Numerical modelling of coastal changes between Chennai and Ennore, India

Accretion due to harbour breakwaters

M.Sc. Thesis

TU Delft
Delft University of Technology
Faculty of Civil Engineering and Geosciences
Hydraulic and Offshore Engineering section

D. Franssen
January 2000
Numerical modelling of coastal changes between Chennai and Ennore, India

Accretion due to harbour breakwaters

M.Sc. Thesis
D. Franssen
January 2000

Supervision

Prof. ir. K. d'Angremond
Dr. ir. J. van de Graaff
Dr. ir. A.J.H.M. Reniers
Dr. ir. J.A. Roelvink

Delft University of Technology
Faculty of Civil Engineering and Geosciences
Hydraulic and Offshore Engineering Section
Abstract

In this report a study is presented concerning the morphological impact of breakwater structures perpendicular to a coast. This study was initially associated with the Indian coast between Chennai (formerly known as Madras) and Ennore in India.

Like many ports, the harbour of Chennai had, and still has a considerable morphological impact on the surrounding coast. By obstructing the longshore transport, the presence of the breakwaters caused accretion and erosion. Construction of a new harbour near Ennore, 20 km north of Chennai, started in 1997. During construction of the harbour similar effects like near the Chennai Port are visible.

Morphological consequences of these harbours can be simulated and predicted by a morphodynamic model. The objective of this study is to design a morphological model of the coast between Chennai and Ennore, and calibrate this model by means of the recorded coastline advancement near the Chennai Port. The time period for these morphological simulations is set to 25 years. Use is made of DELFT2D-MOR, a software package developed by WL | Delft Hydraulics. This numerical program offers a wide range of tools for implementing physical processes that determine coastal morphology.

The objective mentioned above could not be met within the timeframe of this study. In the course of this study, the implementation of breakwater structures appeared to be more complicated than expected. Hence a greatly simplified representation of the real situation has been modelled: simulations have been carried out for a breakwater, perpendicular to the shore where only waves determine the morphological behaviour. The following conclusions can be drawn:

- The present definition of structures like breakwaters in the computational area of DELFT2D-MOR is not suitable for morphological calculations. The breakwaters do not obstruct the longshore sediment transport properly. Most of the incoming sediment does not settle, but "disappears" near the breakwater. A large portion of the longshore sediment drift therefore does not contribute to beach accretion. These structures have to be implemented as closed boundaries within the computational area.
- If the breakwaters are implemented as closed boundaries, DELFT2D-MOR is able to simulate the accretion process due to the presence of a breakwater. The numerical model responds well to input with respect to varying particle size, and varying bottom roughness.
- The erosion process in DELFT2D-MOR is not simulated properly. Although coastline retreat is taking place, the erosion mainly generates unrealistic steep beach slopes.
- The single line theory of Pelnard-Considère yields the same results with respect to coastline advancement near the breakwater, compared to DELFT2D-MOR. The similarity with the computational results only holds for the accreting coastline.
Samenvatting

Dit rapport behelst een studie over de morfologische gevolgen van een golfbreker loodrecht op de kust. Aanvankelijk was deze studie toegespitst op de kust tussen Chennai (voormalig Madras) en Ennore in India.

Zoals vele havens, heeft de aanleg van de haven bij Chennai duidelijke consequenties gehad op de morfologie van de aangrenzende kust. Het jaarlijkse langtransport van sediment wordt door de golfbrekers tegengehouden, hetgeen sedimentatie en erosie van de kust teweegbrengt. Onlangs is een haven aangelegd in Ennore, 20 km ten noorden van Chennai. Tijdens de aanleg hiervan zijn ook sedimentatie en erosie waarnembaar.

Deze verschijnselen kunnen gesimuleerd en voorspeld worden met behulp van numerieke kustmorfologische modellen. Het doel van deze studie is het ontwikkelen van een model voor de kust tussen Chennai en Ennore, en dit model te kaliberen met waargenomen aanzanding bij de Haven van Chennai. De simulatieperiode is gesteld op 25 jaar. Voor dit onderzoek is gebruik gemaakt van DELFT2D-MOR, een numeriek model ontwikkeld door WL | Delft Hydraulics. Dit programma biedt verschillende mogelijkheden om verscheidene kustmorfologische processen te simuleren.

De bovengenoemde doelstelling kon niet gerealiseerd worden binnen het tijdsbestek van deze studie. In de loop van het onderzoek, bleek de implementatie van golfbrekers gecompliceerder dan aanvankelijk verwacht werd. Dientengevolge is een sterk geschematiseerde kust in kaart gebracht: simulaties zijn uitgevoerd voor de situatie van een golfbreker, loodrecht op de kust waar alleen golven het morfologisch klimaat bepalen. De conclusies van deze simulaties zijn:

- De huidige definitie van golfbrekers in het rekenmodel van DELFT2D-MOR is niet geschikt voor morfologische berekeningen. In plaats van het sediment tegen te houden, verdwijnt sediment uit het numeriek model wanneer het de golfbrekers nadert. De golfbrekers moeten daarom gedefinieerd worden door de randen daadwerkelijk te sluiten.
- Als voldaan is aan bovenstaande voorwaarde, is DELFT2D-MOR goed in staat de morfologische gevolgen van een golfbreker te simuleren. Het programma reageert goed op variërende invoergegevens als korrelgrootte en bodemruwheid.
- De aanpassing van de waterlijn, daar waar erosie plaats vindt, wordt niet juist gesimuleerd. In plaats van een teruggang, neemt de bodemhelling zodanig toe, dat onrealistische hellingen ontstaan.
- Wat betreft de kustgroei, levert de theorie van Pelward-Considère vergelijkbare resultaten als DELFT2D-MOR. De gelijkenis geldt alleen voor aanzanding.
Acknowledgements

This thesis completes my study program at the Delft University of Technology. The realisation of this report would not be possible without the support and help I had during the last year, that I spent mostly on the faculty of Civil Engineering.

First of all I would like to thank my supervisors for their help, advises and guidance.
I'm also thankful to Koen Sweers, Gert-Jan Liek and Tim Janssen for their support during the last year. I especially thank Thijs van Berkel for his help-desk like assistance in converting various numerical data to comprehensive plots and graphics.
Contents

Abstract.................................................................................................................. iii
Samenvatting........................................................................................................... v
Acknowledgements.............................................................................................. vii
List of figures and tables......................................................................................... xiii
Notation .................................................................................................................. xv

1 INTRODUCTION ......................................................................................... 1

1.1 OBJECTIVE ......................................................................................... 1

1.2 STRATEGY ......................................................................................... 1

1.3 OUTLINE ........................................................................................... 3

2 SITE CONDITIONS & MODEL INPUT .............................................. 5

2.1 INTRODUCTION ................................................................................. 5

2.2 SITE CONDITIONS ........................................................................... 5

2.2.1 Tides & currents ........................................................................... 5

2.2.2 Waves .......................................................................................... 5

2.2.3 Shoals ........................................................................................... 6

2.2.4 Dredging activities and river discharges ....................................... 6

2.2.5 Ennore creek ............................................................................... 6

2.2.6 Coastline development ................................................................... 7

2.3 INPUT MODEL .................................................................................. 7

2.3.1 Single breakwater ........................................................................ 8

2.3.2 Computational area ....................................................................... 9

2.3.3 Waves .......................................................................................... 9

2.3.4 Bathymetry ................................................................................ 10

2.3.5 Sedimentology ............................................................................ 10

2.3.6 Simulation time ........................................................................... 11
5.5 INPUT MODULE FLOW ......................................................... 32
5.6 INPUT TRANSPORT MODULES TRSTOT/TRSSUS .......................... 33
  5.6.1 Physical and numerical constants .......................................... 34
6 MORPHOLOGICAL SIMULATIONS .................. 37
  6.1 INTRODUCTION .......................................................... 37
  6.2 HYDRAULIC SIMULATION ............................................... 37
  6.3 ACCRETION NEAR BREAKWATER ....................................... 39
    6.3.1 TRSTOT vs. TRSSUS ................................................... 39
    6.3.2 TRSSUS-test cases ..................................................... 39
    6.3.3 TRSSUS with cross-shore routine-test cases ......................... 40
    6.3.4 Transport rates & coastal orientation ................................ 41
    6.3.5 Outbreaking .......................................................... 42
    6.3.6 Profile development .................................................. 43
    6.3.7 Varying wave-climate ................................................ 43
    6.3.8 Bottom roughness ..................................................... 43
    6.3.9 Particle size .......................................................... 43
  6.4 ACCRETION & EROSION ................................................. 44
    6.4.1 Pelnard-Considère .................................................... 44
    6.4.2 Coastline advancement ................................................. 46
7 CONCLUSIONS AND RECOMMENDATIONS ....... 49
REFERENCES ............................................................... 51
APPENDICES ............................................................. 53
List of figures and tables

Figure 1.1  Coastal area Chennai - Ennore .................................................. 2
Figure 2.1  Schematisation of Chennai harbour ........................................ 8
Figure 2.2  Predominant wave direction $\varphi = 15^\circ$ ............................ 9
Figure 2.3  Location cross-section for beach profile .................................. 10
Figure 2.4  Beach profile, bed level and sea level are related to Chart Datum (CD) ................................................................................................. 10
Figure 3.1  Refraction, Davis [1996] ......................................................... 15
Figure 3.2  Wave diffraction at a detached breakwater, Shore Protection Manual [1984] ...................................................................................... 16
Figure 3.3  Radiation stresses for oblique approaching waves .................... 17
Figure 3.4  Wave-induced mean water level variations and currents in the breaker zone ..................................................................................... 18
Figure 3.5  Velocity profile undertow ......................................................... 19
Figure 3.6  Current caused by diffracted waves ........................................ 20
Figure 3.7  Cross-shore transport components (in DELFT2D-MOR) on a beach profile .................................................................................. 21
Figure 3.8  Morphodynamic effect of harbour breakwaters ...................... 21
Figure 3.9  Re-distribution of sediment due to increased/decreased beach slope ......................................................................................... 22
Figure 3.10 Offshore-directed flow due to breakwater .............................. 22
Figure 4.1  Structure DELFT2D-MOR ..................................................... 24
Figure 5.1  Process tree for morphological calculations ............................ 27
Figure 5.2  Sketch of the grid-configuration .............................................. 28
Figure 5.3  The FLOW grid and close-up breakwater with bathymetry ....... 29

