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Sand transport at the middle and lower shoreface of the Dutch coast

Simulations of SUTRENCHE-model and proposal for large-scale laboratory tests

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WL | delft hydraulics
Sand transport at the middle and lower shoreface of the Dutch coast

Simulations of SUTRENCH-model and proposal for large-scale laboratory tests

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CLIENT:
RIKZ, Rijkswaterstaat

TITLE:
Sand transport at the middle and lower shoreface of the Dutch coast

ABSTRACT:
The study was aimed to:
- evaluate the knowledge of sand transport processes in deeper water of the North Sea,
- evaluate the mathematical models available for simulation of these processes and associated morphology,
- to make a proposal for improvement of the knowledge of sand transport processes through large-scale experiments and implementation of this knowledge in models.
These latter models will be used to give advice on the morphological effects of large-scale mining of sand in the coastal waters of the North Sea.

The study comprises of the following elements:
- description of SUTRENCH model, UNIBEST-TC 2.0 model and DELFT-2D/3D model,
- validation of SUTRENCH model based on additional laboratory and field tests,
- sensitivity computations using SUTRENCH model for short-term and long-term morphology of sand mining pits,
- calibration of OBS-transport meter in wave tunnel of WL | DELFT HYDRAULICS,
- proposal for large-scale experiments in Delta-flume of WL | DELFT HYDRAULICS to improve wave-related transport components of models and to obtain data of cross-shore trench migration for validation of models.

REFERENCES:
Letter of RIKZ/OS 977489 dated 25 November 1997 by the National Institute for Coastal Management (RIKZ) of Rijkswaterstaat,
Letter of RIKZ/OS 977046 dated 23 September 1997 by the National Institute for Coastal Management (RIKZ) of Rijkswaterstaat.

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KEYWORDS
- Sand transport on shoreface
- Sand transport models
- Morphology of sand mining pits

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Preface

Within the framework of the Rijkswaterstaat research-programme COAST*2000, which runs between 1995 and 2000, morphologic knowledge of the Dutch Coast is used to generate advises to the several Directorates of Rijkswaterstaat. Within the participating projects C2000*DEEP and C2000*SUP advises are being prepared for the Main directorate and the North Sea directorate respectively. Central question to be answered is:

What will be the morphologic effects (on the long term) of natural sand losses and large scale sand mining on the lower shoreface of the Dutch Coast?

For both the “natural” morphologic development of the lower shoreface and large scale extraction sand mining areas, information about the (yearly averaged) sediment transports at the lower shoreface in front of the Dutch coast (around 20 m below Dutch Ordnance Level = NAP) is necessary. Therefore, research at sediment transports at the lower shoreface takes a central place within the research programme COAST*2000. Application of process based and behaviour oriented models (these are models in which the behaviour of natural systems is modelled rather than the processes) play a central role in these research efforts.

Process based models (e.g. UNIBEST-TC, SUTRENCH (2DV) and DELFT3D) are able to calculate the sediment transports at deep water induced by currents and wave action. The (large scale) behaviour oriented models (e.g. PONTOS and ASMITA) assume some equilibrium condition based on which, in combination with dispersion coefficients, the exchange of sediment between several coastal sections is determined.

To address short term environmental and policy issues it is thought that primarily process based models will have to be applied. However, deployment of process based models is hampered by the fact that, especially in deeper water, present knowledge of bedload transport and suspended sediment transport is still very limited. This is illustrated by the fact that the direction and magnitude of the (residual) sediment transports at deep water in front of the Dutch coast are not well known (Van Rijn, 1995). Also to address long term environmental and policy issues, knowledge of the (residual) transports of deeper water are necessary. It is clear that expansion of the knowledge of the occurring transport processes in deeper water are imperative to the improvement of existing theories.

In order to gain knowledge of sediment transport processes and to enlarge the accuracy and reliability of the model calculations, existing and supplementary data is necessary. Within the framework of the COAST*2000 research programme, field data of the morphologic development of existing shipping channels and sand mining areas are being collected and analysed. Despite that, supplementary data are still needed. Valuable data can be obtained by performing a field experiment, but this is a time and money consuming exercise. Moreover, present measuring techniques are not yet adequate to measure bedload transport; investments in measuring techniques are needed. However, on the short term (1-2 years) laboratory research can be a valuable tool in order to gain new data. The relationship between data, model calculations and central questions to be answered is schematised in the figure below:
By assignment of the National Institute for Coastal and Marine Management (RIKZ) of the Rijkswaterstaat a study at the “process knowledge of the lower shoreface of the Dutch coast” has been performed by WL | DELFT HYDRAULICS (contract RKZ-490). This study took place within the framework of the project C2000*SUP. Results of this study will be used for the short term advises with respect to the environmental effects of large scale sand mining and for the long term advises with respect to the large scale behaviour of the lower shoreface.
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B  Formulations of bed-load and suspended load transport model for equilibrium conditions
I Introduction

The National Institute for Coastal Management of Rijkswaterstaat has by Letter of RIKZ/OS 977489 dated 25 November 1997 commissioned WL | DELFT HYDRAULICS to evaluate the knowledge of sand transport processes in deeper water of the North Sea, to evaluate the mathematical models available for simulation of these processes and associated morphology and to make a proposal for improvement of the knowledge of sand transport processes through large-scale experiments and implementation of this knowledge in models. These latter models will be used to give advice on the morphological effects of large-scale mining of sand in the coastal waters of the North Sea.

The central theme in the present study is the knowledge and modelling of sand transport processes on the middle (between -8 m and -20 m depth contours) and lower shoreface (seaward of -20 m depth) of the Dutch coast with the aim of predicting the morphological behaviour of large-scale sand mining pits. The results can be used as guidelines based on which coastal managers can get a quick impression of the impacts of proposed sandmining alternatives.

The available knowledge of sand transport processes on the middle and lower shoreface is summarised in Chapter 2. The morphological behaviour of the mining pits is presented in Chapters 3 and 4.

The process-models available for simulation of sand transport processes and associated bed level changes along dredged pits and trenches are: SUTRENCH model, UNIBEST-TC 2.0 model and DELFT-2D/3D model. At present stage of research the SUTRENCH model is most suited for cross-currents over relatively long trenches, because of the two-dimensional vertical approach, modelling the turbulence- and wave-induced mixing and the gravity-induced settling of the sand particles. The DELFT-2D/3D model can also model the vertical structure of the transport processes, based on a depth-integrated approach. The UNIBEST-TC 2.0 model is based on a local equilibrium approach. Given the superiority of the SUTRENCH model for small-scale sedimentation and erosion processes in wave-current conditions, the attention will be focused on this model in the present study.

Data that has become available since previous studies, will be analysed to further validate the model for wave-dominated conditions. Sensitivity computations will be carried out to identify the most relevant process and input parameters. The model will also be used to explore the effect of trench geometry and dimensions on the long-term trench behaviour. This information can be used to determine practical trench/pit dimensions and to design future field experiments of trial pits/trenches.

At present stage of research the cost-effectiveness of field experiments related to sand mining pits in relatively deep water is not optimum, because self-registering in-situ process instruments are not yet available and results will not readily be available, as monitoring of morphological evolution over several years is involved. To explore the possibility of optical sand concentration instruments for field and laboratory measurements in the near-bed region, a commercially available OBS-transport meter will be tested in the wave tunnel in the present study. To overcome the problems of a field experiment, a proposal will be made for large-scale experiments in the Delta-flume, based on an evaluation of available process knowledge.
Summarising, the study comprises of the following elements:

- summary of available knowledge of sand transport on the middle and lower shoreface (Chapter 2),
- description of SUTRENCH model, UNIBEST-TC 2.0 model and DELFT-2D/3D model (Chapter 3),
- validation of SUTRENCH model based on additional laboratory and field test (Chapter 3),
- sensitivity computations using SUTRENCH model for short-term and long-term morphology of sand mining pits (Chapters 3 and 4),
- calibration of OBS-transport meter in wave tunnel of WL | DELFT HYDRAULICS (Chapter 5),
- proposal for large-scale experiments in Delta flume of WL | DELFT HYDRAULICS to improve wave-related transport components of models and to obtain data of cross-shore trench migration for validation of models (Chapter 6),
- summary, conclusions and recommendations (Chapter 7).

