2D and 1D numerical model simulations for the effect of a single detached breakwater on the shore

MSc thesis
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Abstract

The development of the coastal area is needed for tourism and commercial purposes for a country located nearby a coast. Coastal development may lead to local erosion of the shoreline.

Coastal defence structures, like an offshore breakwaters, may be used to solve this problem. An offshore breakwater is a breakwater parallel to the shore and located at a specific distance from the original shoreline. To understand the influence of the breakwater on the hydrodynamic processes as well as on the morphological evolution, physical models or mathematical models are developed. The mathematical models do not suffer from the scale effect.

In this study, a morphodynamic model DELFT2D-MOR, which is a two-dimensional horizontal model, and an one-line model UNIBEST have been used. The DELFT2D-MOR model is a coupled model system which consists of four modules. The four modules are the wave module (HISWA), the flow module (TRISULA), the transport module and the bottom module. A sensitivity analysis is made to investigate the impact of the offshore breakwater geometry on the hydrodynamic processes and the morphological evolutions. It was found that the breakwater geometry affects the hydrodynamic processes as well as the morphological evolution (salient or tombolo). Based on sensitivity analysis, the wave-current interaction was found to affect the morphological evolutions. The agreement between the results of the model and the literature results was quite satisfactory.

After getting insight in the validity of the DELFT2D-MOR model, a case study of a single detached breakwater situated at the northern coast of Egypt was carried out. The wave climate was analyzed to get the morphological equivalent one-wave condition and two-wave condition. The choice of the bottom roughness in the one-wave condition simulation appeared to affect the hydrodynamic processes and the morphological evolutions. Applying the two-wave condition in the 2D-computations gave a step forward for the morphological computations in which the two-wave condition simulation gave more authentic results than the one-wave condition simulation.

The choice of the wave-current induced shear stress appeared to affect the hydrodynamic processes in the 2D-model. The results of wave, longshore current and the sediment transport in the UNIBEST model are compared to the 2D-model results. It was found that the longshore current was higher due to the longshore current formulation used in the UNIBEST model. This in turn affects the sediment transport rate. Tombolo formation cannot be simulated in the UNIBEST model for a case with normally incident waves since the effects of the circulation cells are not taken into account. However, it can be simulated for a case with obliquely incident waves.
Acknowledgment

It is great that I have the opportunity in this acknowledgment to express my sonorous indebtedness and thankfulness to Prof. Dr. Ir.: L. C. van Rijn for his kind supervision, pursuit of excellence and creative suggestions. I am drastically indebted and thankful to Ir.: H. J. Verhagen for his fruitful discussion, moral support and precious remarks. Special thanks and appreciation to Ir.: K. J. Bos for his ceaseless guidance and friendship throughout this work.

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Personally, I am very grateful to my family, especially my parents, for every thing they taught me and for all sacrifices they made in my upbringing. I also would like to thank my fiancee Neveen Mohammed for her exceptional love and support. I may never be able to repay them, but fortunately, I have a lifetime to try.
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A \quad \text{wave action} \quad (\text{J}. \text{sec}/\text{m}^2)

A(\theta) \quad \text{an empirical coefficient} \quad (\text{1}/\text{year})

2a \quad \text{one-dimensional directional action spectrum} \quad (\text{J}/\text{m}^2)

b \quad \text{length of the section along which tombolo is connected to the structure} \quad (\text{m})

c \quad \text{half of tombolo base measured along the shoreline} \quad (\text{m})

C \quad \text{constant in Bijker's bed load formula} \quad (-)

C_d \quad \text{Chezy-coefficient} \quad (\text{m}^{1/2}/\text{s})

C_{gb} \quad \text{diffraction coefficient} \quad (-)

C_{gb} \quad \text{wave group velocity at breaking} \quad (\text{m}/\text{s})

C_{max} \quad \text{max. Courant number} \quad (-)

C_r \quad \text{reflection coefficient} \quad (-)

C_1 \quad \text{coefficient} \quad (\text{m}^2/\text{year}^\text{m})

C_2 \quad \text{coefficient} \quad (\text{1}/\text{m}^\text{n})

C_a \quad \text{sediment concentration in the bottom layer} \quad (-)

C_b \quad \text{bed level celerity} \quad (\text{m}/\text{s})

C_x \quad \text{group velocity in x-direction} \quad (\text{m}/\text{s})

C_y \quad \text{group velocity in y-direction} \quad (\text{m}/\text{s})

c \quad \text{distance of tombolo apex from the shoreline} \quad (\text{m})

D \quad \text{depth of profile closure (layer thickness)} \quad (\text{m})

D_b \quad \text{energy dissipation due to wave breaking} \quad (\text{J}/\text{m}^2)

d \quad \text{depth of water measured at tombolo apex} \quad (\text{m})

D_{50} \quad \text{median grain size} \quad (\text{m})

d_s \quad \text{average thickness of sediment entrapped layer} \quad (\text{m})

E \quad \text{wave energy density} \quad (\text{J}/\text{m}^2)

F_x \quad \text{external pressure in x-direction} \quad (\text{N}/\text{m}^2)

F_y \quad \text{external pressure in y-direction} \quad (\text{N}/\text{m}^2)

f_m \quad \text{mean wave frequency} \quad (\text{1}/\text{sec})

f_w \quad \text{wave friction factor} \quad (-)

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VIII

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List of symbols (continued)

- $g$: acceleration of gravity $\quad (m/s^2)$
- $H_o$: wave height at deep water $\quad (m)$
- $H_d$: diffracted wave height $\quad (m)$
- $H_i$: incident wave height $\quad (m)$
- $H_{\text{max}}$: maximum wave height $\quad (m)$
- $H_r$: reflected wave height $\quad (m)$
- $h_B$: water depth at the breakwater $\quad (m)$
- $h_b$: water depth at the breaker line $\quad (m)$
- $h$: water depth $\quad (m)$
- $K$: wave number vector $\quad (1/m)$
- $K_1$: coefficient $\quad (-)$
- $K_2$: coefficient $\quad (-)$
- $k_s$: bottom roughness $\quad (m)$
- $k_x$: effective dispersion coefficient in x-direction $\quad (-)$
- $k_y$: effective dispersion coefficient in y-direction $\quad (-)$
- $k$: wave number $\quad (1/m)$
- $L_B$: breakwater length $\quad (m)$
- $L_o$: wave length at deep water $\quad (m)$
- $\Delta l$: grid increment measured in the transport direction $\quad (m)$
- $m_0(\theta)$: zero-order moments of the action density spectrum $\quad (J/m^2)$
- $m_1(\theta)$: first-order moments of the action density spectrum $\quad (J/sec.m^2)$
- $n$: Manning’s coefficient $\quad (m^{1/3} s)$
- $Q$: Einstein integral term $\quad (-)$
- $Q_B$: sediment volume behind the breakwater $\quad (m^3)$
- $Q_b$: local fraction of breaking waves $\quad (-)$
- $Q_o$: total sediment volume behind the breakwater at the equilibrium state $\quad (m^3)$
- $Q(t)$: sediment volume of the tombolo at time $t$ $\quad (m^3)$
- $Q_s$: longshore sediment transport rate $\quad (m^3/s)$

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- $Q_{so}$: longshore transport along a straight shoreline parallel to the x-axis (longshore direction) $(m^3/\text{year})$
- $Q_s(\theta)$: longshore transport as a function of the shoreline orientation $(m^3/\text{year})$
- $q_b$: lateral sediment transport $(m^3/s/m)$
- $r$: bottom layer thickness $(m)$
- $S$: sediment transport $(m^3/s/m)$
- $S_b$: bed-load transport $(m^3/s/m)$
- $S_s$: suspended-load transport $(m^3/s/m)$
- $S_{tot}$: total sediment transport $(m^3/s/m)$
- $S_{xx}$: radiation stress component in the direction of wave propagation $(N/m)$
- $S_{xy}$: radiation shear stress acting on a plane normal to the shore $(N/m)$
- $s_l$: variation of the transport as a function of the shoreline orientation $(m^3/\text{year})$
- $s_o$: wave steepness (-)
- $T$: wave period in deep water (sec)
- $t$: time (sec)
- $\Delta t$: time step (sec)
- $U_c$: current-velocity vector $(m/s)$
- $U_c$: depth-averaged current-velocity $(m/s)$
- $U_{cw}$: wave-current velocity vector $(m/s)$
- $U_{cw}$: shear stress velocity due to wave and current $(m/s)$
- $U_w$: orbital velocity vector $(m/s)$
- $u$: depth-averaged water flow velocity in x-direction $(m/s)$
- $\hat{u}_o$: maximum orbital velocity $(m/s)$
- $V$: longshore current $(m/s)$
- $\theta$: angle of wave propagation $(-)$
- $V$: volume of sand accumulated in a tombolo $(m^3)$
- $v$: depth-averaged water flow velocity in y-direction $(m/s)$
- $w_s$: sediment fall velocity $(m/s)$
- $X$: attachment width at the breakwater $(m)$

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<th>Symbol</th>
<th>Description</th>
<th>Units</th>
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<td>$x$</td>
<td>longshore coordinate (one-line model)</td>
<td>(m)</td>
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<td>$\Delta x$</td>
<td>grid size in x-direction</td>
<td>(m)</td>
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<tr>
<td>$y$</td>
<td>cross-shore coordinate (2D-model)</td>
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<tr>
<td>$\Delta y$</td>
<td>grid size in y-direction</td>
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<tr>
<td>$Y_B$</td>
<td>offshore distance from the original shoreline</td>
<td>(m)</td>
</tr>
<tr>
<td>$Y_b$</td>
<td>breaker line distance from the original shoreline</td>
<td>(m)</td>
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<tr>
<td>$y$</td>
<td>cross-shore coordinate (one-line model)</td>
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<td>$\Delta y$</td>
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<tr>
<td>$\alpha$</td>
<td>structure seaward angle</td>
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<tr>
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<td>$\alpha_n$</td>
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<td>$\beta$</td>
<td>power of the used transport formula</td>
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<td>$\theta$</td>
<td>shoreline orientation</td>
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<td>$\theta_{b}$</td>
<td>angle between breaking wave crest and x-axis</td>
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<tr>
<td>$\theta_{bs}$</td>
<td>breaking wave angle to the shoreline</td>
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<tr>
<td>$\theta_e$</td>
<td>shoreline angle when sediment transport equals to zero</td>
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<tr>
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<td>angle between shoreline and x-axis</td>
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<tr>
<td>$\kappa$</td>
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<td>$\lambda$</td>
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<tr>
<td>$\rho_w$</td>
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<td>$\omega_o (\theta)$</td>
<td>mean frequency as a function of spectral direction</td>
<td>(1/sec)</td>
</tr>
</tbody>
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1.1 Introduction

Without human interference the shoreline is nearly in a dynamic equilibrium. Human activities can affect this equilibrium. For example harbours might be built to increase the income of a country and to enhance its economy. The breakwaters which are built for harbours, affect the stability of the shoreline. At the up-drift side of the breakwaters, accretion will occur while at the down-drift side of the breakwaters, erosion can be expected. These phenomena are induced by the interruption of the longshore sediment transport by the breakwaters. Another problem appears due to dams construction. Before construction of the river dams, the position of the river delta shoreline is in equilibrium between the sediment supplied by the river and the transport of sediment along the coast. After dams construction sediment yields from the river will cease. In the absence of sediment from river, currents, waves and winds are actively eroding the delta.

These kind of problems can be solved by coastal engineering. There are different ways to solve these problems such as sand nourishment, groins, revetments, seawalls, and offshore breakwaters. Offshore breakwaters have been constructed in several countries (South Africa, Italy, Spain, France, and Egypt). Both emerged and submerged offshore breakwater are built. The emerged offshore breakwater height is higher than the water depth. The submerged offshore breakwater height is less than 40% to 50% of the water depth.

The offshore breakwaters, also called detached breakwaters, are approximately parallel to the shore. Since the offshore breakwater blocks the incoming wave energy, a sheltered area is created in which the wave activity is diminished. However, some wave action is always observed due to diffraction, transmission (waves penetrate the structure due to the porosity, and some overtopping), and breaking over the structure. Due to the reduction of wave energy in the lee side of the breakwater the sediment transport capacity decreases and the sediment will deposit creating a salient or tombolo.
Waves coming from deep water will be affected by shoaling, refraction, reflection and breaking. Wave breaking causes wave set-up. At the sheltered area the wave set-up is smaller than at the exposed area. Due to this difference in wave set-up, currents and circulations occur in the vicinity of the breakwater tips (two dimensional flow). In front of the structure, wave breaking may be rather strong because of the slope of the structure. Undertow occurs wherever wave breaking exists. This current might disturb the sediment in front of the structure and generates scour holes (three dimensional flow). Scour holes are also generated at the tips of the structure due to the current and circulation.

Physical and mathematical models have been utilized to simulate the hydrodynamic and morphological effects of a single detached breakwater. Physical model results are reliable qualitative results but from economical point of view it is quite expensive. Therefore, 2D and 3D numerical models are developed for quantitative prediction of the shoreline response. 2D models are generally available while 3D-models are still in progress of development.

1.2 The study objectives
Nowadays, the development of mathematical models is still going on since they are cheaper and give faster results in comparison with physical models. Furthermore, they don't suffer from scale effects. However, these mathematical models should give authentic results.

DELFT2D-MOR is a two-dimensional horizontal model and a coupled model system which is developed by Delft Hydraulics. This model will be used in this study to predict the effect of a single detached breakwater on the shore. Focusing on the effect of a detached breakwater, the sensitivity analysis of varying the offshore distance of the breakwater will be made. From this sensitivity analysis, the effect of wave-current interaction on the wave pattern, the flow pattern and the sediment transport pattern will be studied. This effect will be compared to situations without wave-current interaction. As well, the effect of wave-current interaction on the morphological evolution at the equilibrium state will be studied. Also, the effect of the breakwater geometry on the morphological evolutions will be studied.

1.2 A single detached breakwater effect on the shore
The validity of the model will be checked by comparing the results from the 2D-model and literature results. After getting insight in the confidence of the model results, a field case situated in the Mediterranean sea (Egypt) will be studied. The simulations with one-wave and two-wave condition are compared. Morphological evolutions resulting from variable bottom roughness values in simulation with one-wave condition are studied as well as the hydrodynamic processes.

Another objective from this study is to verify a behaviour-oriented one-dimensional model (UNIBEST). This verification may be done by making a reasonable schematization of the problem and to find the dominant physical phenomena and parameters involved. Results from two-dimensional model will be used for this verification.

1.3 Report structure

The structure of this report is made for seven chapters including the introduction chapter. The results of the models are collected at the end of the report (Figures). Chapter 2 comprises the literature review about the effect of the single detached breakwater on the waves, the current and the morphology in the vicinity of the breakwater. It contains some empirical relationships of the incipient shore response, shoreline formation and sediment transport accumulation behind the structure. Moreover, it includes a briefly description for the mathematical models used for the shoreline changes.

The set-up of DELFT2D-MOR model is described, Chapter 3. Both of the mathematical description and the numerical procedure of the various modules are discussed. To check the validity of the model, it is made a sensitivity analysis to the influence of the breakwater geometry on the wave pattern, the flow pattern and the sediment transport pattern as well as the shoreline configuration. These are included in Chapter 4 which comprises the comparison between the literature results and the model results.

A case study situated in the Mediterranean sea of Egypt, Chapter 5, is calibrated and simulated by the 2D-model with an equivalent one-wave condition as well as two-wave
conditions. An appendix contains the wave conditions of this case at the seaward boundary. Chapter 6 comprises the one-line model simulations for the wave transformation, longshore current and sediment transport rates which are compared with the 2D-model results. Some cases also in this Chapter are chosen which are reached to the tombolo formation in the 2D-model to simulate them in the one-line model. Finally, Chapter 7 includes the conclusions and the recommendations.
CHAPTER 2
LITERATURE REVIEW

2.1 Introduction
One of the methods for coastal defence is the construction of a detached breakwater. Detached breakwaters locally reduce incident wave energy and alter wave direction to create a sheltered area. The sheltered area is acting as a sediment trap. In case of obliquely incident wave, the longshore sediment transport capacity in the sheltered area decreases due to the fact that the wave-induced velocities and turbulence are reduced. However, the diffracted waves may have sufficient energy to maintain sand in suspension and to transit behind the breakwater.

Sediment may be trapped temporarily and then removed when wave conditions change. This state can be a dynamic equilibrium state. If wave conditions are relatively constant, which rarely occur, a state of inert equilibrium may be attained.

The shoreline response depends on many parameters such as structure length, distance offshore, nearshore wave and tide conditions and sediment characteristics. Offshore breakwaters affected by waves as well as waves and currents have been studied. Earlier studies made clear that offshore breakwaters are quite effective in non-tidal situations. These breakwaters operate successfully in such countries which do not have a tide in their sea. This study will concentrate on the hydrodynamic and morphological effects of an offshore breakwater situated at the northern coast of Egypt, Mediterranean sea where only a very small tidal current is present.

In the following sections, the hydrodynamic, morphological effects of offshore breakwaters and mathematical models for shoreline changes will be discussed.
2.2 Hydrodynamic effects of offshore breakwaters

2.2.1 Wave diffraction

Once the waves meet an obstacle such as an offshore breakwater, the crest of the waves will bend around the tips of the breakwater and enter the lee side of the breakwater. This phenomenon is defined as wave diffraction. Wave height variation alongshore from the exposed area, which has a higher wave height, to the sheltered area, which has a lower wave height, can be expected. The wave height of the diffracted waves is determined by the length of the offshore breakwater and the wave length. From SPM (1984), the diffraction coefficient \( C_d \) can be determined.

\[
C_d = \frac{H_d}{H_i}
\]  

(2-1)

in which:

\( H_d \) = diffracted wave height (m)

\( H_i \) = wave height of the incident wave which is not disturbed by the obstacle (m)

2.2.2 Wave reflection

Vertical impermeable offshore breakwater will fully reflect the incident wave. The interaction between the reflected and incident wave leads to a standing wave. Propagation of reflected waves into the sheltered area may bring about increased peak orbital velocities which have an effect on sediment movement. Under oblique waves, the reflection may increase the longshore current and the local sediment transport.

Reflected wave energy is determined by a reflection coefficient which depends on the seaward slope of the structure, slope roughness and wave steepness. The reflection coefficient \( C_r \) is usually presented as a function of Iribarren number \( \xi_o \):

\[
\xi_o = \frac{\tan \alpha}{\sqrt{\frac{H_o}{L_o}}}
\]  

(2-2)
in which:
\[ \alpha = \text{structure seaward angle (°)} \]
\[ H_0 = \text{wave height at deep water (m)} \]
\[ L_0 = \text{wave length at deep water (m)} \]
\[ H_o/L_0 = \text{wave steepness (-)} \]

The general relationship of the reflection coefficient is written as:

\[ C_r = \frac{H_r}{H_i} \quad (2-3) \]

in which:
\[ H_r = \text{reflected wave height (m)} \]
\[ H_i = \text{incident wave height (m)} \]

2.2.3 Wave transmission
Permeable and low-crested structures produce wave transmission to the sheltered area. The transmitted waves increases the turbulence in the sheltered area. Consequently, the sediment located in the sheltered area is disturbed by the effect of orbital velocities induced by wave transmission. The parameters which affect the transmission coefficient are structure porosity, relative water depth (structure height relative to water depth) and wave steepness.

2.2.4 Wave set-up
Wave set-up is defined as the changes in the water level due to the variation of the radiation stress. Radiation stress can roughly be described as the wave-induced contribution to the mean horizontal transfer of horizontal momentum. The variation of the radiation stress is equivalent to a mean force exerted by the waves on the water column through which they propagate. In equilibrium conditions, normal to the shoreline, the net force is balanced by a horizontal hydrostatic pressure gradient. The following formula shows strictly the balance between the radiation stress gradient and the depth-integrated mean pressure gradient.
\[ \frac{dS_{xx}}{dx} + \rho g h \frac{dh}{dx} = 0 \]  
(2-4)

in which:
- \( S_{xx} \) = radiation stress component in the direction of wave propagation (N/m)
- \( \rho_w \) = mass density of water (kg/m³)
- \( g \) = acceleration of gravity (m/s²)
- \( h \) = water depth (m)
- \( \eta \) = water level variation (m)

This formula has been solved analytically with some approximations to derive the water level variation in the nearshore zone. The result gives:

\[ \eta_{\text{max}} = \frac{3}{8} \gamma^2 h_b \]  
(2-5)

in which:
- \( \eta_{\text{max}} \) = maximum wave set-up in the surf zone (m)
- \( h_b \) = water depth from still water level (m)
- \( \gamma \) = wave breaking index (0.5-1.25) (-)

Circulation cells can be expected at the tips of the structure due to water level differences from the exposed area to the sheltered area.

### 2.2.5 Wave-induced currents

Wave-induced current is generated due to the effect of the oblique breaking wave. The net force thrusts the water to move making the longshore current. It is mostly concentrated in the surf zone in which it is uniform alongshore direction and parallel to the shoreline. The offshore breakwater construction affects the longshore current distribution in the surf zone which is due to the wave energy reduction in the sheltered area. Another longshore current distribution can be generated in the sheltered area but with order of magnitude less than in the exposed area.
2.3 Morphological effects of offshore breakwaters

The main goal of offshore breakwaters is to dissipate wave energy and to create a sheltered area causing favourable sedimentation patterns. Due to the construction of an offshore breakwater, local changes in the wave and current patterns occur. Consequently, the littoral sediment transport is modified resulting in coastline changes. The resulting beach forms are known as salients and tombolos. Tombolos might become a serious obstacle to littoral drift. Beaches are exposed to normally and obliquely incident waves. Each of them has a direct effect on the morphology if that waves meet an obstacle.

For normally incident waves, as shown in the sketch below, there is no wave-driven current. Since an offshore breakwater generates a sheltered area, eddies are generated due to the water level gradient created by wave set-up difference. These eddies generated at the structure extremities may have a velocity larger than the critical velocity of the bottom material. Turbulence created by these eddies may disturb the sediment and force it to migrate into the sheltered area with less turbulence.

For obliquely incident waves, as shown in the sketch below, longshore sediment transport is generated by the interaction between wave-driven longshore current and wave action. At the lee side of the offshore breakwater, longshore sediment transport decreases due to the fact
that the stirring effect is reduced (orbital velocity at the bottom is relatively small behind the offshore breakwater). Sediment is trapped at the lee side of the structure coming from both up-stream and down-stream of the structure due to the eddies. Further down-stream of the structure a severe erosion of the nearshore beach will occur to compensate the required sediment transport capacity. Scour holes may also occur at the tip of the structure due to the eddies which have a velocity (including a turbulent velocity) higher than the critical velocity of the bottom materials.

![Diagram of wave action and erosion](image)

Based on a physical model of a detached breakwater subjected to normally incident waves, Mimura et al. (1983) observed that erosion occurred at the extremities of the breakwater and the adjacent beach, Fig.(2.1) and Fig. (2.2). The eroded sand from that locations was transported mainly by the current which developed shoreward of the breakwater. Mimura et al. measured the bottom levels after six hours of wave action, Fig. (2.1) and another six hours of wave action Fig. (2.2). In the first six hours a large amount of sand was transported to the lee side of the breakwater and the rate decreased to one-fifth in the second six hours. Mimura et al. concluded that the bottom topography was near the equilibrium condition after the second six hours wave action.