Table 6.1  Recorded and calculated coastline advancement at breakwater ... 46
Notation

Roman symbols:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>Chezy coefficient</td>
<td>$[m^{1/2}/s]$</td>
</tr>
<tr>
<td>$C_{\text{max}}$</td>
<td>maximum Courant number</td>
<td>[-]</td>
</tr>
<tr>
<td>CD</td>
<td>Chart Datum</td>
<td>[m]</td>
</tr>
<tr>
<td>c</td>
<td>sediment concentration</td>
<td>[-]</td>
</tr>
<tr>
<td>$c_0$</td>
<td>propagation speed of waves in deep water</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$c_{\text{fc}}$</td>
<td>friction coefficient induced by waves</td>
<td>[-]</td>
</tr>
<tr>
<td>$c_{\text{fw}}$</td>
<td>friction coefficient induced by current</td>
<td>[-]</td>
</tr>
<tr>
<td>$d_{50}$</td>
<td>median particle size [50% by weight smaller in size]</td>
<td>[m]</td>
</tr>
<tr>
<td>$d_{90}$</td>
<td>particle size [90% by weight smaller in size]</td>
<td>[m]</td>
</tr>
<tr>
<td>E</td>
<td>wave energy per unit surface area</td>
<td>[J/m$^2$]</td>
</tr>
<tr>
<td>eppsl.</td>
<td>coefficient bottom slope</td>
<td>[-]</td>
</tr>
<tr>
<td>facA</td>
<td>coefficient wave-asymmetry</td>
<td>[-]</td>
</tr>
<tr>
<td>facU</td>
<td>coefficient undertow</td>
<td>[-]</td>
</tr>
<tr>
<td>$f_w$</td>
<td>Jonsson's friction factor</td>
<td>[-]</td>
</tr>
<tr>
<td>g</td>
<td>gravity acceleration</td>
<td>[m/s$^2$]</td>
</tr>
<tr>
<td>H</td>
<td>total water depth [h+η]</td>
<td>[m]</td>
</tr>
<tr>
<td>H</td>
<td>wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_0$</td>
<td>deep water wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_b$</td>
<td>breaker height</td>
<td>[m]</td>
</tr>
<tr>
<td>h</td>
<td>local water depth</td>
<td>[m]</td>
</tr>
<tr>
<td>$h_b$</td>
<td>breaker depth</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_{\text{rms}}$</td>
<td>root mean-square wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_s$</td>
<td>significant wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>$K_d$</td>
<td>diffraction coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>$K_r$</td>
<td>refraction coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>$K_s$</td>
<td>shoaling coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>$k_s$</td>
<td>Nikuradse roughness length</td>
<td>[m]</td>
</tr>
<tr>
<td>n</td>
<td>ratio of wave group velocity to wave celerity</td>
<td>[-]</td>
</tr>
<tr>
<td>n</td>
<td>Manning coefficient</td>
<td>[s/m$^{1/3}$]</td>
</tr>
<tr>
<td>r</td>
<td>bottom roughness</td>
<td>[m]</td>
</tr>
<tr>
<td>S</td>
<td>sediment transport</td>
<td>[m$^3$/yr]</td>
</tr>
<tr>
<td>S</td>
<td>radiation stress</td>
<td>[N/m]</td>
</tr>
<tr>
<td>t</td>
<td>time</td>
<td>[s]</td>
</tr>
<tr>
<td>$T_d$</td>
<td>return time of a wave in the computational area</td>
<td>[s]</td>
</tr>
<tr>
<td>$T_p$</td>
<td>peak wave period</td>
<td>[s]</td>
</tr>
<tr>
<td>u</td>
<td>velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>Visc.</td>
<td>eddy viscosity</td>
<td>[m$^2$/s]</td>
</tr>
<tr>
<td>w</td>
<td>fall velocity of the grains</td>
<td>[m/s]</td>
</tr>
<tr>
<td>w</td>
<td>mass sediment density</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>z</td>
<td>elevation above bed level</td>
<td>[m]</td>
</tr>
</tbody>
</table>
1 Introduction

In this report a study is presented concerning the morphological impact of breakwater structures perpendicular to a coast. This study was initially associated with the Indian coast between Chennai (formerly known as Madras) and Ennore in India. Figure 1.1 illustrates the location of this particular part of the coast along the Bay of Bengal.

The construction of the Chennai Port started in 1876. The morphological impact on the surrounding coast of this harbour has been considerable. By obstructing the northbound net annual transport of approximately 450,000m³/year, Haskoning [1989], the presence of the breakwaters caused accretion and erosion, south and north of the harbour respectively. Erosion caused a retreat of the coastline, and loss of houses and infrastructure. To this moment all sorts of measures are applied in order to halt this process. However, the present measures are incapable to cope with this problem in a sustainable way.

Construction of a new harbour near Ennore started in 1997. The creation of this harbour, better known as the Ennore Coal Port Project, is designed and supervised by Haskoning Consulting Engineers in association with Rites of India. During construction of the harbour, effects of erosion and accretion are visible, similar to those observed after the construction of the Port of Chennai.

1.1 Objective

These coastal developments can be analysed and explained by means of theories of coastal morphology and historical data of the coastal changes. The prediction of the coastal behaviour in the future, with or without coastal protection measures, is more complicated. The objective of this study is designing a morphological model of the coast between Chennai and Ennore, and calibrating this model by means of recorded coastal changes near Chennai. The time period for morphological simulations is set to 25 years.

1.2 Strategy

DELF2D-MOR, a software package developed by WL | Delft Hydraulics is used to design a morphological model. This numerical program offers a wide range of tools for implementing physical processes that determine coastal morphology. It is not possible to take account of all contributing phenomena of morphodynamics within the timeframe of this study. By increasing the number of processes to be simulated, the model requires a larger computational effort and becomes less attractive.

In this study the data of the accretion process near the Chennai Port are used for calibration purposes. The advancement of the coastline near the southern breakwater has been recorded in the past. If the same accretion rate along a
1.3 Outline

In the next chapter the coastal characteristics between Chennai and Ennore are discussed. A brief description of the relevant theory on coastal morphology is enclosed in Chapter 3. Chapter 4 presents the structure of DELFT2D-MOR, whereas Chapter 5 discusses the application of the model in this study. In Chapter 6 the morphological calculations are discussed. Chapter 7 deals with the conclusions and recommendations of this study.
2 Site conditions & model input

2.1 Introduction

In the following paragraph some local conditions are presented. Most of these data will not be used in this study, but are included for informational purpose. This information can be used for further study after the modelling of the present coastal configuration.

Figure 1.1 illustrates the location and bathymetry of the particular coast. This figure is based on a map of the Naval Hydrographic Office of India, see Appendix A.1.

2.2 Site conditions

2.2.1 Tides & currents

The tide in the Bay of Bengal is semi-diurnal. The average values of the water levels for the area between Chennai and Ennore are:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean High Water Spring</td>
<td>+ 1.13 m CD</td>
</tr>
<tr>
<td>Mean High Water Neap</td>
<td>+ 0.82 m CD</td>
</tr>
<tr>
<td>Mean Sea Level</td>
<td>+ 0.65 m CD</td>
</tr>
<tr>
<td>Mean Low Water Neap</td>
<td>+ 0.42 m CD</td>
</tr>
<tr>
<td>Mean Low Water Spring</td>
<td>+ 0.12 m CD</td>
</tr>
</tbody>
</table>

Levels are related to Chart Datum (CD), which is approximately the lowest astronomical tide. Since the main tidal components M2 and S2 approach more or less perpendicular to the coastline, vertical tides are noticeable, but horizontal tides are limited, Haskoning [1989].

Currents in the Bay of Bengal are mainly induced by currents in the Indian Ocean and monsoon winds in the Bay of Bengal. The north-east monsoon directs the current southward, parallel to the coast, while the current is oriented northward during the south-west monsoon.
As for the monsoon driven currents, no reliable information is available. Flow velocities in the order of magnitude of 0.4 m/s have been measured at a depth of 15 m.

2.2.2 Waves

For the design of the new port near Ennore several studies have been performed. In the period '84-'95 various wave rider buoy measurement campaigns have been launched. The latest campaign concerned a directional wave rider buoy, which was installed 1 km outside the harbour of Chennai at a depth of 15.6 m CD.
siderable part of these volumes is recycled in the dredging process by means of the longshore transport.
In the future, the additional intake of a new power station will increase the inward directed flow, and hence the amounts of suspended sediment transport entering the creek. (However, the longshore drift will still be mainly responsible for sedimentation of the mouth). If the dredging and dumping policy does not change and the dredged quantities are deposited at the shore, these volumes will stay within the coastal transport system between Chennai and Ennore.

2.2.6 Coastline development

The construction of the Chennai Port interrupted the existing longshore transport system, resulting in an accumulation of sediment along the southern breakwater. This process has been recorded, as can be seen in Appendix A.3. These data should be treated with care. To compensate for seasonal variations, the surveys were executed at approximately the same time every year. However, only a short section of the accreted coastline has been recorded. The actual alignment of the entire accreted coastline is not recorded. Moreover, the advancement of the coastline seems quite arbitrary, e.g., the advancement between 1899 and 1900 is large compared to previous and subsequent recordings.

Whilst the southern breakwater of the Chennai Port was accreting, severe erosion occurred north of the harbour. At several places rock and other types of revetments have been applied to protect the retreating coast. These revetments reflect waves more than the original beach, generating an increase of wave heights. This yields considerable scour near the toe of the revetments, endangering their stability.

In the present situation the erosion zone extends approximately 7.5 km north of the fishing harbour, see Figure 1.1. It is difficult to determine the amount of sand that is bypassing the port of Chennai, and the amount that contributes to nourishment of the eroding coast. The yearly accretion and erosion rates have been estimated at 200,000 m$^3$ and 400,000 m$^3$ respectively, Haskoning [1995]. Earlier research performed by WL | Delft Hydraulics yielded a net wave-driven northbound sediment transport of 450,000 m$^3$/year.