The study has been performed by Ir. D.J.R. Walstra, Ir. S.G.J. Aarninkhof and Prof. Dr. L.C. van Rijn of WL | DELFT HYDRAULICS and Ir. B. Grasmeijer (Chapter 5) of Dep. of Phys. Geography of Univ. of Utrecht in cooperation with Drs. S. Hoogewoning of RIKZ of Rijkswaterstaat.
2 Sand transport on the middle and lower shoreface along coast of Holland

2.1 Introduction

This chapter summarizes the knowledge of sand transport processes on the middle and lower shoreface (seaward of the -8 m depth contour). Research in 1995 (Van Rijn, 1995) has resulted in estimates (and variation ranges) of the net annual longshore and cross-shore transport rates at the -20 m depth contour at several stations along the coast, based on state of the art mathematical computations. These results are briefly discussed and compared to transport rates derived from available field data of the middle and lower shoreface (dumping Hoek van Holland 1982, dumping Wijk aan Zee 1982, trial trench Scheveningen 1964, Simon Stevin pit 1981). Available data of test pits/trenches on the upper shoreface (surf zone) like the BP-trench near Wijk aan Zee (Nelis 1987) were not considered.

The shoreface is herein defined, as follows:

- upper shoreface landward of the -8 m depth contour; where wave-driven processes (shoaling and wave breaking) are dominant; this zone is also known as the surf zone;
- middle shoreface between -8 and -20 m depth contours, where wind-, density- and tide-driven flows are controlled by bottom friction; the currents generally are parallel to the coast with during storms a secondary circulation (in transects normal to coast) superimposed on the main longshore current yielding a spiral type of fluid motion with landward flow in the surface layers and seaward flow in the near-bed layers;
- lower shoreface seaward of -20 m contour, where the currents are controlled by pressure gradients and wind forces in combination with Coriolis forces (Ekman spiral, geostrophic flows).

The boundaries of the middle and lower shoreface are conform previous definitions (Van Rijn, 1995). Some examples of shoreface profiles are shown in Figure 2.1.1.

The fluid in the shoreface zone may be homogeneous (well-mixed) or stratified with a surface layer consisting of relatively low fluid density (fresh warmer water) and a bottom layer of relatively high density (saline colder water). Strong horizontal density-related pressure gradients may occur in regions close to the river mouth of the Rhine/Meuse Estuary. In micro-tidal environments (such as Atlantic Shelf, Gulf of Mexico Shelf) the tidal currents generally are less important (<0.5 m/s) than wind-driven currents. In meso-tidal environments like the North Sea both tide- and wind-induced currents are important.

The semidiurnal tide along the Holland coast of the North Sea propagates northwards and the tidal range roughly varies between 1 and 2 m (meso-tidal). The horizontal tide becomes more asymmetric in northern directions; the peak flood and peak ebb depth-averaged velocity are about 0.6 and 0.5 m/s in depth of 20 m near Hoek van Holland (Table 2.2.2) and about 0.75 and 0.45 m/s near Den Helder. The flood duration is about 5 hours and the ebb duration is
about 7 hours along the Holland coast (Van Rijn, 1995); these values are reasonably constant along the coast.

The wave climate is rather constant along the Holland coast; the dominant wave direction is south-west. Some values of the probability of occurrence (duration in % of time) for waves in deep water are:
- south-west (180°-270°): 15% waves of 1-2 m, 4-5% between 2-3 m, 1-2% between 3-5 m;
- north-west (270°-360°): 10% between 1-2 m, 4-5% between 2-3 m, 1-2% between 3-5 m.

The median size of the bed material (Van Rijn, 1995) on the lower shoreface (20 m depth) varies between 0.15 mm (near den Helder) and 0.25 mm (near Hoek van Holland). For the upper shoreface (depth of 8 to 10 m) these values vary between 0.15 mm (Noordwijk) and 0.2 mm (Egmond).

Sand can be transported by wind-, wave-, tide- and density-driven currents (current-related transport), or by the oscillatory water motion itself (wave-related transport). The waves generally act as a sediment stirring agent, whereas the sediments are transported by the mean current. Wave-related transport may be caused by the deformation of short waves under the influence of decreasing water depth (wave asymmetry). Low-frequency waves interacting with short waves may also contribute to the sediment transport process (wave-related transport), especially in shallow water in the surf zone.

In friction-dominated deeper water on the lower shoreface zone the transport process generally is concentrated in a layer close to the sea bed and mainly takes place as bed-load transport in
close interaction with small bed forms (ripples). Bed-load transport is dominant in areas where the mean currents are relatively weak compared to the wave motion (small ratio of depth-averaged velocity and peak orbital velocity). Net sediment transport by the oscillatory motion is relatively small in depths larger than 15 m (Van Rijn, 1995, 1997), because the wave motion tends to be more symmetrical in deeper water.

Suspension of sediments on the lower shoreface can be generated by ripple-related vortices. Suspended load transport will become increasingly important with increasing strength of the tide- and wind-driven mean currents due to the turbulence-related mixing capacity of the mean current (shearing in boundary layer). By this mechanism the sediments will be mixed up from the bed-load layer to the upper layers of the flow. On the lower shoreface of the Dutch coast the suspended sand transport seems to be slightly dominant during storm conditions (Chapter 3).

Soulsby (1987) concluded that the most important contributions to the long-term sediment transport are made by fairly large but not too infrequent waves, combined with tidal currents between mean neap and maximum spring tide. Weak currents and low waves give a small contribution, because their potential for sediment transport is low, although their frequency is high. Extreme conditions also are relatively unimportant, since their frequency is too low, although their transport potential is high.

2.2 Estimates of net annual sand transport along the Dutch coast based on model computations

Estimates (based on model computations) of net annual longshore and cross-shore sand transport rates (at a depth of 20 m) in various stations (sand size of $d_{50}=0.25$ mm) along the coast of Holland have been presented by Van Rijn (1995, 1997). The results and error ranges (based on sensitivity computations varying input parameters) are also given in Table 2.2.1. The location of the stations is given in Figure 2.2.1. The peak depth-mean velocities due to tide plus wind effects used to determine the transport rates are given in Table 2.2.2. The peak tidal velocities increase along the shore in northern direction.

The model approach (UNIBEST-model) representing the hydrodynamic and sand transport processes in four different cross-shore profiles (Fig. 2.2.1) along the coast of Holland is briefly described hereafter.

The UNIBEST-model consists of 3 sub-models: wave propagation model, vertical flow structure model and sand transport model.

The wave propagation model computes the wave energy decay along a wave ray based on shoaling, refraction and energy dissipation by bottom friction and wave breaking. The near-bed instantaneous velocities are computed as time series representing irregular wave groups (including wave asymmetry and bound long wave effects).

The vertical flow structure model computes the vertical distribution of the horizontal flow velocities for a given depth-averaged velocity vector (input, see Table 2.2.2), wave height and
period, fluid density gradient and wind shear stresses (surface). The streaming in the wave boundary layer due to transfer of momentum by viscous and turbulent diffusion is taken into account. The effect of wave breaking resulting in a longshore current and a cross-shore return current (undertow) and the Coriolis effect are taken into account. The model was improved by including the roller effect and a better vertical eddy viscosity coefficient (constant) with respect to the modelling of the longshore current in the surf zone.

The sand transport model which computes the magnitude and direction of the bed load and suspended load transport, uses as input parameters: the time series velocity data and the time-averaged velocity data computed by the wave model and the vertical structure model. The various velocity components are composed to an instantaneous velocity vector, which is converted to an instantaneous bed-shear stress applying a friction factor. The suspended load transport is computed from the time-averaged velocity and sand concentration profiles.

The sediment transport rates are computed for schematised wave and corresponding current conditions. Tidal averaging is applied to obtain the tide-averaged transport rate for each wave direction and wave height class. The tide-averaged transport rate is multiplied by the percentage of occurrence of each specific wave condition, resulting in the weighted transport rate. Adding all individual weighted values, yields the mean annual sediment transport rate.

The output results of the mathematical model are net cross-shore and longshore transport rates at various locations along the shore, given in Table 2.2.1.

