwash model, the effect of one typhoon from the NNE is simulated. The typhoon influence on the sand losses is studied because the sand losses due to one low frequent and high magnitude storm can be significant. For the 2-D modelling however the only deep sea wave condition that is used in the model is the NNE winter condition during strong winds. The following parameters are used:

\[ \Rightarrow \text{deep sea wave height } H_s = 3.5 \text{ m} \]
\[ \Rightarrow \text{deep sea mean wave period } T_m = 7.0 \text{ s} \]
\[ \Rightarrow \text{deep sea mean wave direction } \theta_m = \text{NNE (335°)} \]
Table A-5: Wave direction/energy relationship

<table>
<thead>
<tr>
<th>Direction</th>
<th>Total yearly wave energy (kN/m/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NNE</td>
<td>21,000</td>
</tr>
<tr>
<td>N</td>
<td>800</td>
</tr>
<tr>
<td>S</td>
<td>400</td>
</tr>
<tr>
<td>SSW</td>
<td>200</td>
</tr>
<tr>
<td>SW</td>
<td>1,000</td>
</tr>
</tbody>
</table>

XI-1.3 Model strategy: layouts and set-up

To determine the wave conditions around the Wai San Ting sandbar, the deep sea wave conditions must be transferred to the shallow water area of the intertidal sandbar. The wave height and direction must be determined in progressively finer detail. This is achieved by starting with a large scale model with a grid size of 100m x 400m and progressed to a 10m x 40m grid spacing. The aim of the wave modelling finally was to model the wave action on a fine grid of 10m x 40m, from where the wave information could be transferred to the 20m x 20m flow model, which covers a part of the Wai San Ting area. Then the radiation stress currents can be included in the estimation of the overall (overwash) current pattern.

To achieve this goal, three wave models are used: a regional model, covering the Yunlin coast and the Penghu Islands (100m x 400m), a local model (1) covering the whole Wai San Ting sandbar (37.5m x 150m) and a local model (2), which covers a part of the Wai San Ting from the surf zone at the north side to the intertidal sheltered area at the lee-side (10m x 40m). Although the grid size of local model (2) provided a good wave field, it was not small enough to model the surf zone area and the barrier crest. Concentrating on washover events these areas are very important. In the surf zone, for instance, the wave set-up contributes to the water level elevation responsible for overwash. Therefore the grid size in the wave direction is reduced to 10m and perpendicular to this direction to 40m. Smaller grid sizes will increase the computation time too much.

Regional wave model

The regional wave model is set-up to model the waves coming from deep sea from the NNE. This regional model, which has a grid size of 100m x 400m, covers the total area of the Yunlin coastline and the Penghu Islands. The model orientation on the map of Taiwan is shown in Figure A-14 and the regional model area is shown in Figure A-15. In Figure A-15 also the local wave model 1 is plotted. The details of the model are described in Table A-6.
Figure A-14: Taiwan with regional wave model
Figure A-15: Regional wave model: wave height and direction; $H_s=3.5\text{m}$; $T_m=7.0\text{s}$; wind=$13\text{m/s (N)}$.
Local wave model 1

This local model is set-up for the N direction. Boundary conditions of waves from the NNE regional model where transferred to the model west boundary of the local model 1. Best model results are obtained when the waves enter the model perpendicular to a model boundary. When transferring the waves from the regional to the local model 1, the mean wave direction of the NNE wave is modified by -22.5 degrees (= N) to place the waves entering the west boundary perpendicular again. The model area is shown in Figure A-16, where also the area of local model 2 is plotted. The model details can be found in Table A-6. The model orientation relative to the regional model is shown in Figure A-15.

Local wave model 2

The local model 2 covers a part of the Wai San Ting from the surf zone across the bar to the lee-side and has a grid spacing of 10m x 40m. The model area is shown in Figure A-17 and the details of this local model are specified in Table A-6. Again the west boundary contains transfer wave data obtained from local model 2. The mean wave direction is now modified by -25 degrees relative to the local model 2. The orientation of the model is NNW and the orientation relative to local model 2 can be observed in Figure A-16, where the model area of local model 2 is plotted in the local model 1 area.