The alignment of the original coastline between Chennai and Ennore before breakwater constructions started in 1876 is illustrated in Appendix A.4. The assumption that this coastline had an orientation of 15° N is reasonable, taking into account the orientation of the coast, which is not affected by erosion yet, and the location of the root of the southern breakwater of the Chennai Port.

2.3 Input model

In the previous chapter it was stated that a model is designed for simulating the accretion rate near the Chennai Port. In this section the configuration and the input of this model will be discussed briefly. In reality the monsoons and other phenomena that are mentioned in Section 2.2 strongly influence the sediment transport system along the coast near Chennai.
the computational area north of the breakwater, or even disregard the entire area where erosion will take place. However, both lateral boundaries (perpendicular to the shore) induce disturbances that propagate through the computational area, in both directions. For this reason, both boundaries should be located well outside the area of interest. This is the reason why the computational area is extended in southern and northern direction.

2.3.3 Waves

For practical reasons only one constant wave condition throughout the entire simulation time is used in most cases. Obviously these waves approach the coast from south-east direction, in order to generate an annual net northbound transport. The following wave conditions have been used:

\[
\begin{align*}
H_s &= 1 \text{ m} & \text{significant wave height} \\
T_p &= 7 \text{ s} & \text{peak period, being the period at which the wave energy spectrum reaches its maximum} \\
\text{Dir.} &= 15^\circ & \text{oblique wave attack, relative to shoreline}
\end{align*}
\]

This angle is assumed to be the predominant wave direction throughout the season near Chennai, and is derived by the coastal orientation near the breakwater of Chennai in 1900, see Appendix A.3. The coastal alignment in 1900 is schematised in Figure 2.2.

![Figure 2.2](image)

**Figure 2.2** predominant wave direction \( \varphi = 15^\circ \)

Near the breakwater, the longshore sediment transport is reduced to zero. As will be discussed in the next chapter, the longshore transport due to waves is zero, if the angle of oblique wave attack is reduced to zero as well. The accreting coast near the breakwater therefore will have the same orientation as the wave crests (angle of oblique wave attack near breakwater is zero). At this point the coast has an orientation of \( 15^\circ \) with respect to the original coastline, which equals the angle of incident wave attack which is used for morphological calculations.

2.3.4 Bathymetry

The slope of the coast is based on the beach profile that is present 15 km south of Chennai, where the cross-shore profile is assumed to equal the cross-shore profile near Channai, before the harbour was constructed. The cross-section where the beach slope is determined is illustrated in Figure 2.3.
90% by weight of the sample is smaller. To a good approximation, the mean sediment particle size is interchangeable with $D_{50}$ for most beach sediments. In this study, most morphological simulations are carried out with the following sediment parameters:

\[
\begin{align*}
D_{50} & = 200 \, \mu m \\
D_{90} & = 300 \, \mu m \\
\text{fall velocity} & = 0.023 \, \text{m/s}
\end{align*}
\]

These are common values for sandy coasts.

2.3.6 Simulation time

In the first stages of this study, the objective was to design a morphological model, in order to predict the coastal changes between Chennai and Ennore for a period of 25 years. By simulating the morphological processes for this period, the impact of the Ennore Port and the impact of different types of coastal protection measures for the eroding coast can be assigned. Longer-term simulations are possible, but increase the computational effort as well. The simulations in this study, which will be discussed in Chapter 6, only concern the morphological consequences of a single breakwater. These simulations are carried out for a period of 25 years as well. The same period as the initial objective is chosen, in order to determine whether the model will produce consistent and correct results. In other words, during a simulation of 25 years many things can "go wrong". These faults should be analysed and corrected. Simulating the coastal development near the breakwater for a period of 25 years requires 6 days of calculation time.
3 Relevant theory

3.1 Introduction

In this section a brief overview is presented concerning the theoretical background of the simulated morphodynamic processes near breakwater structures. As mentioned before, only the influence of waves is accounted for in this study. Many other natural processes like tides contribute to the dynamic behaviour of a coast.

In this study only the interaction of waves and wave-induced currents is responsible for suspension, transport and deposition of sediment. These phenomena will be discussed briefly in the following paragraphs.

3.2 Waves

As a result of the variability of the wind forces offshore, an irregular wave field is generated which approaches the coast in a certain direction. A wave field also may be generated by a storm, which occurred well outside the region where the waves reach the coast. These waves are called swell, and are characterised by their relative large period.

Waves play a dominant role in the nearshore zone. A large part of their energy is dissipated here. When entering shallow water, waves are subjected to shoaling, bottom friction, breaking, refraction and diffraction.

3.2.1 Shoaling, bottom friction and breaking

When a wave propagates in water that is gradually becoming shallower, the bottom (friction) will affect the wave when the water depth becomes less than half the wave length. Nearing the breaker line shoaling occurs: the wave celerity (c) and the wave length (\( \lambda \)) decreases while the wave height (H) increases.

The transformation of waves entering shallow water can be described using the Airy theory. In this theory the wave profile is simplified to a linear sinusoidal form. The expressions for wave energy and wave power are given by Equations (3.1) and (3.2). The wave power is the rate at which energy is transmitted in the direction of wave propagation across a vertical plane perpendicular to the direction of wave propagation and extending over the entire depth.

\[
E = \frac{1}{8} \rho g H^2 \tag{3.1}
\]

where:
- \( E \) = wave energy per unit surface area \([J/m^2]\)
- \( \rho \) = mass density of water \([kg/m^3]\)
- \( g \) = acceleration due to gravity \([m/s^2]\)
- \( H \) = wave height \([m]\)
between the wave crest and the depth contours. This phenomenon is called refraction, which is illustrated in Figure 3.1. Again if the water depth becomes too small the wave will break.

![Diagram showing wave refraction](image)

**Figure 3.1 Refraction, Davis [1996]**

This phenomenon can be analysed by means of the Airy theory and Snel's law for geometrical optics. When waves approach the coast obliquely, and again assuming the conservation of wave power, the wave height can be predicted by:

\[
\frac{H_1}{H_0} = K_{sh} K_r
\]  \hspace{1cm} (3.6)

where:
- \(H_1\) = wave height at a certain depth [m]
- \(H_0\) = wave height at deep water [m]
- \(K_{sh}\) = the shoaling coefficient [-]
- \(K_r\) = the refraction coefficient [-]

Using Snel's Law the varying direction of waves can be predicted as well. So once at deep water the wave height, wave period and direction is known, the height and direction at a certain depth can be predicted.

The computation procedure indicated above is easily carried out for coasts with a simple bathymetry. In reality no coast is straight with parallel depth contours, waves are neither linear, sinusoidal nor regular. Moreover, the three-dimensional characteristics of waves are difficult to describe. Nowadays computers are able to deal with complex calculations. DELFT2D-MOR incorporates the application HISWA to describe wave-behaviour. This program will be discussed later in Section 4.4.
depths. As will be discussed later, DELFT2D-MOR accounts for diffraction patterns by incorporating the directional energy distribution of waves.

### 3.3 Wave set-up/set-down

In the previous sections the transport of energy has been discussed briefly. Waves entering an area also transfer a horizontal momentum to this area. This flux of momentum due to the presence of waves is referred to as radiation stress. Changes in radiation stress produce resulting forces on the water column through which the waves propagate.

![Diagram of Radiation stresses for oblique approaching waves](image)

**Figure 3.3 Radiation stresses for oblique approaching waves**

Figure 3.3 shows the components of radiation stress along two balance areas (water columns) for oblique approaching waves. The right area refers to an axis system, perpendicular to the coast. For the left area the following Equations hold:

\[
S_{xx} = (2n - \frac{1}{2})E \tag{3.8}
\]

\[
S_{yy} = (n - \frac{1}{2})E \tag{3.9}
\]

where:

- \(S_{xx}\) = the radiation stress component in the direction of wave propagation \([N/m]\)
- \(S_{yy}\) = the radiation stress component parallel to the wave crests \([N/m]\)
- \(n\) = the ratio of the wave group velocity to wave celerity \([-\] \)
- \(E\) = wave energy, see Equation (3.1) \([J/m^2]\)

Since radiation stress components can be quantified by vectors, the components corresponding to the axis system perpendicular to the coast can be determined using the force equilibrium on the area, once the angle between wave crest and coastline (\(\phi\)) is known.
rent pattern in this area are caused by the longshore gradient in wave-induced set-up, see also Figure 3.6.

Sedimentation and erosion initially take place in the breaker zone, and the profile will change accordingly. This is schematised in Figure 3.9.

Figure 3.9 Re-distribution of sediment due to increased/decreased beach slope

The cross-shore transport mechanism will redistribute the material. Eventually the accreting beach will reach the tip of the breakwater and by-passing of sand will develop.

The decreasing longshore currents induce a water motion offshore, see Figure 3.10. This phenomenon, which is called outbreaking, also contributes to the redistribution of material in cross-shore direction.

Figure 3.10 Offshore-directed flow due to breakwater
3.5 Sediment transport

3.5.1 Longshore transport

The basic equation for sediment transport is given by:

\[ S_x = \int_0^{h+\eta} u(z)c(z)dz \]  

(3.11)

where:
- \( S_x \): longshore sediment transport \([\text{m}^3/\text{sm}]\)
- \( u(z) \): longshore velocity \([\text{m/s}]\)
- \( c(z) \): time averaged concentration \([-]\)
- \( h \): local water depth \([\text{m}]\)
- \( \eta \): instantaneous water elevation \([\text{m}]\) (maximum value = half the wave height)

The current and the entrainment of sediments in the vertical determine longshore transport caused by wave action. If tidal, wind-driven or other currents are absent, the current is determined by the wave-driven forces while the entrainment of sediments is defined in various formulae by a stirring parameter.

This parameter is strongly related to the bottom friction due to waves. Because of the orbital motion of waves, no velocity profile can develop like profiles in rivers. This results in larger bottom shear stresses, and therefore an increasing capability for stirring up material.

In Appendix B.5.2 the Bijker formula is enclosed. Bijker introduced the wave influence via a modification of the bottom shear stress in the Kalinske-Frijlink bed load transport formula for currents only, and coupled this to a modified Einstein formula for suspended sediment transport in currents.