2.6 A single detached breakwater effect on the shore
Figure 2.1 Accretion and erosion after the first six hours wave (Mimura et al., 1983)

Figure 2.2 Accretion and erosion after the second six hours wave (Mimura et al., 1983)

A single detached breakwater effect on the shore
2.4 Empirical relationships

2.4.1 Incipient shore response

Offshore breakwaters are most effective, if there is a certain offshore distance relative to their lengths yielding incipient shore response. Nir (1982) emphasized that the ratio of the offshore breakwater length ($L_b$) to offshore distance from the original shoreline ($Y_b$) is the controlling factor for coast morphology at breakwaters. The higher the ratio ($Y_b/L_b$) the lower the amount of sand accumulated behind the structure. Accretion is very small or does not occur at all if the ratio ($Y_b/L_b$) is equal to or greater than two (Fig. 2.3). This critical value is substantially smaller than that given by Inman and Frautschy (1965) who proposed 3 to 6 based on their studies in California.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nir (1982)</td>
<td>$Y_b/L_b \geq 2$</td>
</tr>
<tr>
<td>Inman and Frautschy (1965)</td>
<td>$Y_b/L_b = 3 \text{ to } 6$</td>
</tr>
</tbody>
</table>

Table (2.1) : summary of the incipient shore response relationships

![Figure 2.3 Average thickness of sediment entrapped between original shoreline and an offshore breakwater as a function of $Y_b/L_b$ (Nir, 1982)](image)

2.8 A single detached breakwater effect on the shore
2.4.2 Shoreline formation

Dally and Pope (1986) suggested a three-phase design process. The first one is to do a desktop study employing various empirical relationships to identify design alternatives. The second one is to make either physical model or numerical model to evaluate alternatives. Finally, field tests may be performed to study the design aspects. In fact the third suggested phase by Dally and Pope is rather expensive. However, this can be done by measuring the wave properties and morphology changes at the existing breakwater instead of making a full scale model. Empirical relationships can be extended and improved if numerical or physical models are calibrated and verified.

Based on results of field studies, Nir (1982) found that most of the tombolos reached to approximately one half of their equilibrium state within a period of one or two years. These tombolos reached to their equilibrium state in about five years after the end of construction. The area of the stable tombolo covers between 40% to 75% of the rectangular area between the breakwater and the original shoreline depending on the $Y_B/L_B$ ratio. The sediment forming the tombolo came mostly from the nearby beaches which suffered severe erosion during the first three to four years after the construction. Nir analyzed field observations from different locations at the Mediterranean coast. Based on his results, the relation between the offshore distance to the breakwater length ratio and the average tombolo sand layer thickness can be represented as, (Fig. 2.3).

$$d_t = 1.80 - 0.80 \frac{Y_B}{L_B}$$ (2-6)

in which :

$d_t$ = average thickness of sediment entrapped layer (m).

Gourlay (1981) presented a relationship based on physical model results and field study results for tombolo formation. He reported that the tombolo formation can only occur if the structure is located inside the surf zone and :
From SPM (1984), the following relationship for tombolo formation is given as:

\[
\frac{L_B}{Y_B} \geq 0.67
\]  
(2-7)

Dally and Pope (1986) suggest two categories to control shoreline response, those used for assuring tombolo development and those used for developing salients. They reported that the development of tombolos results from the intersection of two diffracted wave crests which does not occur before the associated undisturbed portions of the waves reach the shoreline. The relationship for tombolo formation is:

\[
\frac{L_B}{Y_B} \geq 1.5 - 2.0
\]  
(2-9)

Bishop (1982) proposed for tombolo formation the following relationship:

\[
\frac{L_B}{Y_B} \geq 1.0
\]  
(2-10)

Dally and Pope (1986) paid their attention for finding a relationship of salient formation. The following relationship was derived for salient formation:

\[
\frac{L_B}{Y_B} \leq 0.5
\]  
(2-11)

From equations (2-9) and (2-11), it can be concluded that the ratio of \( L_B/Y_B \) between (0.50 to 1.50) gives the transition stage between a salient formation to a tombolo formation.

Rosen and Vajda (1982) have studied the equilibrium state of a salient formation (Fig. 2.4) and tombolo formation (Fig. 2.5) via a small scale moveable bed model. They first have studied dimensionless relationships between the relevant significant physical parameters which affect the salient and tombolo formation.

2.10 A single detached breakwater effect on the shore
Noda (1984) conducted a series of moveable-bed (0.2 mm sand) laboratory experiments evaluating physical parameters controlling the development of tombolos and salients, especially due to cross-shore transport. Noda used both storm and swell-type waves and concluded that maximum deposition occurs when:

$$0.5 \leq \frac{Y_B}{Y_b} \leq 1.0$$  \hspace{1cm} (2-12)

in which:

$$Y_b = \text{breaker line distance from the original shoreline (m)}$$

Well-developed tombolos were observed to form when:

$$\frac{Y_B}{Y_b} = 0.56$$  \hspace{1cm} (2-13)

Harris and Herbich (1986) found that the tombolos generally form when:

$$\frac{L_B}{Y_B} > 1.0$$  \hspace{1cm} (2-14)

While they found that salients form when:

$$\frac{L_B}{Y_B} < 1.0$$  \hspace{1cm} (2-15)

Figure 2.4 Equilibrium relationships for salient parameters (Rosen and Vajda, 1982)
All empirical relationships for shoreline formation mentioned above are summarized in Table 2.2.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Relationship</th>
<th>Beach response</th>
</tr>
</thead>
<tbody>
<tr>
<td>1- Gourlay (1981)</td>
<td>$L_B/Y_B \geq 0.67$</td>
<td>tombolo</td>
</tr>
<tr>
<td>2- Bishop (1982)</td>
<td>$L_B/Y_B \geq 1.0$</td>
<td>tombolo</td>
</tr>
<tr>
<td>3- Noda (1984)</td>
<td>$Y_B/Y_b = 0.56$</td>
<td>tombolo</td>
</tr>
<tr>
<td>4- SPM (1984)</td>
<td>$L_B/Y_B &gt; 2.0$</td>
<td>tombolo</td>
</tr>
<tr>
<td>5- Dally and Pope (1986)</td>
<td>$L_B/Y_B \geq 1.5 - 2.0$</td>
<td>tombolo</td>
</tr>
<tr>
<td></td>
<td>$L_B/Y_B \leq 0.50$</td>
<td>salient</td>
</tr>
<tr>
<td>6- Harris and Herbich (1986)</td>
<td>$L_B/Y_B &gt; 1.0$</td>
<td>tombolo</td>
</tr>
<tr>
<td></td>
<td>$L_B/Y_B &lt; 1.0$</td>
<td>salient</td>
</tr>
</tbody>
</table>

Table 2.2: summary of shoreline formation relationships

2.4.3 Sediment accumulation behind the structure
Sonu and Warwar (1987) found that dredging operations behind the offshore breakwater enhanced the trapping of the arriving littoral transport. They also found a relationship between the cumulative volume of a tombolo formation and time in Santa Monica, California, This relationship is as follows:

$2.12$ A single detached breakwater effect on the shore
\[ Q(t) = Q_o (1 - e^{-At}) \]  

in which:
- \( Q(t) \) = sediment volume in the tombolo at time \( t \) (m³)
- \( Q_o \) = total sediment volume at the final state of equilibrium (m³) = 1,600,000 m³ for Santa Monica breakwater
- \( t \) = number of years just after breakwater construction (year)
- \( A \) = an empirical coefficient (1/year) = 0.104/year for Santa Monica breakwater

They concluded that, without dredging operations, the Santa Monica breakwater would have reached 90% of the equilibrium volume in 22 years. The dredging events delay the development of the equilibrium state.

Nir (1982) has also assessed the quantities of sand accumulating in a growing tombolo from the following relationship:

\[ V = \frac{d \cdot c(2a + b)}{3} \]  

in which:
- \( 2a \) = length of the section along which tombolo is connected to structure (m)
- \( b \) = half of tombolo base measured along shoreline (m)
- \( c \) = distance of tombolo apex from shoreline (m)
- \( d \) = depth of water measured at tombolo apex (m)

Rosen and Vajda have also represented in Figure 2.5 the volume of trapped sand in terms of \( L_B/Y_B \).

Laboratory tests of Harris and Herbich (1986) provided the following relationship for the sediment volume \( Q_B \) moved into the sheltered volume of the breakwater, defined as \( Y_B L_B h_B \), as a function of relative breakwater length \( Y_B/L_B \) :

\[ \frac{Q_B}{Y_B L_B h_B} = e^{(0.320 - 1.920 \frac{Y_B}{L_B})} \]  

A single detached breakwater effect on the shore
in which $h_B$ stands for water depth on the seaward edge of the breakwater. This formula was found applicable for $Y/L_a$ between 0.5 and 2.5.

It can be inferred that the breakwater layout such as the breakwater length and the offshore distance has a major influence on the shoreline formation and even on the incipient shoreline response. The relationships related to the accumulated sand behind the breakwater have limited value and do not include the sediment properties. Moreover, these relationships are simplified and related to specific-site conditions.

2.5 Mathematical models for shoreline changes

As mentioned before, offshore breakwaters influence the wave, current and sediment transport rates. As a result of that influence, the morphology in the vicinity of the structure will change and reaches to a new equilibrium state. Shoreline changes have been modelled by numerical models. Numerical simulation methods for shoreline changes and beach topography have been developed specifically to predict the impacts of coastal structures on the neighbouring beach. The mathematical models which have been used for the shoreline changes are one-line, two-dimensional and quasi three-dimensional models. In the following sections, these models will be briefly discussed.

2.5.1 One-line model

The purpose of the one-line model is to simulate the essential features of shoreline change over a moderate to long-term period on either relatively local scale or large scale. Mimura et al. (1983) have made physical and mathematical models to simulate the influence of a single detached breakwater on waves, nearshore currents and bottom topography changes. They used the measured quantities to develop their one-line model.

The governing equation for the shoreline position $y$ is:

$$\frac{\partial y}{\partial t} + D \left( \frac{\partial Q_s}{\partial x} + q_b \right) = 0$$

(2-19)

in which:

- $x$ = longshore coordinate (m)
t = time (s)
D = depth of profile closure (layer thickness) (m)
Qs = longshore transport rate (m³/s)
qb = the volume rates of supply (+ve) or loss (-ve) in on- or offshore direction (m³/s/m)

The longshore sediment transport rate is taken as:

\[
Q_s = \frac{H_b^2 C_{gb}}{16(\rho_s - 1)(1 - \lambda)} (K_1 \sin 2\theta_{bs} - 2K_2 \frac{\partial H_b}{\partial x} \cot \beta \cos \theta_{bs})
\]

in which:
\[
C_{gb} = \text{wave group velocity at breaking (m/s)}
\]
\[
\rho_s = \text{sand density (kg/m³)}
\]
\[
\rho_w = \text{water density (kg/m³)}
\]
\[
\lambda = \text{sand porosity (-)}
\]
\[
\theta_{bs} = \text{the breaking wave angle to the shoreline (°)}
\]
\[
\tan \beta = \text{beach slope (-)}
\]
The formulae of \( C_{gb} \), \( \theta_{bs} \) and D are as follows:

\[
C_{gb} = \sqrt{\frac{H_b}{g}}
\]

\[
\theta_{bs} = \theta_b - \theta_s = \theta_b - \tan^{-1}\left(\frac{\delta y}{\delta x}\right)
\]
in which:
\[
\theta_b = \text{angle between breaking wave crest and x-axis (°)}
\]
\[
\theta_s = \text{angle between shoreline and x-axis (°)}
\]
The angles definition is demonstrated in the sketch below:

A single detached breakwater effect on the shore
The D-value is represented by an expression given by Hallermeier (1978) as follows:

$$D = 2.28H_o - 68.5\left(\frac{H_o^2}{gT^2}\right)$$  \hspace{1cm} (2-23)

in which:

- $T$ = wave period in deep water (s)

The sediment transport rate formula, Equation(2-20), has been found to be of great importance for describing the shoreline change near structure, where diffraction dominates. Also this equation includes the effects of two different driving forces on the longshore sand transport. The first term corresponds to the transport induced by obliquely incident wave. The second term corresponds to the transport induced by a longshore variation in breaking wave height which generates an extra longshore current due to wave set-up variation. The quantities $K_1$ and $K_2$ are treated as parameters in order to calibrate the model. It has been found that ratios of $K_2/K_1$ in the range of 0.5 to 1.5 give a proper balance between the two driving forces produced by obliquely incident waves and variation in breaking wave height (Kraus, 1983 and Mimura et al., 1983).
It can be concluded that the current induced by the variation of wave height contributes considerably to the longshore sand transport behind the detached breakwater. Mimura et al. have used the measured shoreline position (after seven hours) behind the detached breakwater as a reference for comparison with the calculated shoreline position for various quantities of $K_1$, $K_2$ and simulation time (Fig. 2.6). They found that the value ($K_2 = 0.5 K_1$) gives a realistic shoreline change.

![Figure 2.6 Shoreline position as measured from physical model and computed from mathematical model (Mimura et al., 1983)](image)

2.5.2 Two-dimensional model

In order to evaluate the effect of an offshore breakwater on the bathymetric variation, a two-dimensional horizontal model can be used. The cross-current transport mechanism generated by undertow is ignored in the two-dimensional model (Nicholson et al., 1996) and hence the bottom profile evolutions may be unrealistic. A hypothetical case of a single offshore breakwater placed on a plane sloping (1:50) beach profile, with a length of 300 m parallel to the shoreline and subjected to normally and obliquely incident waves ($H_{rms} = 2.00$ m) has been tested by Bos et al. (1996) using Delft Hydraulics morphological models DELFT2D-MOR and DELFT3D (three-dimensional results will be mentioned in the following section). The Bijker's formula was used in the 2D-computations. For normally incident waves, the equilibrium state was reached after 50 day while the steady state for obliquely incident waves
was reached after 40 day. Bos found two circulation cells in the lee of the breakwater due to the set-up difference induced by alongshore gradient in the wave height. The order of magnitude of the circulation velocities was 0.90 m/s at the breakwater extremities. These flows are able to transport sediment toward the lee of the breakwater (Figure 2.7).

Figure 2.7 Initial sediment transport pattern (left) and sediment transport pattern after 200 hours (right) (Bos et al., 1996).

For obliquely incident waves, a longshore current is generated with order of magnitude of 0.50 m/s. Two circulation cells in the lee of the breakwater are generated but at the upstream side this cell is very much smaller and weaker compared to the down-stream cell. At the up-stream side, sediment is trapped into the lee of the breakwater by the longshore current. At the down-stream side, erosion occurs due to the longshore current combined with the high velocity of the circulation cell (Figure 2.8).
Figure 2.8 Initial flow pattern (left), initial sediment transport pattern (middle) and sediment transport pattern after 40 day (right), (Bos et al., 1996).

Bos has estimated the accumulated sand in the lee of the breakwater in case of both normally and obliquely incident waves over 50 day (Figure 2.9).

Figure 2.9 Deposition behind the breakwater and duration relationship (Bos et al., 1996).
2.5.3 Quasi three-dimensional model

Quasi three-dimensional models have been recently used because they give more sophisticated results. The action of cross-shore currents is taken into account in the three-dimensional computations for the prediction of detailed bathymetry variations. Also the undertow due to the effect of the breaking waves is included. In the Q3D computations of Bos, the cross-shore sediment transport was dominating in both cases of normally and obliquely incident waves (Figure 2.10). Very irregular contour lines occurred in the resulting bathymetry which affected the progress of the computation.

An intercomparison of the results produced by five coastal area morphodynamic models has been done by Nicholson et al. (1996) in which DELFT2D-MOR was one of the five models used. Comparing these results with the laboratory and field data, he found that all models predicted the formation of a double salient in the lee of the breakwater which advanced steadily towards each tip of the structure. From this intercomparison it was concluded that the area models are able to reproduce the major morphological features associated with offshore breakwaters. Furthermore, it was concluded that the choice of sediment transport formula had a pronounced influence on the resulting morphology.

Figure 2.10 Initial sediment transport pattern for normally (left) and obliquely (right) incident waves, (Bos et al., 1996).
Without doubt the one-line model has several advantages. It is economic and saves computation time which are very important nowadays. However, that model needs to be calibrated and verified well to reach to a specific accuracy. The calibration and verification can be done by field data, physical models or mathematical models.
CHAPTER 3
SET-UP OF DELFT2D-MOR MODEL

3.1 Introduction
The variation of the seabed is due to sediment transport variation which in turn depends on waves and currents. Delft Hydraulics numerical model DELFT2D-MOR is a coupled model system which stands for DELFT2D morphological modelling system. This model is a two-dimensional horizontal model. The modelling system consists of a set of modules which are used to compute the bottom changes. These modules are the wave module (computes wave field parameters for a given flow field and a given bed level), the flow module (computes the flow field development for a given bed level and wave forces), the transport module (computes the sediment transport for a given flow and wave field development) and the bottom module (computes the bed level variation for a given sediment transport gradients).

Each module has output data which are used as input data by the other modules. For example, data generated by the wave module will be used as input data in the flow module. Such a coupled system needs a communication channel between the modules. The data computed by one module which are of interest for one of the other modules will be written to a central communication file which has a nefis structure (neutral file system). This communication is executed by the main module.

In the following sections, the main module, the wave module, the flow module, the transport module and the bottom module are discussed.

3.2 The main module
The main module controls the execution of the process simulation such as process starting, splitting up of the total simulation time into time intervals, splitting up of the process into subprocesses, activating and deactivating the process modules and process stopping. The two splitting up procedures with respect to time and process are described by a process tree to be supplied by the user.
3.2.1 The process tree

The process tree, as shown below, consists of a set of nodes and branches. A branch connects two nodes, one of which will be the parent node and one the child node. A node can be a parent and child at the same time (e.g. node 4). A node without a parent node will be the root node (node 5). The process starts at the root node, by activating the subprocess connected to the root node (node 4). This subprocess starts its controlled subprocesses, starting at the subprocess with the lowest number (node 1). If this one stopped, the next child subprocess (node 2) will be activated, and so on till all child subprocesses (node 1, 2 and 3) are stopped. Each process can be executed more than one time.

![Process Tree Diagram]

Example of a specified-process tree in the main module

Each branch of the process tree corresponds to a controller which controls the execution of the process to be associated with the child node of that branch. Each controller has a control criterion which must be specified. The control criterion might be a number of execution, a specific accuracy or simulation time of the process itself.

An elementary subprocess represents the execution of a set of modules (waves, currents, sediment transport and bed level variation).

In the scheme below, the procedure for the morphological computation is illustrated. Two
main alternatives (A and B) for the computation of the bed level changes can be found. The first one (A) is implemented as an initial computation of waves and current on the initial bottom if there is no information available of the waves and the flow fields. The wave and current computations are implemented under quasi-stationarity, meaning that the bottom does not change during the wave and current computation. The wave-induced forces resulting from the wave computation, are used by the flow module for the current computation. It is possible to use these results for another wave and current computation if wave-current interaction has to be taken into account (loop C). The sediment transport is computed under the same condition of quasi-stationarity is implemented. The computed sediment transport rates are used to compute the bottom changes which are superimposed on the original bathymetry (loop E) in order to get a new bottom.

If wave and current information are available, loop B can be followed for the flow field computation. Here it is assumed that for small changes in the bottom level the wave height and flow patterns remain constant. This results in a simple computation of the new velocities by dividing the specific flow rate of each location by the new depth values ($V_{\text{new}} \cdot h_{\text{new}} = V_{\text{old}} \cdot h_{\text{old}}$). By application of this "continuity correction" the computation time reduces considerably. Knowing the new flow field, the transport rates and bottom changes are computed.
3.3 The wave module

The wave module which is implemented in the morphological system DELFT2D-MOR is the HISWA model. HISWA stands for HIindcast Shallow water WAVes. This model is a prediction model for stationary, short-crested waves in shallow water (Holthuijsen et al., 1989). The wave module is the process module which simulates the propagation of waves and predicts the distribution of wave parameters and current driving terms of directionally spread waves over a two-dimensional bathymetry.

3.3.1 Physical background

The wave propagation in HISWA is determined on a grid (based on Eulerian approach) instead of along wave rays as is the approach in conventional techniques of linear wave theory. It was found that the wave ray approach is numerically inefficient when the propagation of nonlinear wave and dissipation has to be taken into account. The reason for this is in the latter case the effects of wave propagation of different spectral components need to be integrated in the spectral domain. In the Eulerian approach of wave propagation all wave information is available at the mesh-points of a regular grid.

The wave computation in HISWA is based on the wave action balance. The wave action depends on wave energy density and the relative frequency of waves and currents. The wave energy (E) is a function of the spatial coordinate (x, y) and the spectral direction (θ).

The wave action is defined, as :

\[ A(\omega, \theta; x, y, t) = \frac{E(\omega, \theta; x, y, t)}{\sigma} \]  \hspace{1cm} (3-1)

\[ \sigma = \omega - K U_c \]  \hspace{1cm} (3-2)

in which :

- \( A \) = wave action (J.sec/m²)
- \( \omega \) = wave frequency (1/sec)
- \( E \) = wave energy density (J/m²)
\( \theta \) = spectral direction (\(^\circ\))

\( \sigma \) = the relative frequency of wave and current (1/sec)

\( x, y \) = coordinate in cross-shore and longshore direction respectively (m)

\( t \) = time (sec)

\( K \) = wave number vector (1/m)

\( \mathbf{U}_c \) = current velocity vector (m/s)

The action balance equation, which replaces the conventional wave energy balance equation in the adopted Eulerian approach, is written as:

\[
\frac{\partial A}{\partial t} + \frac{\partial (c_x A)}{\partial x} + \frac{\partial (c_y A)}{\partial y} + \frac{\partial (c_\theta A)}{\partial \theta} + \frac{\partial (c_\omega A)}{\partial \omega} = T \tag{3-3}
\]

The local rate of change of action density is represented by the first term on the left hand side of equation (3-3). The other terms on the left hand side represent the net transport of action in the x-, y-, \( \theta \)-, and \( \omega \)-domain respectively. The total effect of generation and dissipation of action is represented by the action source \( T \).

Two simplifications have been done to solve equation (3-3). The first one is based on the fact that in coastal situations the travel time of waves through the area of interest is often small compared to the time scale of the local wind field. The situation can then be treated as stationary. This simplifies the wave module considerably since it permits the removal of time as an independent variable. The second one is to parameterize the balance equation of wave action. Such parameterization can be applied either to the source function \( T \) alone or to the spectral balance equation as a whole. The source function parameterization is sufficient for ocean wave models. However, it is not sufficient in the HISWA module. Therefore, the parameterization of the complete action balance has been implemented. Two directional wave functions are defined: the directional action spectrum \( A_0(\theta) \) and a mean wave frequency as a function of spectral direction \( \omega_0(\theta) \). Zero- and first-order moments of the action spectrum in frequency domain have been chosen for the parameterization as follows:

\[
A_0(\theta) = m_0(\theta) \tag{3-4}
\]
\[ \omega_0(\theta) = \frac{m_1(\theta)}{m_0(\theta)} \]  

in which:

- \( A_0(\theta) \) = one-dimensional directional action spectrum (J/m²)
- \( \omega_0(\theta) \) = mean frequency as a function of spectral direction (1/sec)
- \( m_0(\theta) \) and \( m_1(\theta) \) = zero- and first-order moments of the action density spectrum

The moments \( m_n \) of the action density spectrum are defined as:

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

Note that \( A_0(\theta) \) is a directional spectrum in the sense that it presents the directional distribution of frequency-integrated wave action density.