Table A-6: Details of wave models

<table>
<thead>
<tr>
<th></th>
<th>Regional wave model</th>
<th>Local wave model 1</th>
<th>Local wave model 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model orientation*</td>
<td>NNE (112.5°)</td>
<td>N (90.0°)</td>
<td>NNW (65.0°)</td>
</tr>
<tr>
<td>Model origin (latitude, longitude)</td>
<td>119.731°E 24.194°N</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Grid size (dx*dy,m)</td>
<td>100 * 400</td>
<td>37.5 * 150</td>
<td>10 * 40</td>
</tr>
<tr>
<td>Model dimensions (x*y,km)</td>
<td>105 * 86</td>
<td>36 * 26.25</td>
<td>9.6 * 5.2</td>
</tr>
<tr>
<td>Model dimensions (x*y, grid points)</td>
<td>1050 * 215</td>
<td>960 * 175</td>
<td>960 * 130</td>
</tr>
</tbody>
</table>

XI-1.4 Boundary conditions

To transfer wave data from the Tong Chi location to the local study area around the Wai San Ting, the wave data at Tong Chi is used to specify the west boundary condition of the regional wave model. The west boundary is therefore an essential boundary condition. The north and the south boundaries conditions of this model are specified as symmetric, and the east boundary as absorbing, thereby allowing all wave energy to pass out of the model without reflections. Essential, symmetric and absorbing boundary conditions are described in Appendix VIII.

*: angle between true north and the model y-axis, measured clockwise
Figure A-16: Local wave model 1; wave height and direction
Figure A-17: Local wave model 2: wave height and direction
By choosing smaller models (local models 1 and 2) inside the regional model, the boundary conditions are transferred from the regional model, via local model 1, to the barrier shore, which is modelled by local model 2. Table A-7 shows the combinations of boundary types used for the three wave models.

**Table 7: Boundary conditions**

<table>
<thead>
<tr>
<th>Boundaries</th>
<th>North</th>
<th>West</th>
<th>South</th>
<th>East</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regional wave model</td>
<td>symmetric</td>
<td>essential</td>
<td>symmetric</td>
<td>absorbing</td>
</tr>
<tr>
<td>Local wave model 1</td>
<td>symmetric</td>
<td>transfer data regional model</td>
<td>transfer data regional model</td>
<td>transfer data regional model</td>
</tr>
<tr>
<td>Local wave model 2</td>
<td>transfer data local model 1</td>
<td>transfer data local model 1</td>
<td>transfer data local model 1</td>
<td>transfer data local model 1</td>
</tr>
</tbody>
</table>

At the essential (west) boundary of the regional model the following constant wave parameters are specified:

- Wave height \( H_s = 3.5 \) m
- Mean wave period \( T_m = 7.0 \) s
- Mean wave direction \( \theta_m = \text{NNE (0°)} \)
- Max. deviation from \( \theta_m \), \( \theta_d = 60° \)
- No. of grid points in \( \theta \)-dir. \( n_{dir} = 13 \)
- Grid spacing in \( \theta \)-direction \( \Delta \theta_d = 10° \)
- Directional spreading index \( n = 4 \)

The mean wave direction is defined relative to the model x-axis measured positive anti-clockwise. The directional spreading from wave energy is defined by \( \theta_d \), the number of grid points in \( \theta \)-direction, the grid spacing in the \( \theta \)-direction and the directional spreading index \( n \) (all described in Appendix VII).