3.5.2 Cross-shore transport

Cross-shore sediment transport is initiated by different phenomena:
- asymmetry of waves
- net current, due to tidal forces or undertow
4 DELFT2D-MOR

4.1 Introduction

The numerical program DELFT2D-MOR, developed by Delft Hydraulics, has been used to simulate the morphological impact of a single breakwater attached to the shore. This program offers a wide range of tools for implementing physical processes that determine coastal behaviour. It has proved to yield good results in several studies performed in the field of coastal morphology.

In the previous chapter some hydrodynamic processes were discussed, which act on a sandy coast with a long breakwater built perpendicular to the coast. These processes can be simulated to a great extent by the numerical program. In this chapter the structure, possibilities and limitations of the program are discussed briefly.

4.2 Modular structure

The current velocities computed by DELFT2D-MOR are depth-averaged. These currents act on a horizontal plane, which is divided in separate cells by a computational grid. For each cell the following physical processes and their interactions are determined:

- Waves
- Flow
- Sediment transport
- Bed level variations

Each of these processes is incorporated in a separate module. A main module controls their interactions, and all data are written to or read from a curvilinear computational grid. In this chapter all these modules are discussed briefly. A more detailed description is enclosed in Appendix B.

4.3 Control module MAIN

The output of one module may be used as input for another. For example, wave forces used in the flow module yield currents. Relevant output data are written to a central file that provides the communication between the modules. The control module MAIN activates or de-activates the modules by means of a process tree, which will be discussed in Paragraph 5.2. The following scheme illustrates the relations and interactions between the modules.
• shoaling
• refraction due to bottom variations
• refraction due to current variations
• energy loss due to depth-induced breaking
• energy loss due to bottom friction
• energy gain due to local wind generation

Refraction
Refraction is accounted for as follows. A curving wave ray implies that the direction of wave propagation changes while travelling along the ray. In other words, the energy transport continuously changes direction while travelling through the area. This can be conceived as the energy travelling not only through the geographic area, but also (and simultaneously) from one direction to another. This permits the approach that has been taken in HISSWA: the energy propagates not along rays but across a grid covering the area, while refraction is accounted for by shifting energy from one direction to another, Delft Hydraulics [1997].

Diffraction
Diffraction is not accounted for in the WAVES module. However, the lack of wave energy penetrating behind structures like breakwaters, is to some extent compensated by distributing the wave energy over a directional sector. The wave energy will enter a lee of an obstacle via the directional spreading.

Wave set-up/set-down
Wave-induced set-up/set-down can not be computed simultaneously with waves in HISSWA. However, the calculated radiation shear stresses are used by the FLOW module, which will be discussed in the next paragraph. This module computes the wave-induced water level variations and currents. By following loop C in Figure 4.1 the new depths and currents serve as new input values for wave calculations by HISSWA. The results of this iterative procedure determine the wave set-up.

For more information about the implementation of the physical phenomena in HISSWA reference is made to Appendix B.3.

4.5 Module FLOW
This module simulates the non-steady flow and water level variation, using the program TRISULA. The incorporated physical phenomena are:

• tidal force
• bed shear stress on the bottom
• wave-induced stresses and mass fluxes
• wind shear stress on the water surface
• density differences
• Coriolis force
5 Model settings

5.1 Introduction

The various modules of DELFT2D-MOR require a large amount of input parameters. This chapter does not take account of all the model settings. Most of the parameters are set to their default value and will not be mentioned separately. These settings have proven to yield good results in previously performed studies on coastal morphology.

5.2 Input control module MAIN

As discussed in the previous chapter, this module controls the interaction of the physical processes and the total simulation time by means of a process tree (see Figure 5.1). This tree assigns the sequence, the iterations, and the simulation time of all modules used. The control functions concern:

- process starting
- splitting up the total simulation time into time intervals
- splitting up the process into subprocesses
- activating and deactivating the process modules
- process stopping

The tree consists of a set of nodes and branches. The branches connect the child and the parent node. The node without a parent node is the root node, nr. 8. A simulation run starts at the root node by activating the subprocess of its child node (node 7). This node activates its subprocesses too, starting at the lowest number (e.g. node 1).

![Process tree for morphological calculations](image)

*Figure 5.1 Process tree for morphological calculations*
5.3.1 FLOW grid

This grid is the main computational grid for morphological calculations. It covers the area of interest, but is extending south and north, in order to prevent disturbance caused by boundary effects. The influence area of these boundary increases in time. This means that for simulation runs of 25 years, the boundaries should be located at greater distance of the area of interest compared to simulations of 5 years.

By applying a curvilinear grid the computational effort can be minimised: near the boundaries large grid cells are chosen, since no detailed information is required here. The grid of the total computational area in Figure 5.3 shows decreasing grid cell dimensions towards the coast and near the breakwater structure, and growing dimensions towards the boundaries. This diminishes the number of grid cells, and hence the computational effort.

![Diagram of FLOW grid and close-up breakwater with bathymetry](image)

*Figure 5.3 The FLOW grid and close-up breakwater with bathymetry*

Some numerical restrictions are:

- The grid spacing must vary smoothly over the computational region, to minimise inaccuracy errors. A maximum factor for the mesh size between consecutive cells is set to 1.3.
will be discussed in the next section, wave calculations are based on a different grid definition. The information of the computed current field is automatically converted to the WAVES grids by means of spatial interpolation. To ensure that the dry points in the FLOW-grid are interpolated properly onto the WAVES-grid, the dry point area has to be 2 cells wide.

5.3.3 The WAVES grids
The WAVES module uses two types of rectangular grids. An input grid for bottom definition, and a computational grid. Both grids are nested in a larger one, which determines their boundary condition. Fine grids are applied to cover the breaker zone and areas where obstacles like breakwaters are present. These are nested in coarse grids, which cover the whole computational area.

**Bottom grid**
The dimensions of the grid cells are equal to the ones of the computational grid. The depth values are obtained from the curvilinear FLOW grid by means of interpolation.

**Computational grid**
This grid computes the propagating waves in 3 dimensions: x-, y-, and θ-direction. Detailed information is enclosed in Appendix B.3. θ is the spectral wave propagation direction.
The dimensions of the nested grid cells are chosen as 25m in x- (wave propagation-) direction, in order to establish a high resolution in the breaker zone. For numerical stability the mesh size in y-direction should be chosen larger. In order to reduce the number of cells, and hence computational effort, a cell size of 100m can be chosen. However, better results are accomplished if the WAVE grid matches the FLOW grid as much as possible. Therefore the dimensions are chosen equal to the minimum FLOW grid cell size, which are applied around the breakwater: 25 m x 50 m. The maximum sector size, to encompass as much as possible of the directional wave energy distribution is determined by the conditions of numerical stability:

\[
\frac{\Delta x}{\Delta y} \leq \cot(\theta)
\]  
(5.2)

where:

- \(\theta\) = directional sector for energy distribution, at each side of direction of wave propagation \([\circ]\]
- \(\Delta x\) = grid cell size in x-direction = 25m
- \(\Delta y\) = grid cell size in y-direction = 50m

For the given values of \(\Delta x\) and \(\Delta y\), \(\theta\) is set to 60°, so the total sector is 120° wide. This grid definition is by far not needed near the lateral boundaries, where the FLOW grid cell dimensions are very large. However tests with 2 nested grids, with a coarse and fine definition in longshore direction, did not reduce the computational effort.

For the coarse grid, \(\Delta x\) and \(\Delta y\) are set to 100 and 200m respectively.
"wet" side of the grid, see Figure 5.2. A water level of +0.65m CD is imposed, which is the mean sea level. This is also the initial water level condition for the whole computational area.
The reflection coefficient of this "sea-boundary" is determined by the following relation:

\[ \alpha = T_d \sqrt{\frac{H_{\text{average}}}{g}} \]  \hspace{1cm} (5.3)

where:

- \( \alpha \) = reflection coefficient
- \( T_d \) = time needed for a disturbance in water level to travel through the computational area and back [s]
- \( H \) = total water depth [m]
- \( g \) = acceleration of gravity [m/s^2]

\( T_d \) is derived by dividing the travel distance of the wave by its average celerity (= \( \sqrt{gH_{\text{average}}} \)). This yields \( \alpha = 1100 \). Deviations of this value did not influence the spin-up time significantly, and will not be discussed further. In case of open lateral boundaries where velocities are imposed, a similar kind of procedure is followed, which can be found in the FLOW manual, Delft Hydraulics [1996]a.

Similar to the input of WAVES, the numerical and physical parameters and coefficients are set to their default value, which have proven to give good results in earlier studies. Unless mentioned otherwise, the following physical constants are applied:

\[ g = \text{acceleration of gravity} = 9.81 \text{ [m/s}^2\text{]} \]
\[ \rho = \text{water density} = 1023 \text{ [kg/m}^3\text{]} \]
\[ C = \text{Chézy coefficient} = 65 \text{ [m}^{1/2}/\text{s]} \]
\[ v_{\text{hor}} = \text{horizontal eddy viscosity} = 1.0 \text{ [m}^2/\text{s]} \]

The values mentioned above are specified uniformly over the area, since actual values, e.g. bottom roughness, are not known.

The simulation time of FLOW is set to 1 hour, with a time step of 15 seconds. The stable current field which is reached over this period is written to the communication file.

### 5.6 Input transport modules TRSTOT/TRSUS

As discussed in Section 4.2, transport module determines the sediment transport rates using the time-dependent wave and current field and a bed level that remains constant during the computation. The magnitude of the sediment transport will be computed by using the Bijker formula, which is discussed briefly in Appendix B.5.2.
the direction of the bottom slope. The resulting transport vectors are added to the calculated bed load transport vectors, determined by the Bijker formula. More information on the cross-shore transport routine is enclosed in Appendix B.6.1.
6 Morphological simulations

6.1 Introduction

In this chapter the simulations of an accreting beach near the breakwater will be discussed. Appendix A.5 illustrates the computational area for these simulations. The schematisations, which are made for designing the model, have been discussed in Section 2.3. Appendix C.1 shows a 3D sketch of the breakwater and surrounding beach profile. Section 5.3.2 deals with the implementation of the breakwater in DELFT2D-MOR. In Chapter 5 the various model settings are discussed as well.