The computations are carried out for each wave component separately on the basis of two evolution equations:

- For the zero-order moment of the action density spectrum for each spectral direction:

\[ \frac{\partial(c_{0x}^* m_0)}{\partial x} + \frac{\partial(c_{0y}^* m_0)}{\partial y} + \frac{\partial(c_{0\theta}^* m_0)}{\partial \theta} = T_0 \]  

- For the first-order moment of the action density spectrum for each spectral direction:

\[ \frac{\partial(c_{0x}^{**} m_1)}{\partial x} + \frac{\partial(c_{0y}^{**} m_1)}{\partial y} + \frac{\partial(c_{0\theta}^{**} m_1)}{\partial \theta} = T_1 \]

in which \( c_{0x}^* \), \( c_{0y}^* \) and \( c_{0\theta}^* \) in equation (3-7) and \( c_{0x}^{**} \), \( c_{0y}^{**} \) and \( c_{0\theta}^{**} \) in equation (3-8) are the propagation speeds through \((x, y, \theta)\)-space of \( m_0 \) and \( m_1 \) respectively. The propagation speeds \( c_x \) and \( c_y \), in both equations, represent rectilinear propagation including shoaling, whereas \( c_{\theta} \), in both equations, represents refraction. Generation and dissipation of \( m_0 \) and \( m_1 \) are represented by the source functions \( T_0 \) and \( T_1 \) respectively, representing the effects of wave generation and dissipation.
The physical effects which are modelled in HISWA are:
- bottom and current refraction
- directional spreading
- dissipation due to bottom friction
- dissipation due to wave breaking
- wave blocking due to currents
- wave generation by wind

### 3.3.2 Numerical procedure

The independent variables of the two evolution equations, partial differential equations of the first order are the two horizontal coordinates, $x$ and $y$, and the spectral wave direction $\theta$. The dependent variables are the action density $A(x,y,\theta)$ and the mean frequency $f_m(x,y,\theta)$ for each spectral wave direction. The computation is implemented in a direction, $x$-axis, roughly parallel to the main wave propagation direction. If the boundary condition at the up-wave boundary is known, the dependent variables can be computed at each point $(x,y,\theta)$. The computation starts at the up-wave boundary, $x = 0$, and proceeds in positive $x$-direction. After the state variables in all points on a line in $y$-direction have been determined, the computation proceeds with the next line in the grid. The computation is only stable if the following stability condition is satisfied:

$$\frac{|c_x \Delta x|}{|c_y \Delta y|} \leq 1.0$$  \hspace{1cm} (3-9)

In cases without current this is equivalent with:

$$\frac{\Delta y}{\Delta x} \geq \tan \theta$$ \hspace{1cm} (3-10)

in which:
- $\theta$ = half of the directional energy distribution sector ($^\circ$)
- $c_x$ = group velocity of the mean frequency in $x$-direction (m/s)
- $c_y$ = group velocity of the mean frequency in $y$-direction (m/s)
\[ \Delta x = \text{grid size in x-direction (m)} \]
\[ \Delta y = \text{grid size in y-direction (m)} \]

**Grid definition**

HISWA operates with different grids, each may have a different origin, orientation and grid size. Input, computational and output grids can be chosen.

**Input grid**

The water depth, flow field, friction coefficient and wind field (if present) have to be provided on the input grid. In the region outside the input grid HISWA assumes that the bottom level and friction coefficient are identical to those at the nearest boundary of the input grid. Current and wind velocity outside the input grid are neglected.

**Computational grid**

The size of the computational grid must obey the stability condition, equation (3-9). The orientation of the x-axis of this grid must be more or less equal to the main wave direction in the region. The up-wave boundary should be chosen such that refraction effects have not yet influenced the wave field. If open boundaries are used at the lateral sides, the computational grid must be larger than the area where the wave parameters are to be computed. The width in y-direction must be larger than that of the area of interest, because along each lateral side of the grid a region exist where the wave field is disturbed by an import of zero energy from the lateral boundaries. The length in x-direction is given by the distance between the up-wave boundary and the most down-wave point of interest.

**Output grid**

The information on the output grid are obtained from the computational grid by spatial interpolation. In the morphological system, the output grid of the HISWA module (flow module grid) with the interpolated wave parameters will be used in the flow module computations as input grid.
3.3.3 Fundamental limitations of the wave module

The fundamental limitations of HISWA are consequences of the assumptions and simplifications in the mathematical model and the method of computation. These limitations are the applied spectrum which is always considered to be integrated over the frequencies, the basic formulations do not include time variations and that the driving forces are assumed to be stationary. Furthermore, diffraction is not taken into account.

The diffraction phenomenon is very important in this study. The diffracted waves have an effect on the flow field around the extremities of the breakwater which in turn affects the sediment transport. Moreover, the diffracted waves have a sufficient energy to maintain sand in suspension and transit it to the sheltered area.

3.4 The flow module

The flow module which is used in the morphological system, is TRISULA. It allows a 2D horizontal or 3D flow field computation, and takes into account wave-effects and density effects by salinity and non-uniform temperature distribution. A fixed bed level will be used. Within DELFT2D-MOR the 2D-version of the flow model will be used. The flow module is the physical subprocess which simulates the unsteady flow and water level variation from the tide, the wave or meteorological forcing. In this module an overall flow field over a given area is computed.

3.4.1 Physical background

The unsteady shallow water equations, depth-averaged, are solved in two dimensions in the 2D-version of the flow module. The system of equations are the horizontal \((x,y)\) momentum equations and the continuity equation. These equations result from the depth averaging of the complete three-dimensional Reynolds’ equations representing conservation of momentum and conservation of mass. The basic equations for homogeneous, constant fluid density, flow in shallow water are:
- the continuity equation:

\[
\frac{\partial h}{\partial t} + \frac{\partial (hu)}{\partial x} + \frac{\partial (hv)}{\partial y} = 0
\]  
(3-11)

- The momentum equation in x-direction:

\[
\frac{\partial (hu)}{\partial t} + \frac{\partial (hu^2)}{\partial x} + \frac{\partial (hv)}{\partial y} + gh \frac{\partial (h+z_b)}{\partial x} + k_x h \left( \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right) - \frac{\tau_{b,x}}{\rho_w} - \sum \frac{F_x}{\rho_w} = 0
\]  
(3-12)

- the momentum equation in y-direction:

\[
\frac{\partial (hv)}{\partial t} + \frac{\partial (hv^2)}{\partial x} + \frac{\partial (hv)}{\partial y} + gh \frac{\partial (h+z_b)}{\partial y} + k_y h \left( \frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right) - \frac{\tau_{b,y}}{\rho_w} - \sum \frac{F_y}{\rho_w} = 0
\]  
(3-13)

in which:

\( u, v \) = depth-averaged water flow velocities in x, y directions (m/s)

\( h \) = local water depth (m)

\( t \) = time (sec)

\( \rho_w \) = water density (kg/m\(^3\))

\( \tau_b \) = bed-shear stress (N/m\(^2\))

\( F_x, F_y \) = external pressure (wind, waves, Coriolis) (N/m\(^2\))

\( z_b \) = bed level above reference (datum) (m)

\( k_x, k_y \) = effective dispersion coefficients representing the integration effects (m)

The bed-shear stresses usually are related to the depth-averaged velocities, as follows:

3.10 A single detached breakwater effect on the shore
in which:

\[ V = \sqrt{u^2 + v^2} = \text{the magnitude of the resultant velocity (m/s)} \]

\[ C = \text{Chezy-coefficient (m}^{1/2}/\text{s)} \]

The vertical momentum equation is reduced to the hydrostatic pressure relation. Vertical accelerations are assumed to be small compared with the gravitational acceleration and are not taken into account.

The main physical phenomena which are included in the TRISULA are:

- tidal forcing
- the effect of the earth’s rotation (Coriolis force)
- wind-shear stress on the water surface
- bed-shear stress on the bottom
- influence of waves on the bed shear stress (2D only)
- wave-induced stresses and mass fluxes (2D only)

### 3.4.2 Numerical procedure

The accuracy of the solution depends not only on the numerical scheme which is used, but also on the way in which the bottom topography, the geographical area and the physical processes are modelled. The numerical scheme that is applied is the Alternating Direction Implicit (ADI) technique, Stelling (1984). The water levels are calculated implicitly along grid lines in each direction in an alternating way. This means that water levels and velocities in the x-direction are implicitly solved in the first half time step, while water levels and velocities in the y-direction are implicitly solved in the second half time step. The horizontal velocities are computed by substituting the calculated water levels in the momentum equations.
Grid definition
A staggered grid is used in the TRISULA model. The water level points are defined in the centre of a cell, whereas the velocity components are defined on the faces of this cell. In this way the grids for the water levels and velocities are staggered.

3.5 The transport module
The main function of the transport module is to determine the sediment transport components for a two dimensional horizontal area at a set of times, using the time dependent flow, wave fields and a fixed bed level available on the communication file. The sediment transport computations are computed on a computational grid, being the grid defined by the flow module. The magnitude of the sediment transport will be computed by use of a selected sediment transport relation.

Because of the fact that sediment transport formulae all have a limited validity, several options in the transport module for sediment transport formulae are available: Ackers-White, Van Rijn, Meyer-Peter-Muller, Englund-Hansen, Bijker and Bailard. Sediment transport in this study is mainly dependent on both currents and waves. The combination of waves and currents can give a high rate of sediment transport. It should be clearly kept in mind that the sediment transport occurs if the velocity is higher than the critical velocity of the bed material. Waves and currents both have positive effects on suspension and transport respectively. In the Bijker formula, waves and current have been included. Therefore, Bijker’s formula will be discussed in the following sections.

3.5.1 Bijker’s formula
Bijker (1971) developed a transport formula for a combination of currents and waves by modifying a transport formula for currents only. The approach of Bijker was to introduce the wave influence via a modification of the bottom shear stress in an existing sediment transport formula for currents. He used the Kalinske-Frijlink formula for bed load transport and coupled this to the Einstein formula for suspended sediment transport. The time-averaged instantaneous velocity due to wave is small relative to the longshore current velocity. However, the waves contribute primarily to the stirring up of sediment from the bottom to
Bottom shear stress due to current and waves

Bottom shear stress depends on the velocity. Wave-current shear stress is related to the resultant velocity from orbital velocity and current velocity. That is what Bijker did to modify the shear stress. At a specific height above the bottom level, Bijker calculated separately, as vectors, the velocities of the wave and current to obtain the resultant velocity, $U_{cw}$.

\[
U_{cw} = U_c + U_w
\]  

(3-16)

in which:
- $U_{cw}$ = wave-current velocity vector (m/s)
- $U_c$ = current velocity vector (m/s)
- $U_w$ = orbital velocity vector (m/s)

The shear stress for combined waves and a current is denoted by:

\[
\tau_{cw} = \rho_w \kappa^2 U_{cw}^2
\]  

(3-17)

in which:
- $\rho_w$ = water density (kg/m$^3$)
- $\kappa$ = Von Karman constant (-)

The direction of this bottom shear stress changes in time because it depends on the orbital velocity direction. Stirring up of the bed material does not depend on the direction of the shear stress. As mentioned before, to stir up sediment only the critical velocity has to be exceeded irrespective the direction of the shear stress. For the stirring up of material the time-averaged value of the total bottom shear stress is used:

\[
\bar{\tau}_{cw} = \tau_c \left[ 1 + \frac{1}{2} \left( \frac{U_o}{U_c} \right)^2 \right]
\]  

(3-18)

in which:
\[ \tau_c = \rho_w g \left( \frac{U_c}{C} \right)^2 \]  
\[ \xi = C \sqrt{\frac{f_w}{2g}} \]

- \( U_c \): the depth-averaged current-velocity (m/s)
- \( C \): Chezy Coefficient (m\(^{1/2}\)/s)
- \( \hat{u}_o \): the maximum orbital velocity (m/s)
- \( f_w \): wave friction factor (-)
- \( g \): gravitational acceleration (m/s\(^2\))

The average total bottom shear stress can also be found from the following relation:

\[ \overline{\tau_{cw}} = \tau_c + \frac{1}{2} \tau_w \]  
\[ \tau_w = \frac{1}{2} \rho_w f_w \hat{u}_o^2 \]

in which:

- \( \tau_w \): maximum shear stress due to waves (N/m\(^2\))

**Bijker's bed load transport formula**

Bijker assumed that the bottom transport occurred in a layer with a thickness equal to \( r \). In case that the bottom roughness is unknown, Bijker suggested that a roughness equal to half of the height of the ripples on the bottom could be taken. Bijker replaced the shear stress due to current by the time-averaged shear stress due to both waves and currents in Kalinske-Frijlink formula to reach to the following formula:
Set-up of DELFT2D-MOR model

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\[ S_b = \frac{b d_{50} U_c \sqrt{g}}{C} \exp \left( \frac{-0.27 \Delta d_{50} C^2}{\mu U_c^2 \left[ 1 + \frac{1}{2} \left( \frac{\xi a_o}{U_c} \right)^2 \right]} \right) \]  

(3-23)

in which:

- \( b \) = constant depends on wave height and water depth (-)
- \( d_{50} \) = median grain size (m)
- \( \Delta \) = relative sediment density \([\rho_s - \rho_w] / \rho_w \) (-)
- \( \mu \) = ripple factor = \( (C/C_{90})^{3/2} \) (-)
- \( C \) = 18 log (12 h/r) (m\(^{1/2}\)/s)
- \( C_{90} \) = 18 log (12 h/d\(_{90}\)) (m\(^{1/2}\)/s)

**Bijker suspended load transport formula**

Bijker modified the suspended load transport formula of Einstein by changing the shear stress due to a current into the time-averaged shear stress due to current and waves.

The concentration distribution is given by:

\[ c(z) = c_{a} \left( \frac{r}{(h-r)} \right) \left( \frac{h-z}{z} \right)^{z_*} \]  

(3-24)

in which \( r \) is the bottom layer thickness, \( h \) is the water depth, \( z_* \) is the exponent of the concentration distribution with the modified bed-shear velocity and \( c_{a} \) is the bed load concentration.

Bijker assumed that the concentration in the bottom layer, \( c_{a} \), is constant over the entire thickness \( r \) and given by:

\[ c_{a} = \frac{S_b}{r} \int_{0}^{u(z)} dz \]  

(3-25)

\( u(z) \) has been taken as the Prandtl-Von Karman logarithmic velocity distribution.

The exponential part \( z_* \) is defined by:

---

A single detached breakwater effect on the shore 3.15
in which $w_s$ is the fall velocity of the sediment while the shear stress velocity can be computed from:

$$U_{cw} = \sqrt{\frac{\tau_{cw}}{\rho}} = \sqrt{\frac{\tau_c}{\rho} \left[ 1 + \frac{1}{2} \left( \frac{\bar{u}_o}{U_c} \right) \right]} \quad (3-27)$$

The suspended load transport follows from:

$$S_s = \int_r^h c(z) u(z) dz \quad (3-28)$$

The equation of $S_s$ has been solved numerically and after using the total Einstein integral term $Q$ (van der Velden, 1989), it can be shown that:

$$S_s = 1.83 \ Q \ S_b \quad (3-29)$$

Once both the bed load transport, $S_b$, and the suspended load transport, $S_s$, are known the total transport, $S_{tot}$, follows by adding $S_s$ and $S_b$

$$S_{tot} = S_b + S_s = S_b (1 + 1.83 Q) \quad (3-30)$$

The magnitude of the sediment transport computed by the specified sediment transport relation in the transport module can be corrected for different effects which are not included in the transport formulae themself:

- Bed level gradient effect.
- Non-erodible layer effect.
- Numerical stability.

More details on this subject can be found in the DELFT2D-MOR manual.
3.6 The bottom module

3.6.1 Physical background

After computation of the corrected sediment transport components \((S_x, S_y)\), the bottom module will use them to compute the bed level variation. In the bottom module, the continuity equation of sediment is solved which follows:

\[
(1 - \lambda) \frac{\partial z_b}{\partial t} + \frac{\partial S_x}{\partial x} + \frac{\partial S_y}{\partial y} = 0
\]  \hspace{1cm} (3-31)

in which:

- \(z_b\) = the bed level (m)
- \(\lambda\) = sediment porosity (-)

3.6.2 Numerical aspects

The continuity equation is solved by use of a finite difference of Forward Time Central Space (FTCS) scheme. The execution of the bottom module is done for one time step for one call of the bottom module. The time step is based on stability and accuracy criteria. The Courant number should not exceed one to maintain the stability of the computation. The time step can be computed from the stability criterium in which the time step varies during the simulation run. The optimal time step can be computed from the following relation:

\[
\Delta t = \frac{C_{\text{max}} \Delta l}{c_b}
\]  \hspace{1cm} (3-32)

in which:

- \(C_{\text{max}}\) = the Courant number \(\leq 1.00\)
- \(c_b\) = the bed level propagation speed (bed level celerity) (m/s)
- \(\Delta t\) = the optimal time step (sec)
- \(\Delta l\) = grid increment measured in the transport direction (m)

The bed level propagation speed can be determined from the following relation:

\[c_b = \frac{\beta S}{(1 - \lambda) h}\]  \hspace{1cm} (3-33)
in which:
\[ \beta = \text{power of the used transport formula (\)} \]
\[ S = \text{sediment transport (m}^3\text{s/m)} \]

For the sediment transport \( S \), it is assumed a simple sediment transport formula as follows:

\[ S = \alpha U^\beta \tag{3-34} \]

\( \alpha \) and \( \beta \) are constant and depend on the used sediment transport formula and \( U \) is the velocity in the flow direction.

The computational area to be used by the bottom module is equal to the computational area defined by the flow module. The boundaries are equal the flow module boundaries too.
CHAPTER 4
SENSITIVITY ANALYSIS

4.1 Introduction
There are many factors which affect the deposition of sand behind an offshore breakwater. These factors are related to the geometry of the structure, the sediment properties and are related to the wave and current conditions. From the literature it can be derived that the main factor which determines the shoreline response is the structure layout such as breakwater length and its offshore distance. Therefore, the shoreline response related to the variation of the structure layout has been studied. The main aim of this analysis is to compare the results from DELFT2D-MOR and the results from the literature to have an idea about the validity of the model.

One of the fundamental limitations of the wave module (HISWA) used in DELFT2D-MOR is that diffraction is not taken into account. Therefore, a diffraction computation is made using a model which includes the diffraction phenomenon in its formulations. The results from this model are compared with the HISWA results to confirm the applicability of the HISWA module in this study in which the diffraction phenomenon is dominant.

4.2 Diffraction computation
Because the wave diffraction is not included in the wave module, a diffraction computation is carried out using Delft Hydraulics model DIFFRAC which mainly takes into account the diffraction phenomenon.

DIFFRAC model
DIFFRAC was developed by Delft Hydraulics to describe the wave behaviour in and around structures in water of nearly uniform depth. The phenomena accounted for are diffraction and reflection. Partial reflection is modelled at reflecting edges of the schematised basins according to defined reflection coefficients input. Directional spreading can be simulated by this model but refraction is not included. The wave penetration computations are based on the phenomenon of diffraction, which is the three-dimensional effect resulting from the
interruption of a wave train by an obstacle. The mathematical model for the computation of wave penetration in areas with an arbitrary shape is based on the linear theory for harmonic water waves.

The simplifications made in the mathematical formulation are: the fluid is ideal, irrotational flow, no energy dissipation is present in the area of wave propagation (no wave breaking and no bottom friction), linear formulation (small wave steepness), simple harmonic wave (regular wave) and water depth in the area of interest is constant.

**HISWA model**

Although the diffraction phenomenon is not included in HISWA, the directional spreading is taken into account to compensate for the lack of diffraction.

**Test case**

A hypothetical case with a single detached breakwater and a horizontal bed will be applied to both DIFFRAC and HISWA model. The input data are:

- Water depth = 4.6 m
- Wave height = 1.0 m
- Wave period = 8.0 sec
- Directional energy distribution sector = 120°
- Offshore distance (Y_B) = 220 m
- Breakwater length (L_B) = 300 m

**Results discussion**

Figure (4.1) illustrates the schematization which is used in the DIFFRAC computation. The arrows in this figure refer to the directional energy distribution. Focusing on the isolines of the wave height in Fig. (4.2), DIFFRAC result, and Fig. (4.3), HISWA result, it can be concluded that the isolines have the same pattern in both the DIFFRAC and HISWA computations. Furthermore, the region influenced by the breakwater, either at up-stream and down-stream side of the breakwater, has a specific distance which is more or less equal in both computations. The longshore wave height variation at a distance close to the breakwater (0.30 Y_B) and at a greater distance from the breakwater (0.50 Y_B), is presented in Fig. (4.4)
for both the HISWA and DIFFRAC computation. In the exposed area the wave height is relatively equal in both models, Fig.(4.4). In the sheltered area part of the wave height results are more or less identical except for the location at the middle of the breakwater length. This is due to the dead zone, the area which has zero wave height. This zone is larger in HISWA than in DIFFRAC and the wave height computed by HISWA is smaller than the wave height computed by DIFFRAC. This is due to the assumptions of HISWA which have been mentioned before (Chapter 3). At a greater distance from the breakwater (0.50 \( Y_B \), Fig.(4.4), the wave height results from HISWA are more or less identical to the computed wave height by DIFFRAC.

**Conclusion**

It can be concluded that the difference in the wave height between the DIFFRAC and HISWA computations is small and decreases for locations further from the breakwater. The wave height behind the breakwater, either in the DIFFRAC computation or the HISWA computation, is relatively small. From morphological point of view the differences in wave height behind the breakwater from the HISWA computation are expected to have only little influence on the morphological computation.

**4.3 Influence of the breakwater geometry on the shoreline configuration**

Sensitivity analysis with DELFT2D-MOR has been made. The aim of this analysis is to investigate the influence of the breakwater layout on the hydrodynamic processes (wave pattern, flow pattern and the sediment transport pattern) as well as the morphological evolutions. The effect of the wave-current interaction has been studied in this analysis as well. To get insight in the validity of the DELFT2D-MOR, the comparison between the model results and the literature results has been done.

For this analysis a hypothetical case with a single detached breakwater is applied. The input data are held constant during the simulation but the offshore distance of the breakwater varies for each case. The choice of the hypothetical case data are based on the case which has been used by Bos (1996). That is to facilitate the comparison between the results of the simulations.
and his results.

A single detached breakwater is placed on a plane sloping (1:50) beach profile. The breakwater has a constant length of 300 m and is subjected to the action of normally incident wave. The area is 2100 m x 790 m as shown in the sketch below. The offshore distance of the breakwater has been varied and its effect on the shoreline configuration and deposition patterns will be studied. The idea of changing the offshore distance is to investigate the location of the breakwater relative to the breaker line. The input data of the breakwater layout, main module, wave module, flow module, transport module and bottom module are as follows:

![Diagram of the breakwater layout](image)

General layout of the hypothetical case.