Other parameters, used in the three models are:

- Nikuradse roughness \( k_m = 0.02 \) m
- Breakings index \( \gamma_1 = 1.00 \)
- Breakings index \( \gamma_2 = 0.80 \)
- Model constant \( \alpha = 1.00 \)

In the regional model also wind generation is specified, as being the wind speed and direction which occurs during the NNE monsoon:

- Wind speed \( S_w = 13 \) m/s
- Wind direction \( \theta_w = 0° \)

The wind direction is defined positive anti-clockwise with respect to the x-axis. The reference level is chosen at +2m TD, to insure that the model area is always flooded.
XI-1.5 2-D wave model results

The initial wave height and direction at the deep sea west boundary of the regional model is 3.5m and NNE respectively. From the regional wave model results, plotted in Figure A-15, it can be observed that the mean wave direction is modified, when the wave approach shallower water near the site, from NNE to N. Looking at the results of the local wave model 1 in Figure A-16 it shows that the wave direction at the model west boundary (x=0) heads from N to S, but due to refraction the wave direction near the Wai San Ting (x=700) is changed to NNW. In this situation the waves approach the barrier normal to the shore. In local wave model 2, shown in Figure A-17, the refraction is much less. The mean direction at the model west boundary is about 15° relative to shore normal. In the surf zone the waves will refract a bit more, decreasing the angle between shore normal and the waves direction and causing a driving force alongshore due to the change in radiation shear stress component $S_{xy}$.

The wave direction is modified from deep sea direction NNE to the NNW direction near the barrier shore. Concentrating on the significant wave height it can be observed that the height, as expected, decreases when the waves approach the shore. Is the wave height at the west boundary of the regional model 3.5m, the wave height in local wave model 2 is reduced to 2.5m. This wave height decay is mainly caused by the bottom dissipation of the wave energy. The largest dissipation occurs however in the surf zone near the barrier shoreface. The waves there decreases from $\pm 2.5m$ at the breaker to $\pm 0.1m$ on top of the sandbar. This wave height decay is shown in Figure A-18, which is a plot of the significant wave height $H_s$, the mean wave period $T_m$ and the mean wave direction $\theta$ (relative to the model x-axis), obtained from the local wave model 2 along the line $(0,65)$ and $(960,65)$.

The results of the wave-induced (overwash) currents are presented in the next section, where the flow modelling is described. A radiation stress field containing the components $S_{xx}$, $S_{xy}$ and $S_{yy}$ are transferred from the local wave model 2 to the hydrodynamic flow model which has the same size and bathymetry as the local wave model 2, but has a grid spacing of 20m*20m.

A calibration of the wave models is performed in Working Paper No. 3: Hydraulic and Morphological studies. In that study wave height measurements at Tong Chi and Taihsi (located in the study area) are compared with the model results.
Figure A-18: Wave height, period and direction along line (0, 65) (960, 65) (local wave model 2)
XI-2 2-D Flow modelling: tide-induced washover on the Wai San Ting sandbar

XI-2.1 Introduction

The purpose of the 2-D flow modelling is to obtain the hydrodynamic conditions (current velocities and water level elevations) which occur during a tide and wave-induced washover event on the Wai San Ting sandbar. Especially the two-dimensional effects of the tidal currents and overwash currents, which flow around and across the sandbar, need to be known to obtain a clear understanding of the overwash process.

To obtain ideal modelling results a very fine definition of the bathymetry and flow conditions is required. For modelling the processes in the surf zone (wave-induced currents) and across the sandbar (overwash currents), a very fine grid size is required (say 20m or less). This would however result in unreasonable large run times, when modelling the entire study area around the Wai San Ting. Modelling the whole area is however essential to be able to ensure that the tidal currents and elevations in the detailed coastal area are correctly driven by the large scale tidal movements in the Taiwan Straits. It is therefore necessary to apply a range of models from large to small scales to eventually come to the required resolution of the model that simulates the tide- and wave-driven overwash currents.

The data of large scale model, with a grid size of 1000m*1000m and two smaller flow models of 250m*250m and 75m*75m are obtained from the Yunlin Hydraulic and Morphological study. For simulating washovers on the Wai San Ting sandbar a section across the bar is modelled with a 20m*20m grid. The boundary conditions for this model are transferred from the 75m model.

