Cross-shore profile development during storm conditions

Analysis of field data gathered at Petten sea defence

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Preface

The present report forms my Master of Science Thesis and is carried out within the framework of my study Environmental Engineering at the University Politecnico di Milano.

This study concerns the cross-shore sediment transport that takes place during storms and the consequences on an existing sea dike, related with the changes in the root mean square wave height and peak period.

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Abstract

Introduction

During storms the sea level rises considerably above the normal high water level and the wave field changes. As a result storm-driven waves reach the coast and erosion occurs. This can be considered as an extreme case of the continuous adjustment of the coastal profile due to the continuous mutation of hydraulic and meteorological conditions, which are difficult to predict. In this way a new cross-shore (in a direction perpendicular to the coast) beach profile is developed and this change may cause problems to existing structures and settlements.

This is a typical cross-shore sediment transport problem, and this matter is extremely important for the Netherlands. In fact, most of the inhabitants of the Netherlands live well below the mean sea level. At some locations the population and the infrastructures are only protected from the sea by a narrow stretch of sandy beaches and dunes, so reinforcement works will be necessary to prevent a break through during storm surges.

Therefore it is necessary to study problems correlated with the failure mechanisms of protecting structures and in particular our objective of study is wave overtopping due to extreme conditions, which can cause structure's collapse.

This situation calls for a detailed knowledge of the erosion processes because the wave height, the sea level and the wave runup are directly connected with the temporal evolution of the bottom profile during a storm.

Problem definition

Compared to moderate conditions, during storm sand is eroded in a more pronounced way from the unprotected coast. This is due to different mechanisms, such as the water level that is higher than normal, the rather high waves, and the storm surge, that may occur sometimes.

When an extreme event as a tempest occurs, short term erosion takes place. The significant profile changes are in that case restricted to the upper part of the profile only and take place in a very short period of time.

In most cases this erosion can be considered to be only a momentary disturbance of the long-term development of coastal profile but even on a stable coast without any long-term erosion, the dynamical adjustment of the cross-shore profile due to short-term developments may lead to safety problems.

To protect the coast from this kind of processes we may have protection structures as dikes. Design criteria for these flood protection works are based commonly on an estimate of some extreme condition, usually a combination of surface level elevation and wave characteristics.

The cross-shore profile in front of the structures may change during storm surges and its evolution in time affects the propagation of waves towards the flood protection works. Hence, the wave conditions to which the structure is exposed may be different from what one would expect on the basis of the cross-shore profile that is encountered during moderate conditions. To deal with this, it is necessary to gain insight into how the cross-shore profile evolves during a storm.

Within this framework it has been simulated, for eight observed storms, the cross-shore profile evolution at the sea defence at Petten. The results obtained with the
simulations have been evaluated in terms of general characteristics and tendencies of profile evolution and changes of wave conditions.

**Summary of results**

The different computations show in some cases an increase in the wave height at the sea defence and in some cases a decrease. This phenomenon depends from the increase during a storm of the crest level of the offshore bar (breaker bar that corresponds to moderate wave conditions). In fact, if this increase lead to a crest level (below MWL) comparable with the wave height in the same area, a relevant fraction of the waves will break. This causes an energy dissipation that results in a smaller (with respect to the wave height used as input in the computations) wave height at the sea defence. On the contrary, if the increase of the crest level is not enough to cause wave breaking, the waves on the bar will only undergo a shoaling effect without losing energy. The result of this is the fact that at the dike there will be a bigger wave height, as there has not been energy dissipation caused by breaking.

Regarding bottom variations, it can be seen that in some zones there is erosion and in some others accretion. Most of the computations show the formation of a scour hole at the toe of the dike of an average depth of 2.5 metres. The sediment eroded in this zone will then settle seaward, leading to the formation of some bars in the range of 150 metres. The offshore bar shifts horizontally in the offshore direction and increases its crest level.

At an average depth of -11 metres all the changes in bottom profile become negligible (at the scales of the considered storms) with respect to what happens closer to the shore. This point means that, from the cross-shore coordinate that characterises it seaward, all the mechanisms that have influence in sediment transport are in balance. In fact it doesn't mean that there the offshore-directed sediment transport rate is zero, as we calculated its value, which is about 80 m³.
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1 Introduction

1.1 General

Low-lying countries like the Netherlands are strongly dependent on safe coastal defences (sea dikes and dunes). In the past, the design of dikes was based more on experience instead of calculation methods based on insight into physics. The increased demand for reliable design methods for flood protection studies in the Netherlands has resulted in increased research in this field. Sea dikes are built to protect hinterland areas (including land reclamation) when resources become endangered by erosion or inundation due to storm surges, waves and wave overtopping. The proper design of these structures is of major importance and it depends on wave characteristics. Therefore, this study deals with the investigation of bottom changes during extreme conditions, because a modification in the bottom profile affects the characteristics of waves. If the latter will change, also the wave propagation and dynamics near the sea defence will be modified, sometimes in a substantial way that can be dangerous for the dike's safety.

1.2 Objective of study

The main objective of this study is to investigate cross-shore profile evolution during a storm. For this purpose, we will use the physical-mathematical model Unibest-TC to simulate morphological changes of the cross-shore profile in front of the sea defence at Petten for eight storms observed in February 1999, November 1999 and January 2000. These observations are part of an extensive field campaign.

For the present study we have selected data on wind and wave conditions during the eight storms and on bottom profiles as they have been shaped by moderate conditions. These profiles serve as initial condition for the model computations. In this his way, the simulations provide storm-induced morphological changes. The model output is analysed to find what morphological change of cross-shore profile is needed to find a substantial increase of the root mean square wave height at our sea defence that can cause a failure mechanism like wave overtopping.

The root mean square wave height is defined as:

\[ H_{rms} = \sqrt{\frac{1}{N} \sum_{i=1}^{N} H_i^2} \]

where \( N \) is the number of wave heights \( H_i \) taken into account and \( H_i \) are defined as the difference between a consecutive positive and negative water level elevation measured in a given sample period.

The method followed to investigate bottom changes in front of the dike consists of two steps. First, there has been data transformation, then simulations with the aid of Unibest-TC. The data transformation is necessary because from the field data we only have time series of quantities (as vertical acceleration and tilting of the sea surface, wind velocity, wind direction) that are correlated with the quantities that we need as input of Unibest-TC. So we have to transform them in order to obtain the required input data for the model, which are \( H_{rms} \), \( T_p \) (peak period, period that corresponds to the
maximum wave energy density), $\theta$ (wave direction with respect to shore normal),
wind velocity and direction.
Moreover, we will average these quantities in time, as we make the assumption that
in the simulations wave conditions are time-invariant during a storm that is
standardised to one day duration.
Vertical acceleration and tilting of the sea surface will be transformed in the root
mean square wave height and wave direction with the use of a procedure described in
Chapter 3. Wind velocity and wind direction data will be averaged over the period of
measure in order to obtain a unique value.
The computations will cover the second part of my work and they will concern
morphological changes and wave changes caused by storms in front of Petten sea
defence.

1.3 History of dike protection

Natural or artificial dunes and dikes are functioning in the Netherlands to protect
upland (population and economic values) against erosion or inundation due to storm
surges. A storm surge is a deviation from a normal elevation of the sea due to the
piling up of water against a coast by strong winds such as those accompanying an
intense storm.
The main purpose of a dike or sea wall is to fix the land and sea boundary. During
moderate weather conditions, sea walls neither promote accretion nor reduce the
regional trend of the coast to erode, but are constructed for protection of the territory
under extreme conditions.
There is still much to learn on the use of dikes and sea walls and their possible
disadvantages related to the disturbance of the natural coastal processes and even
acceleration of beach erosion. However, it should be said that in many cases when the
hinterland becomes endangered by inundation (as in the Netherlands) or by high-rate
erosion (possible increase of sea level rise) leading to high economical or ecological
losses, the dike or sea wall can even be a "must" for survival.
The coast of the Netherlands consists of about 300 km of dunes and about 100 km of
dikes and dams. The original length of sea dikes (until 1932) was about 700 km.
Dams closing the Zuiderzee (1932) and tidal estuaries of the Delta Project (1957-
1986) drastically shortened this length. The primary function of the dunes and dikes
is to protect the hinterland from flooding during periods of storm surges and heavy
wave attack.
To understand the historical development of protection by dikes in the Netherlands, it
is essential to understand the gradual rise of the sea level with respect to the land and
also the deposits of soil by the North Sea and the rivers. About half of the country is
below mean sea level and about 60% of the total population live in low-lying areas.
Without dikes more than half of the country would be flooded.
Under present climate conditions (over the last 100 years) the relative rise in sea level
(due to sea level rise and subsidence) is about 20 cm per century, and the coast is
strongly attacked by tidal currents and storm surges.
Increased sea level rise will have serious implications for the safety of the land
protected by dikes, dunes and other defence structures along the coast and the lower
part of main rivers. Sea level rise increases the risks of overtopping and the ultimate
collapse of these structures during storms, that's why it is important to study with a
computational model which morphological changes are caused by extreme
conditions.
1.4 History of the Netherlands

During the last geological periods, the Holocene, the first coastal barrier (dunes) along the North Sea was formed. Between this barrier and the higher Pleistocene area in the eastern part of the country, sand and silt were deposited. The rivers Rhine, Meuse and Scheldt added river silt, building and shaping the land by means of regular inundation. As a result, the geological profile of the western Netherlands shows sand and silt layers. In consequence of that, most dikes and dams are founded on soft soils and subjected to high settlement. Since a long time ago, people must have lived in the unprotected lower parts of the Netherlands. In the first centuries of the medieval period, people learned to protect themselves and their stock against storm surges by inhabiting naturally higher parts of the land and erecting artificial clay mounds ("terpen") on which their homes and
barns were built. Many such early settlements became the nuclei of present villages and towns. Considering the simple tools available at that time for digging and transportation of soil, and the construction of the terpen, one is impressed by the tremendous task these early settlers carried out.

An important step was made towards improving living conditions and safety by building dikes. In the ninth century, the first dikes were built. This developed in such a way that by the thirteenth century one can speak of a more organised method of dike construction. An area protected against high water levels by surrounding dikes is called a "polder". Dike construction at that time was undertaken by people who directly faced the elements of nature.

At first, the aim of dike construction was only defensive: people protected the land where they lived. In a later phase, the construction of dikes was used in a more offensive way, for example by reclaiming land from the sea. In this way, from the middle of the thirteenth century until today, in total about 550,000 ha of land was reclaimed. However, during the centuries, much of the previously reclaimed land was lost by attack of the sea, mainly due to storm surges, which many times caused destruction of the dikes.

Another phenomenon was the occurrence of landslides along tidal channels (most dikes were situated on loose soils), thus causing the disappearance of dikes. During high storm surges, the sea also eroded this land. Nevertheless, every time there was the spirit of the people to push back the sea. Most of the land was reclaimed again, despite the ever-occurring storm surges.

### 1.5 Design of dikes

The design of dikes and hydraulic structures in the past (until 1953) was mainly based on a trial and error methodology, rather than a deep understanding of the physics behind the phenomenon.

Sea dikes were generally constructed with clay and showed an asymmetrical trapezoidal cross-section with the steep slope at the inner side. In more recent times, a dike with wave energy absorbing outer berm was developed and sand was applied as core material. The height and the strength of dikes and structures had to be related to a certain storm surge level and wave action, especially the magnitude of the wave run-up. The run-up increases with the steepness of the outer slope.

![Fig. 1.2: Sea dike](image)

Previous storm surge levels were sometimes marked in stones of buildings. For more than a century, water levels have been continuously recorded. Wave characteristics and wave run-up were observed by eye so that reliable data were not available. Only the swag marks on the slope of dikes after storms gave an indication of the level of wave run-up. Later, these swag marks were obtained by measuring directly the water level. Taking into account the (still) water level, a rough value of run-up could be derived. With these data, it was possible to improve the design to a certain extent. In general, it can be said that the design was based on the highest observed storm surge level and the estimate of wave characteristics (height and period) and wave run-
up. The experience during centuries was that ever higher storm surge levels and more extreme wave action occurred than those that were encountered in the past. Overflow and overtopping of dikes caused, in most cases, instability of steep inner slopes, which in many cases induced a breach and consequently led to inundation. Depending on the storage capacity of the polder (depth, area and the composition of the soil under and in vicinity of the dike), deep and wide channels can develop in these breaches.

Even if before the great flood of 1953 the most common method of designing dikes used a procedure based on practical experience, around 1920, the first soil mechanical investigations were started in order to improve dike safety. The increase of storm surge levels was also locally due to the impoldering of relatively large tidal areas, which decreased the storage capacity needed for higher water levels. As a consequence and moreover due to erosion of the wave-reducing foreland of dikes, caused by tidal currents and wave action, wave attack and run-up became more severe. The more severe wave attack necessitated an improvement of the grass protection of the outer slope of dikes and this is a method that is used nowadays too, mostly in combination with concrete blocks. But also seaweed, rows of wooden piles and, in more recent years, boulders, pitched stone, concrete blocks, etc. were applied, in a way to make the dike more resistant and more enduring.

1.6 Measurements at Petten

Dikes, dunes and seawalls form nowadays the major line of defence for the Netherlands against extreme storm surges at sea.

Many decades of basic research have provided the tools for the prediction of the offshore wave climate and currents, the transition to the hydrodynamic forces on the structure, the assessment of the stability of the cover layer and the design of an optimum shape to meet other requirements such as overtopping.

Unfortunately, there haven’t been developed yet many design methods that take into account the influence of substantial morphological changes (such as toe erosion) that can occur in front of a dike, on wave height and peak period. To gather data on this phenomenon a field campaign has been carried out at Petten sea defence by Rijkswaterstaat.

![Fig. 12: Location of Petten sea dike along the Dutch Coast](image_url)

The Petten sea dike is a smooth and impermeable structure. It consists of a lower slope 1:4 (protected by basalt blocks), an almost horizontal berm of ca. 20 metres and an upper slope 1:3.5 (protected by asphalt and asphaltic concrete).

Part of the collected data concern wave conditions, wind velocity, wind direction, surface elevation and these data will be used in the present model to gain insight the
morphodynamics of the sea bottom during a storm. The reliability of the instruments under heavy storm conditions has been demonstrated and together with the use of data verification techniques, this measurement campaign resulted in a valuable data set of wave dynamics.
2 The sediment transport problem

2.1 Introduction

The state of the marine environment is a matter of concern for several countries. An important aspect of this is the future evolution of the coastline, where uncertainty arises both from the morphological impacts of human interference and from the expected future climatic variations with the associated rise in sea level. These changes may gradually alter patterns of waves and currents and, hence, the sediment transport pathways in coastal areas. Such changes may profoundly influence the stability of the shoreline itself, with all that this causes for the security and the livelihoods of the large population living near the coast. Changes may be quick or slow: in these cases, we have short-term and long-term erosion respectively. Longshore transport is considered to be significant in long-term evolution, whereas cross-shore transport is considered to be responsible for short-term or seasonal variations. Our study will only focus on cross-shore transport caused by a storm. Coastal management relies increasingly on predictions made by physical-mathematical models of hydrodynamic and sediment transport processes and of the resulting bed level changes, derived from spatial gradients of the sand transport rates. The key element in these models, and of Unibest-TC model too, is the formulation of the sand transport as a function of wave and current conditions for a given bed material.

Sand can be transported by

- wind-, wave-, tide- and density-driven currents (current-related transport),
- the oscillatory wave motion itself (wave-related transport), as caused by the deformation of short waves under the influence of changing water depth (wave asymmetry)
- or by a combination of currents and short waves.

Roughly, the waves act as sediment stirring agents; the mean current then transports the sediments.

2.2 Sediment transport process

In general the sediment transport process can be divided into three stages:

- The stirring up of bottom material bringing it into suspension, or the loosening of this material on the bottom,
- The horizontal displacement of these particles by the water,
- The re-sedimentation of these particles once again.

Each stage depends on the water motion and the sediment characteristics. The water motion is basically different for currents only, for waves only, or for both waves and currents together, like in coastal zones. Waves mainly loosen material on the bottom and stir it up, while currents mainly transport material to another place. Of course these stages are not exclusive: waves may cause wave-driven sediment transport and currents certainly also pick up and stir up material from the bottom.

Very important factors in sediment transport are the sediment characteristics. The main characteristics are the diameter $D$, that for the Dutch coast sand is in a range from 125 to 600 μm, and the mass density $\rho_s$. These parameters should be taken into
account in all transport formulas because it is obvious that they are significant, for example when considering the transport of a fine sand beach or on a gravel beach.

2.3 Definitions

Total sediment load consists of:

- **Bed load transport**, which is defined as the part of the total load that is in more or less continuous contact with the bed during the transport. It primarily includes grains that roll, slide or jump along the bed, thus the bed load must be determined almost exclusively by the effective bed shear acting directly on the sand surface.
- **Suspended load transport**, which is the part of the total load that is moving without continuous contact with the bed as a result of the agitation of fluid turbulence.

The definition of bed load transport is not universally agreed upon. Some regard bed load transport as occurring in the region where the concentrations are so high that grain-grain interactions are important, and grains are not supported purely by fluid forces.

The suspended load transport can be determined by depth-integration of the product of sand concentration and fluid velocity from the top of the bed layer to water surface.

In there, the net (averaged over the wave period) total sediment transport is defined as the sum of net bed load \( (q_b) \) and net suspended load \( (q_s) \) transport rates: \( q_{tot} = q_b + q_s \).

For practical reasons the suspended transport can be subdivided into current-related and wave-related transport components. Thus, the suspended sand transport is represented as the sum of the current-related \( (q_{sc}) \) and the wave-related \( (q_{sw}) \) transport components, as follows:

\[
q_s = q_{sc} + q_{sw} = \int wc \, dz + \int <(V - v)(C - c)> \, dz
\]  

(2.1)

where: \( q_{sc} \) = time-averaged current-related suspended sediment transport rate and \( q_{sw} \) = time-averaged wave-related suspended sediment transport rate (oscillating component), \( v \) = time-averaged velocity, \( V \) = instantaneous velocity, \( C \) = instantaneous concentration and \( c \) = time-averaged concentration. \( <...> \) represents averaging over time and \( \int ... \) represents the integral from the top of the bed-load layer to the water surface.

The current-related suspended transport \( (q_{sc}) \) is defined as the advective transport of sediment particles by the time-averaged (mean) current velocities (longshore currents, rip currents, and undertow currents). This therefore represents the transport of sediments carried by the steady flow.

The wave-related suspended sediment transport \( (q_{sw}) \) is defined as the transport of sediment particles by the high frequency and low-frequency oscillating fluid components (cross-shore component of orbital motion).

2.4 Basic equation of cross-shore transport

The local, time-averaged sediment transport rate can be described using the basic equation of sediment transport, denoted by:
\[ S = \frac{1}{T'} \int_{0}^{h+\eta} \int_{0}^{t} C(z,t) \cdot u(z,t) \, dt \, dz \]  
(2.2)

Where:

- \( S \) = sediment transport rate
- \( T' \) = time over which average is defined
- \( h+\eta \) = surface level elevation, relative to the bed level
- \( C \) = concentration
- \( u \) = flow velocity
- \( z \) = vertical coordinate

With this equation it is possible to calculate either the cross-shore either the longshore sediment transport, depending on the variables used. If the \( u \) is the cross-shore component of the flow velocity, then this formula will lead to the value of the cross-shore transport rate, averaged over a wave period.

### 2.5 Zones of transport

In friction-dominated deeper water *outside the breaker* (surf) zone the transport process is generally concentrated in a layer close to the seabed because the energy dissipation and the turbulence are mainly confined to the near-bed boundary layer. Especially the transport takes place as bed-load transport in close interaction with small bed forms (ripples) and larger bed structures (dunes, bars). Bed load transport dominates in areas where the mean currents are relatively weak in comparison to the wave motion.