### 4.3.1 Breakwater layout

Five simulation runs are done until the equilibrium state is reached based on the following data:

- **Beach slope**: \(= 1:50\)
- **Wave conditions**:
  - **Hs**: 2.83 m
  - **Tp**: 8.0 sec.
Sensitivity Analysis

Breakwater length \((L_b)\) = 300 m
Breakwater width = 20 m
Breakwater crest level above (SWL) = 1.0 m

The variable offshore distances, \(L_b/Y_b\) ratios and water depths are summarized in Table 4.1 for each run. The offshore distances are assumed to make the \(L_b/Y_b\) ratios as indicated in Table (4.1). That is to facilitate the comparison between the model results and the literature results.

<table>
<thead>
<tr>
<th>Run No.</th>
<th>Offshore distance ((Y_b)) (m)</th>
<th>(L_b/Y_b) ratio</th>
<th>Depth at the breakwater (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>120</td>
<td>2.5</td>
<td>2.40</td>
</tr>
<tr>
<td>2</td>
<td>150</td>
<td>2.0</td>
<td>3.00</td>
</tr>
<tr>
<td>3</td>
<td>200</td>
<td>1.5</td>
<td>4.00</td>
</tr>
<tr>
<td>4</td>
<td>300</td>
<td>1.0</td>
<td>6.00</td>
</tr>
<tr>
<td>5</td>
<td>500</td>
<td>0.6</td>
<td>10.0</td>
</tr>
</tbody>
</table>

Table 4.1: offshore distances, \(L_b/Y_b\) ratio and water depths

4.3.2 Main module input

The process tree which is used in this simulation is the same process tree as specified in Chapter 3 and in the sketch below. The elementary subprocesses 1 and 2 are specified to execute the wave and flow modules respectively, while the elementary subprocess 3 is specified to execute the transport and bottom modules respectively at the same branch.

The process tree specified for morphological computations

---

A single detached breakwater effect on the shore
Controllers are specified to control the whole morphological evolution computation. It is specified in Controller I that the wave computation is executed for one time. The flow computation is controlled by controller II ensuring that the computation stops after 10 times of iteration or until the change of the resultant velocity is less than 0.01 m/s in all grid points.

The wave and flow data are used by the transport module to compute the sediment transport rates which in turn are used to update the bed level in the bottom module. Controller III is set to control the execution of the transport and bottom computation. Here it is specified that after each wave and flow computation the sediment transport and corresponding bottom changes are computed 20 times. After 20 of these steps it is assumed that the bottom level variation is significant to influence the wave and flow patterns. Therefore, a new wave and flow field is computed after each 20 transport and bottom steps. The transport and bottom modules are executed with an automatic time step which is computed by the model itself. The total simulation time is 50 days which is prescribed in controller IV. Previous experiences with the model have shown that values of 20 times and 50 days are reasonable.

In the computation, the wave-current interaction is taken into account. However, the flow field is not available in the initial computation. The flow field can be computed by making another process tree for executing only wave and current computation as illustrated in the sketch below. This flow field can then be used in the initial morphological evolution computation.

![Process Tree](image-url)
4.3.3 Wave module input

In the input file of the wave module, there are five groups of options for input and output of the wave module to the communication file. These options determine how the wave module reads the bottom depth, water level and flow field whereas the flow field can be taken into account in the wave computation. To make the wave computation taking into account the flow field, the results of flow computation are interpolated onto the wave module input grid.

Grid generation

It is mentioned in Chapter 3 that input, computational and output grid have to be specified.

Input grid

The input grid for the wave module covers the whole specified area. The axes convention is that x-axis is the cross-shore direction (x-direction nearly parallel to wave propagation) and y-axis is the longshore direction. The grid size is taken equal to 10 m in both directions (referring to section 4.3.6).

Computational grid

The computational grid is taken larger than the input grid. At each lateral side of the input grid the computational grid extends 500 m to overcome that lateral disturbances penetrate into the area of interest. The cause of the lateral disturbances is that the wave field is disturbed by an import of zero energy from the lateral boundaries (see HISWA manual). The grid size, $\Delta y$, in y-direction (longshore direction) is taken 10 m while the grid size, $\Delta x$, in x-direction is chosen according to the numerical stability as follow:

$$\frac{\Delta y}{\Delta x} \geq \tan\left(\frac{\theta}{2}\right)$$  \hspace{1cm} (4-1)

$\theta$ is the directional sector of wave propagation. The directional sector is taken 120° which gives reasonable resolution in x-direction.

The output grid is specified to be the TRISULA grid.
\[ \Delta x = \frac{10}{\tan 60} \approx 5.0 \text{ m} \] (4-2)

**Wave data**

\[ H_{\text{rms}} = 2.0 \text{ m} \implies H_s = 1.414 \times 2 = 2.83 \text{ m} \]

\[ T_p = 8.0 \text{ sec} \]

wave direction = 0°

The spreading factor \( ms \) is taken equal to 2.0 to obtain a relatively wide spectrum which is very important to simulate the wave action behind the breakwater due to diffraction which is not taken into account by HISWA.

**Energy dissipation factors**

These factors affect the wave energy dissipation, such as bottom friction and wave breaking. Bottom friction coefficients are specified in the input file. The average rate of energy dissipation per unit horizontal area due to wave breaking in shallow water (\( D_b \)) is calculated in the wave module according to Battjes and Janssen (1978) via the following formula for energy dissipation:

\[ D_b = \frac{1}{4} \alpha \rho_w g f_m Q_b H_{\text{max}}^2 \] (4-3)

in which:

\[ \alpha \quad = \text{coefficient (-)} \]

\[ f_m \quad = \text{mean wave frequency (1/sec)} \]

\[ H_{\text{max}} \quad = \text{max. wave height (m)} \]

\[ Q_b \quad = \text{the local fraction of breaking waves (-)} \]

\( H_{\text{max}} \) and \( Q_b \) are computed according to the following formulas:

\[ H_{\text{max}} = \frac{0.88}{k} \tanh \left( \frac{\gamma k h}{0.88} \right) \] (4-4)

4.8 A single detached breakwater effect on the shore
\[
\frac{1 - Q_b}{-\ln Q_b} = \left(\frac{H_{rms}}{H_{max}}\right)^2
\]  
(4-5)

in which:

\(k\) = wave number \((1/m)\)

\(\gamma\) is the breaker index which can be estimated by:

\[
\gamma = 0.50 + 0.4 \tanh(33s_o)
\]  
(4-6)

in which:

\(s_o\) = wave steepness \((-)\)

It was found that if \(\alpha\) is taken equal to 0.50, \(\gamma\) can be computed from \(\alpha \gamma^5 = \text{constant}\). This gives a reasonable estimations of the coefficients for morphological evolution computation (Dingemans, 1995). Equation (4-6) estimates the \(\gamma\)-value at \(\alpha = 1\). From this approach the \(\gamma\)-value can be estimated as a first guess as follows:

\[
\gamma = 0.50 + 0.4 \tanh\left\{33 \times \frac{2.83}{1.56 \times 8^2}\right\} = 0.79
\]  
(4-7)

Therefore,

\(\alpha \gamma^5 = 1 \times (0.79)^5 = 3.25\)

For \(\alpha = 0.50\), \(\gamma\) can be estimated from

\(0.50 \times \gamma^5 = 3.25 \Rightarrow \gamma = 0.69\)

It is prescribed in the wave module that \(\alpha = 0.50\) and \(\gamma = 0.69\).

**Flow field data**

The flow field data, such as velocities and water levels, for the wave-current interaction are estimated by making a wave and current computation, as mentioned in section 4.3.2. The output of this computation is used in the initial state of the morphological computation.
Boundary condition for wave computation

Lateral sides are enlarged as indicated in the computational grid to prevent the generation of disturbances as much as possible. On the up-wave, seaward boundary, the wave height $H_s$, the wave period and the wave direction are specified.

4.3.4 Flow module input

Grid generation

The input grid of the area of interest is generated by the programme developed by Bos (1996). The bottom depth at each grid point is generated as well. The grid size is chosen in the sense that the numerical stability must be satisfied.

The Courant number for two-dimensional problems is defined as (Stelling, 1984):

$$C_{\text{max}} = \frac{\Delta t}{\Delta x} \sqrt{\frac{g h}{\frac{1}{\Delta x^2} + \frac{1}{\Delta y^2}}} \leq 10$$

(4-8)

$\Delta x$, $\Delta y$ and $\Delta t$ are chosen to be equal to 10 m, 10 m and 6.0 sec respectively. Thus, the Courant number is estimated as:

$$C_{\text{max}} = 6.0 \sqrt{9.81 \times 10 \left(\frac{1}{100} + \frac{1}{100}\right)} = 8.4 \leq 10.0$$

(4-9)

Bottom friction formulation

The bottom friction formulation to be applied in the flow module is Manning’s formulation. The bottom roughness coefficients ($n$) in each direction ($x$, $y$) are equal to 0.025 m$^{-1/3}$ s.$^{-1}$

Initial water level

Because of the difficulty of the flow module with the initial flooding of the original dry points, it is recommended to specify the maximum water level as an initial water level. The maximum water level is due to the wave action: (maximum wave set-up ($3/8 \gamma^2 h_b$)) - maximum wave set-down ($1/16 \gamma^2 h_b$).
\[
\eta = \frac{3}{8} \times (0.69)^2 \times 2.83 \times \frac{1}{0.69} - \frac{1}{16} \times (0.69)^2 \times 2.83 = 0.61 \text{ m}
\]

(2-10)

It is prescribed in the flow module that the initial water level \( = 0.61 \text{ m} \)

**Boundaries and boundary condition**

The boundaries of the area and the breakwater are specified in the enclosure file in which the velocity at these boundaries are taken zero. It is specified that the lateral boundaries are closed (no longshore current because of normally incident wave), the shoreline is taken as a closed boundary and the breakwater is taken as a closed boundary as well. It is specified in the up-wave boundary that the water level is equal to zero with a weak reflective condition to avoid the reflection of the waves at the boundaries.

**4.3.5 Transport module and bottom module input**

*The used transport formula*

As mentioned in Chapter 3, the used sediment transport formula is Bijker’s formula which takes into account the effect of waves and currents. This formula needs some input parameters such as: the coefficient \( b \) which depends on the wave height and the water depth. The coefficient \( b \) is specified for shallow water \( (b = 5) \) and for deep water \( (b = 2) \); for intermediate water depth the coefficient \( b \) is linearly interpolated. The other input parameters are:

- Grain size \((d_{50})\) = 250 \( \mu \text{m} \)
- \((d_{90})\) = 350 \( \mu \text{m} \)
- Fall velocity of the grains = 0.031 m/s
- Sediment density \((\rho_s)\) = 2650 \( \text{kg/m}^3 \)
- Water density \((\rho_w)\) = 1000 \( \text{kg/m}^3 \)
- Bottom roughness \((\mu)\) = 0.05 m
- Sediment porosity \((\lambda)\) = 0.4 (-)
- Kinematic viscosity \((\nu)\) = 10\(^{-6}\) m\(^2\)/s
Sediment transport rate correction

As mentioned in Chapter 3, the computed sediment transport must be corrected before computing the bed level variation. The correction has to be done to take into account the effect of bed level gradient, non-erodible layers and numerical stability. In the hypothetical case of this study no non-erodible layers are present. Therefore, the sediment transport rate does not need a correction for this effect. However, the computed sediment transport does not contain the effect of bed level gradient and numerical stability. Both effects are taken into account by applying correction terms to the computed sediment transport rate ($S'$).

\[ S = \alpha_s \alpha_n S' \]

in which:

- $S'$ = the computed sediment transport by Bijker's formula (m$^3$/s/m)
- $\alpha_s$ and $\alpha_n$ are the physical slope and numerical stability correction factors respectively. $\alpha_s$ and $\alpha_n$ can be estimated as follows:

a - Correction for physical slope effect ($\alpha_s$)

The effect of the bed level gradient can be taken into account by applying a correction term $\alpha_s$:

\[ \alpha_s = 1 + \alpha_{bs} \frac{\partial z_b}{\partial l} \]  

in which:

- $\Delta l$ = grid increment measured in the transport direction (m)
- $z_b$ = bed level from a specific reference (m)

The coefficient $\alpha_{bs}$ is taken equal to 1.0.

b - Correction for numerical stability ($\alpha_n$)

The numerical scheme used in the bottom module is extended to guarantee the model stability. This extension is obtained by introducing a slope dependent correction to the computed sediment transport rates. The used stability correction expression is:
The coefficient $a_{nn}$ is applied to the time-averaged sediment transport rates:

$$a_{nn} = \frac{a_{st} \beta \Delta l}{2h_{av}}$$  \hspace{1cm} (4-14)\]

in which:

- $a_{st} = \text{coefficient} = 1.0 \text{ (-)}$
- $\beta = \text{power of the used formula} = 5.0 \text{ (-)}$
- $h_{av} = \text{averaged water depth over the time interval used in the bottom computation (m)}$

More details about sediment transport correction can be found in the DELFT2D-MOR manual.

The sediment transport rates are determined at each grid point of the flow module. The automatic time step option is used and the Courant number is equal to 0.95.

**Boundary condition for the transport and bottom modules**

In the transport and bottom modules, the specified boundaries is the same as specified the flow module. The only used boundary is the seaward boundary. This boundary is taken as a fixed bottom level equal to the original bottom level.

### 4.3.6 Grid size selection

It should be clearly kept in mind that the smaller the grid size, the more accurate the results. However, there are some restrictions for choosing the grid size. These restrictions may be the availability of disk space or the computation time. Therefore, two computations have been made for different grid size. An area of 1300 m by 700 m with a grid size of 5.0 m and an area of 2100 m by 790 m with a grid size of 10 m are assumed with a plane sloping bed. The computation of each case is carried out for 50 days with normally incident wave without the existence of the breakwater.
It is found, Fig. (4.5), that the grid size of 5.0 m gives more accurate results than the results from the area with a 10 m grid size. It is known that the morphology does not change in case of normally incident wave (cross-shore sediment transport is not present in the two-dimensional model computations). However, it is observed by making an animation for the area with 10 m grid size that the entire area, especially close to the shoreline, suffers from some erosion. Thus, a cross-section is taken in the middle of the area for both computations. The cross-section from the area with a 5.0 m grid size does not change but the cross-section from the area with a grid size of 10.0 m suffers from some variations at the shoreline, Fig. (4.5). The cause of these variations is due to disturbances (eddies) which appear at the ends of the area. These eddies migrate towards the middle of the area causing the shoreline erosion. This process is observed by the animation. The variations at the shoreline are assumed to be relatively small in comparison to the expected shoreline changes caused by the breakwater. The morphological effects of the breakwater are found to be dominant. Therefore, it is decided to select the 10.0 m grid size in which less disk space and computation time are required.

4.3.7 Computation results

1. Introduction

Two kind of computations have been carried out. The first one is to get the initial state (at \( t = 0 \)) of the wave field, the flow field and the sediment transport field. The second one is the morphological evolution toward the equilibrium state, which is reached after approximately 50 days. The former computations are implemented with the intention to evaluate the effect of the structure on the various hydrodynamic processes. The comparison between wave-current interaction and no wave-current interaction for the initial state is done by comparing the results of both computations at section 1-1 as indicated before in the general layout sketch (see section 4.3). The effect of the different layouts of the breakwater is discussed as well. The equilibrium state results are used to compare these model results with literature results. It should be kept in mind that most of the prototype cases are based on the obliquely incident waves, longshore sediment transport is present, while the hypothetical case is a theoretical case, longshore sediment transport is absent. In the

4.14  A single detached breakwater effect on the shore
following sections, the results of the initial state and equilibrium state will be discussed.

2. Discussion of the initial state results
There are two objectives for implementing these computations. First of all the effect of the wave-current interaction on the results is studied. Secondly, the effect of the breakwater layouts on the wave field, flow field and sediment transport field is established.

**Without wave-current interaction**

*Wave computation*

Figures (4.6), (4.7), (4.8), (4.9) and (4.10) reveal that the dead zone with zero wave height is more or less the same in the different layouts of the breakwater. Wave heights in cross-section 1-1 are discussed later. A big variation of the wave height at the extremities of the breakwater can be observed. This variation is due to the abruptly reduction of the wave height just behind the breakwater (wave height equals to zero) because the effect of the directional spreading is ceased. Due to the wave height variation, a gradient in the radiation stresses at that location is created.

The wave isolines have an irregular shape. Moreover, the region influenced alongshore by the breakwater increases with increasing offshore distance of the breakwater. The wave height at the sheltered area, precisely at the zone close to the shoreline, is the same as in the exposed area when the offshore distance increases. This is due to the directional spreading effect. The reason why the region influenced by the breakwater increases when the offshore distance increases is due to the effect of wave breaking at the breakwater and the location of the breakwater with respect to the breaker line. The bigger the sheltered area, the bigger the region influenced in the longshore direction by the breakwater. It can also be concluded that the directional spreading has a big effect if the offshore distance increases.

*Flow computation*

Figures (4.11), (4.12), (4.13), (4.14), (4.15) show the flow field for the different breakwater layouts. Eddies are generated at the extremities of the breakwater in the different breakwater layouts. However, the order of magnitude of the velocities differ. The velocities at the
breakwater extremities \( (V_{\text{tip}}) \) and closer to the shoreline at the sheltered area \( (V_{\text{shoreline}}) \), section 1-1, are summarized in Table 4.2 for the different layouts.

<table>
<thead>
<tr>
<th>Run</th>
<th>Offshore distance ( (Y_B) ) (m)</th>
<th>Velocity ( (V_{\text{tip}}) ) (m/s)</th>
<th>Velocity ( (V_{\text{shoreline}}) ) (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>120</td>
<td>0.78</td>
<td>0.72</td>
</tr>
<tr>
<td>2</td>
<td>150</td>
<td>0.76</td>
<td>0.78</td>
</tr>
<tr>
<td>3</td>
<td>200</td>
<td>0.80</td>
<td>0.82</td>
</tr>
<tr>
<td>4</td>
<td>300</td>
<td>0.80</td>
<td>0.80</td>
</tr>
<tr>
<td>5</td>
<td>500</td>
<td>0.52</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Table 4.2: summary of the velocities at the breakwater extremity and close to the shoreline.

Close to the shoreline the maximum velocity is 0.82 m/s for a breakwater situated at offshore distance of 200 m and the velocity value decreases for larger offshore distances. That is due to the wave activity behind the breakwater in which the offshore distance affects the distribution of the wave height behind the breakwater. The change in the wave height in the longshore direction ensues the variation in the water level gradient between the different breakwater layouts. Therefore, the velocity values are different. This can be observed from Table 4.2. Very small eddies are generated with velocities up to about 0.40 m/s for 200 m offshore distance (run 3). It can be concluded that the location of the breakwater affects the velocity value of the eddies. The value of the velocity has a maximum (about 0.82 m/s) if the breakwater is situated at the breaker line which is approximately at 205 m offshore. That is due to the water level gradient is higher when the breakwater is situated at the breaker line.

**Sediment transport rates computation**

Bijker’s formula accounts for wave and current effects in the transport computation. That can be observed in Figures (4.16), (4.17), (4.18), (4.19) and (4.20). The wave action combined with the current give sediment transport in the shoreline zone behind the breakwater layout of 200 m, 300 m and 500 m offshore distances. The small eddies observed in the flow field
have sufficient velocity to move the sediment stirred up by wave action at the breakwater extremities where big scour holes are expected to be generated. The reason for the small eddies generation comes from the variation of the wave height at the extremity of the breakwater which in turn creates water level gradient induced by the radiation stress gradient ensuing the small eddies.

For 120 m offshore distance (run 1), most of the sediment comes from the breakwater extremities as well as for 150 m offshore distance. For 200 m, 300 m offshore distances, most of the sediment comes from the breakwater extremities and the nearshore zone. For 500 m offshore distance, most of the sediment comes from the nearshore zone and very little amount of sediment comes from the breakwater extremities. It can be concluded that the breakwater layout has a big influence on the sediment transport rates.

**With wave-current interaction**

**Wave computation**

The wave-current interaction has a strong effect on the wave height computation for offshore distance greater than 150 m. Figures (4.21), (4.22), (4.23), (4.24), (4.25) reveal this effect. Later on the wave heights are discussed in section 1-1. Having taken into account the flow data in the wave computation, it has a smoothing effect on the wave distribution. The variation of the wave height at the breakwater extremities is very small. Moreover, the dead zone inside the sheltered area of the breakwater situated at 200 m, 300 m and 500 m is not visible any more and it is reduced in case of the breakwater situated at 120 m and 150 m offshore. The big eddies have the main effect on the wave height distribution. The eddies decrease the variation of the wave height at the extremities and make the waves to go closer to the breakwater.

**Flow computation**

The flow field does not change which is revealed in figures (4.26), (4.27), (4.28), (4.29) and (4.30). The same eddies appear with more or less the same order of magnitude of velocity. The small eddies are however not visible. The small eddies generated in the computation
without wave-current interaction are due to a small variation of the water level at this location which is due to the wave height variation at the breakwater extremities.

**Sediment transport rates computation**

Because of the redistribution of the wave height due to the effect of the wave-current interaction, slightly different patterns of the sediment transport rates can be observed in Figures (4.31), (4.32), (4.33), (4.34) and (4.35). The effect of the small eddies are not visible in the sediment transport rates field.

**Comparison between wave-current interaction and no wave-current interaction**

Comparison between the results of the computations without and with wave-current interaction can be discussed for various breakwater layouts. Focusing on the wave height distribution, there are a big variation of the wave height at the extremity of the breakwater for computations without wave-current interaction, Figures (4.6 to 4.10). This big variation is not present for the computations with wave-current interaction, Figures (4.21 to 4.25). In the flow field, there are small eddies are generated in the computation without wave-current interaction, Figures (4.11 to 4.15). The small eddies are not present in the computation with wave-current interaction, Figures (4.26 to 4.30). The sediment transport rates depend on the wave height and the velocity. Therefore, there are small eddies generated in the computations without wave-current interaction, Figures (4.16 to 4.20). The effect of the small eddies is expected to ensue an exaggeration of the scour holes deepening at the breakwater extremity, see Bos (1996) results. The sketch below demonstrates this process.

Section 1-1 at the breakwater extremity is used for getting insight in the order of magnitude of the wave height, the flow and the sediment transport rate and comparison the results of wave-current interaction and without wave-current interaction.

---

4.18

A single detached breakwater effect on the shore
Figures (4.36), (4.37), (4.38), (4.39) and (4.40) show results for 120 m, 150 m, 200 m, 300 m and 500 m offshore distances. There is no difference between the two computations of the wave height along the section further offshore from the breakwater. The changes only occur in the area influenced by the breakwater. The wave height is larger with wave-current interaction than without wave-current interaction. This is due to the effect of the current which redistributes the wave height. The velocity is almost the same except for the velocity of the small eddies which are present in case of without wave-current interaction computation. Although the velocity is almost equal in both cases, the sediment transport varies. The reason is due to the wave effect resulting in stirring of the bottom material. As a result, the sediment transport increases because the wave height increases in case of the wave-current interaction.