<table>
<thead>
<tr>
<th>Cross-shore profile (km alongshore from Den Helder)</th>
<th>Net annual sand transport rates (m³/m/year, incl. pores)</th>
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<tr>
<td>Cross-shore at depth of 20 m</td>
<td>Longshore at depth of 20 m</td>
</tr>
<tr>
<td>14, Callantsoog</td>
<td>5 ± 10</td>
</tr>
<tr>
<td>40, Egmond</td>
<td>15 ± 10</td>
</tr>
<tr>
<td>76, Noordwijk</td>
<td>10 ± 10</td>
</tr>
<tr>
<td>103, Scheveningen</td>
<td>0 ± 10</td>
</tr>
</tbody>
</table>

+ north/onshore; - south/offshore

*Table 2.2.1* Best estimates of net annual sand transport rates at a depth of 20 m in profiles 14, 40, 76 and 103 along coast of Holland (all values incl. pores of 40%)
Table 2.2.2 Peak longshore and cross-shore depth-averaged tide- and wind-induced velocities (based on model computations) for profile 14, 40, 76 and 103 along the coast of Holland

<table>
<thead>
<tr>
<th>Profile (km alongshore from Den Helder)</th>
<th>Wind velocity (m/s)</th>
<th>Water depth (m)</th>
<th>Max. flood velocity (m/s)</th>
<th>Max. ebb velocity (m/s)</th>
<th>Local wave height Hrms (m)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Longshore</td>
<td>Cross-shore</td>
<td>Longshore</td>
</tr>
<tr>
<td>14</td>
<td>0</td>
<td>20</td>
<td>0.65</td>
<td>0.05</td>
<td>-0.50</td>
</tr>
<tr>
<td></td>
<td>15 m/s</td>
<td>20</td>
<td>0.80</td>
<td>0</td>
<td>-0.30</td>
</tr>
<tr>
<td>40</td>
<td>0</td>
<td>20</td>
<td>0.65</td>
<td>0.15</td>
<td>-0.55</td>
</tr>
<tr>
<td></td>
<td>15 m/s</td>
<td>20</td>
<td>0.75</td>
<td>0.20</td>
<td>-0.40</td>
</tr>
<tr>
<td>76</td>
<td>0</td>
<td>20</td>
<td>0.65</td>
<td>0</td>
<td>-0.55</td>
</tr>
<tr>
<td></td>
<td>15 m/s</td>
<td>20</td>
<td>0.75</td>
<td>0</td>
<td>-0.45</td>
</tr>
<tr>
<td>103</td>
<td>0</td>
<td>20</td>
<td>0.65</td>
<td>0.05</td>
<td>-0.55</td>
</tr>
<tr>
<td></td>
<td>15 m/s</td>
<td>20</td>
<td>0.75</td>
<td>0.05</td>
<td>-0.45</td>
</tr>
</tbody>
</table>

(+ north/flood, onshore; - south/ebb, offshore; wind 15 m/s from south-west)

2.3 Net annual sand transport rates derived from morphological data along the Dutch coast

Available morphological data of artificial ridges (dumps) and trenches along the Dutch coast have been used to derive net annual transport rates. Four data sets have been used: dumping near Hoek van Holland 1982, dumping near Wijk aan Zee 1982, trial trench near Scheveningen 1964 and the Simon Stevin pit 1981.

Dumping near Hoek van Holland

Information of net sand transport rates can be derived from the morphological behaviour of sand dumpings related to the maintenance of navigation channels near the port of Rotterdam. During the period 1982 to 1991 an artificial sand ridge was made by dumping sand over a length of about 3600 m normal to the peak tidal current and the shore (location Hoek van Holland, see Figure 2.3.1) in an area with depths between 15 m and 23 m on the northern side of the approach channel. In all, 3.5 million m³ of sand was dumped over the period 1982 to 1991 (Rijkswaterstaat, 1996). No dumpings were carried out in sections 4 to 14 over the

The ridge dimensions are: length of about 3,600 m; toe width between 250 m and 370 m; height between 1.3 m and 4 m; slopes between 1:50 and 1:100 on the south flank and between 1:20 and 1:50 on the north flank; $d_{50}$ between 0.15 mm and 0.45 mm.

The landward end of the ridge is about 6,300 m from the shoreline. Figure 2.3.1 shows cross-section 4 (400 m from landward end) and cross-section 28 (2800 m from landward end) over a period without dumpings of sand. The cross-sections are parallel to the coast.

The basic hydrodynamic and morphodynamic data are:

- wave heights; $H=0-1$ m during 50% of time; period of 3 s; $H=1-2$ m during 35% of time; period of 4 s; $H=2-3$ m during 12% of time; period of 5 s; $H=3-4$ m during 2.5% of time; period of 6 s; $H>4$ m during 0.5% of time; period of 7 s;
- currents;
  - peak depth-mean tidal currents (spring) at location Hoek van Holland (profile 118 km) are about 0.55 m/s during flood (to the north) and ebb (to the south); dominant wind-induced forces from south-west direction will increase the flood velocities by about 20% and decrease the ebb velocities by about 20% during storms with wind velocities of 15 m/s (van Rijn, 1995, see Table 2.2.2); the peak tidal current velocities increase in northern direction along the coast;
- sediments;
  - ridge crest; $d_{50}$ between 0.4 mm and 0.45 mm;
  - southwards of southern ridge toe; $d_{50}$ between 0.23 mm and 0.27 mm;
  - northwards of northern ridge toe; $d_{50}$ between 0.17 mm and 0.36 mm;
- morphodynamics;
  - the crests in the middle part of the ridge (section 10 to 25) migrated roughly at a rate of 8 m/year in the direction of the net tide- and wind-induced current (northeastern direction, see Figure 2.3.1); the centre of gravity of the cross-sections in the middle part of the ridge migrated roughly at a rate 3 m/year in the same direction;
  - the crest level was reduced by about 0.07 to 0.1 m/year;
  - south flank slope in section 4 changed from 1:70 to 1:100 over period of 13 years;
  - the north flank slope in section 4 changed from 1:90 to 1:50;
  - the north flank slopes at some sections in the middle part of the ridge became as steep as 1 to 10.

Based on the analysis of detailed volume calculations (per unit length of the ridge), the increase of the net annual longshore sand transport from the ridge toe to the ridge crest was found to be $20\pm5$ m$^3$/m/year (including pores). A clear effect of the water depth and/or ridge height could not be determined. A bed-load transport model was used to compute the net annual longshore sand transport rates across the ridge in section 4 and 20, taking tidal velocity and orbital velocity variations over a representative year into account. Wind-induced velocities were neglected. The model was calibrated by varying the bed roughness ($k_s$) to give the observed increase of the net annual longshore transport rate at the ridge crest in section 4 and 20. The contribution of cross-shore transport was neglected. Optimum agreement was found for $k_s=0.005$ m.
The calibrated model was applied to compute the total net annual longshore transport rates in two sections, yielding:

- 40 to 65 m$^3$/m/year (incl. pores) at ridge crest in section 4 (water depth of 15 m);
- 30 to 50 m$^3$/m/year (incl. pores) at ridge crest in section 20 (water depth of 19 m).

The total transport rates are about 2 to 3 times larger than the transport increase rates (between toe and crest of ridge) derived from the morphological data.

The results of section 20 (depth of 19 m) are in reasonable agreement with the net annual longshore transport rates of Profile 103 (depth of 20 m; about 15 km north of Hoek van Holland) given in Table 2.2.1, the latter being in the range between 10 and 40 m$^3$/m/year.

**Dumping near Wijk aan Zee**

Another dump site of sand off the coast of Holland (location Wijk aan Zee, north of approach channel to port of Amsterdam, profile 50 km, see Figure 2.3.2) is located in relatively shallow water with depths between 13 and 15 m. An artificial shoal with a height of about 1.2 m was made in 1982 by dumping sand (about 1 million m$^3$) and was sounded regularly over a period of 8 years without any additional dumpings (Rijkswaterstaat, 1992). Section 2 and 4 of Figure 2.3.2 show the bed surface over the crest of the shoal. Based on the analysis of the sounding data, the crest (sections 1 and 2, Figure 2.3.2) was found to migrate at a rate between 25 and 40 m/year in the direction of the dominant flood tidal current (in north-eastern direction) over a period of 8 years. The peak flood current is about 0.65 m/s; the peak ebb current is about 0.55 m/s (see Table 2.2.2). The increase of the net annual longshore sand transport rate due to the presence of the shoal can be roughly estimated to vary between 40 and 50 m$^3$/m/year in sections 1 and 2 (Figure 2.3.2). The actual transport rates will be a factor of 2 to 3 (say 80 to 150 m$^3$/m/year), because the net annual longshore transport entering the sections has to be added (see results of dumping site near Hoek van Holland).

These values are somewhat larger than those of Table 2.2.1, yielding net annual longshore transport rates between 35 and 60 m$^3$/m/year for profiles 76 and 40 km at a depth of 20 m. The larger transport rates at the dumping site near Wijk aan Zee can be explained by the smaller depth (13 to 15 m).