Suspension of sediments can be caused by ripple-related vortices. Suspended load transport will become increasingly important with increasing strength of the tide- and wind-driven mean current due to the turbulence-related mixing capacity of the mean flow. By this mechanism the sediments are mixed up from the bed-load layer to the upper layers of the flow.

In the *surf zone* of sandy beaches the transport is generally dominated by the waves through wave breaking and wave-induced currents in the longshore and cross-shore directions. The breaking process together with the near-bed wave-induced oscillatory water motion can bring relatively large quantities of sand into suspension (stirring) which can be transported as suspended load by net (wave-cycle averaged) currents such as tide-, wind-, and density-driven currents. In the cross-shore direction the generation of a net near-bed return current (undertow) balancing the onshore mass flux between the wave crest and trough, may lead to a net offshore drift of sediment. Undertow is one of the most important mechanism under active surf conditions and is actually the transport of sediment caused by the time-mean seaward directed flow near the bottom. The latter is induced by the breaking of waves that compensates onshore mass flux near the surface due to the propagation of breaking bores. The estimation of undertow is important in modelling offshore transport. The undertow velocity near the bottom is the most important in sand transport modelling, since sand concentration becomes large near the bottom.

Field experience over a long period of time in the coastal zone has led to the notion that storm waves cause sediment to move offshore while fair-weather waves and swell return the sediment shoreward. During conditions with low non-breaking waves, onshore-directed transport processes related to wave-asymmetry and wave-induced streaming are dominant, usually resulting in accretion processes in the beach zone. During high-energy conditions with breaking waves (storm cycles), the beach...
and dune zone of the coast are attacked severely by the incoming waves, usually resulting in erosion processes.

2.6 Coastal erosion

A great problem related to sediment transport is the erosion that occurs during extreme events like storms. There are two distinct types of erosion processes that affect a sandy coast, namely erosion due to mean hydraulic conditions and erosion due to extreme hydraulic conditions. Both result in erosion although the related effects occur on different time scales. Generally, mean hydraulic conditions have an impact on large time scales (seasons, years) whereas the influences of extreme events are primarily restricted to small time scales, say hours to days.

During long term developments the factor that affects in the most pronounced way the coastal evolution is the longshore transport, which may be due to various origins as the presence of longshore gradient in wave attack. Due to an increase in longshore transport along a coastal stretch (say in between two cross-shore rays) the total amount of sand between both boundaries will reduce in time. Consequently, the cross-shore profile will shift in landward direction, resulting in erosion. Short term developments takes place occasionally during less frequent extreme events, such as storm surges and hurricanes. These events affect mostly the cross-shore profile evolution, and the changes are usually restricted to an area that is smaller compared to the one affected by the longshore transport.

In most cases this erosion is only temporarily since the mean hydraulic conditions will tend to reconstruct a pre-storm profile, which principally is of a more or less equilibrium slope. These short-terms developments can be considered to be only a momentary disturbance of the long-term development of the coastal profile.

So, even on a stable coast without any long-term erosion the dynamical adjustment of the cross-shore profile due to, especially very infrequent, short-term developments may lead to safety problems or may even result in a break through of the dike and successive inundation of the area behind it.

2.7 Cross-Shore transport in detail

It is now necessary to focus on cross-shore transport, in order to understand better its mechanism and its influence on the coast and bottom profile.

In the following section will be discussed the sediment transport processes in the cross-shore direction and the relevance of the undertow will be pointed out.

Then, in section 2.7.2 will be briefly discussed the forces that act in the nearshore region, as the latter is the most interesting part in a study on the cross-shore profile development during a storm.

2.7.1 Cross-Shore Sediment Transport Processes

Sediment transport at a point in the nearshore zone has a longshore and a cross-shore component (see Figure 2.1) and these two are treated separately hereafter. A focus on cross-shore sediment transport is relatively recent, just think that the first version of the Unibest model was created at the beginning of the 80's. There are still uncertainties in prediction capability and in some cases the limitation on prediction accuracy of cross-shore transport may be due to a lack of good wave data as to a difficulty in understanding in a deep way transport processes.
Cross-shore sediment transport includes both offshore transport, such as occurs during storms, and onshore transport, which dominates during mild wave activity. Transport in these two directions appears to occur in significantly distinct modes and with markedly different time scales. Offshore transport is the one which prediction is easier, as the volumes of sediment that move are considerably greater than in the case of mild wave activity. Therefore the error in the calculation is supposed to be much smaller.

There is considerably greater engineering relevance and interest in offshore transport than in onshore transport, due to the potential for damage to structures and loss of land. This is exactly the case of Petten sea dike.

The cross-shore profile, as it exists in nature, is shaped by the waves. The sediment transport induced by waves modifies the morphology of the sea bottom, which in turn affects the pattern of wave propagation. As a result of this, there will be a change in sediment transport.

The cycle described above in the end results in a profile that agrees with the wave propagation in the sense that the gradient in cross-shore transport becomes negligible. This means that there is a general tendency of bottom profiles to fit the wave conditions, in order to reach a profile that can be said to be in equilibrium. Storm conditions yield another equilibrium profile than mild wave activity. As extreme and moderate conditions follow each other in time, the actual profile will always be on its way to equilibrium. For this reason, we speak of a dynamic equilibrium.

If we compare the equilibrium profile shaped by storm conditions with the one shaped by moderate conditions, it appears that during a storm a greater volume of sediment is moved offshore.

In the case that the moderate conditions are not capable of returning this sediment before the next extreme event occurs, the long-term effect of alternating storms and moderate conditions will be erosion.

In the present study we will not look at this long-term issue, but we will focus on what a storm does to the profile as it is shaped by the moderate conditions preceding it.

Storms mostly occur in the winter season, while mild wave activity is the predominant characteristic of the summer season. Then, we will have two different equilibrium profiles two basic types of shapes, which are modelled by different wave conditions.
In winter the mean wave conditions are more intense than in summer, material is
moved offshore, the beach slope become gentler and as a result one or more offshore
bars may form. The offshore bars protect the beach from heavy wave attack because
the waves break further offshore. The offshore bars also prevent severe sediment
losses offshore and they provide a mechanism for the temporary storage of eroded
sediments.
In summer the mean wave height decreases, the mean wave period usually increases
and beach recovery begins. Sand is transported onshore and a new berm is created by
the run-up. Because of this onshore sediment transport, the bottom slope in the surf
zone becomes steeper.
It is necessary to keep in mind this difference because we have bottom data either for
winter days and summer days.

2.7.1.1 Mean return flow (undertow)
Observation of sediment transport due to random wave on a beach indicates that one
of the most important mechanisms under active surf conditions may be the transport
of sediment by the time-mean, seaward directed flow near the bottom induced by the
breaking of waves.
It has been hypothesised by Dyhr-Nielsen and Sørensen (1970) that the seaward
directed return flow or undertow in the surf zone (which compensates for the
shoreward directed mass flux above water trough level) is driven by the imbalance
between the vertically non-uniform wave momentum flux and the vertically uniform
pressure gradient.
In order to derive the wave-mean cross-shore current, the water column is divided
into three layers, namely a surface layer above the wave trough level, a middle layer
and a bottom layer. Stive and Wind (1986) propose to consider only the are below
wave trough level and to take account of the surface layer effects via an effective
shear stress at the trough level, compensating for the momentum decay above it, and
via the condition that the net undertow must compensate for the mass flux in the
surface layer. This means that the surface layer model is reduced to the formulation
of the effective shear stress and the mass flux. For breaking waves, Stive and Wind
(1986) show that assuming a zero bottom shear stress leads to acceptable predictions
of the current outside the wave boundary layer. This means that for this part of the
current the bottom layer as such can be left out of consideration. So the problem has
been reduced to solving the velocity in the middle layer from the horizontal
momentum balance, both for the case of breaking and of non-breaking waves. In both
cases the prescribed shear stress at the wave trough level provides an upper boundary
condition, whereas the lower boundary condition follows from the zero shear stress
approximation (breaking waves) or from matching with bottom layer solution (non-
breaking waves). The integral condition of continuity can be used to determine the
remaining unknown constant (the bottom shear stress for non-breaking waves, the
mean return current velocity for breaking waves).

2.7.1.2 Cross-shore sediment transport outside the surf zone
When a satisfactory hydrodynamic description has been obtained, the sediment
transport is calculated as the sum of bed load and suspended load transport.
If the near-bed wave-orbital velocity (and the bed shear stress) is larger in the onshore
motion than in the offshore motion, a net sediment transport in the onshore direction
is induced because the bed load transport is a function of the near bottom velocity.
As will be described in the following section, the cross-shore sediment transport is an
order of magnitude larger in the surf zone than outside. Still the cross-shore transport
under non-breaking waves is important for the development of the coastal profile, because at a given point the waves will be non-breaking during calm conditions for much longer time than the relative short periods of storms-induced wave-breaking.

2.7.1.3 Cross-shore sediment transport in the surf zone

The conditions in the surf zone are characterised by the strong energy dissipation and production of turbulence caused by the wave breaking. All the mechanisms that give contributions to the cross-shore sediment transport outside the surf zone are also relevant in the surf zone, but their significance is much smaller because the energy dissipation in the wave boundary layer is small compared to the energy loss due to wave breaking.

The distribution of the shear stresses together with the continuity equation gives the mean velocity profile with the strong offshore-directed undertow near the bed and an onshore mean flow near the surface. The suspended sediment concentration profiles are influenced by the high turbulence level due to wave breaking. The concentration distribution is much more even over the vertical than outside the surf zone. Nevertheless, the concentrations near the bed are the largest, and the resulting sediment transport goes offshore with the undertow.
3 Data

3.1 Required data to run Unibest-TC

In the present study we will use the model Unibest-TC (developed by Delft hydraulics) to simulate cross-shore profile development due to wave-induced sediment transport. For user-specified information on

- Wave conditions
- Water levels
- Currents
- initial profile
- Sediment characteristics

the model computes the profile evolution as a function of time. Hereafter, this model input is looked at a bit closer.

Waves

For the computation of the cross-shore transport and the possibly resulting bottom evolution, wave data during a storm for the area of interest are indispensable and these are $H_{ms}$ and $T_p$. Furthermore, they have to cover the period for which we want to run the simulation.

Water levels

To calculate the wave transformation and to determine the zone of sediment movement, information on water level is required. This information should relate to the water level variations due to astronomical tide and due to wind and barometric pressure.

Currents

The sediment transport in the area of Petten sea defence is often strongly influenced by currents, so information on currents is essential. Data on current velocities can be obtained by direct measurements or by use of numerical flow models.

Nearshore topography

A bathometric survey of the bottom profile as it is shaped by moderate conditions usually provides the main information regarding the morphology of the zone near the dike.

Sediment

Information on sediments relates to the percentages of sand and other material, e.g. silt, shells, organic matter and the grain size distribution of the sand fraction.

3.2 Available field data

At the Petten sea dike measurements are carried out by the survey department of Rijkswaterstaat Noord-Holland (Ministry of Transport, Public Works and Water Management). These measurements are used to gain insight in coastal processes and
dike design within various projects. For this purpose, a large number of instruments is placed on a line, perpendicular to the dike. Every year the configuration of instruments is evaluated and if necessary adjusted (location and type of instrument etc.).

The data available for my work are time-series of hydrodynamic and atmospheric conditions gathered with the instruments that will be described in Section 3.3. These data are of particular importance because they are field data gathered during storm conditions. In fact, usually storms have been studied just with laboratory data or wave hindcasting, which is the prediction of wave characteristics using meteorological information.

In some cases, these time-series need processing to obtain proper input for Unibest-TC: this will be explained in sections 3.4 and 3.4.

The measurements start in 1994 and finish in season 1999-2000. My study will be focused on the years 1999 and 2000, for which I have the corresponding measured bottom profiles.

### 3.3 Instrumentation

The different measure instruments used at Petten to gather data on storms are placed on a line, as can be seen from Fig. 3.1. At the left it can be seen the dike, while if we move on the right direction we move in a seaward direction. The measure instruments have a name that is composed of two letters and a number. The first two letters are MP, are equal for every instrument and they mean "meet punt" or measure point. Then, there is a number that indicates the order in which the instruments were placed on the line.

From the figure we can also see different shapes of the instruments: for example the circles are the buoys, the yellow lines the step gauges, the rectangle placed on the top of the dike is a videocamera and the blue point are pressure sensors.

Hereafter there is an explanation of all the instruments.

![Instrument configuration: 1999-2000](image)

*Fig. 3.1: Petten ray with the instrument configuration in the season 1999-2000. MPi are the names of the different instruments.*

**Buoys:**
- A waverider is a floating buoy that records the vertical acceleration of the water surface. From the records a reliable estimation of the vertical elevation of the water surface can be made for frequencies in the range of wind waves.
• A directional buoy is a development of the waverider. It measures the tilting of the water surface. The records of this buoy yield an estimate of the vertical water surface elevation and the direction of wave propagation.

![Diagram of a buoy](image)

**Figure 3.1:** Waverider buoy for measurements of (non-directional) wave spectra. The buoy has a vertically oriented accelerometer inside and usually transmits data to shore by radio. The buoy is softly moored to the bottom to keep the position.

- Gauges:
  - A step resistance gauge is basically a vertical staff with electrodes spaced at discrete intervals piercing the water surface. Depending on the water surface elevation the electrodes make contact. The records of this gauge yield a stepwise estimate of the vertical water surface elevation.
  - A run-up gauge is a gauge that measures the wave run-up over a structure.

A *wind vane* is a tool for measuring wind direction. It spins and points in the direction from which the wind is coming.

An *anemometer* is a precision instrument that gives accurate measurements of wind run or mean wind speed.

A *pressure sensor* is a pressure transducer-sensing device for water level measurement. A relative transducer is vented to the atmosphere and pressure readings are made relative to atmospheric pressure. An absolute transducer measures the pressure at its location. The readings are then corrected for barometric pressure taken at the surface.

A *radar-levelmeter* is a level indicator that uses the technology of radar for level measurements. It is used for applications that are difficult or impossible to measure using other technologies. Radar gauges are less affected by such factors as temperature, pressure, and wind compared to non-contact ultrasonic type level gauges and thus provide very stable measurements in particular environments.

A *flow velocity meter* is an instrument that measures flow velocity. It is made with wands and a propeller rotor that turns because of the flow. The basic principle of the sensor is simple, there are multiple bundles of fiber-optics, assembled into a propeller-driven rotor, that gate a beam of infrared light from a photo diode to a photosensitive transistor. The rate of rotation of the propeller rotor is directly proportional to water speed, therefore, pulses produced by the photo transistor over a given time are also directly proportional to water velocity.
### Table 3.1: Overview of all the instruments

<table>
<thead>
<tr>
<th>Location (Code)</th>
<th>Measuring period</th>
<th>Sensor</th>
<th>Distance to dike crest [m]</th>
<th>Local Depth to datum (NAP) [m]</th>
<th>Elevation sensor to datum (NAP)</th>
<th>Variables measured</th>
<th>Sampling frequency [Hz]</th>
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<tbody>
<tr>
<td>M9’1</td>
<td>1994-2000</td>
<td>Directional Waverider</td>
<td>7972</td>
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<td>surface elevation wave direction</td>
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<td>Step gauge Water Level Meter</td>
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<td>M9’3.2</td>
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<td>Waverider</td>
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<td>Directional Waverider</td>
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<td>4</td>
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<td>9.39</td>
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<td>12.80</td>
<td>16.60</td>
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<td></td>
</tr>
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<td>Barometer</td>
<td>-35</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 3.4 Spectral method

What we have from the field data is a time-series of surface level elevation. What we need as input in Unibest-TC is wave information in the form of general, representative characteristics, in particular $H_{rms}$, which is the root mean square wave height and $T_{p}$, which is the peak period. The $H_{rms}$ is defined as:

$$H_{rms} = \sqrt{\frac{1}{N} \sum_{i=1}^{N} H_{i}^{2}} \tag{3.4.1}$$

where $N$ is the number of wave heights $H_{i}$ taken into account and $H_{i}$ are defined as the difference between a consecutive positive and negative water level elevation measured in a given sample period.
The peak period $T_p$ is the wave period that corresponds to the maximum wave energy density.

### 3.5 Data transformation

A random wave field as encountered in nature is a composition of mutually interacting waves with varying periods and heights. Notwithstanding this diversity of properties, natural wave fields exhibit a structure that can be described with a limited number of parameters. For wind waves, for instance, it has been found that wave lengths are approximately Rayleigh distributed. The mathematical formulation of this type of distribution contains only one parameter, being the significant wave height.

Another characterisation of the wave field is the wave energy density spectrum. This is the distribution of wave energy over the frequencies that occur in the wave field (theory on this spectrum can be found in many text-books on surface waves).

The parameters $H_{rms}$ and $T_p$ can be obtained directly from the wave spectrum. The peak period is the reciprocal of the frequency at which the energy density is at its maximum and $H_{rms}$ is related to the total wave energy, which is reflected by the area of the spectrum (energy density integrated over all frequencies).

In the present study, we use the fast Fourier-transformation of the surface elevation in one point as a function of time $\eta(t)$ to find a spectral variance density function $E(f)$ in which $f$ is the frequency in cycles per unit time. $E(f)$ is defined such that its integral, over all positive values of $f$, equals the variance of $\eta(t)$, where the variance of the surface elevation $\sigma_\eta^2(t)$. Because the variance is proportional to the average energy, the spectral variance density function is often called the wave energy spectrum. This energy spectrum therefore indicates how the total energy of the wave field is distributed over the various frequencies.

Most of the characteristics of the wave field can be expressed in terms of moments of $E(f)$ denoted by $m_n$. As we can see $m_0$ is the area beneath the energy spectrum curve, which was equal to $\sigma_\eta^2(t)$:

$$m_0 = \int_0^\infty E(f) \, df = \sigma_\eta^2(t) \quad (3.4.3)$$

We can express $H_{rms}$, which is the root mean square wave height, in terms of $m_0$:

$$H_{rms} = 2\sqrt{2m_0} \quad (3.4.4)$$

As input in Unibest model, we need the $H_{rms}$ and the peak period $T_p$, which is equal to $1/f_m$, the period at which the spectrum has its maximum energy. For this reason, we need to create a Matlab program to transform surface elevation as a function of time to find the $H_{rms}$ and $T_p$.

The first thing to do is to divide our data records in smaller files of 1024 data (a multiple of 2, precisely $2^{10}$) in such a way to obtain a length of time of 800 seconds for the directional wavetherder buoys, which have a sampling frequency of 1.28 Hz. This interval of time is supposed to be meaningful in order to find the correct value of $H_{rms}$.

I will not use data records around the peak of the storm: I will just use casual records assuming that they are significant and , for this reason, that they can represent well the entire storm event.

First of all, it is necessary to de-trend our series of data, in such a way to eliminate the tide from our records. To do this we utilise a Matlab function to find the coefficients of a polynomial $p(x)$ of degree $n$ that fits the data, in a least squares sense. Then we subtract this trend to our surface elevation record. After, we apply the Matlab function that returns the discrete Fourier transform (DFT) of vector of data, computed with a fast Fourier transform (FFT) algorithm. To find the energy spectrum we then use a function that permits us to find the probability density spectrum called $pds$. Integrating
this spectrum over all the frequencies is possible to obtain the order-zero momentum, $m_0$. $H_{rmw}$ is obtained utilising Eq. (3.4.4) and $T_p$ is obtained dividing one by the maximum frequency of the spectrum. Plotting the graph of the spectrum, we realise that the data are very scattered (See Figure 3.4).

![Figure 3.4: Wave spectrum](image)

For this reason is better to find the spectrum for at least 10 different data records of 1024 data. In this way, we obtain 10 different energy spectrum. Averaging them it will be possible to obtain a less scattered function (see Figure 3.5), increasing the precision of $H_{rmw}$ and $T_p$.