Figures (4.36) and (4.37) reveal that the changes are more or less the same for the breakwater situated at 120 m and 150 m. The maximum differences in the wave height are 0.75 m and 1.0 m for 120 m offshore and 150 m offshore respectively. For the breakwater situated at 200 m, 300 m and 500 m offshore as revealed in Figures (4.38), (4.39) and (4.40), that the wave height distribution close to the shoreline coincides in the both cases. However, a big variation in the wave height at the breakwater location is observed because the wave breaks much more in the computation without wave-current interaction than in the computation with wave-current interaction. Another reason can be due to the effect of the directional spreading which does not reach to the point just behind the breakwater in case of without wave-current interaction.

The maximum differences in the wave height reach up to 1.25 m for the breakwater situated at 500 m offshore. Velocity distribution along section 1-1, Fig. (4-36,37,38,39 and 40), are smaller in wave-current interaction than without wave-current interaction. This is due to the different wave height distribution in each case. The wave height variation for without wave-current interaction is quite large in comparison to wave-current interaction. As a result the water level variation alongshore is different. In case without wave-current interaction, the variation in the water level is larger which in turn causes higher velocity. This is why the
velocities for without wave-current interaction are larger than the velocities for wave-current interaction. Also, the net sediment transport getting in the sheltered area is different. This net sediment transport affects the development of the bottom level as well as the deposition zone behind the breakwater.

In nature, the current may affect the wave height distribution around the breakwater. The previous results show that the wave-current interaction has an important role in redistributing the wave height which in turn affects the amount of sediment transport. Since the wave-current interaction is an authentic physical phenomenon, the wave-current interaction is included in the morphological computation of the equilibrium state.

3. Discussion of the equilibrium state results

Introduction

The morphological computations have been made for the different breakwater layouts. The final situation of the morphology for the different layouts will be discussed. The model results are compared with the results from the literature. The results from the literature are the equilibrium relationships for tombolo parameters which come from the analysis of the field data resulting in a curve which fits the field data (Fig. 2.5, Rosen and Vajda, 1982), Harris and Herbich (1986) equation, Equation (2-18), and mathematical model results from previous studies (Bos, 1996).

Sand deposition behind the breakwater

The equilibrium state is reached when the wave forces are perpendicular to the contour lines of the sand deposited behind the breakwater. The wave forces come from the diffracted wave crests which are more or less parallel to the formed contour lines. The equilibrium state is reached when the net sediment transport across the shadow line, which divides the sheltered area and the exposed area, is approaching zero. This is because the effect of the circulation cells is diminished. Another reason is that the curvature of the eddy coincides with the formed bottom contour line curvature. It is found that the equilibrium state for breakwater situated at 300 m and 500 m are achieved after approximately 75 days, and for 120 m, 150 m and 200 m after approximately 50 days.
The results show that when the breakwater is situated at 120 m offshore, a double tombolo system is formed, Fig. (4.41). The zone encompassed by these double tombolo may be filled with sediment by the effect of swash or wind effect which are not included in the model. In Figure (4.42), (4.43) and (4.45), a tombolo is formed in the deposition zone for the breakwater situated at 150 m, 200 m and 300 m offshore respectively. The interval presented in that Figures from (0 - 1.0 m) contains the shoreline in which the case of the breakwater situated at 300 m offshore (run 4) found to be nearly tombolo.

For the breakwater situated at 500 m offshore, a salient is formed, Fig. (4.46). It can be observed from Fig. (4.46) that two scour holes are present at the breakwater extremities which are not present in the other cases. The reason of the scour holes generation comes from the effect of the combined current, about 0.40 m/s, and the stirring effect due to waves. In the other cases the scour holes are present but it is filled up during the deposition zone generation. However, they are not filled up in case of the breakwater situated at 500 m offshore. The reason is that the breakwater is positioned far away from the surf zone which is active from the morphological view point. Therefore, the sediment comes from the nearshore zone will take much time to reach to the scour holes in order to fill them.

It is found that the percentage of the sand trapped behind the breakwater with respect to the available volume is between 55 % to 65 %, Fig. (4.47).

A computation for a breakwater situated at 200 m offshore is done without wave-current interaction. It is found, Fig. (4.44), that the bottom contours have an irregular shape. It can be clearly compared with the case of including wave-current interaction. Generally spoken taking into account the current into the wave computation has a smoothing effect on the morphology which should not be ignored. Since in reality, the bottom contours have also a relatively smooth shape. The following table, Table 4.3, gives a summery of these results.
Comparison between literature results and model results

These results in Table (4.3) are in a good agreement with relationships found in the literature (Chapter 2) yielding a tombolo formation when $L_B/Y_B$ greater than or equal to one while salient forms for $L_B/Y_B$ less than one.

Fig. (4.48) reveals that the model results are in agreement with Rosen and Vajda (1982) results for which the error is estimated (by root mean square) to be about 4%. The curve of Rosen and Vajda is the curve fitting of the field observations. The difference between the model results and the Rosen and Vajda results may be due to the effects of the wave climate in nature which is not represented in the model which uses only one-wave condition of normally incident wave. Moreover, the three dimensional effect which is not included in this model (undertow). However, this variation can be accepted.

With respect to the attachment width at the breakwater, $X_T$, it is found that the results of the model are in a good agreement with the results of Rosen and Vajda (1982), Fig. (4.48).

Another comparison between the model results and Harris and Herbich (1986) results in the same Figure (4.48). It is found that the variation between the model results and their results gives an error about 11 % in which the equation of Harris and Herbich is applicable for $Y_B/L_B$ ratio between 0.5 and 2.5. The variation is quite big between the model results and

<table>
<thead>
<tr>
<th>Run</th>
<th>Distance offshore (m)</th>
<th>$L_B/Y_B$ ratio</th>
<th>Beach response</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>120 m</td>
<td>2.50</td>
<td>double tombolo</td>
</tr>
<tr>
<td>2</td>
<td>150 m</td>
<td>2.00</td>
<td>tombolo</td>
</tr>
<tr>
<td>3</td>
<td>200 m</td>
<td>1.50</td>
<td>tombolo</td>
</tr>
<tr>
<td>4</td>
<td>300 m</td>
<td>1.00</td>
<td>tombolo</td>
</tr>
<tr>
<td>5</td>
<td>500 m</td>
<td>0.60</td>
<td>salient</td>
</tr>
</tbody>
</table>

Table 4.3: summery of the five runs
the Harris and Herbich equation. The reason is that the equation of Harris and Herbich overestimates the volume behind the breakwater for \( Y_B/L_B \) ratio greater than or equals to 0.50. Another reason comes from the model itself, in which the trapped sand is underestimated by the model due to the swash and wind effects which are not included in the model as well as some disturbances occur at the sheltered area which in turn affect the amount of the trapped sand. The underestimation occurs when the breakwater close to the shoreline, precisely when \( L_B/Y_B \) greater than two, whereas the disturbances start to affect the amount of the trapped sand.

Although the wave condition in the hypothetical case (normally incident waves) is different from the natural wave conditions, the model results are nearly the same as the literature results. The reason is that for a tombolo formation, it is required a specific amount of sand for the equilibrium state. The equilibrium mechanism may differ between the hypothetical case and the prototype. In spite of this, once the required sand volume (trapped sediment) for the equilibrium state has fulfilled it will not be trapped sediment behind the breakwater any more. The variation may be observed in the shoreline shape between the hypothetical case and the nature. Later on, Chapter 5, the comparison between simulations with obliquely incident waves and with normally incident waves will be achieved.

In the simulations made by Bos (1996) the wave-current interaction was only included in the initial computation. These simulations, especially after the initial computation where the velocities and water level were taken zero, yield inaccurate bottom level variation and irregular bottom contours. Moreover, the scour holes found at the extremities of the breakwater are exaggerated due to the effects of the big and small eddies in case of no wave-current interaction, as can be seen in the previous sketch (see section 4.3.7). This problem is overcome by fixing the flow computation time to adapt zero time. It means that the associated time of the computed flow is always equal to zero. In this case the wave module successfully reads the flow field.

An additional case of a breakwater located at 75 m offshore is used to check the previous conclusion about the underestimation. It is found, Fig (4.49), that a double tombolo is
formed and a pond encompassed with the double tombolo. But the results of this case have some disturbances due to the disturbances at the ends of the computation area. Therefore, another computation was made for an enlarged computation area (Fig. 4.50).

It is found that these disturbances generate a circulation cell at each end of the area. These two eddies migrate to the middle of the area. For a breakwater situated at 120 m, 150 m, 200 m, 300 m and 500 m, the disturbance does not influence the results at the breakwater location as can be observed in Fig. (4.47). For a breakwater situated at 75 m offshore, the disturbances influence the results of the accumulated sand behind the breakwater, Fig. (4.49).

It can be concluded that the eddies at the extremities of the breakwater situated at 75 m are so weak that the eddies generated by the disturbances overrule them. The area is enlarged by 600 m in both ends with different grid size to decrease this disturbance. This enlargement has been applied by Bos (1996) for the simulation with obliquely incident wave whereas he found some disturbances have occurred. The evidence of making such enlargement was to diminish the effect of the disturbances. However, the enlargement does not enhance the results, Fig. (4.50) and Fig. (4.51) because the disturbance still exist. The enlargement affects only the travelling time of the eddies generated by the disturbances. Another observation is the erosion of the shoreline further from breakwater, precisely at the uninfluenced place. That comes from the eddies generated by the disturbances.

4.4 Conclusions of the sensitivity analysis

Having made the comparison between the DIFFRAC model and HISWA model. It can be inferred that the HISWA model is valid for the morphological computation. However, it may not be valid for the wave computations behind the breakwater which is used for establishing a marina in which the wave height is underestimated. The comparison between the wave-current interaction and no wave-current interaction computations have been made by doing the initial state computation.

It can be concluded that the wave-current interaction has a big effect on the wave pattern, the flow pattern and the sediment transport pattern which in turn results smooth contour lines at the equilibrium state. The comparison between the model results and the literature results gives high assurance of the model results. However, it is found that some disturbances are
generated which in turn affect the estimation of the trapped sediment behind the breakwater. This happens when the breakwater is close to the shoreline.
CHAPTER 5
CASE STUDY

5.1 Introduction
The case study is a single detached breakwater constructed to protect a touristic village. The touristic village is located on the northern coast of Egypt along the Mediterranean sea in which the tide is insensible. The location of the village is precisely at Sidi Krir, about 30 km west of the port of El-Dekhila in Alexandria, Fig. (5.1). The breakwater has been built in 1993. According to a visual observation, the equilibrium state has been reached after two years. The breakwater layout and the morphological evolution is shown in Figure. (5.2).

Field investigation has been achieved by Hydraulics Research Institute in 1995, Egypt. The input data used in this chapter are from that investigation, (HRI, 1995). It was found that a dredging operation has been carried out behind the breakwater. The reason for doing this operation is to remove the sediment which blocked an intake pipe. The intake pipe is used to convey the sea water from the sheltered area to a pond located inside the touristic village for recreation purposes. The position of the intake pipe is radically erroneous. The dredged material was disposed of at the down-drift side of the breakwater. Another observation is referring to a seawall, situated at approximately 450 m upstream of the breakwater, which has collapsed due to the erosion, Fig. (5.2).

The simulation of this field case is made by the DELFT2D-MOR model. The initial situation which is put into the model is the situation just after breakwater construction. After reaching the equilibrium state, the resulting shoreline from the model at the upstream side (no dredging) of the breakwater is compared with the shoreline position from the field investigation, Fig. (5.2). The shoreline shape at the down-stream side of the breakwater is not used in this comparison because it is expected that the dredging operation has affected the morphology down-stream side of the breakwater. Four simulations are carried out. Three simulations are made with different bottom roughness. In these simulations one-wave condition is applied. The fourth run is based on two-wave conditions. The reason for
choosing the bottom roughness as a calibrated parameter is that the bottom roughness is an uncertain parameter in which there is no direct relation which gives an accurate bottom roughness. A comparison between these results and measured field results is achieved.

The model needs the input data for the various modules. Therefore, in the following sections the input data are described.

5.2 Simulation using one-wave condition

5.2.1 Main module input

The input for the main module is identical to that of the input described in the previous chapter. Wave-current interaction is also included.

5.2.2 Wave module input

*Dimensions and grid description*

According to the field investigation of the morphological evolution, the area used for this simulation is 3000 m by 700 m. These dimensions are chosen in the sense that the influence of the breakwater is not noticed at the boundaries in the longshore direction. The width of the area of interest (700 m) is chosen according to HISWA specification. For the model it is required that the location of the up-wave boundary is in deep water. The cross-shore profile is described in Table. (5.1).

The input grid for the wave module is chosen to cover the whole area, which is taken rectangular with dimensions 3000 m by 700 m, where the former size is the distance parallel to the shore, y-direction, and the latter is the offshore-onshore distance, x-direction. This area is divided into square meshes in both directions of 10 m x 10 m, so there are 301 grid points in y-direction and 71 grid points in x-direction.
<table>
<thead>
<tr>
<th>Cross-shore distance (m)</th>
<th>Water depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.000</td>
<td>12.6</td>
</tr>
<tr>
<td>93.50</td>
<td>12.0</td>
</tr>
<tr>
<td>150.0</td>
<td>11.0</td>
</tr>
<tr>
<td>210.0</td>
<td>10.0</td>
</tr>
<tr>
<td>236.5</td>
<td>9.00</td>
</tr>
<tr>
<td>263.0</td>
<td>8.00</td>
</tr>
<tr>
<td>299.5</td>
<td>7.00</td>
</tr>
<tr>
<td>333.0</td>
<td>6.00</td>
</tr>
<tr>
<td>366.0</td>
<td>5.00</td>
</tr>
<tr>
<td>419.5</td>
<td>4.00</td>
</tr>
<tr>
<td>479.0</td>
<td>3.00</td>
</tr>
<tr>
<td>529.0</td>
<td>2.00</td>
</tr>
<tr>
<td>579.5</td>
<td>1.00</td>
</tr>
<tr>
<td>613.0</td>
<td>0.00</td>
</tr>
<tr>
<td>670.0</td>
<td>-1.00</td>
</tr>
<tr>
<td>700.0</td>
<td>-1.50</td>
</tr>
</tbody>
</table>

Table 5.1: average cross-shore profile of the area.

The computational grid is taken larger than the input grid in the lateral longshore direction (y-direction). The reason for this enlargement was explained in the previous chapters. On each lateral side the computational grid is taken 1000 m larger than the input grid. The seaward boundary of the computational grid is taken equal to the seaward boundary of the input grid. The mesh size in y-direction is chosen equal to 10 m which is equal to the input grid but in x-direction the mesh size is chosen equal to 5.0 m to satisfy the numerical stability condition. The output grid is the TRISULA grid.
Wave data and energy dissipation coefficients

In reality, there are four predominant wave conditions (HRI, 1995). These wave conditions are analyzed to get morphological equivalent one-wave and two-wave conditions. In this simulation, the equivalent one-wave condition is evaluated on the basis that the net sediment transport rate is the same as the net sediment transport rate of the four predominant wave conditions whatever the shoreline orientation is (see appendix A, S-φ curves are almost the same).

The equivalent one-wave condition is:

\[ H_s = 1.85 \text{ m}, \quad T_p = 5.8 \text{ sec} \quad \text{and wave direction} = -24^\circ \]

The wave direction is measured relative to the normal to the shoreline (x-direction) in which the positive angle is measured anticlockwise and the negative angle is measured clockwise. The energy dissipation coefficients (\( \gamma, \alpha \)) are estimated according to (Dingemans, 1995). \( \gamma \) is equal to 0.72 when \( \alpha \) is equal to 0.50.

Boundary condition

On the seaward boundary, the wave height \( H_s \), wave period \( T_p \), and wave direction are specified.

5.2.3 Flow module input

Grid generation

The grid size is chosen as 10 m in both sides in which the numerical stability condition is satisfied. The boundaries of the breakwater are specified as dry points. The shape of the breakwater has an inclined part which is schematized as 45° while the offshore distance is maintained, as shown in the sketch. This schematization is due to the flow module (TRISULA) limitations for enclosure specification (see TRISULA...
It is mentioned before that the simulation with one-wave condition is carried out three times with different bottom roughness ($k_s$), which is dealt with as a calibration parameter. The bottom friction formulation to be applied in TRISULA is Manning’s formulation. The Manning’s coefficient ($n$) is estimated according to the following relationship:

$$n = \frac{\frac{1}{6}}{C}$$ \hspace{1cm} (5-1)

in which:

- $n$ = Manning’s coefficient ($m^{1/3}$ s)
- $h$ = water depth (m)
- $C$ = Chezy’s coefficient ($m^{1/2}$/s)

in which Chezy coefficient is estimated from:

$$C = 18 \log \left( \frac{12h}{k_s} \right)$$ \hspace{1cm} (5-2)

in which:

- $k_s$ = bottom roughness (m)

The input data of the three simulation runs for the equivalent one-wave condition are held constant except for the bottom roughness which is specified in Table 5.2.

<table>
<thead>
<tr>
<th>Simulation no.</th>
<th>Bottom roughness, $k_s$ (m)</th>
<th>Manning’s coefficient, $n$ ($m^{1/3}$ s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.01</td>
<td>0.019</td>
</tr>
<tr>
<td>2</td>
<td>0.05</td>
<td>0.023</td>
</tr>
<tr>
<td>3</td>
<td>0.10</td>
<td>0.026</td>
</tr>
</tbody>
</table>

Table 5.2: bottom roughness and corresponding Manning’s coefficient for each simulation.
Boundary condition
Up-stream, down-stream and seaward boundary conditions are created by the model. A velocity boundary is imposed at the up-stream and down-stream sides while a water level condition is described at the seaward boundary. It is specified that this water level is equal to zero with a weakly reflective coefficient.
The velocity boundary is determined by executing wave and current computations in the absence of the breakwater. In the first run, all boundaries values are taken equal to zero. From this run velocity values are picked up from a cross-section located at the middle of the area in which the longshore current is uniform. This procedure is repeated several time to reach to a uniform longshore distribution of the current velocity along the shore.

5.2.4 Transport and bottom module input
Bijker's formula input
The coefficient \( b \) in Bijker's formula is chosen 5 and 2 for shallow water and deep water respectively. The coefficient \( b \) is linearly interpolated for intermediate water depth. The other input related to the sediment characteristics is based on the field observation (HRI, 1995).

Grain size
\[
\begin{align*}
\text{(d}_{50}\text{)} & = 587 \text{ \mu m} \\
\text{(d}_{90}\text{)} & = 1127 \text{ \mu m}
\end{align*}
\]

Fall velocity of the sediment
\( = 0.06 \text{ m/s} \)

Water density \( (\rho_w) \)
\( = 1025 \text{ kg/m}^3 \)

Sediment porosity \( (\lambda) \)
\( = 0.40 (-) \)

Kinematic viscosity \( (\nu) \)
\( = 10^{-6} \text{ m}^2/\text{s} \)

The different bottom roughness parameters are given to the Bijker's formula as well. The sediment transport correction is done as indicated in Chapter 4. The automatic time step option is used and the Courant number is taken equal to 0.95.

Boundary condition
As it is restricted by the model that the boundaries of the transport and the bottom module are the same as the boundaries of the flow module. It is specified in the transport and bottom modules that the bed level at these locations remains constant.
Case Study

5.2.5 Results discussion

1. Introduction

To get insight in the effect of the breakwater on the wave field, the flow field and the sediment transport field with different bottom roughness values, an initial state computation \((t = 0)\) is carried out. A comparison is presented for the different cross-sections which are taken at different location. Section 1-1 is taken far away from the breakwater, section 2-2 is taken at the up-stream breakwater tip while section 3-3 is taken at the down-stream breakwater tip (see the sketch below).

A model sensitivity analysis is presented to select the relevant bottom roughness for the two-wave condition simulation. The model calibration is made in the sense that the resulting shoreline from the model computation at the up-stream side of the breakwater (not affected by the dredging operation) approximately fits with the measured shoreline from the field investigation. The most proper bottom roughness is applied in the final simulation with the two-wave condition. The simulation time of the equilibrium state is roughly estimated based on the results of the initial state computation.
2. Discussion of the initial state results

*Wavefield computation (including flow data)*

Figures (5.3, 4 and 5) show the wave isolines for the different bottom roughness values. It can be noticed that the effect of the bottom roughness merely appears at the up-stream breakwater tip. This effect can be observed from the 1.2 m isoline. That effect may be due to the shape of the breakwater which affects the flow field which in turn affects the wave computation.

*Flow field computation*

Figures (5.6, 7 and 8) represent the flow field for the different bottom roughness values. It can be observed that the velocity value increases considerably when the bottom roughness decreases, Table. (5.3). Circulation cells are generated at the tips of the breakwater, the bigger one at the down-stream side and the smallest one at the up-stream side.

<table>
<thead>
<tr>
<th>Simulation no.</th>
<th>Bottom roughness, $k_s$ (m)</th>
<th>Maximum velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.01</td>
<td>0.40</td>
</tr>
<tr>
<td>2</td>
<td>0.05</td>
<td>0.25</td>
</tr>
<tr>
<td>3</td>
<td>0.10</td>
<td>0.18</td>
</tr>
</tbody>
</table>

*Table. 5.3: maximum longshore current with respect to the bottom roughness, (section 1-1).*

*Sediment transport rate computation*

The bottom roughness has an important effect on the sediment transport rates estimated by the Bijker’s formula. Although the velocity decreases when the bottom roughness increases, the sediment transport rates increase when the bottom roughness increases. This is due to the increasing turbulence at the bed when the bottom roughness increases. This turbulence is due to the eddies generated at the bottom, by the effect of these eddies more sediment is stirred up. The effect of the bottom roughness can be observed in Figures (5.9, 10, 11).
To get insight in the order of magnitude, the wave height, the velocity and the sediment transport rates are shown for the different cross-sections.

At section 1-1, Fig. (5.12), the wave height is the same for the different bottom roughness values but the effect of the bottom roughness can be observed in the velocity and the sediment transport. The reason why the wave height is equal for the different bottom roughness values, is due to the longshore current being relatively normal to the wave direction, which has no effect on the wave characteristics. Another reason could be due to the uniform current distribution in the longshore direction. The velocity gradient is almost zero in the longshore direction and in turn the wave height is not affected by the current velocity. It can be noticed that the bottom roughness has an effect on the velocity value and sediment transport rates. The sediment transport rates are approximately equal for $k = 0.05$ m and $0.10$ m. The reason may be the eddies generated at the bottom has the same effect of stirring up the bottom materials. The sediment transport rates are smaller for $k = 0.01$ m than for $k = 0.05$ m and $0.10$ m.

At section 2-2, Fig. (5.13), the effect of the various bottom roughness values can be observed. The bottom roughness affects the flow field which in turn affects the wave height distribution at the up-stream extremity of the breakwater. The velocity value varies from positive to negative values, which is due to the circulation cell generated at the breakwater extremity. The sediment transport rates are considerably increased for the higher bottom roughness.

At section 3-3, Fig. (5.14), the wave height becomes equal for the various bottom roughness values. The reason is that the effect of the circulation cell generated at the down-stream side of the breakwater, which is too weak to influence the wave height distribution. On the other hand, the velocity gradient is equal and slightly small to affect the wave height distribution. The velocity and the sediment transport rates are also affected at section 3-3.