**Trial trench near Scheveningen**

A trial trench was dredged in the North Sea bed (sand between 0.2 and 0.3 mm) near Scheveningen in March 1964 to obtain information of deposition rates with respect to the construction of a future sewer-pipeline trench. The trial trench was dredged perpendicular to the shoreline between 1 km (local depth of about 7 m below MSL) and 1.7 km (local depth of 10.5 m) from the RSP-baseline on the beach. The length of the trench along the traceline was about 700 m; the bottom width of the trench was about 10 m; the side slopes of the trench were about 1 to 10 and the trench depth below the surrounding sea bed was about 2 m. In all, about 30,000 m$^3$ was dredged. The local peak flood and ebb currents are estimated to be in the range of 0.4 to 0.6 m/s parallel to the shoreline (perpendicular to the trench axis).

The trench was sounded regularly in the period between 7 March and 27 August 1964. Analysis of the bed profiles showed:

- almost symmetrical filling of the trench; net migration was not observed;
- deposition of about 20 to 25 m$^3$/m over 173 days (7 March-27 Aug. 1964) or about 40 to 55 m$^3$/m per year (for currents without waves, summer effect);
• assuming that all incoming sediment is trapped in the trench, the gross (sum of absolute values) transport rates (sum of net flood and ebb transport) will be about 40 to 55 m³/m/yr for tidal currents without waves (calm weather).

Computed transport rates based on the TRANSPOR-model (Appendix D) are in the range of 25 to 70 m³/m/yr for input values: h= 7 to 10 m, peak velocity v= 0.5 to 0.6 m/s (effective duration of 8 hours per day; velocities are larger than initiation of motion during 8 hours per day), d₅₀= 0.00025 m, kₛ=0.05 m (no waves). These values are in reasonable agreement with the observed transport rates of 40 to 55 m³/m/year (calm weather).

**Mining pit Simon Stevin**

The pit was dredged on 26/27 may 1981 (Rijkswaterstaat, 1986) northwards of a dumping site (Loswal Noord near Hoek van Holland) for dredging material from the harbour of Rotterdam. The sea bed is about 15.5 m below MSL; the pit had a depth of about 6.5 m with respect to the surrounding bed and side slopes between 1 to 5 and 1 to 15. The area of the pit was about 100x100 m². The local peak tidal velocities parallel to the coast are about 0.5 to 0.6 m/s. The local bed material is fine sand (median size between 0.1 and 0.2 mm).

Analysis of regular soundings showed a natural deposition rate of about 45,000 m³ (mixture of sand and mud) over the first 520 days immediately after dredging, which is equivalent to about 320 m³/m per year. The deposition rates may be relatively high due to the fact that during the flood current the pit is situated some kilometres downstream of the Loswal Noord dumping site.

Later, the pit was used as a dump site for mud and sand dredged from the harbour docks. Analysis of samples of the deposited material showed the presence of 40% to 60% of sand, yielding a deposition rate of sand of about 130 to 200 m³/m/yr. Assuming that the pit trapped all incoming sand transport from both the flood and ebb directions, the gross sand transport rates will be about 130 to 200 m³/m/yr. These latter values representing conditions with tidal currents and waves are about 3 to 4 times larger than the values for calm weather (observed near Scheveningen), which seems reasonable.

**2.4 Conclusions**

Estimates (based on model computations) of net annual longshore and cross-shore sand transport rates (at a depth of 20 m) in various stations (sand size of d₅₀= 0.25 mm) along the coast of Holland have been presented by Van Rijn (1995, 1997). The net annual longshore transport rates at a depth of about 20 m vary in the range between 25 and 75 m³/m/yr, depending on location along the coast (Den Helder to Hoek van Holland). The net annual cross-shore transport rates at a depth of 20 m are onshore-directed and vary in the range between 0 and 15 m³/m/yr, depending on location along the coast. Net annual longshore transport rates derived from sand dump sites near Hoek van Holland (Rijkswaterstaat, 1996) and near Wijk aan Zee (Rijkswaterstaat, 1992) are in the range of 30 to 100 m³/m/yr for depths between 10 and 20 m. The results obtained from model computations and from both dump sites are of the same order of magnitude, which is a rather encouraging result.
The data of the trial trench near Scheveningen at depths between 7 and 10 m represent gross transport rates of about 40 to 55 m$^3$/m/yr for tidal currents without waves (calm summer weather). The results are in reasonable agreement with transport estimates based on the transport model given in Appendix B using tidal current velocities as input (equilibrium transport model of SUTRENCH).

The data of mining pit Simon Stevin at a depth of about 15 m represent gross annual transport rates of about 130 to 200 m$^3$/m/yr (current and wave conditions).
3 Model descriptions and SUTRENCH simulations

In this chapter an overview is given of the available morphodynamic models which can be applied to simulate the sediment transports and morphological developments on the lower shoreface. The SUTRENCH model was selected out of the described models to carry out additional simulation computations. In total three cases have been investigated with SUTRENCH. The first case was a laboratory experiment by Havinga (1992), the two other cases are in field conditions: the backfilling of a trench at the Danish coast and the development of the Euro-Maas channel. Both field cases have been relatively well monitored so that a good comparison between field data and model results is possible. The Euro-Maas channel case has been used for a sensitivity analysis in which the most important process and model parameters are investigated.

First, an overview of the available numerical models is given in Section 3.1. In Section 3.2 simulation runs of the SUTRENCH model for two cases are described. The results of a sensitivity analysis of the model are discussed in Section 3.3. Finally, Section 3.4 contains the main conclusions and a model evaluation.

3.1 Description of models

This section gives an overview of the available models at WL | DELFT HYDRAULICS to simulate sediment transport and morphodynamic behaviour of trenches or mined areas on the lower shoreface. In total three models are discussed: SUTRENCH, UNIBEST-TC and DELFT3D-MOR. For each model an overview is given of: the modelled basic processes, the basic simplifications and the types of boundary conditions which have to be specified. Furthermore, the suitability of the models for the various characteristic morphological areas (e.g. surfzone, lower shoreface) is indicated.

3.1.1 SUTRENCH model

The SUTRENCH model (Van Rijn, 1986) is a two-dimensional vertical (2DV) mathematical model for the simulation of bed-load and suspended load transport under conditions of combined quasi-steady currents and wind-induced waves over a sediment bed. All processes (parameters) in the direction normal to the computation direction are assumed to be constant.

Basic processes taken into account:

hydrodynamic
- modification of velocity profile and associated bed-shear stress due to presence of waves,
- modification of velocity and associated bed-shear stress due to the presence of non-uniform sloping bottom (only for case without waves),

sediment transport
- advection by horizontal and vertical mean current,
- vertical mixing (diffusion) by current and waves,
- settling by gravity,
- entrainment of sediment from bed due to wave- and current-induced stirring,
- bed-load transport due to combined current and wave velocities (instantaneous intra-wave approach),
- slope-related transport components (bed load),
- effect of mud on initiation of motion of sand,
- non-erosive bottom layers.

Basic simplifications are:

**hydrodynamic**
- logarithmic velocity profiles and associated bed-shear stress in conditions with waves (steep-sided trenches and channels can not be modelled),
- shoaling and refraction of wind waves is not implicitly modelled,
- current refraction (veering) is not implicitly modelled,

**sediment transport**
- steady state sediment mass conservation integrated over the width of the flow (stream tube approach),
- no longitudinal mixing (diffusion),
- no wave-related suspended sediment transport (no oscillatory transport components),
- uniform grain size (no mixtures),

**numerical**
- forward-marching numerical scheme (transport due to near-bed return currents can not be modelled),
- explicit Lax-Wendorf numerical scheme for bed level changes (smoothing effects may occur).

Boundary conditions to be specified, are:
- water depth, flow width (stream tube width, discharge is constant) and bed level along computation domain,
- wave heights along computation domain,
- equilibrium or non-equilibrium sediment concentrations at inlet (x= 0); model has option to generate equilibrium concentrations,
- bed concentration or bed concentration gradient is prescribed as function of bed-shear stress and sediment parameters,
- sediment, settling velocity, and bed roughness.

The SUTRENCH model is a process-based model to be applied at a space-scale of 1 to 5 km and at a time scale of 1 to 100 years (imposed by computation time and available computer memory). Practical application requires detailed knowledge of the sediment composition along the bed and the incoming sediment transport (at inlet boundary x=0).

The SUTRENCH model is applicable in regions outside the surfzone where wave breaking is limited. Since it is a 2DV model all processes normal to the computational direction are assumed constant. This can impose limitations to the applicability of SUTRENCH if 2DH or 3D effects play a dominant role.
Detailed formulations are given in Appendix A. The sand transport model (TRANSPOR) for equilibrium conditions (submodule of SUTRENCH model), which can be used to generate equilibrium sand transport rates at the inlet boundary (x=0), is described in Appendix B. In Section 3.2.2 an overview of previous validation results is given.