![Figure 3.5: Wave spectrum](image)
3.6 Tide

Even if the tide has been calculated as a trend in the surface elevation series, the study has been simplified not taking it into account. Indeed, Unibest model requires tide data for a complete and more accurate study of the subject. Not taking the tide into account will lead to a less precise estimate of the real sediment transport and erosion. This is due to the fact that, in general, a grain of sand inside the surf zone is submitted to both forces of waves and tide. The first acts in a direction almost perpendicular to the shoreline, while the second, on the contrary, acts as a sinusoidal force in a longshore direction (see Figure 3.5).

![Diagram of forces acting on a grain of sand](image)

**Fig. 3.5: Forces acting on a grain of sand**

For this reason, adding the two forces a resultant force is obtained, which forms with the direction perpendicular to the coast a certain angle, depending on the intensity of the two forces that we are adding. This leads to a different phenomenon than the one we are studying, not in the cross-shore direction but in an oblique direction, resulting probably in a more marked sediment transport.
4 Overview of UNIBEST model formulations*

4.1 Introduction

Unibest-TC (TC: Time-dependent Cross-shore) is a module of the program package Unibest, which stands for UNIform BEach Sediment Transport. All modules of this package consider sediment transport along a sandy coast which locally may be considered uniform in alongshore direction. It is designed to compute cross-shore sediment transports and the resulting profile changes along any coastal profile of arbitrary shape under the combined action of waves, longshore tidal currents and wind. Unibest-TC takes the principal cross-shore processes such as wave asymmetry, undertow, gravity and mass-flux below wave troughs into account and can be applied on several coastal problems.

4.2 Coordinate System

The $x$-axis is perpendicular to the shoreline, positive landwards (see Figure 4.1). The $y$-axis is rotated 90 degrees counter-clockwise, relative to the $x$-axis. The $z$-axis is perpendicular to the $x$- and $y$-axis, positive in the upward direction. Wave angles are defined between the $x$-axis and the direction of wave propagation, positive angles counting counter-clockwise.

4.3 Overview of sub-models

Unibest-TC consists of five sub-models:
- Wave propagation model
- Mean current profile model (undertow)
- Wave orbital velocity model
- Bed load and suspended load transport model
- Bed level change model

The wave propagation model computes the wave energy decay along a cross-shore ray including the effects of shoaling, refraction and energy dissipation. The mean current profile model as well as the wave orbital velocity model are local models. The first computes the vertical distribution of the wave-averaged mean current in both longshore and cross-shore direction accounting for wind shear stress, wave breaking, bottom dissipation in the wave boundary layer and the slope of the free surface. The wave orbital velocity model calculates time series of the near-bed wave orbital velocity. These time series contain contributions due to wave asymmetry, wave group related amplitude modulation and bound long waves and are therefore representative for irregular wave groups.

In the sediment transport model one can distinguish between the bed load and suspended load module respectively. It is assumed that the suspended load transport is dominated by the mean current; the suspended sediment flux is computed as the product of the wave-averaged current and concentration profiles, which are obtained from the mean current profile and a time averaged advection-diffusion equation respectively. The bed-load transport is computed as a function of the instantaneous bed shear stress. The near-bed velocity signals, determining the instantaneous shear stresses, are composed of the generated time-series for the near-bed wave orbital velocity plus the time averaged current velocity near the bed.

After the computation of the transport rates along the profile, the bed level changes are computed from the depth-integrated mass balance:

\[
\frac{\partial z}{\partial t} + \frac{\partial \rho_{tot, niv}}{\partial x} = 0
\]
4.4 Wave model

The wave propagation model consists of three first-order differential equations, viz. the time-averaged wave energy balance (Battjes and Janssen, 1978), the balance equation for the energy contained in surface rollers in breaking waves (Nairn et al., 1990) and the horizontal momentum balance from which the mean water level set-up is computed. The refraction of the waves is computed using Snell's law. The three-coupled equations are solved by numerical integration over the cross-shore profile. These equations generate the input required by the local models for the vertical velocity profile, the concentration vertical and the bed-load transport.

The energy balance equation for organised wave energy $E$ reads:

$$\frac{\partial}{\partial x} (EC_x \cos \theta) = -D_w - D_f \tag{4.4.1}$$

where $C_x$ is the wave group velocity, $\theta$ the angle of incidence of the wave field, $D_w$ the dissipation of wave energy due to breaking and $D_f$ the dissipation due to bottom friction. The organised wave energy $E$ is defined according to linear wave theory:

$$E = \frac{1}{8} \rho g H_{rms}^2 \tag{4.4.2}$$

where $\rho$ is the density of water, $g$ the gravitational acceleration and $H_{rms}$ the root mean square wave height:
\[ H_{\text{rms}} = \sqrt{\frac{\sum_{j=1}^{N} H_j^2}{N}} \]  \hfill (4.4.3)

Battjes and Janssen use as a closure for this an expression for the dissipation of organised wave energy based on a bore model:

\[ D_u = \frac{1}{4} \rho g \alpha f_p H_{\text{rms}}^2 Q_b \]  \hfill (4.4.4)

where \( f_p = 1/T_p \) is the peak frequency, \( Q_b \) the fraction of breaking waves and \( \alpha \) dissipation coefficient, which equals 1 in case of a fully developed bore. The model applies a so-called clipped Rayleigh through the surf zone, assuming that the waves smaller than \( H_{\text{rms}} \) are not breaking and Rayleigh distributed, and that all waves larger than \( H_{\text{rms}} \) are breaking. This maximum wave height \( H_{\text{max}} \) is defined as a function of the local water depth, according to:

\[ H_{\text{max}} = 0.88 \frac{\tan h \left( \frac{\gamma k h}{0.88} \right)}{k} \]  \hfill (4.4.5)

where \( k \) the local wave number, \( h \) the local waterdepth and \( \gamma \) a dissipation coefficient.

Setting \( \alpha \) equal to 1, the best fit values for \( \gamma \) turned out to vary systematically with the offshore wave steepness \( S_0 = H_{\text{rms}}/L_0 \), according to:

\[ \gamma = 0.5 + 0.4 \tanh(33.3) \]  \hfill (4.4.6)

Equation (4.4.6) expresses the default relationship for \( \gamma \) as applied by UNIBEST-TC. The fraction of breaking waves \( Q_b \) reflects the percentage of waves larger than \( H_{\text{rms}} \) and is computed iteratively from:

\[ \frac{1-Q_b}{\ln Q_b} = \left( \frac{H_{\text{rms}}}{H_{\text{max}}} \right)^2 \quad \text{for} \quad \frac{H_{\text{rms}}}{H_{\text{max}}} < 1 \]  \hfill (4.4.7)

\[ Q_b = \left( \frac{H_{\text{rms}}}{H_{\text{max}}} \right)^2 \quad \text{for} \quad \frac{H_{\text{rms}}}{H_{\text{max}}} \geq 1 \]

Though "impossible" from a theoretical point of view, it may occur the value of \( H_{\text{rms}} \) exceeds the value of \( H_{\text{max}} \) in very shallow water (frequently in combination with plunging breakers). In those situations the second formulation of Eq.(4.4.7) is applied, which in fact implies that the maximum wave height \( H_{\text{max}} \) in Eq. (4.4.4) is replaced by the local wave height \( H_{\text{rms}} \), hence introducing extra wave dissipation in a region where \( H_{\text{rms}} \) actually is too large. Finally, the wave dissipation \( D_f \) due to bottom friction, which is the second sink term in Eq. (4.4.1), is modelled as:

\[ D_f = \frac{f_w \rho}{\sqrt{\pi}} u_{\text{orb}}^3 \]  \hfill (4.4.7)

where \( f_w \) is a user defined friction factor (FWEE) and \( u_{\text{orb}} \) the amplitude of the wave orbital velocity based on linear wave theory and the \( \text{rms} \) wave height.
Investigation of model performance learned that model predictions of wave height decay were reasonably well in correspondence with wave height measurement through the surf zone, however, the location of initial set-up was predicted too far seaward. For that reason the roller model according to Nairn et al. (1990) has been incorporated in UNIBEST-TC. Instead of being dissipated immediately after the breakpoint, organised wave energy is converted into turbulent kinetic energy first (which can be seen from the development of a roller at the face of a breaking wave), before being dissipated ultimately via the production of turbulence. In this way the dissipation process is delayed, hence shifting the region of wave set-up shoreward direction. This roller model provides the second differential equation of the UNIBEST model, the roller balance equation:

$$\frac{\partial}{\partial x} (2E_c \cos \theta) = D_w - \text{Diss}$$  \hspace{1cm} (4.4.8)

In this expression, \(c\) is the wave propagation speed, \(\text{Diss}\) the dissipation of roller energy and \(D_w\) the dissipation of organised wave energy that acts as a source term for the roller balance equation. The factor "2" in Eq. (4.4.8) originates from additional dissipation of roller energy due to a net transfer of water from the wave to the roller (Stive and De Vriend, 1994). The roller energy \(E_r\) represents the amount of kinetic energy in a roller with area \(A\) and length \(L\), and is defined as:

$$E_r = \frac{1}{2} \rho c^2 \frac{A}{L}$$  \hspace{1cm} (4.4.9)

The roller energy balance is closed by modelling the dissipation \(\text{Diss}\) of roller energy as the power per unit length performed by the shear stress between roller and water surface:

$$\text{Diss} = \beta \rho g c \frac{A}{L} = 2 \beta g \frac{E_r}{c}$$  \hspace{1cm} (4.4.10)

where \(\beta\) is the slope of the face of the wave (normally on the range 0.05-0.10) and \(A\) is written in terms of \(E_r\) via Eq. (4.4.9).

In order to improve the prediction of bar morphodynamics, Roelvink et al. (1995) introduced the concept of breaker delay. The dissipation of organized wave energy as computed from Eq. (4.4.3) and (4.4.4) is only based on local water depth, and disregards the fact that waves need a distance in the order of one wave length to actually start or stop breaking. For that reason they suggest to take into account the bottom elevation some distance seaward of the computational point when determining the wave depth \(h_s\) to be applied in Eq. (4.4.4). To that end they define a reference depth \(h_o\), obtained from weighting water depths seaward of the computational point via weighing function \(W(\xi)\):

$$h_o(x) = \frac{\int_x^\infty W(x-x')h(x')dx'}{\int_x^\infty W(x-x')dx'}$$  \hspace{1cm} (4.4.11)

In this expression, \(h\) is the local water depth and \(X\) is the integration distance. The weighting function \(W\) is given by:
\[ W(\xi) = (X - \xi)^P \]  
(4.4.12)

where \( P \) is a user-defined parameter that determines the shape of weighting function. The integration distance \( X \) is taken proportional to the local peak wavelength \( \lambda_{p} \):

\[ X = \lambda \lambda_{p} \]  
(4.4.13)

where \( \lambda \) is a user-defined coefficient of order one. The third differential equation is the cross-shore momentum equation or set-up equation, which reads:

\[ \frac{\partial \bar{\eta}}{\partial x} = -\frac{1}{\rho gh} \frac{\partial S_{xx}}{\partial x} \]  
(4.4.14)

Here, \( \bar{\eta} \) is the mean wave set-up, \( h = \bar{\eta} - z_s \) the local water depth \( h \) and \( S_{xx} \) the cross-shore radiation stress, which is defined as:

\[ S_{xx} = \left[ (n + n \cos^2 \theta - 0.5)E + 2E, \cos^2 \theta \right] \]  
(4.4.15)

where \( n = c_g/c \), the ratio between the group velocity and the wave propagation speed. The wave direction \( \theta \) is defined as the angle between the \( x \)-axis (perpendicular to the shoreline, positive landwards) and the propagation direction, and is found from Snell's law:

\[ \frac{\sin \theta}{\sin \theta_0} = \frac{c}{c_0} \]

where the subscript \( \theta \) refers to values at the seaward boundary of the model. Finally, the propagation speed \( c \) is defined as:

\[ c = \frac{\omega}{k} \]  
(4.4.17)

where \( \omega = 2\pi f_r \) is the angular frequency, and \( k \) is the wave number, which is solved from the dispersion relation:

\[ \omega^2 = gk \tanh(kh) \]  
(4.4.18)

In order to solve the system for the three unknowns \( E, E_r, \) and \( \eta \), boundary conditions for \( E, E_r, \) and \( \eta \) and \( \theta \) and a bottom profile \( z_b(x) \) are needed. The boundary value of \( E \) is computed from Eq. (5.4.2) via a user-defined wave height at the upwave boundary. In addition, \( \theta \) and \( z_b(x) \) should be given at the upwave boundary, while \( \eta \) is set to zero which is reasonable if the upwave boundary is located outside the surf zone. The roller energy \( E_r \) at the seaward boundary is estimate from Eq (4.4.10), assuming that \( Diss \) equals \( D_{ow} \). Coefficient values must be given for \( \alpha \) (default value 1), \( \gamma \) (default value given by Eq. (4.4.5)), \( \beta \) (optimum value between 0.05 and 0.10) and \( \lambda \) (value of order 1).
4.5 Mean current profile

4.5.1 Introduction

In an alongshore uniform situation, the depth-integrated cross-shore radiation stress balances the wave set-up. The longshore radiation stress gradient, on the other hand, is balanced by a time mean bed shear stress associated with a longshore current. Due to the vertical non-uniformity of the driving forces in the nearshore zone, secondary currents are driven. The imbalance between the cross-shore wave radiation stress gradient (with contributions due to wave height decay and change of momentum in surface rollers) and the pressure gradient due to set-up, drives a circulation current with a shoreward mass flux above the through level and a seaward return flow (or undertow) below the through level. Besides, due to wave boundary layer effects in the momentum balance, a near-bottom streaming in the wave-boundary layer occurs, which is often assumed to be shoreward directed.

In order to determine the velocity distribution of the cross-shore circulation current and the longshore current, the horizontal momentum balance needs to be solved. This is done according to Roelvink and Reniers (1994) who use a quasi-3D model in which account is taken of the effects of wind stress, breaking-induced forcing, surface slope and the wave boundary layer. A parabolic distribution of the eddy viscosity is used in which the effects of turbulence from different sources (slope-driven and wind-driven current, wave breaking and increased turbulence in the wave boundary layer) are combined in a consistent manner.

The quasi-3D model is a direct descendant of the model according to De Vriend and Stive (1987) who identify three layers:

- the surface or trough-to-crest layer, which is represented by boundary conditions on the middle layer;
- the middle layer, from the top of the bottom (wave) boundary layer to the mean water level
- The bottom boundary layer

Svendsen (1985) and Stive and De Wind (1986) were the first to propose only to consider the area below wave trough level and account for the trough-to-crest layer, containing the moving water surface, via an effective shear stress at the trough level, compensating for the momentum decay above it, and via the condition that the net mean flow below trough level must compensate for the mass flux in the surface layer. The modelling of the surface layer is thus reduced to the formulation of the effective shear stress and the mass flux. For simplicity, here we use the mean water level as the upper boundary of the middle layer.

In the following sections, the momentum balance and required turbulence closure (eddy viscosity model) are described.

4.5.2 Momentum balance

For simplicity, the vertical coordinate is scaled according to:

$$\sigma = \frac{z}{h}$$  \hspace{1cm} (4.5.1)

such that $\sigma = 0$ at the bottom and 1 at the surface.

Neglecting the advective acceleration terms for the time mean flow, the momentum balance in $i$-direction, $i = x$ or $y$, reads:
\[ \frac{\partial \tau_i}{\partial \sigma} = R_i, \quad \sigma > \delta \]  

(4.5.2)

\[ \frac{\partial \tau_i}{\partial \sigma} = R + \frac{\partial}{\partial \sigma} \left( \rho \tilde{u}_i \tilde{w} \right), \quad \sigma > \delta \]

Where \( R_i \) is the forcing, \( \delta \) the non-dimensional thickness of the wave boundary layer made dimensionless according to Eq (4.5.1), and \( \tilde{u}_i \) and \( \tilde{w} \) are the oscillating velocity components in \( i \)-direction and vertical direction, respectively.

The non-dimensional thickness \( \delta \) of the wave boundary layer is given by:

\[ \delta = 0.09 \alpha \left( \frac{A}{k} \right)^{0.82} \frac{k_s}{h} \]  

(4.5.3)

With:

\[ \delta_{\text{max}} = 0.5 \]

\[ \delta_{\text{min}} = \alpha \frac{e_{\text{p}} \beta}{h} \]

Here \( A \) is the wave orbital excursion parameter near the bed based on the root-mean-square wave height and peak period and \( z_0 \) is given by \( k_r/33 \) with \( k_r \) equal to the user-defined roughness height RKVAL. The factor \( \alpha \) represents the larger wave boundary layer thickness in irregular waves as compared to regular waves. When using \( \alpha = 1 \), Eq. (4.5.2) reduces to the theoretical expression for monochromatic waves. Here \( \alpha \) is set equal to 20, on the basis of a comparison with measurements in irregular waves.

We make the assumption that the forcing \( R_i \) is dominated by a pressure gradient and that the depth-variation of \( R_i \) can be neglected:

\[ R_i = \rho g h \frac{\partial h}{\partial x_i} \]  

(4.5.4)

The time-averaged shear stress \( -\rho \tilde{u}_i \tilde{w} \) in Eq.(4.5.2) results from the fact that in the wave boundary layer the horizontal and vertical velocities are not exactly 90 degrees out of phase. These stresses grow slow zero at the bed to an asymptotic value \( -\rho \tilde{u}_i \tilde{w} \) in which, from geometrical considerations \( \tilde{w}(\delta) = -\Delta \tau / \rho c \). Here \( \Delta \tau \) is the increase of the instantaneous shear stress through the boundary layer. With the dissipation due to the bottom friction given by \( D_f = \Delta \tau u(d) \) and assuming that the stress \( -\rho \tilde{u}_i \tilde{w} \) decreases linearly to zero across the wave boundary layer, the last term in Eq. (4.5.2) is given by:

\[ \frac{\partial}{\partial \sigma} \rho \tilde{u}_i \tilde{w} = -\frac{1}{\delta} \frac{D_f}{\omega} \frac{k_i}{\omega} \]  

(4.5.5)

in which \( k_i \) is the wave number in \( i \)-direction and \( \omega \) is the angular frequency.

The dissipation due to bottom friction is computed as:

\[ D_f = \frac{1}{2\sqrt{\pi}} \rho f_w u_{\text{orb}}^3 \]  

(4.5.6)
where \( u_{arb} \) is the orbital velocity near the bed based on the root-mean-square wave height and the friction factor \( f_w \) is given by the following relationship (Soulsby, 1994):

\[
f_w = 1.39 \left( \frac{A}{z_0} \right)^{-0.52}
\]

(4.5.7)

\( f_{w,\text{max}} = 0.3 \)

With \( A \) and \( z_0 \) as defined above.

Since we assumed that the depth-variation of \( R_i \) can be neglected, integration of Eq (4.5.2) from the surface downwards yields:

\[
\begin{align*}
\tau_i &= \tau_{i,j} - R_i(1-\sigma) & \sigma > \delta \\
\tau_i &= \tau_{i,j} - R_i(1-\sigma) + \frac{D_i k_i \delta - \sigma}{\omega} & \sigma < \delta
\end{align*}
\]

(4.5.8)

Here \( \tau_{i,j} \) is the known surface shear stress applied at mean water level, which accounts for wind stress and the shear stress introduced by the dissipation in the surface rollers.

The shear stress is related to the velocity gradients by:

\[
\tau_i = \frac{\rho v_i}{h} \frac{\partial u}{\partial \sigma}
\]

(4.5.9)

### 4.5.3 Vertical structure of eddy viscosity

For the computation of the mean current profile, we use an eddy viscosity model (zero equation turbulence model). The eddy viscosity is written as the product of a scale factor and a shape function, which are different from the boundary layer and middle layer respectively. The scale factors may vary with time and location along the profile. The shape functions are parabolic, such that the equations can be solved analytically, and are zero at \( \sigma = 0 \).