In the following paragraphs the results of morphological simulations are presented as follows:
First, a hydraulic calculation will be discussed briefly. Before any morphological calculations may start, the whole computational area has to adapt itself to the imposed hydraulic conditions.
Hereafter, several morphological simulations for a period of 1-2 years will be discussed. Within this simulation period, several problems have been encountered, which had to be solved in order to modify the model settings for simulations of 25 years, which will be discussed after the test cases for a period of 2 years. In addition, some simulations have been carried out in order to determine the sensitivity of the model to varying bottom roughness conditions and grain size.
This chapter will be concluded with a simulation of 25 years, in which erosion has been simulated as well. The results of this calculation are compared with a theory for accreting and eroding beaches due to breakwaters, developed by Pelnard-Considère.

The results of all calculations are presented in the appendices in 2D only.

6.2 Hydraulic simulation

As mentioned above, the whole computational area has to adapt itself to the imposed hydraulic conditions. The time needed for reaching a stable hydrodynamic situation is better known as the spin-up time. In this paragraph the spin-up time for the computational area is discussed briefly, in order to illustrate the response of the computational area to the imposed hydraulic conditions.

After the WAVES module calculates a wave field in the computational area, the FLOW module calculates the wave-induced water level variations and wave-induced currents, see Figure 4.1.
Appendix C.2 illustrates the locations where water levels and current magnitudes are recorded during the FLOW calculation. These "observation points" are located in the area of interest, i.e. in the area near the breakwater where accretion will take place in morphological simulations. It is necessary to deter-
The influence of the direction of waves and the directional energy spreading of waves is apparent. The oblique wave attack induces a longshore current northward. The breakwater acts as a screen which prevents distribution of energy along a wave crest, once the waves have past the breakwater. The result is absorption of all the wave energy at both sides of the breakwater, since reflection of waves is not accounted for in HISWA. At the northern lee-side the gradient of wave height along the coastline results in a southbound flow in the breaker zone. This phenomenon corresponds well to Figure 3.6.

6.3 Accretion near breakwater

6.3.1 TRSTOT vs. TRSSUS

In the first stages of this study the TRSTOT transport module is applied for morphological calculations. This module has generated irregular erosion and accretion patterns, illustrated in upper plot of Appendix C.5. Using TRSSUS these irregularities are diminished, because of a different calculation method for the sediment concentration. The lower plot of Appendix C.5 shows the coastal development calculated using TRSSUS, over the same period, and for the same conditions as imposed while using the TRSTOT module. Details concerning these different calculation methods are enclosed in Appendix B.6.

Only the results of calculations where TRSSUS is applied will be discussed hereafter.

6.3.2 TRSSUS-test cases

Beach profile

In Section 5.6.2 the cross-shore transport routine is discussed briefly. In the first stages of this study a version of DELFT2D-MOR has been used, which did not incorporate the cross-shore transport routine. The consequences of the absence of this routine will be discussed hereafter, illustrated with Appendices C.6-C.8.

The upper plot of Appendix C.6 illustrates the bathymetry and suspended sediment transport vectors along the coast for $t = 0$ years. The lower graph shows the suspended sediment transport vectors for $t = 2$ years. The vectors are significantly smaller after 2 years. This phenomenon occurs along the entire coast and can be explained by the profile development.

The upper graph of appendix C.7 shows the profile development over a period of two years. Within this period the beach profile near the coastline has become steeper. The waves break at a depth of approximately 1.5 m CD. A steeper profile reduces the width of the breaker zone. Meanwhile the number of computational grid cells in cross-shore direction remains constant. This means after a period of 2 years, only 3 grid cells cover the area where breaking occurs. These are too few to simulate the longshore drift properly. The result is a decreasing longshore sediment transport capacity, which is illustrated in Appendix C.8.

The offshore-directed transport is manipulated by varying the dispersion coefficient (see equation B.44), which has been set constant in previous runs to 1 m$^2$/s. This coefficient determines the redistribution of suspended sediment in
sediment “disappears” into the breakwater. This structure is implemented in the computational area by “dry points” which will be discussed hereafter.

**Drypoints**

As discussed in Section 5.3.2, the breakwater is implemented in the FLOW-grid by means of computational cells that are defined as inactive "dry points". The bed level variations near the breakwater are calculated by the BOTTOM module as follows: BOTTOM calculates the bottom changes at all bottom points of the grid using the sediment transport values at the four surrounding water level points, see Appendix B.4.4 for the grid definition. A point is permanently set dry when the four surrounding velocity points of a computational cell are set to zero. The lower figure in Appendix C.10 shows a grid definition, including the presence of dry points and 4 sediment transport vectors. In the upper plot area of Appendix C.9 the vectors at the accreting side of the breakwater are directed parallel to the breakwater. It is very well possible that these vectors actually have a longshore component, as illustrated in the grid in Appendix C.10. Since the depth points at the borders of the dry point area are inactive, these longshore components do not contribute to any changes of these depth points. The result is an area, half a computational cell wide, where sediment "disappears", see Appendix C.10. This is a hypothesis, which has to be verified.

The problem is solved as follows: Instead of defining dry points at computational cells for modelling the breakwater, these grid cells are "cut" out of the computational grid. The breakwater is now represented by closed boundaries. These boundaries do not allow any passage of sediment. The differences between Appendices C.9 and C.11 clearly show the effect of this action. In a period of 2 years considerable accretion has taken place, resulting in a reorientation of the coastline and the depth contours, see the upper plot of Appendix C.11. The accreted volume of sediment after 2 years is roughly \(0.6 \times 10^6\) m\(^3\). This value corresponds well to the volume of sediment that is transported along the shore during the period of 2 years, and is obstructed by the breakwater.

**6.3.4 Transport rates & coastal orientation**

Now that the accretion process is simulated properly on the short term, morphological simulations for a longer period are possible.

The lower plot of Appendix C.12 illustrates the depth contours after 25 years, together with the suspended sediment transport vectors. (The upper plot will be discussed later in this chapter). The lower graph in Appendix C.13 shows the transport rates through several cross-sections along the accreting coast. In the following sections, the recorded transport rates will be evaluated stepwise, according to the numbering of the cross-sections.

**Cross-section 1**

Due to the accretion process, the coast reorients itself towards the orientation of the wave crests. This means the angle of oblique wave attack in the accreting area decreases, and the wave-driven longshore currents and sediment transports decrease as well. After a simulation period of 1 year, this phenomenon is apparent at cross-section 1, see Appendix C.13. 1 year later, a similar trend is visible at cross-section 2.
tion, see the paragraph "cross-section 2". A more detailed study, based on a computational grid with uniform fine grid cells, is required to analyse this feature.

6.3.6 Profile development
Appendix C.17 shows the profile development for cross-section 1. During the first years, the profile remains more or less constant, and progresses in offshore direction.
At t = 8 years a bar develops, just above CD. This feature continues in time, and it is also present at t = 25 years. This is caused by non-temporal irregularities of the coastline. The cross-section where these data are recorded is located across a deviation of the coastline. The irregular coastline after 25 years is visible in the lower plot of Appendix C.12. These "disturbances" start developing after 8 years, see the upper plot of Appendix C.12. The main reason for this phenomenon is the very small (and constant) angle of wave attack, in combination with a coastline orientation that differs from the orientation of both computational FLOW and WAVES grids. Under these conditions irregularities can develop easily. By switching the direction of the longshore sediment drift in time, a more dynamic behaviour of the coast is simulated and these irregularities do not occur. This will be discussed hereafter.

6.3.7 Varying wave-climate
Switching the direction of the longshore sediment drift during a simulation can be achieved by varying the angle of wave attack in time. The result of such an action is illustrated in Appendix C.18. Here the waves approach 15° relative to the coast from Southeast direction, during 9 months. The remaining time of the year the waves approach from the East, which is also 15° oblique to the coast. The total simulation time is 25 years.
The purpose of this simulation is to illustrate the effect of switching the direction of the longshore sediment drift. The net yearly northbound transport is half the value of previous calculations, because sediment now is transported southward during 3 months in a year.

6.3.8 Bottom roughness
The calculated currents in the previously discussed simulations are based on a uniformly defined Chézy coefficient of 65 m$^{1/3}$/s. If the formulation by Manning is used instead, the roughness is depth-dependent, see Appendix B.4.2. With a (default) Manning coefficient of 0.026 s/m$^{1/3}$, the modified Chézy coefficient at a depth of 1 m is 38 m$^{1/3}$/s, which means a higher bottom roughness in this area. The result is a decrease of the longshore current, and a decrease of the longshore drift by 100,000 m$^{3}$/year till 275,000 m$^{3}$/year.

6.3.9 Particle size
In all previously discussed simulations a particle size of $D_{50} = 200$ µm was applied. In this section, a comparison is presented of morphological behaviour for this sediment, and coarser sediment with $D_{50} = 275$ µm. The characteristics of the latter sediment type are presented in Appendix B.6.2.
t = time needed to reach accretion over a length L [year]
L = length of accretion along the breakwater [m]
d = depth at the breakwater head [m]
S = longshore sediment transport [m³/year]
φ = orientation of the accreted coast near breakwater, relative to original coastline

Appendix A.3 illustrates the accretion along the southern breakwater of the Chennai port.
In 1900 the coastline advancement near the breakwater is 590 m. This year is used for determining the longshore transport according to Pelnard-Considère. A depth of 10 m at the head of the breakwater is chosen. The head of the breakwater is located at 900 m from the coastline. A depth of 10 m at 900 m from the coastline agrees well with alignment of the corresponding isoline at other locations along the present coast. Together with the orientation of approximately 15° relative to the original coastline, Equation (6.1) gives:

\[ S = 432,000 \text{ m}^3/\text{year} \]

The theory of Pelnard-Considère also provides relations for estimating the shoreline advancement seaward or shoreline set back:

\[ y = \phi \sqrt{\frac{4at}{\pi}} \left( e^{-u^2} - u \theta \sqrt{\pi} \right) \] (6.2)

where:
\[ \phi = \text{angle of oblique wave attack, equal to } \phi \text{ in Equation (6.1)} \] [-]
\[ a = \frac{S}{\varphi d}, \text{see Equation (6.1)} \] [m²/year]
\[ t = \text{time of accretion/erosion} \] [year]
\[ u = -\frac{x}{\sqrt{4at}} \]
\[ x = \text{the distance from the breakwater along the beach, positive at the lee of the breakwater} \] [m]
\[ \theta = 1 - \frac{2}{\sqrt{\pi}} \int_0^x e^{-u^2} du \] (6.3)

The resulting alignment of the shoreline after 25 years is represented by the thick line in Appendix C.20, together with the calculated beach orientation by DELFT2-MOR.