### 3.1.2 UNIBEST-TC 2.0 model

The UNIBEST-TC 2.0 model is a two-dimensional vertical (2DV) cross-shore mathematical model for the simulation of bed-load and suspended load transport and associated morphological changes under conditions of combined quasi-steady currents and wind-induced (breaking) waves over a sediment bed. In the model it is assumed that all longshore processes (parameters) are constant (uniform coast).

Basic processes taken into account, are (Bosboom et al., 1997):

- **hydrodynamic**
  - wave energy propagation, shoaling and refraction,
  - wave energy dissipation by bottom friction and wave breaking,
  - near-bed currents and asymmetry of orbital velocities,
  - bound-long waves related to wave groups,
  - modification of velocity profile and associated bed-shear stress due to presence of waves,

- **sediment transport**
  - vertical mixing (diffusion) by current and waves (suspended load transport),
  - entrainment of sediment from bed due to wave- and current-induced stirring (suspended load transport),
  - horizontal advective transport (suspended load transport),
  - bed-load transport due to combined current and wave velocities (instantaneous intra-wave approach),
  - slope-related transport components (bed load),
  - varying grain size distribution over the bottom profile (constant in time),

Basic simplifications are:

- **hydrodynamic**
  - deterministic spectral approach for wave field,
  - linear wave theory,
  - parameterised approach for long-period waves,
  - quasi-steady state approach for mean currents,

- **sediment transport**
  - no longitudinal mixing (diffusion),
  - no wave-related suspended sediment transport (no oscillatory suspended transport components),
  - uniform grain size (no mixtures),

- **numerical**
  - implicit scheme for bed level changes.
Boundary conditions to be specified, are:
- mean water depth and bed level along computation domain,
- rms-wave height and period at inlet (x=0),
- mean longshore current at inlet,
- sand transport at outlet (beach boundary),
- sediment, bed roughness and calibration parameters.

The UNIBEST-TC 2.0 model is a process-based model to be applied at a space-scale of 1 to 5 km and at a time scale of 1 to 10 years. UNIBEST-TC is a so-called profile model. It has primarily been designed to model the (cross shore) morphodynamic behaviour of the surfzone under the influence of (breaking) waves, wind and tidal currents.

A detailed overview of the implemented formulations is given in Bosboom et al. (1997). The sand transport model (submodule of UNIBEST-TC 2.0 model) is described in Appendix B.

The simulation cases considered, are:
- small-scale experiments with regular and irregular waves in laboratory flumes (Battjes and Jansen, 1978; Stive and Battjes, 1984; Battjes and Stive, 1984; Stive and Dingemans, 1984; Stive, 1986; Roelvink and Stive, 1989; Roelvink, 1993)
- large-scale LIP-experiments in Delta-flume (Reniers et al., 1995),
- field experiments; bar migration outer-delta Haringvliet (Battjes and Stive, 1984); bar migration surf zone Egmond (Reniers, 1993); bar migration surf zone Terschelling (Roelvink and Meijer, 1995).

3.1.3 Delft-2D/3D model

The DELFT2D/3D non-steady model is a universal field model, simulating waves and currents and bed evolution on a 2DH or 3D grid. In its “morphological” mode the model is a two-dimensional horizontal model (2DH) in which sediment transports are determined based on depth averaged velocity profiles. The bed-load and suspended load transport can be simulated by using a local equilibrium or non-equilibrium (lag effects) approach. The wave-related (oscillatory) bed-load and suspended load transport components are not yet modelled.

Basic processes taken into account, are:

**hydrodynamic**
- wave energy propagation, shoaling, diffraction and refraction,
- wave energy dissipation by bottom friction and wave breaking,
- wave growth by wind input,
- non-steady fluid motion (homogeneous and non-homogeneous) based on continuity and momentum equation,
- advanced turbulence models for modelling of internal shear stresses,
- modification of bed-shear stress due to presence of waves,

**sediment transport**
- vertical mixing (diffusion) by current and waves (suspended load transport),
- entrainment of sediment from bed due to wave- and current-induced stirring (suspended load transport),
• horizontal advective and diffusive transport (suspended load transport),
• bed-load transport due to combined current and wave velocities,
• slope-related transport components (bed load),

Basic simplifications are:

**hydrodynamic**
• deterministic spectral approach for wave field,
• linear wave theory,
• wave induced return currents (undertow) are not yet included in the model,

**sediment transport**
• no wave-related bed-load transport,
• no wave-related suspended sediment transport (no oscillatory transport components),
• uniform grain size (no mixtures),

The DELFT 2D/3D model is a process-based model to be applied at a space-scale of 10 to 100 km and at a time scale of 1 to 10 years. The model can be applied in areas where the 2DH processes are dominant. The model is less suitable for areas (e.g. surf zone) where the vertical velocity profiles do not have an approximately logarithmic shape.

The simulation cases considered, are:

• Validation study under prototype conditions of the Friesche Zeegat (Wang et al., 1995).
• A test case, part of an intercomparison of models in MaST-G6M, aimed at reproducing a semi-circular bay experiment carried out by LNH in Chatou (De Vriend et al., 1993).
• River flowing out into a coastal area with an initially uniform profile, also part of the intercomparison of models in MaST-G6M, (van Oudenhoven, 1992 and De Vriend et al., 1993).
• Validation study aimed at reproducing the results of physical (moveable-bed) model study carried out by WL | Delft Hydraulics in 1982 to investigate the coastal erosion at Keta Lagoon in Ghana (Walstra, 1994 and Roelvink et al., 1994).
• A validation study was carried out to establish the ability of Delft3D-MOR to reproduce the general behaviour of the coast in the vicinity of an offshore breakwater (Bos, 1996).
• In 1996 a study was carried out in which the computed wave dissipation patterns over a measured bar-rip geometry were compared with time-averaged video observations (Graaff, 1996).

### 3.1.4 Comparison of models

The SUTRENCH, UNIBEST-TC 2.0 and DELFT-2D/3D models are all process-based models, simulating bed-load and suspended load transport and associated bed level changes.

The SUTRENCH-2DV quasi-steady state model is based on detailed modelling of the sand transport processes, which are represented by expressions modelling diffusive, advective and settling processes (lag effects). The bed-load transport takes all transport components into account. The wave-related (oscillatory) suspended load transport is not yet modelled. The hydrodynamics are not modelled, but have to be given as input data (wave height along domain, discharge, width of streamtube). The model is generally applicable to sedimentation and erosion problems, provided that the hydrodynamic data are known.
UNIBEST-TC 2.0 is a 2DV model, which gives a detailed description of the hydrodynamic processes (waves and currents) in cross-shore direction, assuming invariant conditions alongshore. The bed-load and suspended load transport processes are represented by assuming local equilibrium conditions (no lag effects). The bed-load transport takes all transport into account. The wave-related (oscillatory) suspended load transport is not yet modelled. The model is only applicable for simulation of cross-shore profile evolution.

The DELFT-2D/3D non-steady model is a universal field model, simulating waves and currents on a 2DH or 3D grid; the bed-load and suspended load transport can be simulated by using a local equilibrium or non-equilibrium (lag effects) approach. The wave-related (oscillatory) bed-load and suspended load transport components are not yet modelled.

The discussed models are in principle all able to model the sediment transport on the lower shoreface. A major limitation of the UNIBEST-TC model is the fact that it is based on a local equilibrium approach. Furthermore, it is not possible to impose a cross-shore time varying tidal current. Both SUTRENCH and DELFT3D-MOR are well equipped to perform morphodynamic simulations of trenches or sand mined areas in deeper water. The construction of a DELFT3D-MOR model is however rather time consuming since a 2DH grid has to be set up, boundaries have to be specified, etc. Moreover, in the present study a number of long term simulation (50 years, see Chapter 4) have to be made. This relatively long time scale lies outside the range on which DELFT3D-MOR can be applied. It was therefore decided to apply the SUTRENCH-model. This model is relatively easy to set up and simulations of 50 years are still well feasible. The trench/area should be relatively long to ensure two-dimensional vertical conditions.

In the next section the SUTRENCH model is used for simulation of two cases.

### 3.2 Simulation results of SUTRENCH model

#### 3.2.1 Introduction

For the present study two cases have been selected to evaluate the SUTRENCH model for conditions of combined waves and currents. The first case is a laboratory experiment (Havinga, 1992) in which the sedimentation of a trench under the influence of a (constant) current in combination with waves was monitored for 25 hours. The second case is a Danish field experiment (Mangor, 1984) in which the natural backfilling of a dredged trench was monitored over a year. First, a summary of previous simulation results of SUTRENCH will be given.

#### 3.2.2 Previous simulation results

Results of earlier simulation computations are summarised hereafter.