In the middle layer, the depth-averaged viscosity \( \nu_i \) has been chosen as a scale factor yielding:

\[
\nu_i = \phi_i \nu_i \sigma(\sigma, -\sigma) \quad \sigma > \delta
\]

(4.5.10)

where \( \sigma = z/h \), the relative height above the bed and \( \delta \) is the scaled boundary layer thickness. The parameter \( \sigma_i \), determining the shape of the viscosity distribution, is specified later on. The parameter \( \phi_i \) follows from the condition:

\[
\int_0^1 \phi_i \sigma(\sigma, -\sigma) d\sigma = 1
\]

(4.5.11)

Hence,
\[
\phi_i = \frac{1}{\frac{1}{2}\sigma_i - \frac{1}{3}}
\]

(4.5.12)

In the wave boundary layer, the eddy viscosity is increased relative to (4.5.10) to account for the increased turbulence in the boundary layer. This eddy viscosity increase is assumed to have a parabolic distribution throughout the boundary layer and is zero at \(\sigma = 0\) and \(\sigma = \delta\). This yields for the eddy viscosity distribution in the boundary layer:

\[
v_i = \phi_i \overline{v_i \sigma (\sigma_i - \sigma) + \phi_i \nu_i \sigma (\delta - \sigma)} \quad \sigma < \delta
\]

(4.5.13)

with \(\nu_i\) is the increased turbulence in the wave boundary layer.

Here the parameter \(\phi_i\) is determined by the condition:

\[
\frac{1}{\delta} \int_0^\delta \phi_i \sigma (\delta - \sigma) d\sigma = 1
\]

(4.5.14)

and thus depends on the boundary layer thickness via:

\[
\phi_i = \frac{6}{\delta^2}
\]

(4.5.15)

It is convenient to write Eq. (4.5.13) in a similar form as Eq. (4.5.10):

\[
v_i = (\phi_i \nu)_i \sigma (\sigma_i - \sigma) \quad \sigma < \delta
\]

(4.5.16)

With:

\[
(\phi_i \nu)_i = \phi_i \overline{v_i} + \phi_i \nu_i
\]

(4.5.17)

\[
\sigma_i = \frac{\phi_i \overline{v_i \sigma} + \phi_i \nu_i \delta}{\phi_i \overline{v_i} + \phi_i \nu_i}
\]

(4.5.18)

Note that the resulting eddy viscosity distribution strongly depends on the relative magnitudes of \(\delta\), \(\sigma_i\), \(\nu_i\), and \(\nu_i\).

### 4.5.4 Integration of velocity profile

**Analytical solution**

We can now obtain the velocity profile by integrating Eq. (4.5.9); by integrating once more, we get the depth-mean velocity. A parabolic eddy viscosity has been chosen (see Eqs. (4.5.10) and (4.5.16)), such that both integrations can be carried out analytically and yield relatively simple, logarithmic expressions.

The first integration yields expressions for the current profile in the boundary layer and in the middle layer. In these equations coefficients occur in terms of the depth-independent forcing \(R\), the known surface shear stress \(\tau_s\), and known streaming term \(\langle D\rangle / \omega\).
The second integration results in a direct relationship between the depth-mean velocity \( \bar{u}_i \), the depth-independent forcing \( R_i \), the known surface shear stress \( \tau_{s,i} \), and known streaming term \( (D, k_i)/\omega \):

\[
\bar{u}_i = (H_h + H - G_h - G)R_i + (G_h + G)\tau_{s,i} + \left( \frac{H_h}{\delta} \right) \left( \frac{D, k_i}{\omega} \right)
\] (4.5.19)

When we apply the expression to obtain the cross-shore current profile, the mean current is known via the condition that the net mean flow in the middle and bottom layer must compensate for the mass flux in the surface layer due to propagating the breaking waves. The unknown forcing (pressure gradient) \( R_i \) is solved from the depth-averaged expression (4.5.19). Having determined the forcing, the current profile can be computed directly using equation (A) in the wave boundary layer and equation (B) in the middle layer. The two equations are given below.

\[
u_i = A_i \left( \frac{B_{b,i}}{\sigma_h} \ln \frac{\sigma}{\sigma_0} - \frac{B_{b,i}}{\sigma_b} + C_{b,i} \right) \ln \frac{\sigma_b - \sigma}{\sigma_b - \sigma_0}, \quad \sigma < \delta \] (A)

where \( u_i \) is the velocity in the bottom layer, given that \( u_i = 0 \) at \( \sigma = \sigma_0 \), and the coefficients \( A_i, B_{b,i}, C_{b,i} \) are defined as:

\[
A_h = \frac{h}{\rho \phi \nu_1}; \quad B_{b,i} = \tau_{s,i} - R_i + \frac{D, k_i}{\omega}; \quad C_{b,i} = R_i - \frac{D, k_i}{\omega}
\]

And:

\[
u_i = u_{b,i} + A_i \left( \frac{B_i}{\sigma_s} \ln \frac{\sigma}{\delta} - \frac{B_i}{\sigma_s} + C_i \right) \ln \frac{\sigma_s - \sigma}{\sigma_s - \delta}, \quad \sigma > \delta \] (B)

where \( u_i \) is the velocity in the middle layer, \( u_{b,i} \) the velocity at the top of the bottom layer \( (z = \delta) \) and the coefficients \( A, B, C \) are given by:

\[
A_h = \frac{h}{\rho \phi \nu_v}; \quad B_i = \tau_{s,i} - R_i; \quad C_i = R_i
\]

In longshore direction, the forcing \( R_i \) follows directly from the known alongshore surface slope. With the forcing \( R_i \) and the depth-averaged viscosity known, the depth-mean current and the current profile can be computed directly from Eq (4.5.19) and Eq. (A) and (B).

**Surface layer effective stress**

In order to solve Eq. (4.5.19), the surface shear stress \( \tau_{s,i} \), with \( i = x \) or \( y \), has to be computed. The wave-induced shear stress translates the momentum decay of the surface layer due to wave breaking to the lower layers and provides the boundary condition for the middle layer. To this wave-induced shear stress, a wind stress is added. The shear stress in the direction of wave propagation, introduced by the surface roller, is given by:

\[
\tau_{s,\text{wave}} = \frac{\text{Diss}}{c}
\] (4.5.20)

in which Diss is the dissipation of roller energy (see Eq. 4.4.10).
For the wind stress in the wind direction, the following relation is used:

\[ \tau_{\text{wind}} = 1.9555 \cdot 10^{-5} u^2 + 0.0024u - 0.0054 + \frac{0.007}{u} \]  \hspace{1cm} (4.5.21)

in which \( u \) is the wind velocity.

**Mass flux and cross-shore mean velocity**

The mass flux in the surface layer is assumed to consist of two parts, one due to the progressive character of the waves and the other due to the surface roller in breaking waves:

\[ q_{\text{drift}} = q_{\text{non-breaking}} + q_{\text{roller}} = \frac{E}{c} + \frac{\rho A c}{T} = \frac{E + 2E_r}{c} \]  \hspace{1cm} (4.5.22)

Here the first part of the right hand side is the mass flux for non-breaking waves and follows from Phillips (1977), whereas the second part accounts for the contribution to the mass of the surface roller. The cross-shore depth-mean velocity in the lower layers, necessary for the computation of the cross-shore current profile, must compensate for the mass flux and is therefore given by:

\[ \frac{u_x}{h} = \frac{q_{\text{drift},x}}{h} = \frac{q_{\text{drift}} \cos \theta}{h} \]  \hspace{1cm} (4.5.23)

**Longshore slope**

In order to compute the forcing \( R_y \), the alongshore surface slope must be derived from the user-defined depth-averaged tidal velocity at a certain reference depth. We use the Chézy formulation that is based upon a logarithmic velocity profile along the complete water column:

\[ \nu_{\text{tide}} = C \sqrt{h \frac{\partial h}{\partial y}} \]  \hspace{1cm} (4.5.24)

with \( C = 18 \log \frac{12h}{k} \)

For the roughness height \( k \) in Eq. (5.5.24) the user-defined roughness (RKVAL) is used.

**4.5.5 Specification of eddy viscosity distribution**

In order to fully define the eddy viscosity distribution the following parameters have to be specified:

- the depth-averaged eddy viscosity \( \nu \) for a combination of turbulence generated by slope-driven currents, wave breaking and wind;
- the parameter \( \sigma \) for which the combined eddy viscosity profile is zero;
- the increased turbulence \( \nu_{\text{in}} \) in the wave boundary layer;
- the boundary layer thickness
These aspects will be specified in the following. First, \( \sigma \) and \( \overline{v} \) are presented for the situation of a purely slope-driven current, a purely wind-driven current, and a turbulence generated purely by wave breaking respectively. Consequently, these results are combined to find \( \sigma \) and \( \overline{v} \) for the situation of turbulence generated by the combination of these sources. Finally, the relations for the increased turbulence \( \overline{v_a} \) in the wave boundary layer thickness are given.

**Depth-averaged turbulence viscosity for purely slope-driven current**

For purely slope-driven currents, the distribution of the eddy viscosity is parabolic, and zero at \( \sigma = 0 \) and \( \sigma = 1 \). This leads to \( \sigma_i = 1 \) and with Eq.(4.5.12) to \( \phi_i = 6 \).

Near the bottom, the velocity varies linearly with distance from the bed, with a gradient:

\[
\frac{\partial u}{\partial \sigma} = \frac{u_*}{\kappa e \sigma_0} \tag{4.5.25}
\]

For a slope-driven current with surface slope \( s_n \), we have near the bed:

\[
\frac{\partial u}{\partial \sigma} = A \left( \frac{B_i}{\sigma, \sigma} + \frac{B_i}{\sigma, \sigma} + \frac{C_i}{\sigma, \sigma} \right) = \frac{h}{\rho \phi_i \overline{v}} \left( -\rho g h s_n \right) = \frac{h}{6 \rho v_i e \sigma_0} \tau_s \tag{4.5.26}
\]

So, with \( \tau_s = \rho u_*^2 \):

\[
\frac{u_*}{\kappa e \sigma_0} = \frac{h}{6 \rho v_i e \sigma_0} u_*^2 \tag{4.5.27}
\]

The depth-averaged viscosity is now expressed as:

\[
\overline{v_i} = \frac{1}{6} \kappa h \sqrt{gh} \left| s \right| \tag{4.5.28}
\]

**Depth-averaged turbulence viscosity for wind-driven current**

In the case of purely wind-driven current, it is unlikely that the viscosity goes to zero near the surface; rather, a maximum is expected there. We model this by assuming a half-parabolic distribution, with \( \sigma_i = 2 \) and hence \( \phi_i = 1.5 \).

Near the bottom, the velocity again varies linearly with distance from the bed:

\[
\frac{\partial u}{\partial \sigma} = \frac{u_*}{\kappa e \sigma_0} \tag{4.5.29}
\]

For wind-driven currents, we have near the bed:

\[
\frac{\partial u}{\partial \sigma} = A \left( \frac{B_i}{\sigma, \sigma} + \frac{B_i}{\sigma, \sigma} + \frac{C_i}{\sigma, \sigma} \right) = \frac{h}{\rho \phi_i \overline{v}} \left( \frac{\tau_{s,i}}{\sigma, \sigma} \right) = \frac{h}{\frac{1}{2} \rho v_i} \frac{\tau_{s,i}}{2 e \sigma_0} \tag{4.5.30}
\]

So:
\[
\frac{u_*}{\kappa e \sigma_0} = \frac{h}{3 v_t e \sigma_0} \left( \frac{u_*^2}{\kappa} \right) (4.5.31)
\]

The depth-averaged viscosity is now expressed as:

\[
v_{i,3} = \frac{1}{3} x h \sqrt{\frac{\tau_s}{\rho}} (4.5.32)
\]

**Depth-averaged turbulence viscosity generated by wave breaking**

The depth-averaged turbulence viscosity due to wave breaking is modelled according to Battjes (1975) as:

\[
v_{i,3} = \alpha_v \left( \frac{\text{Diss}}{\rho} \right)^{1/2} L (4.5.33)
\]

where \( \alpha_v \) is a coefficient and \( L \) is a typical length scale. Validation studies (LIP Delta Flume Experiment, LIP Shear Wave Experiment) indicate that choosing \( H_{rms} \) in combination within \( \alpha_v \) in the range of 0.05-0.10 leads to optimum results. The coefficient \( \alpha_v \) has to be specified by the user.

The distribution over the depth of the breaking-induced turbulence is assumed to be similar to that induced by wind-stress (\( \sigma_e=2 \) and hence \( \phi_e=1.5 \))

**Combination of turbulence viscosity from different sources**

In the case that we have turbulence generated by different sources, a zero-equation turbulence model cannot be derived rigorously. We therefore have to choose a reasonable approximation, which meets a number of constraints:

- the combined viscosity profile must reduce to all of the above mentioned limit cases;
- given that the length scales must be similar for the processes mentioned, it seems reasonable to add turbulence energy from different sources. For the viscosity, since \( \nu_i = l \sqrt{k} \) with \( l \) the length scale of the turbulence and \( k \) the turbulent kinetic energy, we must therefore take the root-mean-square \( rms \) of the different viscosity components;
- to ensure that the equations can be solved analytically, the combined viscosity profile must be of the form described by Eq. (4.5.10)

The second and third constraint cannot be met exactly simultaneously. However, a good approximation is taking the \( rms \) of the respective depth-averaged viscosity contributions instead of the \( rms \) of the viscosity distributions:

\[
v_i = \sqrt{v_{i,\text{current}}^2 + v_{i,\text{wind}}^2 + v_{i,\text{breaking}}^2} (4.5.34)
\]

For a half-parabolic eddy viscosity distribution (\( \sigma_e=2 \) and hence \( \phi_e=1.5 \)) as assumed for both a wind-driven current and the turbulence generated by wave breaking, the viscosity at surface level is equal to 1.5 times the depth-averaged viscosity. With the viscosity at the surface in a purely slope driven current equal to zero, we approximate the viscosity at surface level by taking the \( rms \) of the depth-averaged contributions due to wind and wave breaking only:
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\[ v_{l,\text{surface}} = \frac{3}{2} \sqrt[3]{-\frac{2}{1+\frac{v_{l,\text{wind}}}{v_{l,breaking}}} \frac{2}{1+\frac{v_{l,\text{breaking}}}{v_{l,\text{wind}}}}} \] (4.5.35)

Herewith, the parabolic distribution is completely defined and the parameters \( \sigma \) and \( \phi \) can be determined. We can write \( \sigma \) in terms of the combined depth-averaged viscosity and the viscosity at the surface level:

\[ \sigma = \frac{v_l - \frac{1}{2} v_{l,\text{surface}}}{v_l - \frac{1}{2} v_{l,\text{surface}}} \] (4.5.36)

The parameter \( \phi \) is given by Eq. (4.5.12).

**Increased turbulence in wave boundary layer**

We relate the turbulence in the wave boundary layer to the wave orbital motion and the friction factor. The theoretical result for the wave friction factor for smooth laminar flow is:

\[ f_w = \frac{2}{u_{orb}^2 \sqrt{\nu \omega}} \] (4.5.37)

where \( \nu \) is the molecular viscosity.

Analogous to Eq. (4.5.37) we can express the turbulence in the wave boundary layer as:

\[ \frac{c}{u_{orb}} = \frac{c_f^2 u_f^2}{\omega} \] (4.5.38)

\[ c_f = \frac{f_w}{2} \]

with \( f_w \) determined by Eq. (4.5.7), the orbital velocity \( u_{orb} \) based on the root-mean-square wave height and \( \omega \) the angular frequency corresponding to the peak wave period.

**4.6 Near-bed orbital velocity**

The model of the time variation of the near-bed velocity (orbital motion) due to non-linear short waves and long waves related to wave groups is based on the concept described in Roelvink and Stive (1989). In short, this model consists of two parts:

- A contribution due to wave asymmetry which is computed using Rieckner and Fenton's (1981) method for monochromatic waves, where the mean wave energy and peak period are used as input for the case of random waves.
- A contribution due to long bound waves based on Sand (1982) and an empirical relationship for the phase of the bound long wave relative to the short wave envelope.

The bed load transport model of UNIBEST-TC requires a complete representative time-series of the near-bed velocity in order to compute bed load transport rates.
Therefore a time-series is produced, which has the same characteristics of asymmetry, long waves and amplitude modulation as a random wave field. The shortest time-series that can exhibit all of these features has a length of one short wave group, which is \( m \) waves long. Starting point is a time series of the near-bed velocity in case of regular waves (including wave asymmetry), based on the Rienecker and Fenton model:

\[
U_i(t) = \sum_{j=1}^{n} B_j \cos(j \omega t) \tag{4.6.1}
\]

The amplitudes \( B_j \) are determined numerically (Rienecker and Fenton, 1981), such that the difference between the maximum and minimum velocity of the asymmetric waves equals the difference in case of monochromatic waves. Next this time series \( U_i(t) \) is modulated such that amplitude variation on the time scale of a wave group is taken into account. Finally the contribution due to bound long waves is included. This two-step approach will be outlined below.

### Wave group related amplitude modulation

By adding a second velocity time series that is slightly out of phase with the first according to (4.6.1), the amplitude modulation on the time scale of a wavegroup is introduced yielding a time series \( U_2(t) \):

\[
U_2(t) = \sum_{j=1}^{n} \cos(j \omega t) e^j = \sum_{j=1}^{n} \cos(j \omega t) \left[ \frac{\omega^j}{j!} (1 + \cos(\Delta \omega \cdot t)) \right] \tag{4.6.2}
\]

where \( \Delta \omega = \omega / m \), \( m \) being the number of waves in one wavegroup which is set to 7 in UNIBEST-TC.

Finally, the magnitude of \( U_2 \) is corrected to \( U'_2 \) in such a way that the third moment of \( U'_2 \) equals the third moment of \( U_i \):

\[
U'_2(t) = \left( \frac{1}{T \int_0^T U_i^3 dt} \right)^{1/3} U_i(t) \tag{4.6.3}
\]

### Generation of a time series of a bound long wave

The second step implies the modelling of a bound long wave. In case of random wave field the grouping of the short waves will generate bound long waves. The long wave velocity \( U_j \) is computed according to Roelvink and Stive (1989) who assume that the wave-group related features of a random wave field may be represented by a dichromatic wave train with equal amplitudes \( a_m \) and \( a_n \) respectively, and an accompanying bound long wave with amplitude \( \xi_n \).

Values for \( a_m = a_n \) and \( \xi_n \) have to be found. In order to do so, the schematised wave train is required to have the same total surface variance:

\[
m_n = \frac{1}{8} H_{rms}^2 = \gamma_n a_n^2 + \gamma_m a_m^2 + \gamma_n \xi_n^2 \tag{4.6.4}
\]

Furthermore, Sand (1982) estimates the long-wave amplitude resulting from two waves with equal amplitudes and frequencies of \( f_c \) and \( f_c + \Delta f \) respectively, as:
\[ \xi_{nw} = -G_{nw} \frac{a_n a_m}{h} \] (4.6.5)

where \( G_{nw} \) is a transfer function.

Knowing the cross-shore variation of \( H_{rms} \) (as predicted by the wave propagation model), the amplitudes \( a_n \), \( a_m \) and \( \xi_{nw} \) of both the short wave envelope and the bound long wave can be solved from Eq. (5.6.4) and (5.6.5). Using a long wave approximation (shallow water conditions), the velocity time series \( U_3(t) \) due to the long wave component is described by:

\[ U_3(t) = \hat{u}_i \cos(\omega t + \varphi) \] (4.6.6)

where: \[ \hat{u}_i = \xi_{nw} \sqrt{gh} \] (4.6.7)

and: \[ \omega = \frac{\omega}{m} \] (4.6.8)

In Eq. (5.6.6) the angle \( \varphi \) represents the phase shift between the long-wave and the short-wave envelope, which equals \( \pi \) in the case of a complete bound wave situation. In reality however, it appears that the cross-correlation coefficient is only slightly negative as long as we stay offshore from the surf zone, and that it changes into a positive correlation as we enter the surf zone. Roelvink and Stive (1989) conclude from flume experiments that the value of \( \cos \varphi \) correlates well with the ratio of local wave energy and the incident wave energy as expressed by the square ratio of the local wave energy and the incident wave energy as expressed by the squared ratio of the local wave height over the deep water wave height \( (H_{rms}/H_{rms,0})^2 \). In UNIBEST-TC this empirical relationship is included via:

\[ \cos(\varphi) = C_r \left[ 1 - 2 \left( \frac{H_{rms}}{H_{rms,0}} \right)^2 \right] \] (4.6.9)

where \( C_r \) is the correlation coefficient between wave envelope and long-wave surface variation (which equals 0.25, Roelvink and Stive, 1989) and \( H_{rms,0} \) the incoming wave height at the seaward boundary of the model.