The flow modelling is performed with MIKE21_HD, a hydrodynamic model which simulates the water level variations and flows in response to forcing and resistive functions as tidal water level differences, wind, wave-induced radiation stresses, barometric pressure, coriolis and bed resistance. The water levels and flows are resolved on a two-dimensional rectangular grid covering the area of interest. A description of the model is presented in Appendix VIII.

XI-2.2 Model strategy, layouts and setup

In the study the choice of the 1000m model layout was made after a careful study of the large scale driving forces in the study area (tide, wind and waves) and the availability of high quality data for boundary conditions. In this largest model it was required to model a huge area in comparison with the size of the Wai San Ting sandbar. This was necessary to ensure that the global tidal and wind effects in the Taiwan Straits could be successfully transferred to the detailed local areas around the sandbar. The main criterion was that the large 1000m grid model provided a good description of fluxes and levels along the boundaries of the smaller 250m grid model. The 250m grid model was designed to be used as the main facility for the overall flow patterns and strengths along the Yunlin coast. For finer resolution of the complex Wai San Ting area, a 75m grid model was chosen. Inside this model area, boundary conditions are transferred to a very fine model with a 20m grid size. This high resolution is necessary to be able to model the washover processes which occurs at a specific section of the

---

Wai San Ting sandbar (from the emerged south end of the bar to the large tidal channel in the northern part of the Wai San Ting (Figure 82, Chapter 7). Due to the fact that a very fine grid is used, the model covers just a 3km-section of the sandbar from the surf zone to the lee-side of the bar, where the depth is about 5 meters. A 20m grid model that covers the whole sandbar would lead to excessive run times.

The bathymetries of the 1000m, 250m, 75m and 20m grid models are presented in Figures A-19, A-20, A-21 and A-22 respectively. In Table A-8 the layouts and details of the flow models are presented.

Table A-8: Details of flow models

<table>
<thead>
<tr>
<th>Model orientation&lt;sup&gt;6&lt;/sup&gt;</th>
<th>1000m model</th>
<th>250m model</th>
<th>75m model</th>
<th>20m model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model origin (long.\ latitude)</td>
<td>NNE (115°)</td>
<td>NNE (115°)</td>
<td>N (0°)</td>
<td>NNW (65°)</td>
</tr>
<tr>
<td>119 24' 59.98 E</td>
<td>119 45' 41.10°E</td>
<td>119 58' 00.41°E</td>
<td></td>
<td></td>
</tr>
<tr>
<td>23.12' 59.99&quot;N</td>
<td>23 21' 22.20&quot;N</td>
<td>23 24' 33.66&quot;N</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grid size (dx*dy, m)</td>
<td>1000m * 1000m</td>
<td>250m * 250m</td>
<td>75m * 75m</td>
<td>20m * 20m</td>
</tr>
<tr>
<td>Model dimensions (x*y, km)</td>
<td>138 * 87</td>
<td>85 * 45</td>
<td>25.5 * 19.5</td>
<td>3.0 * 8.2</td>
</tr>
<tr>
<td>Model dimensions (x*y, grid points)</td>
<td>138 * 87</td>
<td>340 * 180</td>
<td>340 * 260</td>
<td>150 * 410</td>
</tr>
</tbody>
</table>

XI-2.3 Model simulation periods and time step

The models simulate the tide-induced currents and water level elevations in the winter (March), because then the typical NE monsoon winds play an important role in combination with the tides. Another reason for choosing the March conditions is because of calibration and verification purposes. The calibration and verification data were only available for the simulation periods March and July. In this study only winter conditions are modelled in combination with spring tide.