The sediment transport rates at the various cross-sections are analyzed to estimate the longshore sediment transport rate per year integrated over the surf zone width. The procedure is done in such a way that the average sediment transport rate is estimated along the cross-section at the location which is not influenced by the breakwater, section 1-1, Table (5.4).
Table 5.4: average longshore sediment transport rates associated with $k_s$ values, section 1-1.

Later on the results of the average sediment transport from this model, DELFT2D-MOR model, will be compared with the one-line model results, UNIBEST model. Table 5.5 represents the sediment getting in the sheltered area, from the up-stream side of the breakwater and down-stream side of the breakwater, which is estimated by the resulting sediment transport rate distribution at section 2-2 and section 3-3.

Table 5.5: average sediment transport rate getting in the sheltered area associated with $k_s$ values, (section 2-2 and section 3-3).

The simulation time for the equilibrium state can be roughly estimated by knowing the required volume behind the breakwater to reach to the equilibrium state. It is concluded from the sensitivity analysis, Chapter 4, that the required volume is between 0.55 to 0.65 of the available volume. The available volume can be approximately estimated as $(L_B \times Y_B \times h_B/2)$ which is equal to $(390 \times 140 \times 3/2 = 81,900 \text{ m}^3)$. Thus, the required volume is roughly between 45,000 $\text{m}^3$ to 55,000 $\text{m}^3$. The simulation time for the equilibrium state can now be estimated, Table 5.6.
<table>
<thead>
<tr>
<th>Simulation No.</th>
<th>Bottom roughness $k_s$ (m)</th>
<th>Required simulation time for the equilibrium state (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.01</td>
<td>125 to 145</td>
</tr>
<tr>
<td>2</td>
<td>0.05</td>
<td>85 to 100</td>
</tr>
<tr>
<td>3</td>
<td>0.10</td>
<td>85 to 100</td>
</tr>
</tbody>
</table>

**Table 5.6**: required simulation time associated with the $k_s$ values (Note that 80 days are equivalent to one year of the wave action).

3. **Discussion of the equilibrium state results**

Figures (5.15, 16 and 17) reveal the morphological evolution at the equilibrium state for the various bottom roughness values. It can be observed that for the bottom roughness of 0.05 m and 0.10 m, the contour lines have an irregular shape. The irregularity of the contour lines for $k_s$ equal to 0.10 m is much more than for $k_s$ equal 0.05 m. The reason is due to the small velocity which has a slight effect on the wave computations. Although the wave-current interaction is included, there is also some irregularity in the contour lines for $k_s$ equal to 0.01 m.

A scour hole can be found at the down-stream side of the breakwater extremity. In the field investigation, Fig. (5.2), this scour hole is not present. The reason for the scour hole generation is due to the effect of the circulation cell velocity which is generated down-stream side of the breakwater, Fig. (5.6, 7 and 8), combined with the stirring effect of the waves. The reason for the absence of the scour hole in reality may be that the waves are acting from various directions causing that the scour hole is filled up due to a sediment transport coming from other directions. The morphological evolution of the bottom roughness of 0.01 m and 0.05 m shows tombolo formation which is in agreement with the field observation, Fig. (5.2, 15 and 16). This is not true for $k_s$ equal to 0.10 m since no tombolo is created, Fig. (5.17).

There is an evidence for the collapsed seawall, at the up-stream side of the breakwater, is that due to the circulation cell generated at the up-stream side of the breakwater combined with the wave action (stirring effect). The circulation cell migrates toward the up-stream side.
causing the shoreline erosion which reached to the seawall position. Figure. (5.18) shows that
the ratio of the trapped sediment is less than 55 %. That is due to the ratio of the breakwater
length to the offshore distance of the breakwater is greater than two \((390/140 = 2.80)\). From
the sensitivity analysis, Chapter 4, for the \(L_B/Y_B\) ratio greater than two the model
underestimates the sand volume behind the breakwater. The reason was explained before in
Chapter 4.

5.2.6 Model calibration
As mentioned before the shoreline from the model computation at the up-stream side of the
breakwater is compared with the measured shoreline from the field observation. Figure.
(5.19) shows this comparison in which the measured shoreline from the field observation has
a specific error which is not known. For \(k_s = 0.10\) m, the shoreline is very irregular but for
\(k_s = 0.01\) m and 0.05 m the shoreline is a bit regular and almost matches with the measured
shoreline from the field observation. However, there are some variations which are due to
the difficulty in reality to measure the exact position of the shoreline because of the swash
effect so that the measured shoreline is presented with an uncertain error. It is found that the
shoreline is eroded outside the area influenced by the breakwater in the model computations.
That is perhaps due to the boundary description of the model. Accordingly, the boundary
definition will be changed in the two-wave conditions simulation to check the erosion of the
shoreline outside the area influenced by the breakwater.

Further upstream of the breakwater at 140 m and 280 m, cross-sections are available to make
another comparison between the model and the field data. Figure. (5.20) reveals that the
profile for \(k_s = 0.10\) m is irregular close to the shoreline but the computed profile
approximately matches the measured profile for all \(k_s\)-values. Generally speaking the
variations in the profile for the different bottom roughness values are not such big. On the
other hand, it is rather difficult to do such calibration between two-dimensional model and
nature because the swash, the undertow and the wind effects are not included in the model.
Additionally, the grain sizes of the bottom material \((d_{50} = 587 \, \mu m \text{ and } d_{90} = 1127 \, \mu m)\)
are rather high to take the bottom roughness equals to 0.01 m. Meaning that the bottom
roughness is dependent of the grain roughness and the ripple roughness which is higher than
0.01 m. Therefore, in the simulation with two wave conditions the bottom roughness is taken equal to 0.05 m.

5.3 Simulation using two-wave condition

5.3.1 Introduction

In this simulation, the waves are acting from two different directions, see the sketch below. The first one generates a longshore current in the same way as for the simulation with one-wave condition which is the direction of the net sediment transport. The second one generates the longshore current in the opposite way of the first one. Two-wave condition affect the morphological evolution. The intention is to simulate the natural behaviour as good as possible. Consequently, it is required to develop a process tree in the main module which can proceed this simulation. In this part the input data related to two-wave condition are mentioned. Meaning that the input data which are mentioned in the previous part, are not repeated.

Sketch represents the used wave conditions and the used cross-sections for the comparison
5.3.2 Main module input

To simulate the wave actions of different wave conditions, a process tree is developed. Figure (5.21) reveals the process tree used. The process tree consists of 18 nodes which are enumerated in a circle and 17 controllers which are enumerated in a square. The elementary subprocesses are enumerated to be 10. The model executes these 10 elementary subprocesses by satisfying the controllers requirements. Wave-current interaction is included in this process tree. For the first wave action, the wave and current computation is made by the elementary subprocesses 1 and 2 while the elementary subprocesses 6 and 7 are described to execute the wave and current computation for the second wave action. Bearing in mind that only one flow field can be written to the communication file. Accordingly, it is specified in the elementary subprocesses 1 and 6 that the flow field is assumed to be zero. This option is made to ignore the effect of the flow field of a specific wave direction, on the other wave computations. Moreover, the elementary subprocess 3 and 8 are described to execute the wave computations taking the flow field into account. The elementary subprocess 4 and 9 are described to execute the flow field corresponding to the wave direction. Furthermore, the elementary subprocess 5 and 10 are described to execute the transport and the bottom computations. For the first wave action the elementary subprocesses from 1 to 5 are executed and from 6 to 10 for the second wave action, that is including wave-current interaction in the morphological computation.

Controllers have to be specified which are dependent on the type of computation executed, either for the initial state simulation or for the equilibrium state computation. For the initial state simulation, all the controllers are prescribed to execute for one time except for the controllers which are used to control the flow computation. It is prescribed in the controllers 2, 4, 7, and 9 that the computation stops after 5.0 times of iteration or until the change of the resultant velocity is less than 0.01 m/s in all grid points. For the equilibrium state simulation, it is required to prescribe the simulation time in the sense that the two wave actions are acting by the same occurrence (40 days for each wave direction). It is prescribed in controller 17 that the simulation time is equal to 90 day. It was found by applying the optimal time option that the computed optimal time of the second wave condition is slightly
larger than the optimal time of the first wave condition. It is found by prescribing 20 times of execution for controller 5 (first wave condition) and 13 times of execution for controller 10 (second wave condition) that the desired percentage of occurrence is approximately achieved. That could be a drawback of this process tree. However, the effect of the first wave action run lasts for about half day before the effect of the second wave action run. That means each wave run is lasting for about half day for the morphological computation. The evaluated morphology affects the wave forces generated by the other wave direction.

The initial state of the first wave condition is made separately by making another process tree including wave-current interaction, Fig. (5.22). The description of the controller is written on each controller. The elementary subprocesses 1 and 2 are prescribed to execute wave and current computations for wave-current interaction purpose for the wave computation in the elementary subprocess 3. The elementary subprocess 4 is imposed to execute the flow computation in condition that the computation stops after 5.0 times of iteration or until the change of the resultant velocity is less than 0.01 m/s in all grid points. The other controllers are set to execute one time.

5.3.3 Wave and flow module input

The dimensions of the area are 650 m by 3000 m. The dimension in the cross-shore direction is reduced to make the shoreline close to the area boundary. That is to overcome the erosion of the shoreline which occurred further from the breakwater in the one-wave condition simulation. The grid size is taken equal to that of the previous simulation.

The wave data of the two-wave condition, based on 1D-computation (UNIBEST, see appendix A), are imposed on the seaward boundary.

The wave data of the first wave condition are:
- $H_s = 2.60$ m, $T_p = 5.68$ sec, and $\phi = -29.30^\circ$ (clockwise from x-direction)
- The energy dissipation coefficients: $\gamma = 0.76$ and $\alpha = 0.50$

The wave data of the second wave condition are:
- $H_s = 1.00$ m, $T_p = 5.64$ sec, and $\phi = 15.0^\circ$ (anticlockwise from x-direction)
- The energy dissipation coefficients: $\gamma = 0.64$ and $\alpha = 0.50$

A single detached breakwater effect on the shore
For the flow module, the bottom roughness \((k_0)\) is equal to 0.05 m. The boundary conditions are evaluated as in section 5.2.3.

5.3.4 Results discussion

1. **Initial state simulation results**
   The process tree in Fig. (5.22) is applied to evaluate the effect of the first wave condition which has quite big wave height and wave angle compared to the second wave condition. For the second wave condition, the process tree in Fig. (5.21) is applied. It can be observed in Fig. (5.23, 24) that the first wave condition is more active than the second wave condition. The breakwater is located inside the surf zone for the first wave condition while it is located outside the surf zone for the second wave condition. It can be expected that the first wave condition has a dominating impact for the sediment accumulation behind the breakwater. That can be observed from the flow field in Fig. (5.25, 26) and the sediment transport field in Fig. (5.27, 28). The longshore current is higher for the first wave condition than for the second wave condition as well as for the sediment transport rates.

2. **Equilibrium state simulation results**
   It is found that the computational time for this simulation lasts about one week. The equilibrium state reached after about 93 days (about one year), Fig. (5.29). During this simulation, the wave action alters from the first wave condition to the second one. It is required from this simulation that the first wave action should act for 40 days as well as the second wave action. It is found that the first wave condition effect lasts about 39 days and the second one lasts 54 days. That is due to the computation of the optimal time step which is reversely proportional to the sediment transport rate. The period of 54 days (more than 40 days) for the second wave action is not important because the sediment transport rate generated by the second wave condition is quite small to make a big variation in the morphology. The time associated to the wave condition can be achieved by prescribing the fixed time step option in the transport module, but this description may affect the stability of the transport and the bottom computation. Fig. (5.29) shows the morphological evolution at the equilibrium state. The sand formation is tombolo which is in agreement with the nature of the study.
and the contour lines are relatively smooth. From Fig. (5.30), it can be observed that the trapped sediment ratio is less than 55%. It means that the model underestimates the sediment accumulation because the breakwater length to the offshore distance ratio ($L_b/Y_b$) is greater than two.

### 5.4 Comparison between one-wave and two-wave conditions simulation

For the optimal time step estimated by the transport module, it is found after some time of the simulation that the grid point which controls the calculation of the optimal time step, based on the sediment transport rates, is positioned further from the region influenced by the breakwater. That is for the simulation with one-wave condition but this point is positioned in the region influenced by the breakwater for the simulation with two-wave condition. The computational time of the simulation with one-wave condition increases due to the irregular shape of the contour lines, Fig. (5.17), while for the simulation with two-wave condition the computational time mainly increases due to the calling times of HISWA and TRISULA modules. Moreover, the contour lines for the simulation with two-wave condition are hardly irregular, Fig. (5.29).

Another comparison can be observed from Fig. (5.30), the rate of the trapped sediment behind the breakwater for the one-wave condition simulation is higher, at the beginning, than for the two-wave condition simulation. The reason is due to the wave action of the second wave condition in the two-wave condition simulation which brings a rather low amount of the sediment to the sheltered area. It can be observed also from Fig. (5.30) that the curve for the one-wave condition simulation is relatively irregular. That is due to the irregularity in the contour lines which creates a bit disturbance. In both the one-wave and two-wave conditions simulations, eddies are generated close to the shoreline which are present along the shoreline and the region far away from the influence of the breakwater as well. That may be related to the model stability (not to the boundary description). The amount of the trapped sediment at the equilibrium state is more or less equal, Fig. (5.30).

Another comparison of the cross-shore profiles at sections (1-1) to (5-5) can be done (see the sketch in section 5.3.1). Sections 1-1, 5-5 are taken far away from the effect of the
breakwater. It can be observed from Fig. (5.31) that the cross-shore profiles are more or less the same regardless the small accretion which created due to the effect of the second wave condition in the two-wave condition simulation. Figure. (5.32) reveals the comparison between the cross-shore profiles at the extremities of the breakwater as well as at the middle of the breakwater. It can be noticed that the cross-shore profiles resulting from the two-wave condition simulation are smoother than the profiles from the one-wave condition simulation.

At section 2-2, Fig. (5.32), a scour hole is observed due to the effect of the circulation cell generated at the upstream side of the breakwater extremity. The resulting profile from the two-wave condition simulation is quite smoother than the one-wave condition simulation.

At section 3-3, Fig. (5.32), it can be noticed a scour hole at the seaward direction of the breakwater (the middle of the breakwater). This scour hole is present in the two-wave condition while it is absent in the one-wave condition simulation. In reality, this scour hole may be present because the breaking wave over the slope of the structure creates undertow which in turn generates the scour hole at the seaward direction of the breakwater. Consequently, the wave climate condition effect generates the sediment which in turn fills up the scour hole.

At section 4-4, Fig. (5.32), there is another scour hole generated by the circulation cell at the down-stream side of the breakwater. The scour hole at the down-stream side of the breakwater is bigger in the one-wave condition simulation than in the two-wave condition simulation. The reason is that the second wave condition in the two-wave condition simulation generates longshore sediment transport in the opposite direction of the net sediment transport direction which in turn fills up that scour hole. It is expected if the wave conditions increase in the 2D-model simulation (i.e, more than two-wave condition), the scour holes will be completely filled up by the sediment coming from the other directions.
Comparison between simulations with normally and obliquely incident waves

In this part, the comparison between the obliquely incident waves and normally incident waves is discussed. The field case of one-wave condition with $k_s = 0.05$ m is chosen for this purpose. For simulation with normally incident waves, the field case of one-wave condition is prescribed but the wave direction is specified to be normal to the shore. The objective of this part is to compare between the effect of the normally and the obliquely incident waves on the morphological evolution.

Figure. (5.33) reveals the relation between the volume of trapped sediment and the % of trapped sediment behind the breakwater to the time. It can be observed that the effect of the normally and obliquely incident waves are more or less identical. This emphasizes that the volume of the trapped sediment has to be achieved whatever the wave condition is. However, the shape of the shoreline for the simulation with the normally incident waves, Fig. (5.34), is different from the shoreline shape for the simulation with obliquely incident waves, Fig. (5.16). The reason is the presence of the longshore sediment transport in the simulation with obliquely incident waves and the equilibrium mechanism between the two cases is different.

5.5 Conclusion

It can be inferred that the two-wave conditions simulation gives a smoothed shape of the contour lines. On the other hand, the interaction between the two-wave conditions results a realistic morphological evolutions. Wave-current interaction can be included by using the process tree in Fig. (5.22) for the one-wave condition simulation. For the two-wave conditions simulation, the process tree in Fig. (5.21) can be used including wave-current interaction. The drawback of this process tree is the time associated to the wave direction is not achieved. However, it can be overcome by specifying the number of iterations of the transport and the bottom computations. Furthermore, the simulation for the morphological evolutions can be made for more than two-wave conditions. It means that the wave climate can be prescribed in the two dimensional simulation in condition that the time corresponding to the wave direction should be specified with a care.
CHAPTER 6
ONE-LINE MODEL SIMULATIONS AND QUALITATIVE RESULTS

6.1 Introduction
There is no doubt that the one-line model is a preferable tool because the cost and the consuming computational time are less than that of the two-dimensional and quasi-three dimensional models. This is only true under the condition that the one-line model gives reliable results. In this study, the one-line model UNIBEST is used which has been developed at Delft Hydraulics. A comparison between the UNIBEST model results and the DELFT2D-MOR model results is achieved. Furthermore, the offshore breakwater is schematized in the UNIBEST model to examine the shoreline response, either a salient formation or a tombolo formation, which is compared with the DELFT2D-MOR model results. Set-up of the one-line model (UNIBEST) and the comparison between the results of the two models are discussed in the following sections.

6.2 Set-up of UNIBEST model
UNIBEST-package is an acronym of UNIf orm BEach Sediment Transport. The software package is developed in order to simulate the longshore and cross-shore sediment transport processes and related morphodynamics of beach profiles and beach planform shapes (shoreline evolution). The UNIBEST coastal software package consists of four separate modules such as: UNIBEST-CL (a module for CoastLine dynamics), UNIBEST-LT (a module for Longshore Transport), UNIBEST-TC (a module for Time dependent Cross-shore transport) and UNIBEST-DE (a module for Dune Erosion during storm surges). In this study, UNIBEST-LT and UNIBEST-CL modules are used.
UNIBEST-LT module

UNIBEST-LT was designed to compute tide- and wave-induced longshore currents and sediment transports on any beach of an arbitrary profile. The tide is not present in this study. Therefore, the longshore current and sediment transport are only due to waves. In UNIBEST-LT the nearshore dynamics are computed by a built-in random wave propagation and decay model (Battjes and Stive, 1984). The model transforms offshore wave data to the shore, accounting for the principal processes of wave energy changes due to bottom refraction, shoaling and dissipation by wave breaking and bottom friction.

The longshore current distribution across the beach profile is derived from the momentum equation alongshore accounting for the bottom friction and the gradient of radiation stress, equation (6.1) excluding the effect of the tide.

\[
\frac{dS_{xy}}{dx} + \rho_w \frac{g}{C^2} V |V_{tot}| = 0
\]  

in which :

- \( S_{xy} \) = radiation shear stress acting on a plane normal to the shore (N/m)
- \( V_{tot} \) = \( \sqrt{V^2 + U_{rms}^2} \) (m/s)
- \( V \) = longshore current (m/s)
- \( U_{rms} \) = orbital velocity (m/s)
- \( C \) = Chezy coefficient (m\(^{1/2}\)/s)
- \( g \) = gravitational acceleration (m/s\(^2\))
- \( \rho_w \) = water density (kg/m\(^3\))

Wave-current interaction is not included in the computations and the Chezy’s coefficient is computed from the following formula:

\[
C = 18 \log \left( \frac{12h}{k_s} \right)
\]  

The longshore transport rate and its cross-shore distribution can be evaluated according to several total-load sediment transport formulae. The sediment transport formulae which have
been implemented in the model are; CERC, Engelund-Hansen, Bijker, Van Rijn, Bailard and Van der Meer-Pilarczyk formula. In this study, the Bijker's formula is used as in the two-dimensional model computations. More details about UNIBEST-LT module can be found in the UNIBEST manual.

**UNIBEST-CL module**

UNIBEST-CL was designed to compute the shoreline evolution due to longshore sediment transport gradients of an alongshore nearly uniform coast, on the basis of the one-line theory, which was first introduced by Pelnard-Considère (1956).

The shoreline is schematized as a single line and the displacement of this line is described as a function of time and longshore position. The dynamic area of the bottom profile moves in parallel to itself without altering its shape during the process of erosion or accretion. The theory of Pelnard-Considère gives the basic equations describing the shoreline evolution due to longshore sediment transport gradients. These equations lead to the well-known diffusion equation. Both initial and boundary conditions are needed to solve this equation for a specific problem. The x-axis is chosen along the original shoreline, while the shore-normal y-axis is chosen in a direction normal to the original shoreline pointing in offshore direction to create a right-hand coordinate system.

The shoreline evolution is simulated by solving the continuity equation and the equation of longshore transport as a function of the shoreline orientation.

The equation of continuity can be written as:

\[
\frac{h_p}{\partial t} + \frac{\partial Q_s}{\partial x} + q_b = 0
\]  

(6-3)

in which:

- \(Q_s\) = total longshore transport (m\(^3\)/year)
- \(y\) = shoreline position (m)
- \(h_p\) = active profile height (m)
- \(q_b\) = lateral sediment transport, source or sink (m\(^3\)/year/m)

The longshore transport can be written as:
\[ Q_s(\theta) = Q_{s0} - s_1 \frac{d y}{d x} \] 

(6-4)

in which:

- \( Q_s(\theta) = \) longshore transport as a function of the shoreline orientation (m³/year)
- \( Q_{s0} = \) longshore transport along a straight shoreline parallel to the x-axis (m³/year)
- \( s_1 = \) variation of the transport as a function of the shoreline orientation (m³/year)
- \( \theta = \) shoreline orientation with respect to the x-axis (°)

Equation (6-4) is known as the equation of Pelnard-Considéré. When equation (6.4) is substituted in equation (6.3), the following equation is obtained which is the so-called diffusion equation for which analytical solutions can be found.

\[ \frac{\partial y}{\partial t} = \frac{s_1}{h_p} \frac{\partial^2 y}{\partial x^2} \] 

(6-5)

In UNIBEST the sediment transport rate is defined as follows:

\[ Q_s(\theta) = c_1 (\theta - \theta_e) e^{-(c_2 (\theta - \theta_e))^2} \] 

(6-6)

in which:

- \( \theta = \) shoreline orientation with respect to the x-axis (°)
- \( \theta_e = \) shoreline angle when the sediment transport equals to zero (°)
- \( c_1 \) and \( c_2 = \) coefficients determined by least square method, based on number of transport computations for different shoreline (for a selected transport formula) \( c_1 \) (m³/year/°) and \( c_2 \) (1/°)

The shoreline progression is then determined on the basis of the sediment balance of grid cells. Gradients in the longshore transport, resulting in shoreline displacement, can occur due to a curved coast (\( \theta \) is not uniform along the coast) or due to non-uniform coefficients \( c_1 \) and \( c_2 \) along the coast (gradients in the wave climate).
6.3 Computations using the one-line model (UNIBEST)

First of all, a computation for the wave transformation, longshore current and sediment transport rate is made to compare the results with the two-dimensional model results. The computation of the UNIBEST-LT module is done for the field case with one-wave condition and bottom roughness equal to 0.01 m. Another computation is made for the same case using the DELFT2D-MOR model.