Final step is the computation of the time series \( U_3(t) \) of the total orbital velocity, by simply adding the effects due to short-wave envelope and the bound long-wave:

\[ U_3(t) = U_3'(t) + U_3(t) \] (4.6.10)

### 4.7 Bed load transport

#### 4.7.1 General

Separate transport formulations are used for bed load transport, bed load being that part of the load which is more or less continuously contact with the bed, and suspended load transport. In this section, the applied quasi-steady bed load formulation is described. At small shear stresses the bed load transport formulation represents the transport occurring as individual particles moving over a rippled bed,
while at higher shear stresses the formulation represents the sheet flow phenomenon where particles move as bed load in several layers over a plane bed.

4.7.2 Bed-load transport formulations

By correlation of various non-dimensional parameters using a range of datasets of sediment transport in oscillatory flow over horizontal beds, a generalized bed load transport formula has been obtained by Ribberink (1995). This formulation is used in UNIBEST-TC supplemented with corrections to account for slope effects on the transport.

The non-dimensional instantaneous bed-load transport vector \( \Phi_{bd} \), defined as the ratio of bed-load transport rate \( q_b \) and the square root of a parameter representing the specific under-water weight of sand grains, is given by:

\[
\Phi_{bd} (t) = \frac{q_b(t)}{\sqrt{\Delta g d_{50}^3}} = 9.1 \frac{\beta_s}{(1 - p)} \left( \theta'(t) - \theta_{cr}^1 \right)^{1.8} \frac{\theta'(t)}{\theta'(t)}
\]  

(4.7.1)

with:

\( q_b \) = bed-load transport rate in volume per unit time and width including pores

\( d_{50} \) = median grain diameter

\( \Delta \) = relative density \( = (\rho_s - \rho) / \rho \)

\( \rho \) = density of water

\( \rho_s \) = density of sediment \( = 2650 \text{ kg/m}^3 \)

\( p \) = porosity of the sediment \( = 0.4 \)

\( g \) = gravity acceleration

\( \theta' \) = dimensionless effective shear stress

\( \theta_{cr} \) = dimensionless critical shear stress

\( \beta_s \) = slope factor

Note that the computed transport rates include the pores and that for the porosity a constant value of 0.4 is taken.

**Bed shear stress**

The parameter \( \theta'(t) \) is the instantaneous dimensionless effective shear stress due to currents and waves:

\[
\theta' = \frac{\tau_b}{(\rho_s - \rho) g d_{50}}
\]  

(4.7.2)

and represents the sediment forcing as the ratio of the flow-drag force on the grains and the under-water weight of grains. The effective bed-shear stress \( \tau_b \) is that part of the total bed shear stress which is transferred directly to the grains in the bed as skin friction. The form drag induced by bed forms is not effective in relation to bed load transportation.

For the computation of \( \theta'(t) \) a quadratic friction law is applied using intra-wave nearbed velocities of the combined wave-current motion and weighed friction factor \( f_{cw} \).

\[
\theta'(t) = \frac{1}{2} g f_{cw} \frac{u_b(t)}{(\rho_s - \rho) g d_{50}}
\]  

(4.7.3)
in which \( u_h \) is the time-dependent (intra-wave) near bottom horizontal velocity vector of the combined wave current-motion at the top of the wave boundary layer \( z = \delta \). This vector is computed in UNIBEST-TC as the sum of the near-bed oscillating velocity signal and the time-averaged velocity at 1 cm from the bed.

**Slope correction to bed load transport rates**

In the case of a sloping bed not only the effects of slope to the initiation of motion (see below) have to be taken into account, but the transport directly induced by gravity as well when the grains have been set in motion. For that reason, the Bagnold parameter \( \beta_i \) is introduced, which increases the transport rates in case of downslope transport and decreases the case the transport rates in case of upslope transport:

\[
\beta_i = \left( 1 + \frac{dz_h}{ds} \frac{\tan \varphi}{\tan \varphi + \frac{dz_{bh}}{ds}} \right)
\]

with the slope:

\[
\frac{dz_h}{ds} = \frac{u_{bh}}{|u_h|} \frac{dz_{bh}}{dx}
\]

and \( \varphi \) the angle of repose, which may differ from the natural angle of repose. In UNIBEST-TC the angle of repose is function of the cross-shore distance and is specified by the user. It should be remarked that this formulation is only valid for:

\[
\left| \frac{dz_h}{ds} \right| < \tan \varphi
\]

limiting the maximum slope of the bed that can be used in simulations.

Note that in UNIBEST-TC the slope in longshore direction is zero by definition.

**Initiation of motion**

The parameter \( \theta_c \) is the non-dimensional critical shear stress, representing the threshold of motion of sand grains. This threshold parameter is calculated according to the classical Shields curve as modelled by Van Rijn (1993) as a function of the non-dimensional grain size \( D_\ast \). In this way, no iteration is necessary to obtain the critical shear stress, as would be in the case when applying the classical Shields curve. In addition, the threshold parameter is corrected to account for the effect of bed slope \( \alpha \) to the threshold of motion. Therefore:

\[
\theta_c = f(D_\ast, \alpha)
\]

with:

\[
D_\ast = d_{50} \left( \frac{g \Delta}{\nu^2} \right)^{1/3}
\]

\( \nu \) is the cinematic viscosity of water:
\[
\nu = \frac{4 \cdot 10^{-5}}{20 + Te}
\]
(4.7.9)

We follow Van Rijn (1993) who represents the Shields curve as follow:

\[
\theta_{cr} = 0.24 \ D_{cr}^{-1}, \quad 1 < D_c \leq 4
\]
\[
\theta_{cr} = 0.14 \ D_{cr}^{0.64}, \quad 4 < D_c \leq 10
\]
\[
\theta_{cr} = 0.04 \ D_{cr}^{-0.1}, \quad 10 < D_c \leq 20
\]
\[
\theta_{cr} = 0.13 \ D_{cr}^{0.20}, \quad 20 < D_c \leq 150
\]
\[
\theta_{cr} = 0.55 \ D_{cr}, \quad 150 < D_c
\]
(4.7.10)

This representation of the Shields curve is applied in the suspended load and the bed load module to represent the time-averaged and intra-wave threshold parameter respectively. Only in the bed load formulation a slope correction is applied to the critical shear stress.

**Slope correction to the threshold parameter in bed load formulation**

The threshold criterion (Eq. 5.7.10) in the bed load formula is adapted using the Schoklitsch factor to take account of the effect of bed slope on the initiation of motion:

\[
\theta_{cr, \text{slope}} = \frac{\sin (\varphi + \arctan \left( \frac{dz_h}{ds} \right))}{\sin \varphi} \theta_{cr}
\]
(4.7.11)

with \( \theta_{cr} \) according to Eq. (4.7.10) and the bottom slope according to Eq. (4.7.5). This formulation results in an increase of the critical shear stress for upslope movement and a decrease for the critical shear stress for downslope movement.

**Time-averaged cross-and longshore bed load transport rates**

The instantaneous cross-and longshore transport components are obtained from:

\[
q_{hx} = \frac{u_{hx}}{|u_h|} |q_b|
\]
\[
q_{hy} = \frac{u_{hy}}{|u_h|} |q_b|
\]
(4.7.12)

In which \( u_h \) and \( q_b \) are respectively the time-dependent (intra-wave) near-bottom horizontal velocity vector and the bed-load transport vector of the combined wave-current motion.

The net wave-averaged bed-load transport rate is obtained by averaging of the time-dependent transport vector \( q_b(t) = (q_{hx}, q_{hy}) \) over the duration of the imposed near bottom velocity time series.
4.8 Suspended load transport

4.8.1 General

The suspended sediment transport rate ($q_s$) can be computed from the vertical distribution of fluid velocities and sediment concentrations, as follows:

$$ q_s = \int_a^h V C dz $$

(4.8.1)

in which:

$V$ = Local instantaneous fluid velocity at height $z$ above bed (m/s)
$C$ = Local instantaneous sediment concentration at height $z$ above bed (kg/m$^3$)
$h$ = Water depth (to mean surface level), (m)
$\eta$ = Water surface elevation (m)
$a$ = Thickness of bed-load layer (m)

Defining: $V = v + \bar{v}$ and $C = c + \bar{c}$

(4.8.2)

In which:

$v$ = time and space-averaged fluid velocity at height $z$ (m/s)
$c$ = time and space-averaged concentration at height $z$ (kg/m$^3$)
$\bar{v}$ = oscillating fluid component (including turbulent component), (m/s)
$\bar{c}$ = oscillating concentration component (including turbulent component), (kg/m$^3$)

Substituting Eq. (4.8.2) in Eq. (4.8.1) and averaging over time and space yields:

$$ \bar{q}_s = \int_a^h \bar{v} c dz + \int_a^h \bar{v} \bar{c} dz = \bar{q}_{s,c} + \bar{q}_{s,w} $$

(4.8.3)

in which:

$\bar{q}_{s,c} = \int_a^h \bar{v} c dz$ Time-averaged current-related sediment transport rate (kg/sm)

$\bar{q}_{s,w} = \int_a^h \bar{v} \bar{c} dz$ Time-averaged wave-related sediment transport rate (kg/sm)

The current-related suspended transport rate is defined as the transport of sediment particles by the time-averaged (mean) current velocities (longshore currents, rip currents, undertow currents). The current velocities and the sediment concentrations are affected by the wave motion. It is known that the wave motion reduces the current velocities near the bed and strongly increases the near-bed concentrations due to its stirring action. The wave-related suspended sediment transport is defined as the transport of sediment particles by the oscillating fluid components (cross-shore orbital motion).

4.8.2 Suspended load transport formulation
In UNIBEST-TC the wave-related suspended sediment transport is assumed to be small as compared to the current-related sediment transport. The suspended load transport in volume per unit time and width (m³/s) inclusive of pores is therefore computed as:

\[ q_{v,c} = \int_{a}^{h} v_c \, dz / (1 - p) \rho_c \]  \hspace{1cm} (4.8.4)

With \( v \) is the time-mean velocity profile and \( c \) is the time-mean concentration profile. The factor \( p \) is the porosity and is set equal to 0.4. The procedure for computation of the concentration profile is described in the following section.

### 4.9 Time averaged concentration profile

#### 4.9.1 Convection-diffusion equation

\[ w_{s,m} \cdot c + \varphi_d \cdot \varepsilon_{s,cw} \cdot \frac{dc}{dz} = 0 \] \hspace{1cm} (4.9.1)

In which:

- \( w_{s,m} \) = fall velocity of suspended sediment in a fluid-sediment mixture (m/s)
- \( \varepsilon_{s,cw} \) = sediment mixing coefficient for combined current and waves (m²/s)
- \( c \) = time-averaged concentration at height \( z \) above the bed (kg/m³)
- \( \varphi_d \) = damping factor dependent on the concentration

Here, it is assumed that Eq. (4.9.1) is also valid for wave-related mixing. The convection-diffusion equation is solved by numerical integration from a near-bed reference level \( a \) to the water surface. At the reference level a concentration-type boundary condition is used.

#### 4.9.2 Sediment fall velocity and turbulent damping

**Particle fall velocity**

The fall velocity of a sediment particle is computed according to Van Rijn (1993) as:

\[
\begin{align*}
\dot{w}_v &= \frac{\Delta g d^2_{s}}{18 \nu}, & 1 \ \mu m < d_s < 100 \mu m \\
\dot{w}_v &= \frac{10 \nu}{d_s} \left[ \left( 1 + \frac{0.01 \Delta g d^3_s}{\nu^2} \right)^{2} - 1 \right], & 100 \ \mu m < d_s < 1000 \mu m \\
\dot{w}_v &= 1.1 (\Delta g d_s)^{0.5}, & 1000 \mu m < d_s
\end{align*}
\]  \hspace{1cm} (4.9.2)

Here \( d_s \) is the diameter of the suspended sediment, and is a user-defined property. Van Rijn (1987) concluded on the basis of measurements that \( d_s \) should be in the range of 60 to 100% of the diameter of the median bed material size \( d_{50} \). \( \nu \) is the cinematic viscosity.
Fall velocity in a mixture

In high concentration mixtures, the fall velocity of a single particle is reduced due to the presence of other particles. In order to account for this hindered settling effect, the fall velocity in a fluid-sediment mixture is determined as a function of the sediment concentration \( c \) (kg/m\(^3\)) and the particle fall velocity \( w_p \):

\[
w_{s,m} = \left( 1 - \frac{c}{\rho_s} \right)^5 w_p
\]  

(4.9.3)

Damping factor

The damping factor \( \varphi_d \) represents the influence of the sediment particles on the turbulence structure of the fluid. The effect becomes increasingly important for high sediment concentrations, which result in stratification and hence damping of turbulence. The following relation is used (see Van Rijn, 1993):

\[
\varphi_d = 1 + \left( \frac{c}{c_0} \right)^{0.8} - 2 \left( \frac{c}{c_0} \right)^{0.4}
\]  

(4.9.4)

In which \( c_0 \) is the maximum concentration and \( c \) is the actual concentration. The maximum volume concentration is set to 0.65 which amounts to a maximum concentration \( c_0 \) of 1.7\( \times 10^3 \) kg/m\(^3\).

4.9.3 Sediment mixing coefficient

Measurements in wave flumes show the presence of suspended sediment particles from the bed up to the water surface. The largest concentrations are found close to bed where the diffusivity is large, due to ripple-generated eddies. Further away from the bed the sediment concentration decrease rapidly because the eddies dissolve rather rapidly travelling upwards.

Various researchers have tried to model the suspension process by introducing effective wave-related sediment mixing coefficient \( \varepsilon_{s,w,bed} \) (Van Rijn, 1993).

Based on analysis of measured concentration profiles, the following characteristics were observed (Van Rijn, 1993):

- Approximately constant mixing coefficient \( \varepsilon_{s,w,bed} \) in a layer \((z \leq d_v)\) near the bed,
- Approximately constant mixing coefficient \( \varepsilon_{s,w,bed} \) in the upper half \((z \geq 0.5 \ h)\) of the water depth,
- Approximately linear variation of the mixing coefficient for \( d_v < z < 0.5 \ h \).

For the current mixing coefficient, a constant mixing is assumed in the upper half of the water column, which decreases linearly to zero in the lower half of the column. For combined current and wave conditions the sediment-mixing coefficient is modelled as:

\[
\varepsilon_{s,sw} = \sqrt{\left( \varepsilon_{s,w} \right)^2 + \left( \varepsilon_{s,c} \right)^2}
\]  

(4.9.4)

In which \( \varepsilon_{s,w} \) is the wave-related mixing coefficient (m\(^2\)/s) and \( \varepsilon_{s,c} \) current-related mixing coefficient.
Wave related mixing

The mathematical formulation for the wave-related mixing coefficient reads:

\[
\begin{align*}
\varepsilon_{s,w} &= \varepsilon_{s,w,\text{bed}} & z \leq \Delta_s, \\
\varepsilon_{s,w} &= \varepsilon_{s,w,\text{max}} & z \geq 0.5h \\
\varepsilon_{s,w} &= \varepsilon_{s,w,\text{bed}} + \left[\varepsilon_{s,w,\text{max}} - \varepsilon_{s,w,\text{bed}}\right] \frac{z - \delta_s}{0.5h - \delta_s} & \Delta_s < z < 0.5h
\end{align*}
\]  

Equation (5.9.5) is fully defined when the thickness \( \Delta_s \) of the near-bed sediment mixing layer, the mixing coefficient \( \varepsilon_{s,w,\text{bed}} \) in the near-bed layer and the mixing coefficient \( \varepsilon_{s,w,\text{max}} \) in the upper layer are known.

The thickness \( \Delta_s \) of the near-bed sediment mixing layer is modelled according to Kroon and Van Rijn (1993) who has analysed concentration profiles measured in the surf and swash zone near the Dutch coast of Egmond. They found that the mixing coefficient of spilling and plunging breaking waves could be represented by increasing the mixing layer thickness and proposed the following relationship:

\[
\frac{\delta_s}{h} = 0.3 \left( \frac{H_s}{h} \right)^{\frac{1}{2}}
\]  

With:

- \( \delta_{s,\text{minimum}} = 0.05 \text{m} \)
- \( \delta_{s,\text{maximum}} = 0.20 \text{m} \)

Here \( H_s \) is the significant wave height and \( h \) the water depth.

For the mixing coefficient in the near-bed layer the following expression is used:

\[
\varepsilon_{s,w,\text{bed}} = \alpha_b \tilde{U}_s \alpha_s
\]  

In which \( \tilde{U}_s \) is the peak value of the near-bed orbital velocity based on the significant wave height and peak period and the empirical coefficient \( \alpha_b \) equal to 0.004 \( D_s \).

The mixing coefficient in the upper layer reads:

\[
\varepsilon_{s,w,\text{max}} = 0.035 \frac{H_s h}{T_p}
\]  

In which \( T_p \) is the peak period of spectrum. The maximum value for \( \varepsilon_{s,w,\text{max}} \) is set to 0.01 \( \text{m}^2/\text{s} \), while the minimum value for \( \varepsilon_{s,w,\text{max}} \) is the value of \( \varepsilon_{s,w,\text{bed}} \)

Current-related mixing

The current-related mixing coefficient \( \varepsilon_{c,c} \) reads:
\[
\begin{align*}
\varepsilon_{*,c} &= \kappa \beta u_{*,c} z \left(1 - \frac{z}{h}\right) \quad \text{for } z < 0.5h \\
\varepsilon_{*,c} &= 0.25 \kappa \beta u_{*,c} h \quad \text{for } z > 0.5h
\end{align*}
\] (4.9.9)

In which \( \kappa \) is the constant of Von Karman (\( \kappa = 0.4 \)), the coefficient \( \beta \) is the ratio between sediment and fluid mixing coefficient and \( u_{*,c} \) is the bed shear velocity.

The bed shear velocity \( u_{*,c} \) is given by:

\[
u_{*,c} = \sqrt{\frac{g - v}{C}}
\] (4.9.10)

Here \( v \) is the depth-averaged velocity vector and \( C \) is the Chézy coefficient given by:

\[
C = 18 \log \left( \frac{12h}{k_{*,c}(x)} \right)
\] (4.9.11)

In which \( k_{*,c} \) is the current-related bed roughness height, which is a user-defined property (RC).

The coefficient \( \beta \) is the ratio between sediment and fluid mixing coefficient and is represented by (see van Rijn, 1993):

\[
\beta = 1 + 2 \left( \frac{w}{u_{*,c}} \right)^2, \quad \text{with } \beta_{\text{max}} = 1.5
\] (4.9.12)

This equation results in values larger than unity, indicating that the effect of an increase of the sediment mixing coefficient with respect to the fluid mixing coefficient due to centrifugal forces causing the particles to be thrown outside the eddies is dominant over the decrease of the sediment mixing coefficient due to an incomplete response of the particles to turbulent fluctuations.