Simulation periods<sup>7</sup>

<table>
<thead>
<tr>
<th>Model</th>
<th>Simulation periods</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000m model</td>
<td>10/03/92 00h00 - 22/03/92 00h00</td>
</tr>
<tr>
<td>250m model</td>
<td>19/03/92 00h00 - 21/03/92 12h00</td>
</tr>
<tr>
<td>75m model</td>
<td>19/03/92 00h00 - 20/03/92 03h00</td>
</tr>
<tr>
<td>20m model</td>
<td>19/03/92 00h00 - 20/03/92 03h00</td>
</tr>
</tbody>
</table>

<sup>6</sup>: angle between true north and the model y-axis, measured clockwise

<sup>7</sup>: the simulation times are excluding the model warm-up or soft starts periods.
Figure A-19: Bathymetry 1000m flow model (incl. 250m, 75m and 20m flow models)
Figure A-20: Bathymetry 250m flow model (incl. 75m and 20m flow models)
Figure A-21: Bathymetry 75m flow model (incl. 20m flow model)
Figure A-22: Bathymetry 20m flow model
**Time step**

The model time steps were selected based on the Courant criterion and the stability displayed by the model. The Courant criterion is specified in section VII-3. The used time steps are:

- 1000m model: $\Delta t = 180$ s
- 250m model: $\Delta t = 45$ s
- 75m model: $\Delta t = 10$ s
- 20m model: $\Delta t = 6$ s

**XI-2.4 Other model parameters**

**Flooding and Drying**

The flooding and drying facility in MIKE21 is able to include and exclude computational areas dynamically during the simulation. This is an important facility because in the smaller models (75m and 20m models) with their tidal flats (Wai San Ting sandbar) it allows certain areas to be flooded or to dry out. The following values are given for the different models (the first value should be read as "minimum water depth before drying" and the second value as "minimum water depth before flooding"):  

- 1000m model: 0.10m - 0.50m  
- 250m model: 0.10m - 0.50m  
- 75m model: 0.10m - 0.30m  
- 20m model: 0.05m - 0.15m

Small differences between the flooding and drying depth can lead to instability problems of the model. In general it can be considered that with decreasing grid size the difference between the flooding and drying depth can be reduced.

**Eddy viscosity**

The eddy viscosity is specified in section VII-3. Here only the used values are given. In the model the following values are used:

- 1000m model: 30 m$^2$/s  
- 250m model: 30 m$^2$/s  
- 75m model: 10 m$^2$/s  
- 20m model: 2 m$^2$/s

**Bed roughness**

The roughness is specified by two-dimensional maps consisting Manning coefficients (defined in section VII-3). These Manning numbers depend on the depth and the bottom roughness, specified by the Nikuradse roughness $r = 0.02$m. The Manning numbers applied range from 20 m$^{-1/3}$/s (rough) to 70 m$^{-1/3}$/s (smooth).
Appendix

Wind

In all the models the following wind conditions are specified:

- Wind direction: NNE (22.5° relative to true north and model x-axis)
- Wind speed: 8.0 m/s
- Wind friction coeff.: 0.0026 (m/s)^2

Initial conditions

Boundary conditions for the 1000m model were introduced using a soft start of at least 100 time steps from an initial water level at rest at TD. For the 250m, the 75m and the 20m models, the models were started from rest at an initial (horizontal) elevation which matched as closely as possible the elevations in the transferred boundary levels. The start times are therefore set to coincide with a time in the tide when the transfer boundary levels resulted in only small water level slopes across the model.

XI-2.5 Boundary conditions

Boundary conditions required by the flow model need to be provided at every time step and at every grid point along the boundaries of the computational grid. The boundary conditions can be specified as "flux-boundaries" or "level-boundaries". Because no flux (discharge per unit width per time step) data was available in deep water, level-boundaries were specified all around the 1000m model. The 1992 tidal level recording, which were used for the boundary conditions, were provided by the THL (Tainan Hydraulic Laboratory) and are discussed in Working Paper No. 3.