**Comparison**
The resemblance of Pelnard-Considère with the results of DELFT2-MOR with respect to the accreted coastline is apparent. Although the estimated longshore transport according to Pelnard-Considère is 15% higher than the calculated longshore transport in DELFT2D-MOR, the coastline orientation is similar.
coastline is not recorded. In addition, not quantified sand mining activities have been undertaken in the past on the accreted shore. In view of the continuous development of Chennai in the past, and its need for construction materials, it is expected that sand mining activities must have been considerable.

On the other hand, it is stated that the simulation with the used model is a strong simplification of the actual morphological behaviour of the coast. In the simulation only a constant wave attack is considered. The combination of varying wave conditions with monsoon or tide-driven longshore currents will yield different results. Besides these matters, many input parameters (e.g. sediment particle size) are strongly related to the morphological behaviour. These have been set constant throughout the computational area, whereas e.g. the sediment particle size varies throughout the area. The implementation of varying bed material conditions will also yield different results.

At present, accretion also takes place near the Ennore Port, which is located 20 km north of Chennai. By recording this accreting process, and by incorporating more conditions of the real situation, a morphological model can be tuned to the real situation to a better extent.

In the next chapter the conclusions and recommendations of this study will be discussed.
7 Conclusions and recommendations

The aim of this study was to design an operational numerical model to analyse and predict the morphodynamic behaviour of the coast between Chennai and Ennore, and calibrating this model by means of the recorded coastline advancement near the Chennai Port. As was discussed earlier, these objectives have not been met. Within the timeframe of this study several problems have been solved, which were encountered during numerous morphological test cases in the calibration stage. Some of these test cases are discussed in the previous chapter.

Nevertheless, a set-up is presented of a computational area for the present situation, see Appendix C.21. This computational area including the Ennore shoals requires a large computational effort. Taking into account the present development of computers, these kinds of morphological calculations will be operational for long-term simulations on a PC in near future, incorporating various processes.

Conclusions
In this study the impact of a breakwater, perpendicular to the shore where only waves determine the morphological behaviour, has been simulated. The following conclusions can be drawn:

- The present definition of structures like breakwaters in the computational area of DELFT2D-MOR is not suitable for morphological calculations. The breakwaters do not obstruct the longshore sediment transport properly. Most of the incoming sediment does not settle, but "disappears" near the breakwater. A large portion of the longshore sediment drift therefore does not contribute to beach accretion. These structures have to be implemented as closed boundaries within the computational area.

- If the breakwaters are implemented as closed boundaries, DELFT2D-MOR is able to simulate the accretion process due to the presence of a breakwater. The numerical model responds well to input with respect to varying particle size, and varying bottom roughness.

- The erosion process in DELFT2D-MOR is not simulated properly. Although coastline retreat is taking place, the erosion mainly generates unrealistic steep beach slopes.

- The single line theory of Peñard-Considère yields the same results with respect to coastline advancement near the breakwater, compared to DELFT2D-MOR. The similarity with the computational results only holds for the accreting coastline.
References

Davis Jr, R.A. [1996]
The evolving coast, Scientific American Library, New York, USA.

Delft Hydraulics [1994]
Global hindcast study for the Ennore coal port project, India, report phase 1, Delft Hydraulics.

Delft Hydraulics [1996]a
delft3D-flow user manual, version 0.1, release 2.48, Delft Hydraulics.

Delft Hydraulics [1996]b
An introduction to DELFT2D-MOR, release 2.07, Delft Hydraulics.

Delft Hydraulics [1997]
delft3D-waves user manual, version 1.02, Delft Hydraulics.

Haskoning [1989]
Paradip-Ennore Coal Transport Feasibility Study, final report Appendix 5.4 Morphological Impacts, Haskoning in association with Tebodin & Rites.

Haskoning [1995]
volume V, Annex 7: Coastal morphology and coastal protection, Haskoning in association with Rites.

Holthuijsen, L.H., Booij, N. and Herbers, T.H.C. [1989]
A prediction model for stationary, short crested waves in shallow water with ambient currents, Coastal Engineering, 13:23-54.

Naval Hydrographic Office of India [1991]
Map: Mamallapuram to Point Pudi, 1:150,000.

Nipius, K.G. [1998]

Raudviki, A.J. [1990]
Loose boundary hydraulics, 3rd edition, Pergamon press.

Shore Protection Manual [1984]
Coastal engineering research centre, Department of the army, Waterways experiment station, Corps of engineers, Vicksburg, Mississippi, USA.

Stelling, G.S. [1984]

van der Velden, E.T.J.M. [1989]
Coastal engineering, volume II, Delft University of technology.
Appendices

A. MAPS
A.1 Map Naval Hydrographic Office of India
A.2 Wave height rose
A.3 Recorded accretion near Chennai
A.4 Orientation original coastline at Chennai
A.5 Computational area

B. DELFT2D-MOR
B.1 Introduction
B.2 The control module MAIN
B.2.1 Data communication
B.3 The wave module WAVES
B.3.1 Introduction
B.3.2 General structure
B.3.3 Physical background
B.3.4 Numerical procedure
B.3.5 Grid definition
B.4 The FLOW module TRISULA
B.4.1 Introduction
B.4.2 Physical background
B.4.3 Numerical procedure
B.4.4 Grid definition
B.5 Transport module
B.5.1 Introduction
B.5.2 Bijkers transport formula
B.6 TRSTOT/TRSSUS
B.6.1 TRSSUS + Cross-shore routine
B.6.2 Particle fall velocity
B.7 The bottom module BOTTOM
B.7.1 Introduction
B.7.2 Physical background
B.7.3 Numerical procedure

C. RESULTS SIMULATIONS
C.1 3D-view of breakwater in computational area
C.2 Location observation points and cross-section
C.3 Hydraulic spin-up calculation
C.4 Hydraulic spin-up calculation, Waves&Currents
C.5 TRSTOT vs. TRSSUS
C.6 TRSSUS 2 years, Morphological development
EXPLANATION

<table>
<thead>
<tr>
<th>Item:</th>
<th>Represents:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of bar</td>
<td>Height / Speed</td>
</tr>
<tr>
<td>Direction of bar</td>
<td>Direction</td>
</tr>
<tr>
<td>(to centre of rose)</td>
<td></td>
</tr>
<tr>
<td>Length of bar</td>
<td>Occurrence (%)</td>
</tr>
<tr>
<td>Number in centre of</td>
<td>Occurrence (%) in</td>
</tr>
<tr>
<td>wave</td>
<td>lowest class</td>
</tr>
</tbody>
</table>

Significant wave height (m)

Wave height rose at CD -20m contour near Ennore waves + swell all year, Delft Hydraulics [1994]  Appendix A.2
Position of shoreline south of Chennai harbour since 1876  Appendix A.3
depths [m] related to Chart Datum

Chennai port schematised as single breakwater

Computational area

DELFT2D-MOR
Plot computational area

Appendix | A.5
B. DELFT2D-MOR

B.1 Introduction

DELFT2D-MOR is a numerical model system for morphological studies. It is based on 4 separate modules for the following physical processes:
- waves
- flow
- sediment transport
- bed level variations
This modular structure of the program ensures maximum flexibility since various combinations of the different modules are possible. The general structure of the model as well as the separate modules are described in this Appendix.

B.2 The control module MAIN

The general structure of the compound morphological model is illustrated in Figure B.1.

![Diagram of the model DELFT2D-MOR]

Figure B.1 Structure of the model DELFT2D-MOR

A user-supplied process tree must specify routes that should be followed in the morphological model. This tree is a scheme to describe a hierarchical system and consists of a set of nodes and branches. The graphical representation of such a tree is usually top-down. An example of a process tree is given in Figure B.2.
B.2.1 Data communication

The coupling of the various modules requires a file into which relevant data used by the various modules can be stored. In DELFT2D-MOR this file is called a communication file. This file has a nefis structure (Neutral File System). All the data relevant to the various modules are (over)written to and can be read from this communication file. This file can also be used for post processing of the data. Besides the communication file, extra output files are generated by the independent modules (WAVES, FLOW, TRSSUS, BOTTOM). Information about specific features of each subprocess can be found here.

B.3 The wave module WAVES

B.3.1 Introduction

The WAVES module is the physical process module, which simulates the propagation of waves and as a result predicts the distribution of wave parameters and current-driving terms. Waves play an important role in the morphological evolution of a coastal area. They stir up the bottom material and bring it into suspension. Furthermore, waves generate currents that are able to transport the bottom material.

The WAVES module in DELFT2D-MOR makes use of the HISWA model. HISWA, an acronym for Hindcast Shallow water WAVes, is a numerical model for the prediction of stationary, short crested waves in shallow water.

B.3.2 General structure

The user prescribes the input and output of WAVES by means of switches. These switches determine the way the bottom depth, water level and current velocity are accounted for. In general the next procedure is followed:

Each time the WAVES module is activated by the process tree, the bottom and FLOW data are read from a user-specified file containing this data. In general this will be the communication file. Since the FLOW data computed by TRISULA (model used by the FLOW module) are available at a staggered grid (see Figure B.3), the data must first be interpolated onto the HISWA input grid. Next the HISWA computation can be executed, after which the results must again be interpolated back onto the TRISULA grid.

B.3.3 Physical background

In HISWA the wave propagation is determined across the grid according to the Eulerian approach of the action balance of the waves. The wave action is a function of the spatial co-ordinates (x, y) and of the spectral wave direction (θ).

In this approach all wave information is available at the mesh-points of a rectangular grid.