4.9.4 Reference concentration near the bed

The reference concentration \( c_0 \) (in kg/m³) is given by:

\[
c_0 = 0.015 \rho_s \frac{d_{50}}{a} \frac{T^{1.5}}{D_0^3}
\] (4.9.13)

In which \( D_0 \) is the dimensionless particle diameter, \( T \) is the dimensionless bed-shear stress parameter and the reference level \( a \) is the thickness of the bed-load layer.

The reference level \( a \) is given by the maximum value of the current-related roughness \( k_{*,c} \) and wave-related roughness \( k_{*,w} \), which are both defined by the user in the input file (RC and RW respectively).

The bed shear stress parameter is defined as follows:

\[
T = \frac{\tau_{b,cr}}{\tau_{b,cr}}
\] (4.9.14)

In which:

45
\( \tau_{b,sw} = \) Time-averaged effective bed-shear stress (N/m²)
\( \tau_{b,cr} = \) Time-averaged critical bed-shear stress according to Shields (N/m²)

The time-averaged critical shear stress is computed as:

\[
\tau_{b,cr} = (\rho_s - \rho)gd_s\theta_{cr}
\]  
(4.9.13)

With \( \theta_{cr} \) according to Eq. (5.7.10). Note that no bed slope correction is applied to the critical shear stress, as opposed to the critical shear stress for the bed load formula.

### 4.10 Bed shear stress formulations

#### 4.10.1 General procedure for the bed shear stress

**Bed shear stress due to waves**

The bed shear stress \( \tau_{b,w}(t) \) due to waves alone is related to the wave friction coefficient \( f_w \) by:

\[
\tau_{b,w}(t) = \frac{1}{2} \rho f_w |u_{orb,\delta}(t)| u_{orb,\delta}(t)
\]  
(4.10.1)

In which \( u_{orb,\delta}(t) \) is the instantaneous oscillating velocity just outside the wave boundary layer. The wave friction factor \( f_w \) is assumed to be constant over the wave cycle and is modelled according to Swart (1974):

\[
f_w = \exp \left[-6 + 5.2 \left(\frac{A_{\delta}}{A_{s}}\right)^{0.19}\right]
\]  
(4.10.2)

\( f_{w,max} = 0.3 \)

With \( \hat{A}_{\delta} \) is the peak value of near-bed orbital excursion according to linear wave theory and based on the significant wave height and peak wave period. The wave-related bed roughness height \( k_{s,w} \) is slightly different in the bed load and suspended load module.

The mean (time-averaged) value of the magnitude of the bed shear stress can be written as:

\[
<|\tau_{b,w}|> = \frac{1}{2} \rho f_w <u_{orb,\delta}^2(t)> = \frac{1}{4} \rho f_w \hat{U}_{\delta}^2
\]  
(4.10.3)

Where \( \hat{U}_{\delta} \) is the peak value of near-bed orbital velocity according to linear wave theory and based on significant wave height and peak wave period.

**Bed shear stress due to currents**

The current-related bed shear stress in the presence of waves can be modelled by assuming a two-layer system for the velocity \( v \):

- Logarithmic velocity profile affected by the form roughness \( k_s \) inside the near-bed mixing layer
• Logarithmic velocity profile affected by the apparent roughness \( k_a \) (\( k_a > k_s \), due to the influence of waves on the current profile) outside the near-bed mixing layer. With the roughness height \( z_0 \) defined by \( k/30 \), the expression for rough turbulent flow \( v(z) \) are:

\[
\begin{align*}
\begin{cases}
    v(z) = \bar{v} & \text{for } z \geq \delta \\
    v(z) = \nu(z) & \text{for } z < \delta \\
    \nu(z) = \nu(\delta) \\
    \nu(\delta) = \bar{v} \\
\end{cases}
\end{align*}
\]

\[
\frac{\ln \left( \frac{30z}{k_a} \right)}{-1 + \ln \left( \frac{30h}{k_a} \right)}
\]

\[
\frac{\ln \left( \frac{30z}{k_s} \right)}{\ln \left( \frac{30\delta}{k_a} \right)}
\]

\[
\frac{\ln \left( \frac{30\delta}{k_s} \right)}{-1 + \ln \left( \frac{30h}{k_s} \right)}
\]

Where \( \bar{v} \) is the depth-averaged velocity vector and the thickness \( \delta \) of the near-bed sediment mixing layer is equal to 0.1m. The minimum value of \( \delta \) is the apparent roughness height \( k/30 \).

The shear stress velocity (at \( z = \epsilon z_0 = \epsilon k/30 < \delta \)) is then given by:

\[
v_\epsilon = \kappa \nu(ez_0) = \bar{v} \frac{\kappa \ln \left( \frac{30\delta}{k_a} \right)}{-1 + \ln \left( \frac{30h}{k_a} \right) \ln \left( \frac{30\delta}{k_s} \right)}
\]

The bed shear stress defined at \( z = z_0 \), due to currents in the presence of waves then reads:

\[
\tau_{b,c} = \rho \nu^2 = \bar{v} \frac{\rho \kappa^2 \ln \left( \frac{30\delta}{k_a} \right)^2}{-1 + \ln \left( \frac{30h}{k_a} \right)^2 \ln \left( \frac{30\delta}{k_s} \right)^2}
\]

In the absence of waves, and thus having a logarithmic velocity profile along the complete water column, the bed shear stress is equal to:

\[
\tau_{b,c,\text{no waves}} = \bar{v} \frac{\rho \kappa^2}{-1 + \ln \left( \frac{30h}{k_a} \right)^2} = \rho \frac{\bar{v}^2}{g} \left( \frac{v}{C} \right)
\]

With \( C = 18 \log \frac{12h}{k_s} \).
This can also be written as:

$$\tau_{b,c,\text{no waves}} = \frac{1}{8} \rho f_c v^2$$  \hspace{1cm} (4.10.8)

With

$$f_c = 0.24 \left[ \log \frac{12h}{k_c} \right]^{-2}$$

Combining Eqs. (4.10.7) and (4.10.8) gives for the bed shear stress due to currents in the presence of waves:

$$\tau_{b,c} = \frac{1}{8} \alpha_r \rho f_c v^2$$

With:

$$\alpha_r = \left[ \frac{\ln \left( \frac{30\delta}{k_a} \right)}{\ln \left( \frac{30\delta}{k_c} \right)} \right]^2 \left[ \frac{-1 + \ln \left( \frac{30h}{k_s} \right)}{\ln \left( \frac{30h}{k_a} \right)} \right]^2$$  \hspace{1cm} (4.10.9)

Instead of in terms of the depth-averaged velocity, the bed shear stress due to currents in the presence of waves can also be expressed on terms of the velocity at the level \( z \) above the wave boundary layer. We therefore substitute Eq. (4.10.4) in Eq. (4.10.9). This yields with an unchanged definition of the current friction factor \( f_c \):

$$\tau_{b,c} = \frac{1}{8} \alpha(z) \rho f_c v(z)^2$$

With:

$$\alpha(z) = \left[ \frac{-1 + \ln \left( \frac{30h}{k_s} \right)}{\ln \left( \frac{30z}{k_c} \right)} \right]^2$$  \hspace{1cm} (4.10.10)

**Bed shear stress due to the combined wave-current motion**

The time-averaged bed shear stress under combined wave current motion now becomes:

$$\tau_{b,m} = \langle \tau_{b,w} \rangle + \tau_{b,c}$$  \hspace{1cm} (4.10.11)

For the time-dependent shear stress a slightly different formulation is used for the case of combined waves and currents. We follow the approach as suggested by Grant and Madsen (1979) who assume that the bed shear stress can be expressed as a quadratic function of the combined wave-current velocity \( u_b(t) \) at some height \( z \) above the bed (above the wave boundary layer):
\[ \tau_{n,\text{cw}}(t) = \frac{1}{2} \rho f'_{\text{cw}} |u_n(t)| u_n(t) \]  

(4.10.12)

Here \( f'_{\text{cw}} \) is a (skin) friction factor for the combined wave-current motion.

In principle the problem is 2DH since waves and currents may interact under an arbitrary angle. The bed shear stress \( \tau_n \) and near-bed velocity \( u_n \) are vectors in the same direction with varying magnitudes and varying directions during the wave cycle.

The above mentioned quadratic friction law is used together with a formulation for the weighted friction coefficient for currents and waves \( f'_{\text{cw}} \). Following Van Rijn (1993) the wave-current friction factor \( f'_{\text{cw}} \) is computed from the friction factors for "waves alone" and "currents in the presence of waves", weighted linearly with the relative strength of the near-bed net current and oscillatory velocity amplitude.

\[ f'_{\text{cw}} = \alpha f'_{\text{w}} + (1 - \alpha) f'_{\text{c}} \]  

(4.10.13)

With:

\[ \alpha = \frac{\langle u_n \rangle}{\langle u_n \rangle + U} \]  

(4.10.14)

In which \( \langle u_n \rangle \) is the time-averaged or mean current near-bed velocity at the level \( z \) (but above the wave boundary layer) and \( \hat{U} \) is the velocity amplitude of the wave-induced oscillatory flow near the bed (without mean current).

### 4.10.2 Bed shear stress in suspended load model

The suspended load module requires the magnitude of the time-averaged bed shear stress in the combined wave-current motion in order to compute the reference concentration. The bed shear stress due to the combination of waves and currents is computed as the sum of the bed shear stress due to waves and currents respectively (Eq. 4.10.11). Equations (44.10.3) and (4.10.11) are used to obtain the bed shear stress due to waves and currents. The current friction factor and the wave friction factor are defined by Eq. (4.10.8) and Eq. (4.10.2) respectively.

The so obtained current- and wave-related bed shear stresses are total bed shear stresses, i.e. they have to be multiplied by efficiency factors to obtain the effective bed shear stress \( \tau'_{n,\text{eff}} \). The efficiency factor for currents is given by:

\[ \mu_n = \frac{f'_{\text{c}}}{f'_{\text{c}}} \]  

(4.10.15)

In which the grain-related \( f'_{\text{c}} \) is obtained from Eq.(5.10.8) using \( k_c = 3 \, d_{50} \) and \( f'_{\text{c}} \) by using by using \( k_c = RC \) (current related roughness) in Eq.(5.10.8).

The efficiency factor for waves is given by (Van Rijn, 1993):

\[ \mu_w = \frac{0.6}{D_w} \]  

(5.10.16)
4.10.3 Bed shear stress in bed load model

The bed load model requires instantaneous shear stress in the combined wave-current motion which are computed using Eqs.(4.10.12-14) using the mean and oscillatory velocity components just outside the wave boundary layer \((z=\delta)\). The necessary current- and wave-related friction factors in Eq. (4.10.13) are computed by using Eqs. (4.10.8) and (4.10.2) respectively. As in the suspended load module \(f_s\) is obtained by using \(k_s=\text{RC}\) in Eq.(4.10.8).

As opposed to the procedure in the suspended load module, the grain-related friction factor \(f_s\) in Eq. (4.10.8) is obtained by Eq. (4.10.8) using \(k_s=3 \cdot d_w\) for \(\theta<1\) and \(k_s=3 \cdot \theta \cdot d_w\) for \(\theta>1\). This same grain-related roughness value is used for \(f_s\) in Eq. (5.10.2) instead of the user-defined wave-related roughness (RW). Therefore, the use of the wave-related efficiency factor is superfluous.

For the computation of the roughness height \(k_s\) in the sheet flow regime (\(\theta>1\)) a mean bed-shear stress (or Shields parameter) is necessary, which itself is dependent on \(k_s\). Therefore an iterative procedure is followed in which a mean bed-shear stress just outside the wave boundary layer \((z=\delta)\) is first estimated with input of the grain roughness \(k_s=3 \cdot d_w\) using Eqs. (5.10.11), (5.10.3) and (5.10.10) for the computation of the mean bed-shear stress. With this bed-shear stress a new estimate \(k_s\) is obtained using \(k_s=3 \cdot \theta \cdot d_w\) and the computation is repeated until the solution converges and changes less than 1\% during the last iteration. Note that the current-related roughness height \(k_s\) used in Eq. (5.10.10) is set equal to the user-defined value (RC).

References


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5 Results and analysis

5.1 Preliminary hypothesis

Unibest-TC has been applied taking into account many hypothesis:

- Negligible longshore transport gradient,
- Tidal currents and variations are not considered,
- The coast may be thought uniform in the alongshore direction,
- The parameters $H_{rms}$, $T_p$, wave direction, wind direction, wind velocity are supposed to be constant at the seaward boundary during the total computational time,
- There are groins in the area but we disregard them,
- The bottom files are the ones shaped by moderate conditions. That's the reason why it is possible to use them to see changes during storms.

5.2 Computations

For the computations with Unibest-TC two different kinds of data files are used: files with wave and wind conditions and files with bottom profiles.

Wave and wind conditions data are gathered during storms, as there is no problem for the measure instrument to bear extreme conditions. As they are storm data, they are collected during the storm season, which roughly goes from December until the end of February. The bottom data are on the contrary collected in a fair weather day, with ships or with the use of the polar track.

The idea of my study is to match measured storm conditions, represented by the wind and wave data, with the bottom profile that exists under normal conditions. In this way, it will be possible to investigate the changes that a profile shaped by moderate conditions undergoes during extreme conditions and the wave changes caused by this remodelling of the sea bottom.

The storm conditions are taken constant in time in order to have the worst conditions possible. Therefore for the total computational time we will have constant $H_{rms}$, $T_p$, wave and wind direction, wind velocity.

The total time of the computations is usually around one day and this choice is made for two reasons:

- The first is that the storm (wind and wave) data are collected during a period of 24 hours: this could mean that none of the measured storms lasted more than one day.
- The second is that if we prolong the calculation for a period of time longer than one day, there are no more substantial changes in the profile.

One of the biggest problems encountered during the calculations has been the setting of the time step $[\Delta t]$ and of the spatial step $[\Delta x]$. In fact these two parameters influence the stability of the computation, so it is necessary to choose suitable values for them, especially minding about their ratio, $\Delta x / \Delta t$. Therefore a change in $\Delta x$ has to come along with an equivalent change in $\Delta t$ in order to have good results in the calculations.
5.3 Results

To have a general view on the computations it is useful to read these three tables, one containing the wave data used in the computations (input), another one containing the data used in the different calculations and the last one containing the main results obtained (output).

The first table represents wave and wind characteristics of the eight storms taken into account. These storms are considered to be representative of extreme conditions, therefore we can use them to compute the changes in bottom profile from a qualitative point of view.

For each of these storms I have computed bottom profile and wave parameters evolution.

Tab. 5.1: Overview of wave data used in the computations

<table>
<thead>
<tr>
<th>Day</th>
<th>Hrns</th>
<th>Tp</th>
<th>Wave direction</th>
<th>Wind velocity</th>
<th>Wind direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>risp shore-normal</td>
<td>[deg]</td>
<td>[m/s]</td>
</tr>
<tr>
<td>29-jan-00</td>
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<td>0.901</td>
<td>17.087</td>
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<td>12.558</td>
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<td>-2.8</td>
<td>11.808</td>
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<td>22-feb-99</td>
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<td>7.273</td>
<td>-0.454</td>
<td>17.741</td>
<td>341.540</td>
</tr>
</tbody>
</table>

Wind data are missing for the 16th and 17th of February 1999, then I used wind data of the 22nd, which was the nearest day in time.

The second table specifies the different simulations. In the first column there is the computation number, in the second one there is the bottom profile used, in the third column there is the number of the transect and in the last one it can be seen the day chosen for the storm conditions.

Tab. 5.2: Description of the simulations

<table>
<thead>
<tr>
<th>Computation number</th>
<th>Bottom profile</th>
<th>Transect</th>
<th>Storm</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>24-feb-00</td>
<td>20.830</td>
<td>29-jan-00</td>
</tr>
<tr>
<td>20</td>
<td>6-jul-00</td>
<td>20.830</td>
<td>29-jan-00</td>
</tr>
<tr>
<td>21</td>
<td>6-jul-00</td>
<td>20.830</td>
<td>29-jan-00</td>
</tr>
<tr>
<td>28</td>
<td>30-nov-00</td>
<td>20.830</td>
<td>29-jan-00</td>
</tr>
<tr>
<td>38</td>
<td>10-mar-99</td>
<td>20.830</td>
<td>6-feb-99</td>
</tr>
<tr>
<td>39</td>
<td>10-mar-99</td>
<td>20.860</td>
<td>6-feb-99</td>
</tr>
<tr>
<td>46</td>
<td>19-jul-99</td>
<td>20.800</td>
<td>6-feb-99</td>
</tr>
<tr>
<td>47</td>
<td>19-jul-99</td>
<td>20.815</td>
<td>6-feb-99</td>
</tr>
<tr>
<td>48</td>
<td>19-jul-99</td>
<td>20.860</td>
<td>6-feb-99</td>
</tr>
<tr>
<td>55</td>
<td>13-set-99</td>
<td>20.800</td>
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</tr>
<tr>
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<tr>
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<tr>
<td>60</td>
<td>20-oct-99</td>
<td>20.860</td>
<td>6-feb-99</td>
</tr>
</tbody>
</table>
The last table represents output parameters of interest obtained in my computations. All of these parameters are meant after one day of storm conditions.

**Tab. 5.3:** Results obtained in the computations (after one day of constant extreme conditions). All the measures are in metres. * At the toe of the dike, ** if positive it means in the offshore direction, *** from the toe of the dike.

<table>
<thead>
<tr>
<th>Computation number</th>
<th>Bottom profile</th>
<th>Transect</th>
<th>Storm</th>
</tr>
</thead>
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<table>
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<th>Computation</th>
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<th>Increase in the height of the offshore bar</th>
<th>Horizontal shifting of the offshore bar*</th>
<th>Depth of negligible bottom changes</th>
<th>Increase in Hmax*</th>
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I analysed the computational results to obtain insight into:

- General characteristics (qualitatively) of the profile evolution
- Wave propagation as a function of time, including
  - Pattern of wave height
  - Wave height and period at the toe of the dike
  - Wave set-up at the toe of the dike
- Time-averaged offshore directed sediment transport rate
- Total erosion-accretion as a function of the distance from the toe of the dike.

Furthermore, in Appendix B, can be find a table where the model results are processed in order to have wave-related parameters that may be relevant for design purpose.

### 5.3.1 Wave variations

One of the main goals of this study is to investigate how wave induced bottom changes affect wave propagation towards the sea defence. For this purpose, we'll focus only on the results in terms of wave height at the coordinate x=0, toe of the dike, as this part is the most interesting in the whole cross-shore profile.

The more important result obtained is the fact that \( H_{rms} \) does not always increase at the toe of the dike after one day of storm, but it has been found in some simulations that it can also increase. This discovery is in contrast with the common sense that lead us to think that wave conditions at the toe of the sea defence during a storm can only get worse, with respect to the wave conditions in the same point at \( t = 0 \) days. This means that we suppose that, after one day of constant storm conditions, at the toe of the dike the \( H_{rms} \) will be higher than the one we have in the same point at the beginning of the storm.

The cause of this anomalous behaviour can be seen in Figure 5.1. In this graph are plotted the bottom profile and the \( H_{rms} \) in two different moments: at the beginning of the storm \( (t=0) \) and after one day of extreme conditions \( (t=1) \).

It can be noticed that the bottom profile evolves in such a way that the offshore bar (the breaker bar corresponding to moderate wave conditions, usually located around ~350 metres offshore) increases its height. This change in the cross-shore profile will affect the wave propagation.
Fig. 5.1: Computation number 067. It can be noticed the remarkable increase in height of the offshore bar that causes wave breaking.

Depending on the crest height of the bar in relation to the wave height, waves can undergo a shoaling effect or break: the case showed in Figure 5.1 seems to be the latter. In fact, looking at the representation of H_{rms} at t=1 day, it can be seen that, while the waves approach the bar, they start to undergo a shoaling effect, so the H_{rms} will increase. Then the waves reach the area around the crest of the bar, in which the H_{rms} becomes comparable with the water depth: therefore a fraction of the waves will break.