*Unidirectional and tidal currents without waves*
- adjustment of concentration profiles in a flume,  
  (water depth= 0.25 m, current velocity= 0.67 m/s, sand= 0.23 mm),
- adjustment of sand concentration profiles in Western Scheldt Estuary,  
  (water depth= 9.5-11.2 m, current velocity= 0.8 m/s, sand= 0.16 mm),
• migration and sedimentation of a trench in a flume, see Figure 3.2.1 (water depth= 0.39 m, current velocity= 0.51 m/s, sand= 0.16 mm),

• sedimentation of a trial dredge channel in Western Scheldt estuary, see Figure 3.2.2 (water depth= 7.5-10.5 m, current velocity= 0.5-1.1 m/s, sand= 0.18 mm),

• Sedimentation of a trial dredge channel in Eastern Scheldt Estuary, see Figure 3.2.3 (water depth= 20-22 m, current velocity= 0.7-1.15 m/s, sand= 0.3 mm),

Delft Hydraulics, report R975 part II, 1977: Numerical model for non-steady suspended transport (in Dutch),

Journal of Hydraulics Division, ASCE, Hy 5, 1979: Model for suspended sediment transport, by Kerssens, P.J.M, Prins, A. and Van Rijn, L.C.,

Delft Hydraulics, report R1267 part V, 1980: Computation of siltation in dredged trenches,

Delft Hydraulics, report S488 part IV, 1985: Sutrench-model; Two-dimensional vertical mathematical model for suspended sediment transport by currents and waves,

Rijkswaterstaat Communications, No. 41, 1985: Sutrench-model; Two-dimensional vertical mathematical model for sedimentation in dredged trenches by currents and waves, by L.C. van Rijn and G.L. Tan,


• erosion of underwater sill in Eastern Scheldt Estuary, (water depth= 10-20 m, current velocity= 1-2 m/s, sand= 0.24 mm),


• trial dredge pit near Barrow in Furness, England, see Figure 3.2.4 (water depth= 3-9 m, current velocity= 0.5-0.7 m/s, sand= 0.1-0.2 mm)

Delft Hydraulics, Report H1208, 1991, Design of the Access channel to Barrow in Furness, Task 5,

• sedimentation in a trial dredge channel in Asan Bay, Korea, see Figure 3.2.5 (depth= 7-11 m, current vel.= 0.7-0.8 m/s, sand= 0.2 mm),

• siltation in a trial dredge channel near Bahia Blanca, Argentina, see Figure 3.2.6 (depth= 11 m, current vel.= 0.3 m/s, mud),

Delft Hydraulics, report S488 part IV, 1985: Sutrench-model; Two-dimensional vertical mathematical model for suspended sediment transport by currents and waves,

Journal of Waterway, Port, Coastal and Ocean Engineering, ASCE, Vol. 112, No. 5, 1986: Sedimentation of dredged channels by currents and waves, by L.C. van Rijn,
Unidirectional and tidal currents with waves

- adjustment of sand concentration profiles in a flume,
  (water depth = 0.24 m, current vel. = 0.18 m/s, waves = 0.075 m, sand = 0.1 mm),
  *Delft Hydraulics, report S488 part IV, 1985: Sutrench-model; Two-dimensional vertical mathematical model for suspended sediment transport by currents and waves,*

- migration and sedimentation of a channel in a flume, see Figure 3.2.7
  (water depth = 0.24 m, current vel. = 0.18 m/s, waves = 0.1 m, sand = 0.1 mm),

- sedimentation in navigation channel Eurochannel near Rotterdam,
  (water depth = 19-22 m, current vel. = 0.7-1.1 m/s, waves = 0.5-4 m, sand = 0.21 mm)
  *Delft Hydraulics, Report Z2268, 1997, Morphological impact of large-scale marine sand extraction, by D.J.R. Walstra and others.*

- sedimentation in navigation channel Eurochannel near Rotterdam,
  (water depth = 19-22 m, current vel. = 0.7-1.1 m/s, waves = 0.5-4 m, sand = 0.21 mm)

The main conclusion from the results of these simulation runs is that the sedimentation in trenches can be quite well simulated, provided that representative incoming transport rates are known. The evolution of the downstream trench slope can only be accurately modelled if the acceleration effects on the velocity profile are taken into account (non-logarithmic profiles). Neglecting these effects, the computed erosion and migration rate of the slope are too small.

So far, only one field data set (Eurochannel data) for combined current and wave conditions has been studied. The SUTRENCH model has not extensively been used to simulate the bed evolution for conditions of combined currents and waves. Two new cases related to sedimentation in a trench are considered in the following sub-sections.

### 3.2.3 Available data for coastal zone simulations with SUTRENCH

As stated in the previous section the SUTRENCH model has not extensively been used to simulate the bed evolution for conditions of combined currents and waves. Two new cases related to sedimentation in a trench are considered. The first case is a laboratory basin experiment (Havinga, 1992) related to the sedimentation of a trench under the influence of a (constant) current in combination with waves over 25 hours. The second case is a Danish field experiment (Mangor, 1984) related to the natural backfilling of a dredged trench over a year. In the current section a description is given of the two cases. In the Sub-sections 3.2.4 and 3.2.5 the performed SUTRENCH simulations of these two cases are described.
**Basin experiment**

The experiment has been carried out in a wave-current basin. A channel with a sediment bed consisting of fine sand ($d_{50} = 100$ µm, $d_{90} = 130$ µm) was present at the end of the basin. The movable bed surface was at the same level as the cement floor of the surrounding basin. Irregular waves were generated by a directional wave generator. The wave spectrum (JONSWAP form) was single-topped with a peak frequency of 0.4 Hz. The water depth was about 0.4 m in all tests. Three different wave conditions were used with significant wave heights of 0.07, 0.1 and 0.14 m for each wave direction. In all, three wave directions were considered $60^\circ$, $90^\circ$ and $120^\circ$ (angle between wave orthogonal and current direction). A pump system was used to generate a current in the channel. Guiding boards were used to confine the current in the movable-bed channel (width = 4 m). The guiding boards were placed normal to the wave crests in all experiments to allow free passage of the waves. Three different current velocities (0.1, 0.2 and 0.3 m/s) were generated by varying the pump discharge. The velocity distribution across the channel was found to be almost uniform (current alone). The vertical distribution of the velocity in the middle of the channel was perfectly logarithmic in all tests (current alone). The vertical distribution of the turbulence intensity was found to be in good agreement with values reported in the literature (current alone). Details have been presented by Havinga (1992).

During one of the tests (T10.20.90; wave height of about 0.1 m, current velocity of about 0.2 m/s and wave approach angle of 90$^\circ$, normal to channel) a trench was present in the movable-bed channel and the sedimentation in the trench was recorded by performing regular soundings (over about 25 hours) in three sections. The results of section 2 are shown in Figure 3.2.8. The dimensions of the initial trench profile are: depth of about 0.2 m, bottom width of about 0.5 m, side slopes of about 1 to 8, see Figure 3.2.8. The trench (longest axis) was situated normal to the current and parallel to the wave propagation direction, see Figure 3.2.8. The basic data upstream of the trench are:

- water depth = 0.42 m
- significant wave height = 0.105 m
- peak wave period = 2.2 s
- depth-mean velocity = 0.245 m/s
- angle between current and waves = 90$^\circ$
- median particle size of bed = 0.0001 m
- fall velocity of suspended sediment = 0.006 m/s
- suspended sand transport = 0.018 to 0.024 kg/s/m
- ripple height = 0.007 m
- ripple length = 0.084 m
- velocity and sand concentration profiles, see Table 3.2.1 and Figure 3.2.12.
Table 3.2.1 Basic data of basin experiment upstream of trench
Danish field data

In connection with the dual landing of a 30 inch gas pipeline and a 20 inch oil pipeline in the spring of 1982 at the Danish North Sea coast near Kørøgaard Plantation, a 1.6 km long trench was dredged through the entire littoral zone (see Figure 3.2.9). After placement of the pipelines the trench was left for natural backfilling. The dredging and backfilling were monitored closely together with wind, wave and current conditions over the four month dredging and installation period. After termination of construction the trench was left for natural backfilling. This backfilling was monitored for one year together with the environmental conditions. The study was carried out by the Danish Hydraulic Institute (Mangor, 1984 and Mangor et al., 1984).

At the site a sandy coast is present, the cross-shore profile has three characteristic bars (see Figure 3.2.9). Wind and wave conditions were collected for a period of 16 months, orbital velocities and currents were measured for about half a year. During the three month’s dredging period, soundings were carried out frequently. Backfilling was monitored for a period of one year during which period five complete soundings were made and bottom samples were collected. The collected data consists of bathymetric surveys, bottom samples, wind wave and water level data.