In case of the 250m and 75m models the fluxes and levels were transferred from the larger model. At a level-boundary the tide level is given and at a flux-boundary the flux perpendicular to the boundary is given as input, these at every time step and grid point along the boundary. The following Table A-9 shows the boundary types used in the models:

Table A-9: Boundary types in the flow model

<table>
<thead>
<tr>
<th>HD model</th>
<th>West boundary (x=0)</th>
<th>South boundary (y=0)</th>
<th>East boundary (x=x_{max})</th>
<th>North boundary (y=y_{max})</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000m</td>
<td>LEVEL</td>
<td>LEVEL</td>
<td>closed</td>
<td>LEVEL</td>
</tr>
<tr>
<td>250m</td>
<td>LEVEL(^T)</td>
<td>FLUX(^T)</td>
<td>closed</td>
<td>LEVEL(^T)</td>
</tr>
<tr>
<td>75m</td>
<td>FLUX(^T)</td>
<td>LEVEL(^T)</td>
<td>closed</td>
<td>FLUX(^T)</td>
</tr>
<tr>
<td>20m</td>
<td>FLUX(^T)</td>
<td>LEVEL(^T)</td>
<td>FLUX(^T)</td>
<td>LEVEL(^T)</td>
</tr>
</tbody>
</table>

\(^T\): Transfer boundaries

XI-2.6 2-D results of tide-induced overwash

Due to the fact that the area of interest is restricted to the Wai San Ting sandbar, no results of the 1000m and 250m model are shown in this study. The two-dimensional maps displaying the flow fields and water level elevations of the 1000m and the 250m model are shown in the
Working Paper No. 3. In that study also the calibration and verification of the model results are presented. Unfortunately no calibration data is available for the area covered by the 20m model, implying that 20m model can not be calibrated or verified. However, comparisons between measurements and the model results within the 1000m, 250m, and 75m model areas showed that the higher the resolution (e.g. the finer the grid), the better the results (e.g. the smaller the differences with the measured data), due to the improved bathymetry resolution. This implies that the 20m model results should be reasonably acceptable, because the boundary conditions were transferred from the calibrated and verified larger models. In general the measured current speeds and water levels agreed well with the computed currents and levels. In cases where significant differences occurred a new calibration of the model was performed. A general accepted tool for calibration purposes is changing the bed roughness of the model, to get a good agreement between the model results and measurements.

75m model results

Tidal currents:
Figures A-23 and A-24 show the peak flood and peak ebb vector plots for the spring tide in March. It can be observed that the peak ebb flow (from N to S) is stronger than the peak flood flow over the whole region because of the wind, shielding of the flood flow by Wai San Ting. This tendency is caused by the forcing of the ebb flow by the strong NNE winds. It is during this period of the tide that overwash occurs.

High velocities are clear in SW direction along the Wai San Ting on the ebb tide as they approach the discontinuity at the southern tip and join the north-south flow system. At this location, the peak flood tide floods around the tip of the Wai San Ting rather than pass it and there is far less of a discontinuity in the flow field compared to the flood tide.

Washover currents:
Washover flows occurs at high tide, when the water level in front of the barrier exceeds the crest height and a water level difference across the bar occurs. Figure A-25 shows a plot of the water levels on either side of the sandbar for spring and neap tide. The Figure shows that assuming the sandbar becomes flooded at elevations above +0.5m TD the water level on the north side is always higher than that on the south side (the same is true for summer conditions), caused by a difference in phase and tidal amplitude (shallow water at south side, relative deep water at the north side).