Looking at the graph 5.1 it can also be seen how the value of the H_{rms} before and after the bar is different, and the value after the bar is smaller than the one just before the bar. This means that a fraction of the waves have undergone breaking. If we plot the fraction of breaking waves, Q, at t=0 and at t=1 day (Fig. 5.2) we can notice that Q in correspondence of the crest of the bar has a value of 0.23, meaning that the percentage of breaking waves is 23%, value that is significant.

Fig. 5.2: Computation number 067. The fraction of breaking waves at t=1 day in correspondence of the offshore bar has a value of 0.23.

The effect of wave breaking discussed above will then influence the H_{rms} that we find at the toe of the dike. This phenomenon in fact causes a loss of energy and thus a decrease
of $H_{\text{rms}}$ after the bar that will lead to a smaller wave height at the sea defence, even if there is a shoaling effect (as waves approach the shore) that further changes the $H_{\text{rms}}$. As can be seen from Figure 5.3, in $x=0$ the $H_{\text{rms}}$ at $t=1$ day is smaller than the one at $t=0$.

![Bottom profile and $H_{\text{rms}}$](image)

**Fig. 5.3:** Computation number 067. The wave height at $t=1$ day is smaller than the one at $t=0$.

The fact that there is a decrease in the wave height can also be seen from the graphs of the wave set-up (see Figure 5.4). The wave set-up is the elevation of mean water level caused by wave action: it balances the gradient of the cross-shore directed radiation stress. In fact, the latter changes if there is a change in the average water level along a profile perpendicular to the coast and the resultant radiation stress is counteracted by a static horizontal pressure gradient resulting from a water surface slope.

![Set-up](image)

**Fig. 5.4:** Computation number 067. The value of the wave set-up at the end of the computation has grown with respect to the value of the beginning.

According to the equation of equilibrium of these two quantities, if the wave set-up increases it means that the cross-shore component of the cross-shore radiation stress is decreasing and thus $H_{\text{rms}}$ is reduced.

We can then think that the evolution during a storm of this offshore bar is really significant as it can influence in a marked way the $H_{\text{rms}}$ that we find at the dike. This result is very important because it shows that the wave height during an extreme event can also decrease, depending on the depth (below MWL) of the offshore bar.

If the offshore shoal increases its height of an amount that does not cause wave breaking on the bar itself, the wave height will have another behaviour, plotted in Figure 5.5.
Fig. 5.5: Computation number 057. In this case the increase in height of the offshore bar is not sufficient to cause wave breaking.

In this figure can be seen the bottom profile and the $H_{rms}$ at the beginning of the storm ($t=0$) and at the end ($t=1$ day).

This figure shows that in this case the waves approaching the bar will just undergo a shoaling effect in the zone in correspondence of the crest of the bar, without breaking and thus without losing energy. The result of this will be an increase in the $H_{rms}$ at the toe of the dike, usually followed by the breaking on the wall of the sea defence. Waves in most cases will not break before the dike for two reasons:

- The increase in the crest height of the inner bars (the bars that are in a range of 150 metres from the shore) during one day of storm is not so pronounced, as can be seen from Figure 6.5. Therefore waves in most of the cases will not break on the inner bars, just undergoing a shoaling effect, or at most only a negligible fraction of them will break.

- The scour that is formed in most of the computations at the toe of the dike (See always Figure 5.5) will further contribute to avoid wave breaking in the nearshore region.

If we plot the wave set-up also for the case of non-breaking waves on the offshore bar, we obtain the result shown in Figure 5.6.

Fig. 5.6: Computation number 057. The value of the wave set-up at the end of the computation has diminished with respect to the value of the beginning.
As can be seen, the wave set-up after one day of storm conditions has decreased. This means that the cross-shore component of the cross-shore radiation stress has increased, thus leading to an increase in $H_{\text{rms}}$. As can be seen from table B in the Appendix B, the average value of this increase is around 0.32 metres, value that is not negligible and can influence in an important way the security of the dike. In fact, usually design criteria for sea dikes are based on $H_{\text{rms}}^2$, which means a value of 0.102 m$^2$.

Therefore, we can conclude also in this case that the principal morphological change that influences most the wave height development during a storm is the increase in height of the offshore shoal.

What happens if also the increase in height of the nearshore shoals (bars created by the sediments eroded at the toe of the dike that settle seaward due to the undertow) causes wave breaking? The behaviour of the wave height will be different in the case of wave breaking or no breaking on the offshore bar. Figure 5.7 represents the evolution of the bottom profile and wave height during one day of storm in the case of wave breaking in the offshore and nearshore bar. It can be noticed that in the first case this further breaking on the nearshore shoal, summed to the breaking effect on the offshore bar, will lead to an even smaller wave height at the sea defence.

![Bottom profile and Hrms 075](image)

**Fig. 5.7:** Computation number 075. This graph is an example of double breaking: on the offshore and on the onshore bar.

If we plot also the fraction of breaking waves in correspondence of the nearshore shoal, we can notice that a significant amount of waves break on this bar, causing an important loss of energy (see Figure 5.8).

![Bottom profile and fraction of breaking waves 075](image)

**Fig. 5.8:** Computation number 075. The fraction of breaking waves on the inshore shoal increases at $t=1$ day.
In the second case, in which the waves do not break on the offshore shoal, the effect of breaking on the nearshore bar does not cause enough energy loss to reduce \( H_{\text{rms}} \) at the toe of the dike (See Figure 5.9).

![Bottom profile and \( H_{\text{rms}} \) 057](image)

**Fig. 5.9:** Computation number 057. This graph shows the phenomenon of breaking on the inshore shoal in the case of no breaking on the offshore shoal.

This can be due to the fact that just a small fraction of the waves will break as can be seen from Figure 5.10, in which is plotted the evolution of the bottom profile that leads to the formation of the nearshore bar and the fraction of breaking waves, at \( t=0 \) and at \( t=1 \) day.

![Bottom profile and fraction of breaking waves 057](image)

**Fig. 5.10:** Computation number 057. The fraction of breaking waves on the inshore shoal is negligible at \( t=1 \) day.

Therefore it can be concluded that in this second case, even if a fraction of the waves will break on the nearshore bar, the loss of energy caused by the breaking is not sufficient to reduce the \( H_{\text{rms}} \) at the toe of the dike.

![Breaking at offshore and nearshore bar](image)

**Tab. 5.4:** Combined effects of breaking
Finally, it is useful to look at the behaviour of $H_{\text{max}}$ as it is a function of the local water depth. We can notice from the different computations that $H_{\text{max}}$ always increases, even in the case of wave breaking on the offshore and inshore bar: in this latter case, of course, its increase will be less than the case of no breaking. The mean value of this increase is around 1.5 metres, value that is significant for the design of the dike.

![Hmax 055](image)

**Fig 5.11:** Computation number 055. It can be seen how the maximum wave height increases after one day of storm.

5.3.2 Bottom variations

It can be seen from the different computations that the bottom profile undergoes important changes if submitted to extreme conditions and these alterations are not just restricted to the area close to the dike, but are relevant also at some distance from the dike.

This is due to the fact that Petten sea defence has a shallow foreshore: for example the depth at a distance 400 metres from the shore has an average value of 7.5 metres.

Looking at the bottom profile in a direction that goes from the toe of the dike seaward, the first thing that we can observe in most of the different simulations is the formation of a scour hole at the toe of the dike.

In Figure 5.12 it can be seen an example of this phenomenon for the computation number 057. Each one of the lines represented is the cross-shore profile between 0 and -70 metres in different instants of time. From this graph we can notice that the scour deepens with time and the bottom profile translates vertically because of the erosion.

![Bottom profile 057](image)

**Fig. 5.12:** Computation number 057. It can be noticed in this figure the formation of a scour at the toe of the dike.
This scour is mainly due to the undertow caused by the waves breaking on the wall of the dike, while locally there is also an influence of the backflow from the slope of the dike. In fact, the wave breaking on the dike creates:

- a high turbulence level, which brings the sediments of the bed into suspension and
- the seaward-directed current called undertow.

Sediment transport will thus occur in the offshore direction, with the consequent scour at the toe of the dike.
The seaward sediment transport will have another result, as can be seen from Figure 5.13: the formation of one or more bars that are usually located around 90 metres from the dike.

![Bars 048](image)

**Fig. 5.13:** Computation number 055. Inshore bars that form for the settling of sediments eroded at the toe of the dike.

This leads to the conclusion that the sediment eroded from the toe of the dike will settle in this zone. Looking closer to these bars, we can also see that their mean behaviour is to become steeper and higher with the passing of time.

As we move offshore it can be noticed that in a certain point, usually located around 180 metres from the dike, there is an inversion in the behaviour of the cross-shore profile (see Figure 5.14). This is a characteristic that can be found in most of the graphs (See appendix B). Seeing in an offshore direction, accretion stops in this point and erosion begins: sediments begin to be eroded from the bottom, taken into suspension by the waves and then they settle seaward.

![Bottom profile 046](image)

**Fig. 5.13:** Computation number 046. Peculiar point in which accretion stops and erosion begins again.
If we look closer at this point, we can notice that it is peculiar: in fact it is the intersection of the lines that represent the evolving in time of the bottom profile (see Figure 5.14).

Fig. 5.14: Computation number 046. Zoom of the peculiar point. It can be observed the intersection of the three profiles.

This phenomenon deserves attention: the fact that the profiles in different times intersect each other in that point could mean that there the offshore directed sediment transport rate equals zero. If we prove this last assertion to be true, we could then divide the cross-shore profile in two closed parts, shown in Figure 5.14, independent one from the other.

To verify this, we can use the following procedure. If we define \( L(x, t, T) \) to be the onshore directed sediment transport rate at location \( x \), averaged over a period of time starting at \( t \) and with duration \( T \), it holds that

\[
L(x, t, T) = \frac{1}{T} \left( \int_{\xi=x}^{0} b(\xi, t + T) d\xi - \int_{\xi=x}^{0} b(\xi, t) d\xi \right)
\]

where \( b(x, t) \) is the bottom level at location \( x \) and at time \( t \), relative to an arbitrary horizontal reference level. Note that \( x \) equals 0 at the toe of the dike and is positive landward.

Therefore if \( L \) calculated (between \( t=0 \) and \( t=1 \) day) in the \( x \)-coordinate that define the singular point (called \( P \) in Figure 5.15) has a negligible value, we can then prove that in point \( P \) the sediment transport rate is negligible.

Fig. 5.15:

Hypothetical bottom profile evolution and hypothetical \( L \).
In figure 5.16 is plotted the rate of sediment transport for the computation number 038. On the $x$-axis we find the distance from the dike ($x$ positive in a landward direction) and on the $y$-axis the value of $L$. Positive values of $L$ mean onshore-directed sediment transport rates and negative values offshore-directed sediment transport rates. Looking at this graph we realize that the value of $L$ calculated in point P is different from zero of a quantity that is about 75 m$^3$/day (negative, so it is in an offshore direction). If we make this calculation for more bottom profiles, we discover that its average value is around 65 m$^3$/day. Therefore the profile cannot be divided in two independent areas, separated by a zero offshore transport point, but each part of it interacts with the others.

![Bottom profile 038](image1)

![L(x,0,1)](image2)

Fig. 5.16: Computation number 038. Bottom profile evolution and calculated $L$.

If we continue looking at the profile going in the offshore direction, we can see that the existing offshore bar (the breaker bar corresponding to moderate wave conditions) shifts seaward and increases its height and steepness after one day of storm conditions (see figure 5.17). In this process, the volume of the bar increases. The material required for this originates from the offshore transport at point P in combination with the erosion just seaward of this point. So there is no smoothening of the irregularities during a storm but, on the contrary, a growth.
It can be noticed from Figure 5.17 that the mean location of this bar is around 50 metres from the dike and it can be also seen from Table 6.3 that this shoal moves in the seaward direction over some 50 metres and it increases its height with about 1.3 metres.

![Bottom profile 055](image)

**Fig. 5.17:** Computation number 055. Bottom profile evolution and increase of the offshore bar

Another interesting result of the different computation has been the discovery that usually there is a point around -11 meters depth where there are no more morphological changes. In Figure 5.18 is possible to see an example of this.

![Bottom profile 075](image)

**Fig. 5.18:** Computation number 075. Bottom profile evolution and point in which there are negligible morphological changes (point A)

So, from that point seaward, at the scales of the considered storms, the profile changes seaward of the 11m depth contour are negligible compared to what happens closer to the shore. This means that in this zone the sediment transport gradient is zero. However if we calculate itself for some of the simulations, we discover that its value is between 30 and 150 m³/day, value not negligible at all. In Figure 5.19 is shown one example of the calculation for the simulation number 075.
Fig.5.19: Computation number 075. Offshore-directed sediment transport rate

In this graph is shown the rate of sediment transport and it can be noticed that its value seaward of the point $x=-500$ m is almost constant and has a positive and constant value of $\approx 125$ m$^3$/day. This means that the sediment transport in that zone is onshore-directed, until -440 metres, location of the crest of the offshore bar at $t=1$ day (see figure 5.18). This onshore-directed sediment transport also contributes to the accretion and steepening of the offshore bar. This mechanism therefore explains why seaward of point A there are negligible changes in the bottom profile.

The reason of the existence of this onshore-directed sediment transport could be the wave asymmetry.
6 Conclusions and recommendations

6.1 Introduction

The first objective of this study was to investigate morphological and hydrodynamical changes in front of Petten sea defence during storm conditions. The conclusions and recommendations, which are given in this chapter, concentrate on the results obtained with the computations (conclusions) and advices for further studies on the case (recommendations).

6.2 Conclusions

From the various computations it is clear that a scour hole is created just before the dike. This is due to the seaward sediment transport near the bottom caused by the combination of wave breaking and undertow.

The second important result obtained is the formation, growth and migration of bars. If there is no bar in the nearshore region, the sediment transport will create one (or more) with the sediments eroded at the toe of the dike. If there is one, like the offshore bar (breaker bar that corresponds to moderate wave conditions) it can be noticed that it will become steeper and higher.

Regarding the offshore bar, it will increase its height and will move seaward. Seaward of the 11m depth contour, at the scales of the considered storms, the profile changes are negligible compared to what happens closer to the shore. As there are no morphological changes but the value of the offshore sediment transport is different from zero, we can conclude that in this point all the mechanisms that influence sediment transport are in balance.

The bottom changes influence the wave field. There is a general increase in the wave height at the toe of the dike, except in the case in which there is breaking on the offshore bar. From the eight storms considered in our study, the increase has a non-negligible average value of 0.32 metres.

Regarding the wave period, as UniBest-TC is based on linear wave theory, it is assumed not to change when waves propagate towards the shore.

6.3 Recommendations

1. Apart from the deep water wave conditions, the wave energy that reaches the toe of the dike depends on dissipation due to wave breaking at the offshore bar and at the toe of the dike. As the offshore bar tends to increase its height during a storm, breaking at this location may gradually increase. An opposite effect is encountered near the toe of the dike, where ongoing erosion results in an even larger fraction of waves that break on the outer slope of the dike. This process continues until the cross-shore profile has reached an equilibrium shape that matches the storm conditions. The actual wave conditions at the toe of the dike depend on the net effect of the above mechanisms. Based on the studied cases, no general statement can be made on this net effect, and how it evolves during a storm. Making a general statement requires a systematic study on the relation between deep-water wave conditions and profile evolution.

2. In the present study, wave conditions at the seaward boundary of the cross-shore profile have been taken constant over the duration of a storm. In reality however,
these conditions vary with time. Roughly, storm shows initially a worsening of conditions and a decay at the end. This effect may be included in a future follow-up on this study to obtain a more precise picture of profile dynamics and wave conditions at the toe of the dike under storm conditions.

3. Longshore tidal flow is not taken into account in the present analysis. In fact, phenomena that act in longshore direction have not been considered at all. This limitation can be avoided when two horizontal dimensions are considered in stead of only one, for instance with a combination of a two (or three) dimensional morphological model (including tidal and wind-driven flow, waves, wave-current interaction and sediment transport). As the present study has a preliminary, introductory character, application of such a sophisticated model was beyond its scope. One argument in favor of extensive modeling is that breaker bars, which according to our analysis may alleviate the wave attack on the sea defence, commonly show interruptions in longshore direction. In such cases the wave conditions at the toe of the dike vary along it.

4. In most of the assessed cases, there is one location in the cross-shore profile where the bottom level remains practically unchanged during the storm. The time-averaged offshore-directed sediment transport rate shows a value that doesn't equal zero. It must be noted however, that these computations are rather rough as they are based on a discrete representation of the bottom profile on a rather course grid (mesh size 5 meters or more).
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Appendix A: Graphs of the simulations

In the following pages will be showed the graphs representing the simulations made. In each of them are represented five quantities:

- The bottom profile at the beginning of the computations (t=0 days),
- The bottom profile after half day of constant storm conditions (t=0.4 or t=0.5 days), depending on the time step used for the computations,
- The bottom profile after one day of storm conditions (t=1 days), as it has been shaped by the waves,
- The root mean square wave height at the beginning of the simulation (t=0 days),
- The root mean square wave height after one day of storm conditions (t=1 days).

In this way it will be easy to compare wave height changes with bottom changes and the contrary.

Fig. 1: Bottom profile and wave height for the computation number 007.

Fig. 2: Bottom profile and wave height for the computation number 020.
Fig. 3: Bottom profile and wave height for the computation number 021

Fig. 4: Bottom profile and wave height for the computation number 028

Fig. 5: Bottom profile and wave height for the computation number 038
Fig. 6: Bottom profile and wave height for the computation number 039

Fig. 7: Bottom profile and wave height for the computation number 046

Fig. 8: Bottom profile and wave height for the computation number 047
Fig. 9: Bottom profile and wave height for the computation number 048

Fig. 10: Bottom profile and wave height for the computation number 055

Fig. 11: Bottom profile and wave height for the computation number 056
Fig. 12: Bottom profile and wave height for the computation number 057

Fig. 13: Bottom profile and wave height for the computation number 058

Fig. 14: Bottom profile and wave height for the computation number 059
Fig. 15: Bottom profile and wave height for the computation number 060

Fig. 16: Bottom profile and wave height for the computation number 066

Fig. 17: Bottom profile and wave height for the computation number 067
Fig. 18: Bottom profile and wave height for the computation number 068

Fig. 19: Bottom profile and wave height for the computation number 074

Fig. 20: Bottom profile and wave height for the computation number 075
Fig. 22: Bottom profile and wave height for the computation number 080

Fig. 23: Bottom profile and wave height for the computation number 081

Fig. 24: Bottom profile and wave height for the computation number 085
Fig. 25: Bottom profile and wave height for the computation number 090

Fig. 26: Bottom profile and wave height for the computation number 091

Fig. 27: Bottom profile and wave height for the computation number 092
Fig. 28: Bottom profile and wave height for the computation number 093

Fig. 29: Bottom profile and wave height for the computation number 094

Fig. 30: Bottom profile and wave height for the computation number 100
Fig. 31: Bottom profile and wave height for the computation number 101

Fig. 32: Bottom profile and wave height for the computation number 110
Fig. 32: Bottom profile and wave height for the computation number 110

Fig. 33: Bottom profile and wave height for the computation number 120
Fig. 34: Bottom profile and wave height for the computation number 121
Appendix B: Table of Results in terms of significant wave height and peak period

In the following table the model results are processed in order to have wave-related parameters that may be relevant for design purpose.

<table>
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<th>Computation number</th>
<th>Hrms used [m]</th>
<th>Increase in Hrms*[m]</th>
<th>Decrease in Hrms*[m]</th>
<th>Hs (=1.4*Hrms) [m]</th>
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<th>Tp [s]</th>
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Table B: Summary of the results in terms of Hs and Tp.
Appendix C: Using Unibest-TC model

Unibest-TC consists of the following sub-programs:

- **Main-TC**, which is the shell program from which all sub-programs can be called,
- **Pre-TC**, which prepares a new or edits an existing definition file (extension dat),
- **Uni-TC** that is the simulation module of Unibest-TC. It reads the model set-up from a data definition file (extension dat),
- **Batch-TC**, with facilitates a sub-sequent execution of Uni-TC, in a way to execute a large number of Unibest-TC simulations,
- **Viz-TC** that can be used to inspect results of Uni-TC,
- **Ani-TC**, with which the output files can be animated in time.