The tide is semi-diurnal with a tidal range of approximately 1 m. The main directions of the tidal and storm surge currents are North-South and the maximum tidal currents at mean spring lie in the order of 0.1 to 0.3 m/s. The sea bed soils consist of alternating layers of fine and medium to coarse sand, covering sandy gravel. The median grain size, d50, of the fine sand was found to be 0.14 to 0.21 mm whereas the median to coarse sand has a d50 in the range of 0.28 to 0.80 mm. In the trench also bottom samples were taken, these can be sub-divided into two different categories. Category 1 can be characterised as dark greyish green mud with varying contents of fine sand, silt and shell fragments. The median grain size, d50, lies in the range of 0.02 to 0.1 mm. Category 2 is fine to medium grey sand with varying small contents of mud. The median grain size ranges from 0.14 to 0.23 mm. Category 1 was found in areas of the trench were practically no backfilling had occurred. Category 2 was found on the bottom and sides of areas of the trench which experienced considerable backfilling.

In total backfilling was monitored during six periods. The measured profiles for CH1700 for these periods are shown in Figure 3.2.10.

From Figures 3.2.10 it can be found that in the period from 22 March 1982 to 14 April 1982 considerable backfilling occurred, whereas in the period from 31 May 1982 to 4 December 1982 no significant sedimentation of the trench was measured. To limit the computational efforts in the present study, only the period from 22 March to 14 April 1982 was simulated.

3.2.4 Simulation computations for Basin experiment

Model setup

The computational domain of the SUTRENCH simulations starts at x=13.0 m and ends at x=26.0 m (for a definition of the co-ordinates is referred to Figure 3.2.11). The computational resolution (dx) was set to 0.1 m. This leads to 131 grid points in the
computational domain. In the vertical 20 layers were defined. The model width was taken as 1 m. A timestep of 15 min was applied.

To prevent numerical instabilities at the upstream boundary (x=13.0 m), the first two computational points were defined as a non-erodable layer. This is done because SUTRENCH may develop an erosive area just inside the computational domain, if there are strong gradients in the suspended sediment close to the inlet.

The hydrodynamic boundary conditions and the sediment characteristics were obtained from the measurements (see previous section). In Figure 3.2.11 an overview is given of the model setup with the applied parameter settings of the base run (RUN T09, see next section).

**Calibration computations**

The model was calibrated by adjusting the bed concentration to give the measured suspended transport rate at the inlet (x=0). The correction factor for the bed concentration ($c_b$) was set to 2. This model setup was used as the base run (RUN T09). The main input parameters are listed in Table 3.2.2 and are also shown in Figure 3.2.11. The reference level of the bed concentration was set at the level of the ripple crest. The effective bed roughness values were estimated to be equal to 2 times the observed ripple height (by calibration).

<table>
<thead>
<tr>
<th>Model dimensions</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>13.0 to 26.0 [m]</td>
</tr>
<tr>
<td>Grid resolution (horizontal)</td>
<td>0.1 [m]</td>
</tr>
<tr>
<td>Number of layers (vertical)</td>
<td>20 [-]</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Hydrodynamic boundary conditions</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Discharge, $Q$</td>
<td>0.103 [m$^3$/s]</td>
</tr>
<tr>
<td>Wave height, $H_s$</td>
<td>0.105 [m]</td>
</tr>
<tr>
<td>Wave period, $T_z$</td>
<td>2.16 [s]</td>
</tr>
<tr>
<td>Wave direction, $Dir$</td>
<td>90° (perpendicular to current direction)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sediment characteristics</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Median grain size, $d_{50}$</td>
<td>0.000100 [m]</td>
</tr>
<tr>
<td>90% grain size, $d_{90}$</td>
<td>0.000130 [m]</td>
</tr>
<tr>
<td>Sediment fall velocity, $w_s$</td>
<td>0.006 [m/s]</td>
</tr>
<tr>
<td>Reference level, $z_a$</td>
<td>0.004 [m]</td>
</tr>
<tr>
<td>Wave related roughness height, $r_w$</td>
<td>0.01 [m]</td>
</tr>
<tr>
<td>Current related roughness height, $r_c$</td>
<td>0.01 [m]</td>
</tr>
</tbody>
</table>

(Table 3.2.2 Model parameters for Basin experiment.)

This base run is used to modify the main input parameters to investigate their influence on the resulting profile development. First, the results of the base run are discussed after which some simulations are carried out in which the sensitivity of the model to variations in the main input parameters is investigated. Secondly, the input parameters related to the sediment characteristics are varied. The suspended transport rates at the inlet are kept constant, as this represents the known measured transport rates. Finally, simulations are made with an optimal representation of the upstream concentration- and velocity profiles. Thus adjusting the suspended transport rates at the inlet boundary.
In Table 3.2.3 an overview is given of the performed simulations. The transport rates applied at the inlet (x=0) are given in each figure.

<table>
<thead>
<tr>
<th>Run</th>
<th>Remarks</th>
<th>Figures</th>
</tr>
</thead>
<tbody>
<tr>
<td>T09</td>
<td>Base run (see also Table 3.2.2) Correction coefficient for $C_a = 2$ [-]</td>
<td>3.2.12</td>
</tr>
<tr>
<td>T10</td>
<td>Decreased wave and current related roughness height $r_w=r_c=0.005$ [m]</td>
<td>3.2.13</td>
</tr>
<tr>
<td>T11</td>
<td>Increased wave and current related roughness height $r_w=r_c=0.02$ [m]</td>
<td>3.2.14</td>
</tr>
<tr>
<td>T12</td>
<td>Decreased fall velocity $w_s=0.004$ [m/s]</td>
<td>3.2.15</td>
</tr>
<tr>
<td>T13</td>
<td>Increased fall velocity $w_s=0.008$ [m/s]</td>
<td>3.2.16</td>
</tr>
<tr>
<td>T14</td>
<td>Corrected to minimum measured suspended transport Correction coefficient for $C_a=1.64$ [-]</td>
<td>3.2.17</td>
</tr>
<tr>
<td>T15</td>
<td>Corrected to maximum measured suspended transport Correction coefficient for $C_a=2.2$ [-]</td>
<td>3.2.18</td>
</tr>
<tr>
<td>T16</td>
<td>Increased discharge with 5% $Q=1.082$ [m$^3$/s]</td>
<td>3.2.19</td>
</tr>
<tr>
<td>T17</td>
<td>Increased discharge with 5% and corrected to minimum measured suspended transport $Q=1.082$ [m$^3$/s] Correction coefficient for $C_a=2.2$ [-]</td>
<td>3.2.20 to 3.2.23</td>
</tr>
</tbody>
</table>

Table 3.2.3 Overview of performed simulations for the basin experiment.

**Base run**

In Figure 3.2.12 the main results of the base run are shown. The two graphs at the top compare the measured and calculated velocities and the concentrations respectively at the upstream boundary. The velocities seem to be underestimated slightly whereas the concentrations are somewhat too high. The middle graph shows the initial transports (i.e. at $t=0$). As can be seen the initial profile has been smoothed to avoid numerical instabilities. The bottom graph shows the initial profile and the measured and computed bottom profile at the end of the experiment (25hr30). The middle graph reveals that the bottom transport is negligible to the suspended transport. It can be seen that SUTRENCH overestimates the sedimentation in the trench. There is however a reasonable correspondence between the measured and calculated profile.

**Variation of sediment characteristics**

wave- and current related roughness height

To investigate whether the applied wave and current related roughness heights are realistic two calculations have been made in which both parameters are set to 0.005 m and 0.02 m respectively. The transport rates at the upstream boundary are kept constant by correcting the bed concentration by means of varying the $c_a$-factor. A decreased value ($r_w=r_c=0.005$ m
(50%) and correction coefficient for $c_a$ is 4.36, RUN T10) leads to an improved concentration profile (compare Figures 3.2.13 and 3.2.14). The influence on the velocity profile is minimal, a slight increase can be observed near the bottom. The resulting profile after 25hr30 shows that the sedimentation in the trench is overestimated significantly. The migration of the trench is also overestimated. An increased value ($r_w=r_c=0.02$ m (200%) and correction coefficient for $c_a$ is 0.73, RUN T11) results in an underestimation of the near bottom velocities and an overestimation of the concentration profile at the upstream boundary, see Figure 3.2.14. The morphological development is in good agreement with the measurements. The upstream slope of the trench is well predicted in terms of migration and rotation compared to the base run.