Figure A-26 shows the initial state of the washover current. The crest height in the 75m model as well as in the 20m model are set to a constant height of +0.5 TD. Due to the fact that hardly no data of the geometry of the crest was available, the crest could not be modelled very accurate. In Figure A-24 it can be noticed that except for the southern end the whole area of the Wai San becomes flooded. Where along the crest overwash occurs depends for a great deal on the geometry of the crest. Certain areas will not be flooded when the crest height is locally higher than the maximum water level. However, looking at the bathymetry plot in Figure A-21 it can be concluded that due to the fact that the south side of the sandbar is very shallow and wide, a considerable amount of sediment is transported to that side by overwash currents. The shallow water area at the south side extends from the main land side (east) up to the beginning of the southern tip, which remains above the water. This implies that overwash current will also occur over that length of the crest (± 10km). This satisfies the assumption that the crest will be overwashed over a 10km stretch and that the chosen crest height is reasonable.
Figure A-24: 75m flow model; current velocity and surface elevation; spring tide March (1992); peak ebb
Figure A-23: 75m flow model; current velocity and surface elevation; spring tide March (1992); peak flood
Figure A-25: Tidal elevation in front side and back side of the Wai San Ting; spring tide (March, 1992)
Figure A-26: Vector plot velocities; 75m flow model: moment of overwash
Appendix

20m model results

The 20m model is specially set-up to examine the overwash currents and elevations on a fine model grid. The boundary conditions are all transfer boundaries from the 75m model. A problem occurred at the west and east boundary of the model, caused by the flooding and drying. After all, at every time step and grid point the boundary must contain water. The west and east model boundaries however cross the crest of the bar and fall dry when the water level is lower than the crest. This problem is solved by "dredging" channels across the barrier along the west and east boundary. The depth of the channels is chosen in such a way that at every time step the boundaries are flooded. Looking at the tide curve in Figure A-25 the depth of the channel must be lower than LLW (= -1.1m TD), plus an extra depth for the flooding and drying (+0.05m). Thus the channel depth must be equal or lower than -1.15m TD. The width of the channels is 2 grid points (40m).

To make sure that the channels do not effect the overwash current, the roughness is increased to a Manning number of 10 (very rough). Comparisons of overwash currents computed with and without the channels showed that no differences occurred in the area between the channels from x = 20 to x = 130.

Washover current

Figure A-27a, b and c shows the washover event at three time steps at an interval of 30 minutes:

\[ t = 09h 20m \] Just before overwash occurs; water level raised to +0.5m TD (=crest level); water level in front already higher than at the backside of the bar (Figure A-25)

\[ t = 09h 50m \] Overwash current reached mid-section of the crest; alongshore tidal current at north side effects washover current only along shoreface

\[ t = 10h 20m \] Total crest area flooded and overwash current is fully developed. Note the southward flow direction just in front and just at the back side, while the current on top of the crest is perpendicular to the length-axis of the Wai San Ting.

2-D effects

In the above description of the three-stage overwash process it is concluded that in the 20m model the overwash current direction is perpendicular to the length-axis of the Wai San Ting sandbar, except for a small area parallel to the shoreface (direction influenced by longshore current) and just at the end of the crest, where the current is directed more southwards (deeper water). This trend is confirmed by the 75m model. In Figure A-26 it can be seen that on the total overwash crest length the overwash current is directed normal to the length-axis of the barrier.
Appendix

(Gridspacing 20 m)

L

(Gridspacing 20 m)

L

(Gridspacing 20 m)

1 m/s

Fig. A-27a

Fig. A-27b

Fig. A-27c

3 stage overwash 20m flow model: moment of overwash

A-65
Wave-induced washover

The radiation stresses from the local wave model 2 are used in the hydrodynamic model to determine the wave-induced overwash conditions. The model size is the same as the local wave model 2 but the grid size is 20m*20m. The results are shown by Figure A-28, which is a vector plot of the wave-generated currents.

To model the wave-induced currents and water level set-up a 2-D radiation stress map with the three radiation stress component $S_{xx}$, $S_{xy}$ and $S_{yy}$ must be specified over the whole model area. The initial water level is set to +2m TD, to insure that no areas of the model become dry. The boundary conditions are obtained using the programs WAVCUR and WAVTRN, described in Appendix VIII. The radiation stresses at the wave model boundaries are transferred by these programs to currents/fluxes and water levels.