The flow module of DELFT2D-MOR can be operated with different bottom shear stress formulations to account for the wave-current induced bottom shear stress. Recently, it was found that the choice of the bottom shear stress has a considerable influence on the flow field, (Soulsby et al 1993a and 1993b), which in turn affects the sediment transport rate. Soulsby et al. have made an intercomparison between the various formulations and observed that the Fredsoe (1984) formulation appeared to perform quite well while the Bijker-method (1967), which has been used for the DELFT2D computations, gives a considerable overestimation of the bottom shear stresses. As well, it was recently found that the Fredsoe (1984) formulation gives realistic results compared to the field observation (Bosboom, 1996). However, this formulation may not be relevant for the other situations.

6.3.1 Input data for the UNIBEST-LT

In UNIBEST-LT the cross-shore profile (far away from the structure) which has been used in the two-dimensional model computation, Table (5.1), is specified here with 10 m grid size. It is specified in the wave scenario in UNIBEST-LT that one-wave condition is acting for 80 days a year. The significant wave height is equal to 1.85 m, wave period is equal to 5.8 sec and the wave direction is equal to -24° (with respect to an axis ,y-direction in UNIBEST model, normal to the shoreline). The sediment transport formula used is Bijker's formula. The sediment input data for the Bijker’s formula is taken the same as in the two-dimensional model computations.
6.3.2 Comparison between the results from UNIBEST-LT and DELFT2D-MOR

Figure (6.1) shows the comparison between the wave height, longshore current and the sediment transport rate results from the UNIBEST-LT module and the DELFT2D-MOR model. BR67 stands for the Bijker (1967) bottom shear stress formulation which has been used in the previous computations. Additionally, a computation is done using Fredsoe (1984), FR84, bottom shear stress formulation. It can be observed that the choice of the bottom shear stress formulation has no influence on the wave height results of the 2D-model computation. Moreover, the wave height results of the UNIBEST-LT module are more or less the same as the DELFT2D-MOR results.

Concerning the longshore current and longshore sediment transport results, it can be observed from Figure. (6.1) that there are quite big variations between the results from the one-line model and the 2D-model as well as between the 2D-model results. The maximum longshore current velocity resulting from the various computations are presented in Table. (6.1).

<table>
<thead>
<tr>
<th>Run no.</th>
<th>Used Model</th>
<th>Longshore current (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>DELFT2D-MOR using BR67</td>
<td>0.40</td>
</tr>
<tr>
<td>2</td>
<td>DELFT2D-MOR using FR84</td>
<td>0.54</td>
</tr>
<tr>
<td>3</td>
<td>UNIBEST-LT</td>
<td>0.87</td>
</tr>
</tbody>
</table>

Table. (6.1) : maximum longshore current of the different models.

It can be noticed that the velocity using FR84 formulation is 35% higher than the velocity using BR67 formulation in the DELFT2D-MOR. The reason is that, the bottom shear stress of Bijker (1967) is overestimated resulting in more flow resistance. For UNIBEST results, the longshore current is much higher than the DELFT2D-MOR results. The reason is strictly due to the different longshore current formulations used in the UNIBEST model, equation (6.1). In UNIBEST model, the longshore current formulation does not contain the effect of the wave-induced shear stress which in turn increases the flow resistance. In spite of all that, it is difficult to choose the most realistic longshore current because there are no available...
field measurements.

As a result of the velocity deviations between the models, it is found that the computed sediment transport rates also show rather large deviations, Table (6.2).

<table>
<thead>
<tr>
<th>Run no.</th>
<th>Model used</th>
<th>Long. sediment transport rate $(10^{-5} \text{ m}^3/\text{s}/\text{m})$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>DELFT2D-MOR using BR67</td>
<td>4.30</td>
</tr>
<tr>
<td>2</td>
<td>DELFT2D-MOR using FR84</td>
<td>6.40</td>
</tr>
<tr>
<td>3</td>
<td>UNIBEST-LT</td>
<td>21.1</td>
</tr>
</tbody>
</table>

Table (6.2) : maximum sediment transport rates from the different models.

The longshore sediment transport is integrated over the 80 days representing one year and over the cross-shore distance (surf zone), Table (6.3).

<table>
<thead>
<tr>
<th>Run no.</th>
<th>Model used</th>
<th>Long. sediment transport rate (m$^3$/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>DELFT2D-MOR using BR67</td>
<td>46,000 (excl. pores)</td>
</tr>
<tr>
<td>2</td>
<td>DELFT2D-MOR using FR84</td>
<td>68,000 (excl. pores)</td>
</tr>
<tr>
<td>3</td>
<td>UNIBEST-LT</td>
<td>113,000 (excl. pores)</td>
</tr>
</tbody>
</table>

Table (6.3) : integrated longshore sediment transport rates from the different models.

The UNIBEST model overestimates the sediment transport which is due to the higher velocity.

It can be expected that the morphological equilibrium state computed by the UNIBEST-model may be reached very fast due to the overestimation of the sediment transport rates.
6.4 UNIBEST-CL simulations for the morphological evolution

In this part, the UNIBEST-CL module which accounts for the shoreline evolution, is used. The aim is to compute the tombolo formation and to compare these results with those of the 2D-model. Three cases are selected which have been examined by the 2D-model. The choice is based on the tombolo formation generated in these cases. These three cases are the breakwater situated at 120 m offshore and 300 m offshore with the normally incident wave, the third one is the field case with obliquely incident waves (one-wave condition and $k_x = 0.05$ m).

Problem schematization

In UNIBEST-CL module, the local sediment transport is specified in a table for the points indicated in the sketch below (breakwater domain). This table contains the parameters of the S-Φ curve of each selected point behind the breakwater, prescribing the relation between the sediment transport rate and the shoreline orientation based on the diffracted waves behind the breakwater. The S-Φ curve parameters are $c_1$, $c_2$, and the shoreline orientation at zero sediment transport rate (equilibrium shoreline angle, $\theta_e$). The wave conditions, such as wave height and wave direction as well as the wave period, are picked up from the 2D-model results and analyzed to compute the S-Φ curve parameters. Outside the breakwater domain, a file is defined which contains the sediment transport rate and the shoreline orientation relationship (S-Φ curve). The longshore sediment transport at the model boundaries is zero (normally incident waves). The wave condition for this case is picked up as well from the 2D-model including the wave dissipation.

For the breakwater situated at 120 m offshore, the points for defining the local sediment transport table parameters are selected just behind the breakwater while they are selected at the breaker line for the breakwater situated at 300 m.

The same schematization is used for the field case with obliquely incident wave. The selected points are located just behind the breakwater. More details about this schematization can be found in the UNIBEST manual.

6.8 A single detached breakwater effect on the shore
Problem schematization for the normally incident wave

6.4.1 UNIBEST-CL simulations for normally incident wave

For the breakwater situated at 120 m offshore

Input data

The length of the area used in the 2D-model simulation, is prescribed here with 20 m grid size alongshore. It is defined at the boundaries of the area that the sediment transport rate is equal to zero. The local sediment transport table which comprises the coefficients of $c_1$, $c_2$ and $\theta_e$ (equation 6.5) is described. These coefficients may vary over the x-domain along the model, Table. (6.4). The wave conditions at the selected points are picked up from the computation of the initial state. Inside the breakwater domain, the coefficients are linearly interpolated. Outside this domain, the coefficients of the file described are prescribed.
One-line model simulations and qualitative results

<table>
<thead>
<tr>
<th>Longshore distance (m)</th>
<th>$\theta_e$ (°)</th>
<th>$c_1$ (m³/year°)</th>
<th>$c_2$ (1°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>805</td>
<td>-0.300</td>
<td>-0.0182</td>
<td>0.0149</td>
</tr>
<tr>
<td>865</td>
<td>0.240</td>
<td>-0.0185</td>
<td>0.0149</td>
</tr>
<tr>
<td>905</td>
<td>20.27</td>
<td>-0.0065</td>
<td>0.0167</td>
</tr>
<tr>
<td>925</td>
<td>51.40</td>
<td>-0.0002</td>
<td>0.0178</td>
</tr>
<tr>
<td>1055</td>
<td>0.000</td>
<td>0.000</td>
<td>0.0000</td>
</tr>
<tr>
<td>1175</td>
<td>-51.40</td>
<td>-0.0002</td>
<td>0.0206</td>
</tr>
<tr>
<td>1195</td>
<td>-20.20</td>
<td>-0.0071</td>
<td>0.0186</td>
</tr>
<tr>
<td>1225</td>
<td>-0.800</td>
<td>-0.0191</td>
<td>0.0163</td>
</tr>
<tr>
<td>1285</td>
<td>0.330</td>
<td>0.0191</td>
<td>0.0163</td>
</tr>
<tr>
<td>1325</td>
<td>0.000</td>
<td>0.0191</td>
<td>0.0163</td>
</tr>
</tbody>
</table>

Table. (6.4) : local sediment transport table parameters for the breakwater at 120 m offshore.

Figure. (6.2) indicates the S-Φ curve which is used outside the breakwater domain.

**Simulation results**

The wave duration for this case is assumed to be 50 days (sensitivity computations shown equilibrium after 20 days). Results of the shoreline evolution are shown after 4.0 days, 20 days and 50 days of simulation time in Figures (6.3, 6.4 and 6.5) respectively. Figure (6.3) shows the longshore transport distribution and shoreline position along the longshore direction. It is clear that the double salient is formed after 4.0 days of wave actions. Figures (6.4) and (6.5) demonstrate the shoreline evolution at the equilibrium state whereas the equilibrium state is achieved after 20 days of wave actions. This can be observed from the upper part of Figure (6.4) and (6.5) that the sediment transport gradients in the longshore direction is small. Also it can be conclusively seen in Figure (6.5) that the adjacent shore zone is eroded. However, the tombolo is not formed. The effect of the circulation cells which appears in the 2D-model simulation seems to play an important role in the formation of the tombolo for this case with normally incident waves. In the one-line simulation this effect is

6.10 A single detached breakwater effect on the shore
not included, and the shoreline development stops after equilibrium coastline angles have
been reached. Additionally, the sediment trapped behind the breakwater comes mostly from
shoreline erosion. The controlling factor for the equilibrium state in the 2D-simulation is the
effect of the circulation cells, but in the one-line model simulation the equilibrium coast angle
is the predominant factor.

*For the breakwater situated at 300 m offshore*

**Input data**

The same area is used as in the previous simulation but the location of the selected points
behind the breakwater is now chosen at the breaker line, about 100 m (approximately equals
to the wave length) landward of the breakwater. The wave conditions are picked up from the
2D-model results and analyzed to evaluate the S-Φ curve outside the breakwater domain, Fig.
(6.6), and to determine the coefficients of the local transport table.

The coefficients of the local transport table are determined from the wave conditions in the
initial state as well as in the equilibrium state, Tables (6.5, 6.6).

<table>
<thead>
<tr>
<th>Longshore distance (m)</th>
<th>$\theta_e$ (°)</th>
<th>$c_1$ (m³/year/°)</th>
<th>$c_2$ (1/°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>800</td>
<td>0.500</td>
<td>-0.0725</td>
<td>0.0148</td>
</tr>
<tr>
<td>850</td>
<td>8.300</td>
<td>-0.0678</td>
<td>0.0154</td>
</tr>
<tr>
<td>900</td>
<td>16.20</td>
<td>-0.0487</td>
<td>0.0162</td>
</tr>
<tr>
<td>950</td>
<td>13.60</td>
<td>-0.0224</td>
<td>0.0167</td>
</tr>
<tr>
<td>1000</td>
<td>8.600</td>
<td>-0.0178</td>
<td>0.0166</td>
</tr>
<tr>
<td>1050</td>
<td>-1.400</td>
<td>-0.0195</td>
<td>0.0161</td>
</tr>
<tr>
<td>1100</td>
<td>-9.900</td>
<td>-0.0182</td>
<td>0.0166</td>
</tr>
<tr>
<td>1150</td>
<td>-14.90</td>
<td>-0.0258</td>
<td>0.0166</td>
</tr>
<tr>
<td>1200</td>
<td>-15.40</td>
<td>-0.0526</td>
<td>0.0161</td>
</tr>
<tr>
<td>1250</td>
<td>-6.500</td>
<td>0.0695</td>
<td>0.0153</td>
</tr>
<tr>
<td>1300</td>
<td>0.800</td>
<td>-0.0780</td>
<td>0.0149</td>
</tr>
</tbody>
</table>

**Table. (6.5)**: local sediment transport table parameters for the breakwater at 300 m offshore
(estimated from the initial wave condition).
The reason is to investigate the effect of modified waves behind the breakwater due to bottom changes. Three runs are made; the first one using the initial wave condition, the second one using the final wave condition and the third one using the initial and the final wave conditions.

**Simulation results**

**Run 1 (using the initial wave condition)**

In the case of the breakwater situated at 300 m offshore, the wave duration is assumed to be 75 days. It can be observed in Figure (6.7) that a salient is formed instead of a tombolo after 75 days of wave actions (equilibrium state). Also, it can be conclusively observed that the equilibrium state is reached because the sediment gradients are zero, Fig. (6.7).
Run 2 (using the final wave condition)
Results are shown after 10 days and 75 days of simulation time in Figures (6.8) and (6.9) respectively. Figure (6.8) reveals that a double salient is formed as well as erosion at the adjacent shore zone after 10 days of wave actions. Figure (6.9) demonstrates the shoreline evolution after the equilibrium state is reached (longshore transport gradients are small). A salient is formed but the shape of the shoreline evolution is different from that of the previous run. This is due to the different equilibrium coast angle, Table (6.6).

Run 3 (using the combination of the initial and final wave conditions)
This run has been done because the modified wave patterns during the morphological evolution are not incorporated in the used one-line model. However, the sand formation is salient as well, Figure (6.10), in which the equilibrium state is reached after 150 days. The formation of the tombolo is not generated because the effect of the circulation cells is not included in the UNIBEST-CL module.

6.4.2 UNIBEST-CL simulations for obliquely incident wave (field case)
Input data
The field case of a single detached breakwater situated at the touristic village of Sidi Krir is simulated by the UNIBEST-CL. The case of an equivalent one-wave condition and bottom roughness which is equal to 0.05 m is selected to be simulated. The whole area used in the 2D-simulation is used in the one-line model simulation as well. The grid size used is 15.0 m alongshore. The selected points for defining the sediment transport table coefficients in the breakwater domain are chosen just landward of the breakwater. The wave conditions are picked up from the 2D-model results including the wave dissipation. The region outside the breakwater domain is described by the $S-\Phi$ curve, Fig. (6.11). The sediment transport rate of 100,000 m$^3$/year is imposed at the model boundaries ($x = 0$), Fig. (6.11). The coefficients of the local transport table are prescribed, Table (6.7).
Table (6.7): local sediment transport table parameters for the breakwater domain of the field case.

From Table (6.7), it can be expected that the tombolo formation will be generated. That comes from the $c_1$-values at the middle of the breakwater (1455 m - 1595 m) which are equal to zero. Meaning that the sediment transport abruptly reduces to zero (just behind the breakwater) so that the sediment transport will be blocked; whereas outside the breakwater domain the longshore transport is 100,000 m$^3$/year.

Simulation results
Results after 8.0 days and 80.0 days of simulation time are shown in Figures (6.12) and (6.13). After 8.0 days of wave actions it can be observed that the shoreline is accreted at the up-stream side of the breakwater and eroded at the down-stream side of the breakwater, Fig. (6.12). After 80.0 days of wave actions, the accretion increased at the up-stream side of the breakwater as well as the severe erosion at the down-stream side of the breakwater, Fig. (6.13). That comes from the tombolo formation which blocks the longshore sediment transport to move behind the breakwater. As a result of the tombolo formation, the shoreline
at the up-stream side of the breakwater is accreted and is eroded at the down-stream side of
the breakwater. This situation is similar to the blocking effect of a groyne.

This process here at the beginning did not happen in the 2D-model simulation because of the
effect of the circulation cells at the up-stream tip and down-stream tip. The effects of these
cells create a local sediment transport rate at the adjacent nearshore which in turn causes the
erosion of the adjacent shore zone. Because of the absence of the circulation cells in the one-
line model, the erosion at the up-stream side of the breakwater as computed by the 2D-model
does not exist.

6.5 Conclusion
It is concluded that the one-line model, UNIBEST-CL module, is less suitable to generate
the tombolo formation, for normally incident waves, which is mainly formed due to the
effects of the circulation cells whereas the equilibrium shoreline generation mechanism of
UNIBEST is based on equilibrium shoreline angles. This approach can be improved by
transferring it to a n-line module in which the flexibility for including the cross-shore
sediment transport is present and for which different degrees of wave shielding can be
imposed for different zones of the coastal profile. Tombolo formation can be predicted for
a case with significant longshore transport along the adjacent coast. On the other hand, the
longshore current formulation should be justified, taking into account the wave-current
induced shear stress.
CHAPTER 7
CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

7.1.1 Two-dimensional model (DELT2D-MOR)

Based on a comparison between the results of the diffraction computations made by the DIFFRAC model (accounting for the diffraction phenomenon) and HISWA (accounting for the directional spreading effect), it can be concluded that the difference in the wave height between the DIFFRAC and HISWA computations is small and decreases for locations further landward from the breakwater. The wave height behind the breakwater, either in the DIFFRAC computation or the HISWA computation, is relatively small. From morphological point of view the differences in the wave height behind the breakwater from HISWA are expected to have only a little influence on the morphological computation. Accordingly, the comparison between the two models gave a confidence for the applicability of HISWA in the morphological simulation. However, it may not be valid for the wave computations behind the breakwater which is used for establishing a marina since the wave height is underestimated by HISWA.

From the simulations made with wave-current interaction and without wave-current interaction, it can be concluded that the wave-current interaction has a strong influence on the hydrodynamic processes giving smoother bottom contour lines. The effect of the wave-current interaction is a redistribution of the wave height behind the breakwater.

After having made the sensitivity analysis on the impact of the structure geometry on the hydrodynamic processes as well as on the morphological evolutions, it can be inferred that the breakwater geometry, especially the variation of the offshore distances, has an influence on the wave pattern, the flow pattern and the sediment transport pattern. As a result of this impact the morphological evolutions change.

Based on the sensitivity analysis, it can be concluded that a tombolo forms when:

\[
\frac{L_B}{Y_B} \geq 1.0
\]  

(7-1)

in which:

A single detached breakwater effect on the shore
Conclusions and recommendations

Chapter 7

\[ L_a = \text{breakwater length} \]
\[ Y_B = \text{offshore distance of the breakwater relative to the original shoreline} \]

while a salient is formed when:

\[ \frac{L_B}{Y_B} < 1.0 \]  \hspace{1cm} (7-2)

Additionally, the percentage of the trapped sand behind the breakwater for the tombolo formation is found to be between 55% to 65% of the available volume. The validity of the model has been checked by comparing its results to results from the literature as well as field data. It is found that the model results are in a good agreement with the literature results and the field data. Therefore, it can be inferred that the model is reasonably valid for predicting the morphological evolution caused by an offshore breakwater although it should be realized that some effects are not included in the 2D-model such as the swash effect and the cross-shore current mechanism.

The bottom roughness in the field case study appeared to affect the hydrodynamic processes such as the wave field, the flow field and the sediment transport field at the area influenced by the breakwater. It can be concluded that the variation of the bottom roughness affects the morphological evolution. It can also be concluded that the sediment transport rate increases when the bottom roughness increases until a specific value of bottom roughness while after this value the sediment transport rate decreases. The irregularity of the morphology increases when the bottom roughness increases. On the other hand, the choice of the wave-induced shear stress was found to affect the hydrodynamic processes as well.

Applying a two-wave condition in the 2D-model computations gives a step forward in modelling the morphological evolution. Meaning that the resulting morphological evolution due to the application of two-wave condition is quite smooth. The scour hole generated by a specific wave condition is filled up by the other wave condition as well as the shoreline erosion generated by a specific wave condition is accreted by the other wave condition. A conclusion from the comparison between the simulation with one-wave condition and the simulation with two-wave condition, can be come to the enhancement of the morphological

7.2 A single detached breakwater effect on the shore
evolution using two-wave condition. Additionally, more than two-wave condition can be applied in the 2D-model computations in which the resulting morphology is realistic.

From economical point of view the 2D-model computations are more expensive than the 1D-model computations because the operation of the 2D-model needs a work station. On the other hand, the time required to prepare and run the 2D-model computations, regardless the 2D-model computations complexity, is quite high compared to the 1D-model computations.

7.1.2 One line model (UNIBEST)

It can be concluded from the comparison between the results of the 2D-model and the results of UNIBEST, that the longshore current and transport are larger in the UNIBEST model. This is because the effect of the wave-current interaction resulting in a larger (apparent) bed-roughness is missing.

Focusing on the morphological computations, UNIBEST model is less suitable for simulating the case with normally incident waves in which a tombolo is formed because the tombolo is formed due to the effect of the circulation cells. Since the equilibrium shoreline generation mechanism of UNIBEST is based on equilibrium shoreline angles. However, the UNIBEST model is able to simulate the case with obliquely incident waves to create the tombolo formation but the shoreline configuration is not the same as computed in the 2D-model because of the circulation cells effect.

From economical point of view the UNIBEST model is quite cheaper than the 2D-model since it operates in the personal computer. The difficulty, preparation and computation which appear using the 1D-model can be solved in short time.
7.2 Recommendations

7.2.1 Two-dimensional model (DELFT2D-MOR)

It is advised to include the wave-current interaction in the morphological evolution in which the resulting morphology is realistic. It is recommended to investigate the existence of the eddies generated close to the shoreline which in turn may cause the shoreline irregularity as well as the shoreline erosion at the region which is not influenced by the breakwater. The wave transmission should be investigated as well. That is by making a sensitivity analysis on the variation of the breakwater height. It is suggested to do another sensitivity analysis on the shape of the breakwater itself.

It is recommended to investigate the wave height distribution behind the breakwater by comparing the results with the field data for marina establishment purpose. Because of the restriction of writing one flow field in the communication file, the computational time increases due to the calling times of the wave module and the flow module. Therefore, the need to write more than one flow field is required to increase the flexibility of the process tree development.

7.2.2 One-line model (UNIBEST)

It is recommended to improve the longshore current formula used in the UNIBEST model which can be done by implementing the wave-current induced shear stress and comparing the model results to the field measurements.

Because of the difficulty to simulate the tombolo formation in cases with normally incident wave in which the effect of the circulation cells are not present, it is recommended to transfer the model to a n-line model in which the flexibility for including the cross-shore sediment transport is present and for which different degrees of wave shielding can be imposed for different zones of the coastal profile.
References


Booij, N. and Holthuijsen, L. H., 1992, "HISWA user manual".


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Delft Hydraulics, 1994, "DIFFRAC user manual".

Delft Hydraulics, 1994, "TRISULA user manual".

Delft Hydraulics, 1994, "UNIBEST user manual".


A single detached breakwater effect on the shore
A single detached breakwater effect on the shore
number of defined points : 12  
minimum hard disk space (kb): 6012  
case : AHMED

number of boundary elements : 613  
maximum hard disk space (kb): 6614

Figure 4.1  
delft hydraulics
DIFFRAC wave height computation behind the breakwater isolines represent wave height (m)

DELFT HYDRAULICS

Q1800.22

SCALE 1:9000.