The action density A is defined as:

\[ A(\omega, \theta, x, y, t) = \frac{E(\omega, \theta, x, y, t)}{\sigma} \]  \hspace{1cm} (B.1)
\[ H_{\text{max}} = \frac{0.88}{k} \tanh \left( \frac{\gamma kh}{0.88} \right) \]  

(B.6)

where:

\[ k = \text{wave number } \quad [1/m] \]
\[ \gamma = \text{breaker index } \quad [-] \]

The breaker index is a function of the wave steepness and can be calculated by:

\[ \gamma = 0.5 + 0.4 \tanh(33s_0) \]  

(B.7)

where

\[ s_0 = \text{wave steepness } = H_s/\lambda \quad [-] \]

The action balance has been simplified in the following manner. Assuming that the time scale of the wave propagation over the model area is small compared to that of the local wind or current field the time dependent parameters are neglected. This means that the first and fifth terms on the left-hand side are left out of the action balance that makes the model stationary. The second simplification consists of the parameterisation of the remaining action balance. Therefore, two directional wave functions are defined: the directional action spectrum \( A_0(\theta) \), resulting from the integration of the action density \( A(\omega, \theta) \) over the total frequency domain, and a mean wave frequency as a function of spectral direction \( \omega_0(\theta) \).

\[ A_0(\theta) = m_0(\theta) \]  

(B.8)

\[ \omega_0(\theta) = \frac{m_1(\theta)}{m_0(\theta)} \]  

(B.9)

where:

\[ A_0(\theta) = \text{one-dimensional directional action spectrum } \quad [J/m^2] \]
\[ \omega_0(\theta) = \text{mean frequency as a function of the spectral direction } \quad [1/s] \]
\[ m_0(\theta) = \text{zero}^{th} \text{ order moment of the action density spectrum } \quad [J/m^2] \]
\[ m_1(\theta) = \text{first order moment of the action density spectrum } \quad [J/sm^2] \]

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

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

(B.10)

When the zero\(^{th}\) and first order moments of the action density spectrum are used, the following two evolution equations remain of the action balance:

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

(B.11)
\[ \theta = \text{half of the directional energy distribution sector} \quad [^\circ] \]
\[ c_x = \text{group velocity of the mean frequency in x-direction} \quad [\text{m/s}] \]
\[ c_y = \text{group velocity of the mean frequency in y-direction} \quad [\text{m/s}] \]
\[ \Delta x = \text{grid size in x-direction} \quad [\text{m}] \]
\[ \Delta y = \text{grid size in y-direction} \quad [\text{m}] \]

B.3.5 Grid definition

HISWA uses different grids for input, computation and output. The bottom grids interpolate depth values, water level and current data of the FLOW grid, wherever the grids overlap. The computational grids do the same with respect to water level and current data. Outside the input grids HISWA extrapolates the water depth and current information by taking the value at the nearest boundary of the input grid. All grids mentioned above are illustrated in Figure 5.2.

The computational grid is a grid in the dimensions x-, y- and \( \theta \). The orientation of the x-axis of this grid has to be chosen so that it is more or less equal to the main wave direction because this is the direction in which HISWA carries out the computation as explained above. The computational grid must be larger than the area of interest, especially when open boundaries are used. In that case a region exists along each lateral side of the grid where the waves field is disturbed as a consequence of the fact that no wave energy enters the model area here. So the y-direction has to be sufficiently large to prevent that these regions fall into the area of interest.

All computed wave parameters are interpolated onto the FLOW grid, and written to the communication file.

B.4 The FLOW module TRISULA

B.4.1 Introduction

The FLOW module is the physical subprocess that simulates the non-steady flow and water level variation from a tidal, a wave or meteorological forcing. The FLOW module in DELFT2D-MOR consists of the TRISULA model which is a program for 2DH or 3D flow computations, including the effects of waves, wind and density differences.

B.4.2 Physical background

The 2DH version of the TRISULA model solves the unsteady shallow water equations in 2 depth-averaged dimensions. In this approach the vertical momentum equation is reduced to the hydrostatic pressure relation. Vertical accelerations are assumed to be small compared to the gravitational acceleration and are not taken into account. The system of equations consists of horizontal momentum equations and a continuity equation. The momentum equations in x- and y-direction are:

\[
\begin{align*}
\frac{\partial (hu)}{\partial t} + \frac{\partial (hu^2)}{\partial x} + \frac{\partial (huv)}{\partial y} + gh \frac{\partial (h+z_b)}{\partial x} - k_s h \left( \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right) - \frac{\tau_{b,x}}{\rho_w} \sum \frac{F_x}{\rho_w} &= 0 \\
\end{align*}
\]  

(B.15)
\[ C = 18 \log \left( \frac{12h}{k_s} \right) \]  
White Colebrook's formulation  
(B.22)

where:
\[ H = \text{total water depth} \quad \text{[m]} \]
\[ k_s = \text{the Nikuradse roughness length} \quad \text{[m]} \]

The main physical phenomena, which are accounted for in TRISULA, are:
- tidal forcing
- the effect of the earth's rotation (Coriolis force)
- density differences
- wind shear stress on the water surface
- bed shear stress on the bottom
- influence of the waves on the bed shear stress
- wave-induced stresses and mass fluxes

For a complete review of the physical phenomena that are taken into account and their implementation, reference is made to the FLOW user manual, Delft Hydraulics [1996].

B.4.3 Numerical procedure

The equations for the water levels are solved with an Alternating Direction Implicit (A.D.I) technique, Stelling [1984]. 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. For more details about this numerical method reference is made to the TRISULA manual and to Stelling [1984].

B.4.4 Grid definition

In the horizontal plane TRISULA uses a staggered grid. Each grid cell contains a water level point, a bottom depth point, a x-direction velocity point (u) and a y-direction velocity point (v). The points in a grid cell all have the same indices (i, j). This is illustrated in Figure B.3.

\[ \text{Figure B.3 Staggered TRISULA grid} \]
$U_{cw} = U_c + U_w$  \hspace{1cm} (B.25)

where:

\begin{align*}
U_{cw} & \text{ wave-current velocity vector} \quad [\text{m/s}] \\
U_c & \text{ current velocity vector} \quad [\text{m/s}] \\
U_w & \text{ orbital velocity vector} \quad [\text{m/s}] \\
\end{align*}

The wave-current velocity is now substituted in the equation for the bottom shear stress:

$\tau_{cw} = \rho_w \kappa^2 U_{cw}^2$  \hspace{1cm} (B.26)

where:

\begin{align*}
\rho_w & = \text{water density} \quad [\text{kg/m}^3] \\
\kappa & = \text{von Karman coefficient} \quad [-] \\
\end{align*}

Due to the oscillating wave motion the direction of this bottom shear stress varies with time. However, the only important condition for the stirring of the bed material is the exceeding of the critical velocity regardless of its direction. So it is sufficient to know the mean shear stress. The time-averaged value of the bed-shear stress finally yields:

$\overline{\tau_{cw}} = \tau_c + \frac{1}{2} \hat{\tau}_w$  \hspace{1cm} (B.27)

where:

$\hat{\tau}_w = \text{maximum shear stress due to the waves} \quad [\text{N/m}^2]$  

This maximum shear stress can be determined by:

$\hat{\tau}_w = \frac{1}{2} \rho_w f_w \hat{u}_0^2$  \hspace{1cm} (B.28)

where:

\begin{align*}
f_w & = \text{Jonsson's friction factor} \quad [-] \\
\hat{u}_0 & = \text{maximum horizontal velocity component of waves,} \\
& \text{just outside the boundary layer} \quad [\text{m/s}] \\
\end{align*}

The equation for the time averaged bed shear stress can be re-written which yields:

$\overline{\tau_{cw}} = \tau_c \left( 1 + \frac{1}{2} \left( \xi \frac{\hat{u}_0}{U_c} \right)^2 \right)$  \hspace{1cm} (B.29)

where: \hspace{1cm} $\tau_c = \rho_w g \left( \frac{U_c}{C} \right)^2$ \hspace{1cm} [N/m$^2$]
\[ z_* = \text{exponent of the concentration distribution} \]
\[ c_a = \text{bed load concentration} \quad \text{[kg/m}^3\text{]} \]

Bijker assumed that the concentration in the bottom layer \((c_a)\) is constant over the entire thickness \(r\) and is given by:

\[ c_a = \frac{S_b}{\int_0^r u(z)dz} \quad \text{(B.33)} \]

in which \(u(z)\) is the Prandtl-von Karman logarithmic velocity profile [m/s]. The exponential part is the Rouse number given by:

\[ z_* = \frac{w_s}{\kappa U_{*cw}} \quad \text{(B.34)} \]

where:
\[ \kappa = \text{von Karman coefficient} = 0.4 \quad \text{[-]} \]
\[ w_s = \text{fall velocity of the sediment} \quad \text{[m/s]} \]

The shear stress velocity can be computed from:

\[ U_{*cw} = \sqrt{\frac{\tau_{cw}}{\rho}} = \sqrt{\frac{\tau_{cw}}{\rho} \left( 1 + \frac{1}{2} \left( \frac{\tilde{u}_0}{U_c} \right) \right)} \quad \text{(B.35)} \]

where \(\tilde{u}_0\) = maximum velocity, see (B.28) [m/s]

The suspended load transport in the layer between the water surface and the bottom layer is determined by:

\[ S_s = \int_r^h c(z)U(z)dz \quad \text{(B.36)} \]

where:
\[ U(z) = \text{velocity profile according to Prandtl-von Karman profile} \quad \text{[m/s]} \]
\[ S_s = \text{suspended sediment transport} \]

The equation of \(S_s\) can be solved numerically and after using the total Einstein integral term, van der Velden [1989], it can be shown that:

\[ S_s = 1.83 \cdot Q \cdot S_b \quad \text{(B.37)} \]

The total transport \((S_{tot})\) can now be calculated by adding the bed load transport to the suspended transport:

\[ S_{tot} = S_b + S_s = S_b(1 + 1.83Q) \quad \text{(B.38)} \]
B.6 TRSTOT/TRSSUS

TRSTOT uses the transport rates following from the applied transport relation. In this module the contribution of the bed load and the equilibrium suspended load are summed together. For each grid point the total sediment transport contribution S is calculated for both directions of the 2D area:

\[ T_x = \frac{1}{T} \int_0^T S_x dt \]

\[ T_y = \frac{1}{T} \int_0^T S_y dt \]  \hspace{1cm} (B.41)

where:

- \( T_{x,y} \) = total average sediment transport per unit width \([\text{m}^3/\text{s}/\text{m}]\)
- \( T \) = time interval imposed by MAIN \([\text{s}]\)
- \( S_{x,y} \) = total sediment transports per unit width, determined by the transport relation \([\text{m}^3/\text{s}/\text{m}]\)

In the TRSSUS module no summation of bed and suspended transport contributions occurs. The bed load transport is separately accounted for, similar to the way in which the total transport is dealt with by the module TRSTOT. The equilibrium suspended sediment contribution will be used in TRSSUS to derive a local equilibrium concentration:

\[ c_{se} = \frac{\frac{\overline{S}_{se}}{\alpha_s \overline{u} \overline{h}}}{\alpha_s \overline{u} \overline{h}} \]  \hspace{1cm} (B.43)

where:

- \( c_{se} \) = the local equilibrium concentration \([-]\)
- \( S_{se} \) = suspended sediment transport contribution, determined by the transport relation \([\text{m}^3/\text{s}/\text{m}]\)
- \( \alpha_s \) = suspended sediment coefficient \([-]\)
- \( \overline{u} \) = local velocity \([\text{m/s}]\)
- \( \overline{h} \) = local water depth \([\text{m}]\)

TRSSUS takes into account the time dependent development of the suspended sediment concentration for all grid points of the computational area. By introducing an advection-diffusion term and an adaptation time for the vertical sediment concentration profile, this module generally produces smaller variations of sediment transport rates between the grid points. The use of the advection-diffusion equation results in a spreading of the accretion or erosion rate over more grid cells, which would be concentrated in one cell if TRSTOT was used.

The concentration computed by the module is the depth-averaged concentration \( c_s \). The advection-diffusion equation to be solved for \( c_s \) reads:
Three coefficients are implemented in DELFT2D-MOR to determine the contribution of the physical phenomena that initiate this type of transport:

- facA for the asymmetry of waves  
- facU for the undertow  
- epssl for the bottom slope

The present version of the model can not calculate the undertow. The coefficient "facU" therefore is set to zero. The other coefficients have to be calibrated for different situations. Since no data are available, the following advised values are used:

\[
\begin{align*}
\text{facA} &= 0.4 \quad [-] \\
\text{facU} &= 0 \quad [-] \\
\text{epssl} &= 0.2 \quad [-]
\end{align*}
\]

The transport due to the asymmetry of waves acts in the wave propagation direction, while the sediment transport due to the bottom slope is calculated in the direction of the bottom slope. The resulting transport vectors are added to the calculated bed load transport vectors, determined by the Bijker formula. In this way the cross-shore transport contribution bypasses the advection-diffusion Equation (B.44) to be solved in TRSSUS. More information on the application of the coefficients for cross-shore transport is to be found in Nipius [1998].

**B.6.2 Particle fall velocity**

Two kinds of sediments are used in the morphological calculations. The parameters are listed below:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>General</th>
<th>Section 6.3.9</th>
</tr>
</thead>
<tbody>
<tr>
<td>(D_{50}) [(\mu m)]</td>
<td>200</td>
<td>275</td>
</tr>
<tr>
<td>(D_{90}) [(\mu m)]</td>
<td>300</td>
<td>400</td>
</tr>
<tr>
<td>Fall velocity [m/s]</td>
<td>0.023</td>
<td>0.036</td>
</tr>
</tbody>
</table>

The fall velocity of sediment, as discussed in Section 6.3.9 is determined by:

\[
\log(1/w) = 0.4949 (\log D_{50})^2 + 2.4113 (\log D_{50}) + 3.7394 \quad (B.49)
\]

where:

- \(w\) = particle fall velocity = 0.037 [m/s]  
- \(D_{50}\) = median diameter = 275 [\(\mu m\)]

Raudkivi [1990] provides a similar relation for the sediment fall velocity.
\[
\frac{Z_{n+1} - Z_n}{\Delta t} = \frac{\left( S_{x_i^{n+\frac{1}{2}}} - S_{x_i^{n+\frac{1}{2}}} \right)}{2\Delta x} + \frac{\left( S_{x_i^{n+\frac{1}{2}}} - S_{x_i^{n+\frac{1}{2}}} \right)}{2\Delta y} + \ldots
\]

(B.53)

Because the FTCS scheme is an explicit scheme, the Courant number should be less than one. On the other hand, the Courant number should not be much lower than one as low Courant numbers will generally induce numerical diffusion. In order to ensure this stability criterion, the time-step used in the BOTTOM module is determined by the transport module. In TRSSUS/TRSTOT a maximum Courant number is specified, which may not be exceeded. This results in a varying time step during the simulation run. For a given Courant number the maximum time step is determined by the minimum value of the stability criterion:

\[
\sigma = c_b \frac{\Delta t}{\Delta x} \rightarrow \Delta t = \frac{\sigma \Delta x}{c_b}
\]

(B.54)

where:
\[
\sigma = \text{user-specified Courant number} = 0.8 \quad [-]
\]
\[
\Delta x = \text{the grid spacing} \quad [\text{m}]
\]
\[
c_b = \text{bed-level celerity} \quad [\text{m/s}]
\]

The smallest value of \(\Delta t\) throughout the field finally determines the timestep for updating the bottom level.

The bed-level celerity \(c_b\) can be seen as the propagating speed of bed-level disturbances and is determined by assuming that the sediment transport relation can be written as:

\[
S = \alpha U^\beta
\]

(B.55)

where \(\alpha, \beta\) are constants [-]. When using some algebraic relations it is possible to re-write the bed-level celerity as:

\[
c_b = \frac{\beta S}{(1 - p)h}

\]

(B.56)

where:
\[
\beta = \text{power of the used transport formula} \quad [-]
\]
\[
S = \text{sediment transport} \quad [\text{m}^3/\text{s/m}]
\]
\[
p = \text{sediment porosity} \quad [-]
\]
\[
h = \text{water depth} \quad [\text{m}]
\]

The optimal timestep now follows from:
waves (schematised)

900 m

10 m

Mean Sea Level

1:500

wave direction

3D-view of breakwater
surf zone

head breakwater

cross-section

bed levels [m] related to Chart Datum

coastline

waves

computational FLOW grid

Location observation points and cross-section

Appendix C.2
top: depths related to Chart Datum

bottom: wave heights (Hrms) and currents

Results wave–current interaction

DELFT2D–MOR

Hydraulic spin–up calculation

Appendix C.4
top: accretion (neg)/erosion (pos) using TRSTOT

bottom: accretion (neg)/erosion (pos) using TRSSUS

Morphological development over 1 year

DELFT2D-MOR
Morphological calculation
Appendix C.5
top: depths related to CD and susp. sediment transport $t = 0$

bottom: depths and susp. sediment transport $t = 2$ years

Longshore suspended sediment transport, TRSSUS

DELFT2D-MOR

Morphological calculation

Appendix C.6
Bed levels and Mean Sea Level are related to Chart Datum

Bed levels and Mean Sea Level is related to Chart Datum

Profile development, TRSSUS, no cross-shore routine

Appendix C.7
Location cross-section

Transport rates

Sediment transport through cross-section

Transport rates, TRSSUS

Appendix C.8
top: bathymetry and susp. sediment transport $t = 2$ years

bottom: accretion (neg)/erosion (pos) $t = 2$ years

Morphological development TRSSUS+cross

Appendix C.9
Volume balance after 2 years:

\((S_{in} - S_{out}) \cdot 2 \text{ years} \approx 0.75 \cdot 10^6 \text{ m}^3 >> 0.075 \cdot 10^6 \text{ m}^3\) (accreted sediment, see Appendix C.9)

Grid definition with dry points and 4 transport vectors
top: depths and susp. sediment transport t = 2 years

bottom: accretion (neg)/erosion (pos) t = 2 years

Morphological development TRSSUS+cross, cut cells
top: depths related to CD and susp. sed. transport \( t = 8 \) years

bottom: depths and susp. sediment transport \( t = 25 \) years

Morphological development TRSSUS+cross, cut cells

DELT2D-MOR

Morphological calculation

Appendix C.12
Transport rates through cross-sections as illustrated in upper figure.

Transport rates, TRSSUS+cross, cut cells
top: scour [m] at closed boundary
bottom: scour [m] at flowboundary

Boundary disturbances after 2 years

DELFT2D-MOR
Morphological calculation

Appendix C.14
Transport rates through cross-sections as illustrated in upper figure.

Transport rates, sediment inflow boundary imposed
Transport rates at $t = 1$ year through cross-section at cross-section parallel to the coast

Suspended sediment transport through cross-section parallel to the accreting shore at $t = 1$ year
Evolving beach profile along cross-section 1, see Appendix C.15

Bed levels and Mean Sea Level is related to Chart Datum

Accreting beach, TRSSUS+cross, cut cells
Transport rates cross-section 4, see Appendix C.13 for locations cross-section in computational area

Equilibrium profiles, recorded at cross-section 1
Depths related to Chart Datum

Profile development over 5 years, recorded at cross-section 4

Profile development for varying particle sizes Appendix C.19
t = 25 years, waves approaching 120° N
depths [m] related to CD
comparison Pelnard-Considère

Morphological development TRSSUS+cross, cut cells

Appendix C.20
Ennore shoals

Ennore

Harbour Chennai

depths [m] related to Chart Datum

<table>
<thead>
<tr>
<th>Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; -2.000</td>
</tr>
<tr>
<td>&lt; -0.650</td>
</tr>
<tr>
<td>&lt; 0.000</td>
</tr>
<tr>
<td>&lt; 2.500</td>
</tr>
<tr>
<td>&lt; 5.000</td>
</tr>
<tr>
<td>&lt; 10.000</td>
</tr>
<tr>
<td>&lt; 15.000</td>
</tr>
<tr>
<td>&lt; 20.000</td>
</tr>
<tr>
<td>&gt; 20.000</td>
</tr>
</tbody>
</table>

Computational area of present situation, without Ennore Port

Appendix C.21