A problem occurred concerning the wave set-up, which remained constant from the shoreface to the east model boundary. With a constant wave set-up no overwash occurs, as was already concluded in section 4.3. This problem is solved by specifying the downstream (east boundary) water level equal to initial reference water level. The west and east boundary condition are now specified as level boundaries with a constant elevation at +2m TD. The north and south boundaries of the model are modified: the surface elevation along these boundaries is adjusted by bringing the surface elevation back the initial water level at the end of the crest.

The vector plot in Figure A-28 shows that the overwash current velocity caused by the wave action occurs at the beginning of the crest. Velocities between 0.2 to 0.4 m/s can be observed. In Figure A-29 the velocity variation and water surface elevation along a line between the grid points (0,120) and (490, 120) is plotted and Figure A-30 shows the cross shore bathymetry of that Wai San Ting section. At this specific cross section the maximum velocity is about 0.4 m/s.

Compared to the tide-induced overwash in a wave-induced overwash situation 2-D effects play an important role. This is caused by the waves, which in shallow water directly react on changes in the bathymetry (refraction, diffraction, shoaling and breaking). This results in a different wave set-up along the barrier shore, which causes changes in the water levels difference across the sandbar. The overwash velocity therefore also changes at various points along the crest.

N.B. After inspection of the north and south boundary conditions along the model it could be seen that the fluxes oscillated strongly near the shoreface of the bar. This instability is probably caused by the irregularities of the bathymetry. No time was available to solve this problem. Therefore only "level conditions" are specified along the model boundaries. This however means that the wave-induced currents can not be combined with the tide-induced currents, because then the wave action must be specified as fluxes, which showed an oscillating pattern.
XI-3 Conclusions 2-D flow and wave modelling

2-D effects

The 2-D effects of washover on the Wai San Ting sandbar are examined to determine the "limitations" of the 1-D approach. From the results it can be concluded that the 2-D effects of the tide-induced overwash are restricted to the areas just in front of the bar, and at the lee-side of the bar. At the front side the interaction of the longshore current and the overwash current causes a current which flows under an angle with the shore. Also just at the end of the crest, where the overwash current flows in deeper, is the current direction not shore normal. However, on the crest the current does flow almost perpendicular to the length-axis of the bar, which shows that the 1-D approach, which assumes perpendicular currents across the bar, is correct.

The direction of the waves, which is assumed to be shore normal in the 1-D overwash model, is in the 2-D wave model also nearly perpendicular to the shore, caused by refraction. This is the case for deep sea waves which approach the bar from the NNE and refract until they reach the bar almost shore normal (NNW). It can be concluded that waves approaching the bar from northerly directions will refract until they have a shore normal direction. Waves from westerly directions will refract more than the waves from northerly directions. This causes a decrease in wave set-up due to the fact that the wave energy is spread out over a larger area than in case the waves approach the shore almost perpendicular. In the 1-D model the wave angle cannot be specified and no refraction is computed. The influence of refraction on the wave set-up can be included by specifying a smaller wave height at the upstream boundary.

During a wave-induced washover the current direction cannot be well defined at the front side of the bar, where due to the irregularities of the bottom (submerged longshore bars) the wave set-up, and therefore the overwash current, varies along the crest. This results in a different angle of the overwash current just in front of the bar. On the crest the wave-induced current does flow almost perpendicular to the length-axis of the bar.

Wave and flow conditions for 1-D model boundaries

Tide conditions:
The tidal water level variations are obtained from the flow model with the 20m grid size. The water levels are taken at two points on both sides of the bar (see T1 and T2 in Figure A-27). The used tide curves taken from locations T1 and T2 for the winter condition are plotted in Figure A-25. The summer tide curves at both sides of the bar are plotted in Chapter 6.

Wave conditions:
The used wave heights and directions for the 1-D overwash model are given in section 6.5.1.
Figure A-29: Wave set-up and overwash velocity: along line (0, 120) (490, 120) Figure A-27
Figure A-30: Cross-shore profile: along line (0,120) (490,120) Figure A-27