Fig.4.2
HISWA wave height computation behind the breakwater isolines represent wave height (m)

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:9000

Fig. 4.3
Comparison between HISWA and DIFFRAC computations of wave height behind the breakwater

DELFt HYDRAULICS

Fig. 4.4
Cross-section at the middle, area of 1300 x 700 and grid size = 5.0 m

Cross-section at the middle, area 2100 x 790 and grid size = 10.00 m

The effect of different grid size "Case without the existance of the breakwater"
Initial wave height $H_{rms}$ (m) Without wave-current interaction 120 m offshore distance

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Run 1 Fig. 4.6
Initial wave height $H_{rms}$ (m)
Without wave-current interaction
150 m offshore distance

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Run 2
Fig. 4.7
Initial wave height $H_{rms}$ (m)
Without wave-current interaction
200 m offshore distance

DELFT HYDRAULICS

DELFT2D  G1800.22
Scale 1:5000
Run 3  Fig. 4.8
Initial wave height $H_{rms}$ (m)
Without wave-current interaction
300 m offshore distance

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Run 4 Fig. 4.9
Initial wave height Hrms (m)
Without wave-current interaction
500 m offshore distance

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Run 5 Fig. 4.10
Initial flow field
Without wave-current interaction
120 m offshore distance

current = 1.0 m/s

DELFT HYDRAULICS

DELFT2D  Q1800.22
Scale 1:5000
Run 1  Fig. 4.11
Initial flow field
Without wave-current interaction
150 m offshore distance

current = 1.0 m/s
Initial flow field
Without wave-current interaction
200 m offshore distance

current = 1.0 m/s

DELFT HYDRAULICS
Run 3  Fig. 4.13
<table>
<thead>
<tr>
<th>Initial flow field</th>
<th>DELFT2D</th>
<th>Q1800.22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without wave-current interaction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>300 m offshore distance</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

`DELFT HYDRAULICS`
Initial flow field
Without wave-current interaction
500 m offshore distance

DELFT HYDRAULICS

DELFT2D  Q1800.22
Scale 1:5000
Run 5    Fig. 4.15
Initial sediment transport
Without wave-current interaction
120 m offshore distance

DELFT HYDRAULICS

Run 1  Fig. 4.16
Initial sediment transport
Without wave-current interaction
150 m offshore distance

DELFT HYDRAULICS
Initial sediment transport
Without wave-current interaction
200 m offshore distance

DELFT HYDRAULICS
Initial sediment transport
Without wave-current interaction
300 m offshore distance

DELFT2D  Q1800.22
Scale 1:5000
Run 4  Fig. 4.19
Initial sediment transport
Without wave-current interaction
500 m offshore distance

DELFT HYDRAULICS
Initial wave height $H_{\text{rms}}$ (m)
With wave-current interaction
120 m offshore distance

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Run 1 Fig. 4.21
Initial wave height $H_{rms}$ (m)
With wave-current interaction
150 m offshore distance

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Run 2 Fig. 4.22
Initial wave height $H_{rms}$ (m)
With wave-current interaction
200 m offshore distance

DELFT HYDRAULICS

DELFT2D  Q1800.22
Scale 1:5000
Run 3  Fig. 4.23
Initial wave height $H_{rms}$ (m)
With wave-current interaction
300 m offshore distance

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Run 4 Fig. 4.24
Initial wave height $H_{rms}$ (m)
With wave-current interaction
500 m offshore distance

DELFT HYDRAULICS

DELFT2D | Q1800.22
---|---
Scale 1:5000
Run 5
Fig. 4.25
Initial flow field
With wave-current interaction
120 m offshore distance

---

DELFT HYDRAULICS

DELFT2D  Q1800.22
Scale 1:5000
Run 1  Fig. 4.26

current = 1.0 m/s
Initial flow field
With wave-current interaction
150 m offshore distance

DELFT HYDRAULICS
Initial flow field
With wave-current interaction
200 m offshore distance

\[ \text{current} = 1.0 \text{ m/s} \]

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Run 3 Fig. 4.28
Initial flow field
With wave-current interaction
300 m offshore distance

DELFT HYDRAULICS

DELFT2D  G1800.22
Scale 1:5000
Run 4  Fig. 4.29
Initial flow field
With wave-current interaction
500 m offshore distance

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Run 5 Fig. 4.30
Initial sediment transport
With wave-current interaction
120 m offshore distance

DELFT HYDRAULICS
Initial sediment transport
With wave-current interaction
150 m offshore distance

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Run 2 Fig. 4.32
Initial sediment transport
With wave-current interaction
200 m offshore distance

DELFT HYDRAULICS
Initial sediment transport with wave-current interaction, 300 m offshore distance.

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Run 4 Fig. 4.34

1.5e-4 m³/m/s
Initial sediment transport
With wave-current interaction
500 m offshore distance

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Run 5 Fig. 4.35
Comparison between wave-current interaction and without wave-current interaction for offshore distance = 120 m
Comparison between wave-current interaction and without wave-current interaction for offshore distance = 150 m

DELFT HYDRAULICS

DELFT2D 01800.22
Run 2 Fig. 4.37
Comparison between wave-current interaction and without wave-current interaction for offshore distance = 200 m

DELFT HYDRAULICS

Run 3  Fig. 4.38
Comparison between wave-current interaction and without wave-current interaction for offshore distance = 300 m

DELFT HYDRAULICS

Run 4 Fig. 4.39
Comparison between wave-current interaction and without wave-current interaction for offshore distance = 500 m
Morphological evolution at the equilibrium state (50 days) for a breakwater situated at 120 m offshore.

DELFT HYDRAULICS

DELFT2D | Q1800.22
Scale 1:5000

Run 1 | Fig. 4.41
Morphological evolution at the equilibrium state (50 days) for a breakwater situated at 150 m offshore

DELFT HYDRAULICS

<table>
<thead>
<tr>
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<tr>
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<td>Fig.</td>
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Morphological evolution at the equilibrium state (50 days) for a breakwater situated at 200 m offshore
Morphological evolution at the equilibrium state (50 days) for a breakwater situated at 200 m offshore (without wave-current interaction)

<table>
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<td>4.44</td>
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DELFTHYDRAULICS
Morphological evolution at the equilibrium state (75 days) for a breakwater situated at 300 m offshore
Morphological evolution at the equilibrium state (75 days) for a breakwater situated at 500 m offshore

DELFT2D | Q1800.22
---|---
Scale | 1:5000
Run 5 | Fig. 4.46

DELFT HYDRAULICS
Time-volume and time-ratio relation for the various breakwater layouts from run 1 to run 5

DELFT HYDRAULICS

Run 1-5

Fig. 4.47
Comparison between literature results and model results

DELFT HYDRAULICS

Fig. 4.48
Morphological evolution at the equilibrium state (75 days) for a breakwater situated at 75 m offshore (without enlargement)

DELFT HYDRAULICS

DELFT2D Q1800.22

Scale 1:5000

Fig. 4.49
Morphological evolution at the equilibrium state (75 days) for a breakwater situated at 75 m offshore (with enlargement)

DELFT HYDRAULICS

DELFT2D | Q1800.22
Scale 1:5000

Fig. 4.50
Time-volume relation for a breakwater situated at 75 m offshore "with and without enlargement of the whole area"
Sidi Krir touristic village location. (HAI, 1995)
The bathemetric survey carried out by Hydraulics Research Institute, 1995

Fig. 5.2

DELFT HYDRAULICS

01800.22
Initial wave height $H_{rms}$ (m)
With wave-current interaction
For $ks = 0.01$ m

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Fig. 5.3
Initial wave height $H_{rms}$ (m)
With wave-current interaction
For $k_s = 0.05$ m

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Fig. 5.4
Initial wave height $H_{rms}$ (m)
With wave-current interaction
For $k_s = 0.10$ m

DELFT HYDRAULICS

DELFIT2D Q1800.22
Scale 1:5000

Fig. 5.5
Initial flow field
With wave-current interaction
For \( k_s = 0.01 \) m

\[ \text{current} = 0.50 \text{ m/s} \]
Initial flow field
With wave-current interaction
For $ks = 0.05$ m

$\rightarrow$ current = 0.50 m/s
Initial sediment transport
With wave-current interaction
For $ks = 0.01$ m

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Fig. 5.9
Initial sediment transport with wave-current interaction for $k_s = 0.05$ m

DELFT HYDRAULICS
Initial sediment transport
With wave-current interaction
For ks = 0.10 m

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000

Fig. 5.11
Comparison between wave height, velocity and sediment transport rate for different bottom roughness

**Fig. 5.12**
Comparison between wave height, velocity and sediment transport rate for different bottom roughness

DELFT HYDRAULICS

Section 2-2

Fig. 5.13
Comparison between wave height, velocity and sediment transport rate for different bottom roughness
Morphological evolution at the equilibrium state (140 day) for $k_s = 0.01$ m one-wave condition simulation

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000

Fig. 5.15
Morphological evolution at the equilibrium state (90 day) for $k_s = 0.05$ m one-wave condition simulation

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</thead>
<tbody>
<tr>
<td>Scale</td>
<td>1:5000</td>
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</tbody>
</table>

Fig. 5.16
Morphological evolution at the equilibrium state (90 day) for $k_s = 0.10$ m
one-wave condition simulation

**DELFT HYDRAULICS**

<table>
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</tr>
</thead>
<tbody>
<tr>
<td>Scale</td>
<td>1:5000</td>
</tr>
<tr>
<td>Fig.</td>
<td>5.17</td>
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</table>
Volume- and ratio-time relationship for different bottom roughness values

DELFT HYDRAULICS
Comparison between the field observation and the model results (shoreline at the upstream side of the breakwater). (Model Calib.)
Comparison between the field observation and the model results (sections at different location at US of the breakwater). (Model Calib.)

Fig. 5.20
The process tree used in the two-wave conditions simulation.
The process tree used for the initial state for the first wave direction (Hs = 2.60 m)
Initial wave height $H_{rms}$ (m)
With wave-current interaction
For the first wave condition

DELFT HYDRAULICS
Initial wave height $H_{rms}$ (m)
With wave-current interaction
For the second wave condition

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000

Fig. 5.24
Initial flow field
With wave-current interaction
For the first wave condition

---
DELFT HYDRAULICS

---
DELFT2D Q1800.22
Scale 1:5000
Fig. 5.25
Initial flow field
With wave-current interaction
For the second wave condition

DELFT HYDRAULICS

Fig. 5.26
Initial sediment transport with wave-current interaction for the first wave condition.

DELFT HYDRAULICS

Fig. 5.27
Initial sediment transport
With wave-current interaction
For the second wave condition

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Fig. 5.28
Morphological evolution at the equilibrium state (93 day) for $ks = 0.05$ m two-wave conditions simulation

<table>
<thead>
<tr>
<th>Cross-shore distance (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
</tr>
<tr>
<td>Longshore distance (m)</td>
</tr>
<tr>
<td>1000</td>
</tr>
</tbody>
</table>

original shoreline ($t = 0$)

**DELFT HYDRAULICS**

<table>
<thead>
<tr>
<th>DELFT2D</th>
<th>Q1800.22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scale 1:5000</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 5.29
Comparison between the volume- and ratio-time relationship for the one-wave and two-wave conditions simulations

DELFT HYDRAULICS

Fig. 5.30
Comparison between the cross-shore profiles (different locations) for the one-wave and two-wave conditions simulations

DELFTHYDRAULICS

Fig. 5.31
Comparison between the cross-shore profiles (different locations) for the one-wave and two-wave conditions simulations

DELFt HYDRAULICS

Fig. 5.32
Comparison between normally and obliquely incident waves for the field case of one-wave condition with $k_s = 0.05m$

DELFT HYDRAULICS

Fig. 5.33
Morphological evolution at the equilibrium state (90 day) for $k_s = 0.05$ m one-wave condition simulation (normal waves)

DELFT HYDRAULICS

DELFT2D Q1800.22
Scale 1:5000
Fig. 5.34
Comparison between the DELFT2D-MOR results and the UNIBEST-LT module results for the field case (one-wave condition, $k_s = 0.01$ m)
S-Phi curve used outside the breakwater domain in the simulation of the breakwater situated at 120 m offshore distance.
The shoreline evolution after 4.0 days of wave action (breakwater situated at 120 m offshore)

UNIBEST Q1800.22

Fig. 6.3
The shoreline evolution after 20 days of wave actions close to the equilibrium state (breakwater situated at 120 m offshore)

UNIBEST Q1800.22

DELFt HYDRAULICS

Fig. 6.4
The shoreline evolution after 50 days of wave action (breakwater situated at 120 m offshore)

DELFT HYDRAULICS

UNIBEST Q1800.22

Fig. 6.5
S-Phi curve used outside the breakwater domain in the simulation of the breakwater situated at 300 m offshore distance.

UNIBEST Q1800.22

DELFt HYDRAULICS

Fig. 6.6
The shoreline evolution after 75 days of wave action (breakwater situated at 300 m offshore) (using initial wave conditions)

DELFT HYDRAULICS
The shoreline evolution after 10 days of wave action (breakwater situated at 300 m offshore) (using final wave conditions)

DELFT HYDRAULICS

Fig. 6.8
The shoreline evolution after 75 days of wave action (breakwater situated at 300 m offshore) (using final wave conditions)

DELFT HYDRAULICS

Fig. 6.9
The shoreline evolution after 150 days of wave action (breakwater situated at 300 m offshore) (using initial and final wave conditions)

DELFT HYDRAULICS
S-Phi curve used outside the breakwater domain in the simulation of the field case (using one-wave condition)
The shoreline evolution after 8.0 days of wave action for the field case (using one-wave condition)

DELFT HYDRAULICS
The shoreline evolution after 80 days of wave action for the field case (using one-wave condition)
1. Wave conditions at the seaward boundary

1.1 Introduction

Wave conditions are taken from a report made by Hydraulics Research Institute (HRI), (Model Investigations for Sidi Krir Touristic Village, 1995). The wave conditions are estimated by analysing the wind climate at the port of El-Dikheila which is far away from Sidi Krir for 30 km. The wave conditions are revealed in Table A.1. The wave climate is given at 30 m water depth which is relatively deep water.

Four directions are chosen which have higher percentage of occurrence (Fig. A.1). These directions are analyzed to estimate a representative wave height for each direction.

The wave period is estimated via a formula which is relevant for the Mediterranean sea (Short waves lecture notes at IHE). This formula is:

\[ T_p = 4 + 2 \times H_o^{0.7} \]  

(A.1)

in which:
- \( T_p \) = peak wave period (sec)
- \( H_o \) = representative wave height at deep water (m)

The procedure which is used to estimate the representative wave height is described in the following section.

1.2 Representative wave height estimation

The predominant wave directions are in north, north-west and north-east as revealed in Table A.1, A.2 and Fig. (A.1). The approach to estimate the representative wave height for each sector is related to the wave energy flux concept as follows:

\[ H_{eq} = \left( \frac{\Sigma p_i H_i^{5/3}}{P_{tot}} \right)^{2/5} \]  

(A.2)

in which:
- \( H_{eq} \) = the representative wave height (m)
- \( p_i \) = the probability of each wave height \( H_i \) (-)
\( p_{\text{tot}} = \) the total probability of each wave direction (-)

Table A.3 shows the evaluation of the representative wave height for the predominant directions. Table A.3 shows the representative wave height, wave period which is computed by Equation (A.1) and the corresponding wave direction at deep water.

**1.3 Wave transformation**

Waves from different directions are transformed to a water depth of 13 m. The transformation is done by using wave energy decay routine in CRESS-package which is developed at IHE. The calculation of the wave energy decay is based mainly on the principal processes of linear refraction and non-linear dissipation by wave breaking and bottom friction. The deep water wave conditions are summarized in Table A.4. The wave angle is estimated with respect to a line normal to the coastline (anticlockwise is +ve and clockwise is -ve).

<table>
<thead>
<tr>
<th>Direction</th>
<th>Wave height (m)</th>
<th>Wave period (sec)</th>
<th>Wave angle (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.78</td>
<td>5.68</td>
<td>-68.00</td>
</tr>
<tr>
<td>2</td>
<td>0.76</td>
<td>5.64</td>
<td>22.00</td>
</tr>
<tr>
<td>3</td>
<td>0.83</td>
<td>5.76</td>
<td>-8.00</td>
</tr>
<tr>
<td>4</td>
<td>1.65</td>
<td>6.84</td>
<td>-38.00</td>
</tr>
</tbody>
</table>

Table A.4 : summary of the representative wave conditions at deep water.

The wave transformation for different directions are separately executed, Fig. (A.2 to A.5). The results of wave conditions at water depth of 13 m are summarized in Table A.5.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Wave height (m)</th>
<th>Wave period (sec)</th>
<th>Wave angle (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.64</td>
<td>5.68</td>
<td>-60.50</td>
</tr>
<tr>
<td>2</td>
<td>0.71</td>
<td>5.64</td>
<td>20.60</td>
</tr>
<tr>
<td>3</td>
<td>0.77</td>
<td>5.76</td>
<td>-7.50</td>
</tr>
<tr>
<td>4</td>
<td>1.45</td>
<td>6.84</td>
<td>-32.00</td>
</tr>
</tbody>
</table>

Table A.5 : summary of the representative wave conditions at 13 m water depth.
Wave conditions at seaward boundary

For the morphological evolution of DELFT2D-MOR, these directions will be reduced to one and two equivalent wave directions. The approach is to get the same sediment transport rate from the different wave directions as the equivalent wave directions. This approach can be done by using unibest-lt programme which is developed at Delft Hydraulics.

Unibest-lt is a module which is an acronym for uniform beach sediment transport-long shore transport. This module is designed to compute tide-and wave-induced longshore currents and sediment transports on an alongshore beach. The longshore sediment transports and crossshore distribution are evaluated according to various formulae, which enables a sensitivity analysis for local conditions. The computational procedure takes into account the wave climate in order to enable an assessment of gross and yearly transport. The available transport formulae are Engelund-Hansen, Bijker, Van Rijn, Bailard and CERC. Bijker formula will be used in this approach to evaluate the equivalent wave directions.

1.4 Equivalent wave conditions

The determination of the equivalent wave conditions is based on the sediment transport rates computations. Changing the coastline angle will affect the sediment transport rates. First of all the sediment transport rate is evaluated due to the effect of the four wave directions with different coastline angle (S-\(\phi\), Fig. (A.6). The function \(Q_s\) is approximated by the analytical function:

\[
Q_s^n = c_1 \theta_e e^{-(c_2 \theta_e)^2}
\]

in which

- \(\theta_r\) = the relative coastline angle
- \(\theta_e\) = the equilibrium angle (sediment transport = 0)
- \(c_1\) and \(c_2\) = coefficients affect S-\(\phi\) curve shape

Applying trial and error to estimate the equivalent wave conditions to reach to the same values of the equilibrium angle, sediment transport rate, \(c_1\) and \(c_2\). Fig. (A.7) and Fig. (A.8) show S-\(\phi\) curve for one equivalent direction and two equivalent directions which are used in the morphological computations.
<table>
<thead>
<tr>
<th>Item</th>
<th>$H_s$ (m)</th>
<th>$T_p$ (sec)</th>
<th>$\phi$ (deg.)</th>
<th>Period (day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Equivalent one wave condition</td>
<td>1.85</td>
<td>5.80</td>
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<td>80.00</td>
</tr>
<tr>
<td>- Equivalent two wave conditions</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-</td>
<td>2.60</td>
<td>5.68</td>
<td>-29.30</td>
<td>40.00</td>
</tr>
<tr>
<td>2-</td>
<td>1.00</td>
<td>5.64</td>
<td>15.00</td>
<td>40.00</td>
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Table A.6 : summery of the equivalent wave conditions at water depth 13.0.
<table>
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<tr>
<th>Wave height (m)</th>
<th>15</th>
<th>45</th>
<th>75</th>
<th>105</th>
<th>135</th>
<th>165</th>
<th>195</th>
<th>225</th>
<th>255</th>
<th>285</th>
<th>315</th>
<th>-15</th>
<th>Total</th>
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<tr>
<td>0.00</td>
<td>5.62</td>
<td>3.11</td>
<td>1.87</td>
<td>1.85</td>
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<td>5.36</td>
<td>8.71</td>
<td>7.36</td>
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</tr>
<tr>
<td>0.25</td>
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<td>0.01</td>
<td>0.01</td>
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</tr>
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<td>0.00</td>
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<tr>
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<td>0.00</td>
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<td>1.62</td>
<td>3.33</td>
</tr>
<tr>
<td>2.25</td>
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<td>0.08</td>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.02</td>
<td>0.37</td>
<td>0.41</td>
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</tr>
<tr>
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<td>3.75</td>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
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<td>0.25</td>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.02</td>
<td>0.13</td>
<td>0.14</td>
<td>1.43</td>
</tr>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.02</td>
<td>0.13</td>
<td>0.14</td>
<td>1.43</td>
</tr>
<tr>
<td>5.25</td>
<td>6.25</td>
<td>0.00</td>
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<td>0.00</td>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.07</td>
<td>0.07</td>
<td>0.70</td>
</tr>
<tr>
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<td>0.00</td>
<td>0.00</td>
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<td>0.00</td>
<td>0.00</td>
<td>0.07</td>
<td>0.07</td>
</tr>
<tr>
<td>Total</td>
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<td>3.35</td>
<td>1.97</td>
<td>1.93</td>
<td>3.00</td>
<td>1.72</td>
<td>1.57</td>
<td>2.38</td>
<td>6.85</td>
<td>14.37</td>
<td>26.26</td>
<td>25.28</td>
<td>100.00</td>
</tr>
</tbody>
</table>

Table A.1 Probability that wave conditions occur in the given wave height and direction class at El-Dikhelia (modified, in %)
Table : (A.2) represents the significant wave height, % of occurrence and wave direction at relatively deep water.

<table>
<thead>
<tr>
<th>Significant wave height (m)</th>
<th>Direction from the North direction (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>from</td>
<td>to</td>
</tr>
<tr>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>0.00</td>
<td>0.25</td>
</tr>
<tr>
<td>0.25</td>
<td>0.75</td>
</tr>
<tr>
<td>0.75</td>
<td>1.25</td>
</tr>
<tr>
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Table : (A.3) represents the equivelant wave condition computation

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<th>Average wave height (m)</th>
<th>Direction</th>
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<td>H_eq (o) (m)</td>
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<tr>
<td>Tp (sec)</td>
<td>5.68</td>
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<tr>
<td>Phi (deg.)</td>
<td>-68</td>
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</tbody>
</table>

Note: clockwise is -ve and counterclockwise is +ve
Figure (A.1) The wave rose at 30 m water depth (HRI, 1995)
Figure A.2 Calculation of wave energy decay for wave direction (15°-45°) from north direction
Figure A.3 Calculation of wave energy decay for wave direction (285°-315°) from north
Figure A.4 Calculation of wave energy decay for wave direction (315°-345°) from north
Figure A.5 Calculation of wave energy decay for wave direction (345°-375°) from north
Figure (A.6) : S-ϕ curve for the four representative wave directions
Figure (A.7) : S-φ curve for the equivalent one wave direction
Figure (A.8) : S-θ curve for the equivalent two wave directions