C.1 Pre-TC: preparing an input file for Uni-TC

In the main window of Pre-TC two main window areas can be distinguished: the "Input options" area and the "Output options" area. The main buttons in the “Input options” zone identify the main input topics that together form the simulation settings for Unibest-TC.

Each topic is discussed here, being the accuracy in creating an input file the most important part in order to obtain a correct computation and avoid instabilities in the calculation.

C.1.1 Input options

Input schematisation

The input schematisation is defined as all the schematisation necessary to drive Unibest-TC. Two types of input schematisations can be distinguished:

1. The schematisations related to hydrodynamic forcing (waves, tide and wind) and
2. The schematisations related to the bed profile (bottom profile schematisation, sediment characteristics, and bottom roughness).

Definition of computational grid

In Unibest-TC a variable grid size can be defined. The first criterion is that the bottom profile must be represented with sufficient accuracy. The second criterion is that the grid resolution must be able to capture the rapid change of wave characteristics in the surf zone.

Definition of the computational grid comprises the definition of a starting point at deep water and a definition of the number of grid points and grid size. The starting point can be defined looking at the starting point of the corresponding bottom file (a file with the definition of the cross-shore profile in the transects between km. 20.785 and km 20.860, in front of Petten sea dike).

The transects chosen for my study are the nearest to the line of instruments, which is the transect at km 20.815 (see Fig. C.1).
The number of grid points is defined taking larger grid sizes faraway from the dike and diminishing the size while approaching to the dike, because we need more precision in the zone near the toe of the dike.

A general guideline is to use a grid size in the order of 5 to 10 m in the surf zone, while in the more deep regions a grid size in the order of 20 to 50 m is acceptable. The computational stepsize \( |dx| \) has to be chosen such that:

- the spatial resolution is sufficiently accurate
- numerical stability is guaranteed
- the maximum number of grid points of Unibest-TC, which is 399, is not exceeded.

The mesh will finish at a finishing point that coincides with the toe of the dike, as this point is the last erodible point of the bottom profile.

**Run parameters**

With these parameters the user can define which computations he intends to perform, which values to adopt for physical features like bottom friction and which values to apply for the different model parameters (e.g. profile simulations).

The model and run parameters are sub-divided into five categories:

- general
- waves
- currents
- sediment
- transport

**General parameters**

The general parameters are seven and they are used to define in general the computation. They do not relate to a specific module but influence most of the modules.
The first one \((DT)\) is the time step and has to be specified in days. It is used in both the hydrodynamic and morphodynamic modules. It has to be chosen such that the boundary condition time series are well represented and that no instabilities occur in morphological profile development. It is better to have a short time-step because the changes caused by a storm are very fast. The values used in my computation varies from two hours to fractions of days.

The second one \((NT)\) is the number of time steps and it determines the total simulation time. It has no influence on the numerical stability of the simulation, however the predictive capabilities decrease with increasing simulation interval.

In my computation the number of time steps is chosen in order to have a total simulation time of about one day. In fact, if we extend the computation for more time, for example for a period of two days, there are no significant changes in the behaviour of the bottom profile and the wave height. It has also to be said that wave and wind data during the storm are gathered just for a period of 24 hours. For these reasons it seemed to me reasonable to choose a computational time of one day.

The third one \((USTR)\) is the transport at shoreward boundary, or the transport rate at the last computational grid point. It is a user-defined parameter and it is given in \(\text{m}^3/\text{hr}\). As Unibest-TC does not have an appropriate description of the swash zone related processes, it can be specified the transport rate at the last computational grid point. The model will then interpolate the transport between the last considered wet cell and the user specified transport at the last computational grid point to obtain transport values for the dry part of the profile. This option should be used with care, unrealistic values can have a significant effect on the profile development. Without sufficient profile data on which this parameter can be calibrated it is advised to set its value to zero (default value), value used in my computation.

The parameter called \(JFR\), is the frequency of output files. \(JFR=1\) means every time step.

The fifth one \((TDNY)\) is the maximum relative wave period. It is defined as \(T_{DNY}=T_p(g/h)^{1.5}\) and is a dimensionless parameter which essentially indicates the non-linearity of the wave field. Higher values for \(T_{DNY}\) indicate that the simulation is continued into shallower water and vice versa. This parameter determine at which water depth the Unibest-TC calculations are stopped, and the minimum water depth is defines to be: \(h_{\text{min}}=g(T_p/T_{DNY})^2\). For the Rienecker and Fenton theory \(T_{DNY}\) should be limited to 25, while in prototype conditions values up to 40 can be used (default value). In my study I'll use a value of 35, meaning that \(h_{\text{min}}\) will be in a range between 0.85 m (using the maximum value for \(T_p\) obtained in the computations) and 0.35 m (using the minimum value for \(T_p\)).

The last parameters are water salinity \((SALIN)\) and temperature \((TEMP)\). They are both used to determine the water density, and have a little influence on model results. They are respectively set to 35 % and 11.5 degrees Celsius (average between the mean temperature of the North Sea in winter, which is 6 °C and the mean temperature in summer, which is 17 °C).

**Wave related hydrodynamic parameters**

As waves are usually the dominating forcing in the nearshore coastal regions, an accurate description of the wave characteristics is of vital importance. Modifications of these parameters affects the wave model but also all the other modules which use wave forcing as input.

The wave related parameters are 5, plus the possibility of computing the breaker delay.
The first parameter (ALFAC) is the wave breaking parameter, for use in dissipation formulation according to Battjes and Janssen (1978). It is used the default value $c=1$ because we consider a fully developed bore. Variations of this parameter influence the wave height prediction over the complete profile.

The second one (GAMMA) is the wave breaking parameter to determine maximum local wave height. It is set to 0.5 according to Stive and Battjes (1985): $\gamma = 0.5 + 0.4 \tanh (33s_0)$ where $s_0$ is the offshore wave steepness $= H_{rms}/L_0$. This parameter can have a significant influence on the predicted wave height and especially the initiation of breaking is influenced.

The parameter BETD is the roller parameter, according to Nairn et al. (1990) and it expresses the steepness of the wave front. It is needed to calculate the dissipation "Diss" of roller energy. The value set for this parameter is 0.1, default value. This parameter only has a small effect on the wave height predictions, however the wave set-up can be influenced significantly.

The fourth parameter (FWEE) is the friction factor and influences the amount of wave dissipation due to bottom friction. Especially, if wave calculations are made over a relatively long distance (3 to 10 km) this parameter can influence the wave height predictions significantly, but this is not our case. Although the value of the friction factor is influenced by the bed forms, it has been found that the default value 0.01, obtained from Delta Flume experiments, gives the best results.

The parameter C_R is the correlation coefficient between wave envelope and bound long waves and it expresses the long and short wave interaction. This parameter allows the phase shift between the long-wave and short-wave envelope: in case of complete bound long wave the two are exactly out of phase (phase shift is -90). However, in general this does not occur in the nearshore region where slightly negative values are present seaward the surf zone and slightly positive inside the surf zone. A negative phase shift will result in increased offshore transport as the short waves are the highest in the trough of the long wave (a positive phase shift has the opposite effect). By increasing the correlation coefficient the effect of the long wave will be enhanced, causing an increased offshore transport outside the surf zone and increased onshore transport inside the surf zone. By setting the coefficient to zero the long wave effect will be eliminated. The default value is 0.25 and for my study I'll use this. It varies from -C_R at deep water up to +C_R at the shoreline.

The breaker delay is the delay in the dissipation process caused by a conversion of the organized wave energy first into turbulent energy and then in a dissipation via the production of turbulence. It is used to improve predictions of bar morphodynamics. The breaker delay can be regarded as an extension to the roller model because it modifies the rate of wave breaking via a modification of the reference depth which is used to determine the local maximum possible wave height. This process needs four parameters to be defined: the number of wave lengths for depth integration, the power in weighing function and the seaward and shoreward boundary for reduction factor.

The number of wave length (F-LAM) over which weighted depth is integrated is usually in a range from 0.5 to 2.5. For my computations I’ll use the value $F-LAM=2$.

The power in weighting function (POW) is set to 1, implying an application of a linear weighting function to compute the reference depth $h_0$. 
DEEP-V and SHALL-V are respectively the seaward boundary and the shoreward boundary (in metres) of the zone along which λ (... is reduced by a factor of sin^2Θ, where Θ indicates the relative distance between the cross-shore locations DEEP-V and SHALL-V.
Θ equals 0 at x=DEEP-V and π/2 at x=SHALL-V.

Current related hydrodynamic parameters
The forcing of the wave model can be a combination of waves, tide and wind. Especially the cross-shore distribution of the cross-shore currents (undertow) is dominated by the wave forcing. The current model has only three parameters that have to be specified by the user: the viscosity coefficient, the friction factor for mean current and the reference depth for tidal velocity.

The first one (FCVISC) is the viscosity coefficient αw of vertical velocity profile. A higher value results in a higher (breaking wave induced) viscosity. A higher viscosity results in lower internal velocity gradients and therefore in flatter vertical velocity profiles. In general, a value of 0.10 gives reasonable results (default value).

The friction factor for mean current computation (RKVAL) is a roughness height, related to the grain size of the bed material or to the ripple height (if present). This parameter is set to 0.01, which is the default value and can be used to optimise velocity results: higher values result in large bed shear stresses and hence lower velocities.

The longshore tidal velocities can be calculated by determining the alongshore slope of the water surface (this slope is assumed to be constant over the cross-shore profile). The depth-averaged tidal velocity is determined using Chezy formulation:

\[ v_{tide} = C \sqrt{h \frac{\partial h}{\partial y}} \]

The parameter h is the reference depth to calculate tidal velocity (DIEPV) and is set to 0 m, because I decided to consider negligible the tide.

Grain size parameters
The grain size parameters are useful to define the granulometric distribution of the sediment, in order to obtain a more accurate prediction of the effective sand transport. The parameters to define are D50, median grain size, D90, grain size diameter of the 90 % of bed material and DSS, grain size diameter of the suspended sediment. In experiments it was found that the suspended sediment particle grain size is about 60 to 70% of D10. Each of these values has to be given in meters. As the grain size of Dutch sand varies in a range from 125 to 600 \( \mu \)m, I used for D50 a value of 240 \( \mu \)m and for D90 a value of 430 \( \mu \)m, assuming that the variation of grain size is linear. Consequently, DSS will be 165 \( \mu \)m.
There is also the option of considering the cross-shore varying grain size, switching on the proper box, but we'll not consider it even if it is an important parameter of sediment transport, because we do not have data on the real grain size of the sea bottom in front of Petten sea defence.

Transport parameters, including slope effects
The transport parameters are related to the sediment transport formulations and are used to calculate bottom changes. Therefore the morphodynamic switch (IBOD) has to be switched on. If multiple time steps are executed, the hydrodynamic and
transport modules will generate output, according to the prescribed boundary condition, based on the initial bathymetry.
The transport parameters are seven and are listed below.

The first one is the current related roughness \((RC)\), and the default value, according to Delta Flume experiments is 0.01 in meters.

The second one is the wave-related roughness for sediment transport computation \((RW)\), and is set equal to \(RC\). In general, the \(RW\) value should be lower than the value for \(RC\).

The third one is the fixed bottom layer \((REMLG)\) or the layer over which the sediment transport is reduced to zero in case of a fixed bed. If the bottom height is less than the fixed bottom added with \(REMLG\), a reduction factor is applied. In this way a gradual decrease in sediment transport is imposed when the bottom profile reaches the fixed layer, which avoids numerical instabilities. The default value is 0.1 and I will use this value in the computation.

Then it is necessary to define the fourth and fifth parameter the most seaward and the most landward location respectively \(XF1\) and \(XF2\), the starting and the ending point of the bottom file.

The final parameters to define are the internal friction angle at location \(XF1\) and \(XF2\), respectively \(TANPHI1\) and \(TANPHI2\). Actually, computed bed load transport rates are corrected for the local bottom slope as a function of the local (subaqueous) angle of internal friction \(\phi\). The value of \(TANPHI1\) and \(TANPHI2\) varies from 0.01 to 0.60. In my calculation I’ll use the default values that are respectively 0.03 (outside the surf zone) and 0.1 (well inside the surf zone).

The slope effect is primarily included to control offshore migrating bars.

**Boundary conditions**

The boundary conditions define bathymetry, hydrodynamic and meteorological conditions, which can be constant or varying. The input of varying boundary conditions can be made entering a file with time series of data (usually an ASCII file with a fixed format).

There are two types of boundary conditions file: the bottom file and the boundary condition files.

The bottom file carries the extension *.bot and can be used for the definition of the initial bottom profile and the definition of a fixed layer. The \(x\)-coordinate is always directed towards the shoreline. The bottom file ends at the \(x\)-coordinate of the toe of the dike, because from that point the bottom can't be eroded.

The boundary condition file (*.ubc) defines time series of wave and wind conditions and of tidal water level (as said in paragraph 3.5 tide is not taken into account).

As Unibest-TC is a so-called profile model it assumes that the modelled coastal section is longshore uniform. Any effects resulting from longshore non-uniformities are not included in the model.

Thanks to a special button, the selected time series can be inspected graphically as \(X-Y\) plot or \(T-Y\) plot.

Unibest-TC needs off-shore boundary conditions for incoming waves, tide and wind, as well as initial bottom profile. Therefore the variables to be supplied are:

- the bottom profile \((Z)\) in coordinate system \((x,z)\), varying for each transect,
- the position of fixed bottom level \((Z_FIX)\) in coordinates \((x,z)\),
• the tidal elevation ($H_0$) in meters,
• the tidal velocity at reference depth DIEPV in m/s,
• the angle of incoming waves (A_WAVE) relative to shore-normal (deg),
• the root mean square wave height (HRMS), taken constant in time,
• the peak wave period (T) in seconds,
• the wind velocity (V_WIND),
• the wind direction relative to shore normal (deg)

Boundary conditions are read from file (time series), or have a fixed value specified in the column 'VALUE'. The fixed value is used if the switch 'Constant' is selected in the column 'Mode', if the switch is set to 'Varying' input is read from an ASCII-file.

C.1.2 Output options

Definition of observation points

Observation points are the points along the profile where output of time series is required. It is necessary to specify which output is desired to be given.

Definition of profiles

In this section are defined the time points at which output over the complete computational grid is required. In the table the times at which output is desired have to be given.

Volumes

In certain regions, for instance around a beach nourishment, it can be of interest to determine the sediment balance of a reference volume. For my study I'll not use this option, as the objective is to compute bottom and wave height changes.

Definition of vertical and intra wave functions

Finally, it is possible to store the vertical distribution of the return flow and sediment concentration in a file, the time series of the near-bed orbital velocity or both. This is done at a user-defined location and user-defined time.

C.2 Uni-TC: Model simulation program

The Uni-TC module is the simulation model of UNIBEST-TC. It reads the model set-up from a data definition file (extension DAT), previously created with Pre-TC, and makes the simulation.

C.3 Batch-TC: Running Uni-TC in batch mode

It is fairly common to execute a large number of UNIBEST-TC simulations. BATCH-TC is the tool that facilitates a sub-sequent execution of Uni-TC.
C.4 Viz-TC: Visualisation of model results

VIZ-TC can be used to inspect results of UNI-TC (both binary and ASCII). It is noted that the file name is determined by the Run-ID. The ASCII file option can also be used to import measured data into VIZ-TC for comparison purposes.

In VIZ-TC there are four client windows:

- One is called "Data" and it is used for data selection,
- another one is "T-function", and it is used to select time series from the selected DAF-files,
- The third one is called "X-function", which is used to .........cross-shore profiles from the selected DAF-files,
- The last one is the "Plot" button, in which the selected data in the "T-function" and "X-function" can be inspected graphically.

C.5 Ani-TC: Animation of model results

With Ani-TC the binary output files can be animated in time. Default the program will animate the computed profile development in time. The user can run an animation or can inspect the results step by step.
Appendix D

Matlab Script

eta1=mp1_1;
p1=polyfit([1:length(eta1)],eta1,1);
eta1=eta1-polyval(p1,[1:length(eta1)]);
y1=fft(eta1);
pds1=y1.*conj(y1)/length(eta1);
freq=1.28*[0:length(eta1)/2-1]/length(eta1);

eta2=mp1_2;
p2=polyfit([1:length(eta2)],eta2,1);
eta2=eta2-polyval(p2,[1:length(eta2)]);
y2=fft(eta2);
pds2=y2.*conj(y2)/length(eta2);
freq=1.28*[0:length(eta2)/2-1]/length(eta2);

da3=mp1_3;
p3=polyfit([1:length(eta3)],eta3,1);
eta3=eta3-polyval(p3,[1:length(eta3)]);
y3=fft(eta3);
pds3=y3.*conj(y3)/length(eta3);
freq=1.28*[0:length(eta3)/2-1]/length(eta3);

eta4=mp1_4;
p4=polyfit([1:length(eta4)],eta4,1);
eta4=eta4-polyval(p4,[1:length(eta4)]);
y4=fft(eta4);
pds4=y4.*conj(y4)/length(eta4);
freq=1.28*[0:length(eta4)/2-1]/length(eta4);

eta5=mp1_5;
p5=polyfit([1:length(eta5)],eta5,1);
eta5=eta5-polyval(p5,[1:length(eta5)]);
y5=fft(p5);
pds5=y5.*conj(y5)/length(eta5);
freq=1.28*[0:length(eta5)/2-1]/length(eta5);

eta6=mp1_6;
p6=polyfit([1:length(eta6)],eta6,1);
eta6=eta6-polyval(p6,[1:length(eta6)]);
y6=fft(eta6);
pds6=y6.*conj(y6)/length(eta6);
freq=1.28*[0:length(eta6)/2-1]/length(eta6);

eta7=mp1_7;
p7=polyfit([1:length(eta7)],eta7,1);
eta7=eta7-polyval(p7,[1:length(eta7)]);
y7=fft(eta7);
pds7=y7.*conj(y7)/length(eta7);
freq=1.28*[0:length(eta7)/2-1]/length(eta7);

eta8=mp1_8;
p8=polyfit([1:length(eta8)],eta8,1);
eta8=eta8-polyval(p8,[1:length(eta8)]);
y8 = fli(eta8);
pds8 = y8.*conj(y8)/length(eta8);
freq8 = 1.28*[0:length(eta8)/2-1]/length(eta8);
eta9 = mp1_9;
p9 = polyfit([1:length(eta9)], eta9, 1);
eta9 = eta9-polyval(p9, [1:length(eta9)]);
y9 = fli(eta9);
pds9 = y1.*conj(y9)/length(eta9);
freq9 = 1.28*[0:length(eta9)/2-1]/length(eta9);
eta10 = mp1_10;
p10 = polyfit([1:length(eta10)], eta10, 1);
eta10 = eta10-polyval(p10, [1:length(eta10)]);
y10 = fli(eta10);
pds10 = y10.*conj(y10)/length(eta10);
freq10 = 1.28*[0:length(eta10)/2-1]/length(eta10);

x(:, 1) = pds1;
x(:, 2) = pds2;
x(:, 3) = pds3;
x(:, 4) = pds4;
x(:, 5) = pds5;
x(:, 6) = pds6;
x(:, 7) = pds7;
x(:, 8) = pds8;
x(:, 9) = pds9;
x(:, 10) = pds10;

mpds = mean(x');
[mpdsmax, i] = max(mpds);
Tp = 1/freq(i);

m0 = 0.5*1.28/1024*sum(mpds(1:end-1)+mpds(2:end));
Hrms = 2*sqrt(2*m0);
Tp
Hrms