**Fall velocity of suspended sediment**

To investigate the effect of variations in the fall velocity of the suspended sediment two calculations have been made in which the fall velocity ($w_s$) was set to 0.004 m/s (60% and correction coefficient for $c_a$ is 1.02, RUN T12) and 0.008 m/s (130% and correction coefficient for $c_a$ is 3.46, RUN T13), respectively. The velocity profile is not influenced by these variations. The suspended sediment concentrations however are somewhat affected as can be seen in Figures 3.2.15 and 3.2.16. A decreased $w_s$ results in an overestimation of the suspended sediment concentration in the upper half of the water column and an underestimation in the lower half. An increased $w_s$ does not seem to affect the concentration distribution significantly. If the transport rates are compared, the (expected) differences are: in case of an increased $w_s$ more sediment settles in the trench, a reduced $w_s$ leads to a stronger lag effect so relatively less sediment will settle in the trench. The resulting bottom profile development after 25hr30 shows a significant sedimentation of the trench in case of an increased $w_s$, both the sedimentation and the migration of the trench are overestimated. A decrease of $w_s$ has the opposite effect.

From the variation of the sediment-related input parameters it can be concluded that the applied settings in the base run (RUN T09) yields reasonable results. The simulation with an increased value for the wave and current related roughness height (RUN T11) did show the best agreement with the measured profile after 25hr30. The velocity and sediment concentration profiles however deviate considerable from the measured values, especially compared to the resulting vertical profiles from the base run.

**Optimisation of velocity and concentration profiles**

**Optimisation of concentration profiles**

In the laboratory basin experiment the total amount of suspended transport has been derived from the measurements using three different extrapolation methods (see for details Havinga, 1992). The suspended sand transport was found to be between 0.018 and 0.024 kg/m/s. Two simulations were made, in which the suspended sediment was respectively scaled to the minimum and maximum suspended sediment. This is done by scaling the bed concentration to the appropriate value by varying the bed concentration correction factor. In RUN T14 the calculated suspended sediment was scaled to the minimum value by setting the correction coefficient for $c_a$ to 1.64. In Figure 3.2.17 it can be seen that, in comparison with the base run, the concentration profile shows an improved agreement with the measured values. Also the resulting bottom profile shows good agreement with the measurements.

By setting the correction coefficient for $c_a$ to 2.2 the maximum suspended sediment transport is calculated (RUN T15). As can be seen in Figure 3.2.18 the concentrations are overestimated. Consequently also the sedimentation of the trench is overestimated.
Optimisation of velocity profiles

In the base run the upstream velocities are somewhat underestimated (Figure 3.2.19). By increasing the applied discharge with 5% (RUN T16) the velocity profile shows an improved agreement with the measurements (see Figure 3.2.19). The concentration profile and the resulting sedimentation in the trench are overestimated. By correcting the bed concentration to the minimum measured suspended sediment transport (correction coefficient for $c_n=1.36$, RUN T17, Figure 3.2.20), the final bottom profile shows good agreement with the measurements.

In the Figures 3.2.21 to 3.2.23 the development of the trench in time is shown (RUN T17). Until 12hr30 the upstream slope of the trench is modelled fairly accurate. The erosion of the downstream slope and the adjacent section is however underestimated. The three last displayed times show that the upstream slope of the trench is calculated somewhat too steep. In general it can be concluded that SUTRENCH is able to give a good representation of the morphological development of the trench. The downstream erosion is however underestimated whereas the upstream slope of the trench is too steep. This is caused by the fact that the influence of the acceleration of the water over the trench slope on the vertical velocity distribution is not modelled in SUTRENCH. This results in an underestimation of the near bed velocities and as a consequence also of the picking up of sediment, resulting in an underestimation of the erosion on the downstream slope.

3.2.5 Simulation computations for Danish field experiment

General

For the present study one cross-section over the dredged trench was simulated at approximately 1700 m offshore (CH1700 in Figure 3.2.9). The orientation of the dredged trench is 310°N whereas the orientation of the coast normal is 285°N. At this cross-section the water depth is 6.5 m whereas the water depth in the trench is 12 m (see also Figure 3.2.9).

From Figures 3.2.10 it can be found that in the period from 22 March 1982 to 14 April 1982 considerable backfilling occurred, whereas in the period from 31 May 1982 to 4 December 1982 no significant sedimentation of the trench was measured. To limit the computational efforts in the present study, only the period from 22 March to 14 April 1982 was simulated.

Computational Domain

The computational domain is from $x=0$ m to $x=129$ m. The computational resolution is set to $dx=1$ m resulting in 130 computational grid points. In the vertical 20 layers are defined. In Figure 3.2.24 an overview is given of the computational setup. To prevent numerical instabilities the first two computational grid points are defined as non-erodable layers. Although the trench lies under an angle 65° with the main current directions (which are approximately parallel to the coast) no current refraction is taken into account. This was done because no accurate current measurements were available for the considered period.
Tide schematisation

The horizontal tide is schematised into two intervals, a southerly and northerly directed current which has an effective duration of 4 hours. The applied current velocities are estimated based on available field data, taking tidal and meteorological effects into account. The values applied are given in the description of the simulations later in this section.

In the simulations a constant water level was applied which approximately coincides with the appropriate mean sea level (MSL).

Wave schematisation

In the measured wave conditions of the simulated period from 22 March 1982 to 14 April 1982 three characteristic periods can be distinguished, as can be seen in Figure 3.2.25 where the measured wave heights for the referred period are shown. From 22 March to 7 April calm conditions have been measured. From 7 April to 13 April a storm was present. During the last two days, the wave heights decrease. In Table 3.2.4 the schematised wave conditions for the SUTRENCH simulations are presented:

<table>
<thead>
<tr>
<th>Period</th>
<th>Wave height, $H_s$</th>
<th>Wave period, $T_p$</th>
<th>Wave Direction</th>
<th>Direction in SUTRENCH</th>
</tr>
</thead>
<tbody>
<tr>
<td>22/03/82 to 07/04/82</td>
<td>0.5 [m]</td>
<td>2.5 [s]</td>
<td>300°N</td>
<td>285°</td>
</tr>
<tr>
<td>07/04/82 to 13/04/82</td>
<td>3.0 [m]</td>
<td>6.5 [s]</td>
<td>315°N</td>
<td>285°</td>
</tr>
<tr>
<td>13/04/82 to 15/04/82</td>
<td>1.0 [m]</td>
<td>3.0 [s]</td>
<td>300°N</td>
<td>285°</td>
</tr>
</tbody>
</table>

Table 3.2.4 Applied wave climate in Danish field experiment

Simulation computations

As can be seen in Table 3.2.4 the selected period contains a severe storm with a duration of approximately 6 days. It is thought that during this storm most backfilling will have taken place. To verify this assumption, a SUTRENCH calculation was made in which the period prior to the storm was simulated. As there is no significant backfilling during this period, it has been omitted in the simulations. Subsequently a base run is defined which is used to investigate the influence of variations in various model parameters in an attempt to improve the predicted backfilling of the trench. The bed roughness values were taken to be 0.05 m to simulate minor bed irregularities (flat ripples and undulations in the surf zone). The reference level for the bed concentration is taken equal (0.05 m) to the bed roughness. The correction factor for the bed concentration was assumed to be 1 (default value).

First, an overview is given of the applied model parameters in the initial SUTRENCH simulations:
Model dimensions

Length 0.0 to 129.0 [m]
Grid resolution (horizontal) 1.0 [m]
Number of layers (vertical) 20 [-]

Hydrodynamic boundary conditions

Discharge (southerly), \( Q \) See various runs [m³/s]
Discharge (northerly), \( Q \) See various runs [m³/s]
Wave height, \( H_s \) See Table 3.2.4
Wave period, \( T_z \) See Table 3.2.4
Angle between wave and current dir., \( \text{Dir} \) 90° (perpendicular to current direction)

Sediment characteristics

Median grain size, \( d_{50} \) 0.000200 [m]
90% grain size, \( d_{90} \) 0.000300 [m]
Sediment fall velocity, \( w_s \) 0.025 [m/s]
Reference level, \( z_a \) 0.05 [m]
Wave related roughness height, \( r_w \) 0.05 [m]
Current related roughness height, \( r_c \) 0.05 [m]

Table 3.2.5  Model parameters for Danish field experiment.

Calculated backfilling of trench during calm conditions (RUN D08)

Calm sea conditions are present from 22 March 1982 to 7 April 1982. The corresponding wave condition is listed in Table 3.2.4. The parameters as listed in the table above are applied in the model. The south-going tidal velocities are 0.2 m/s and north-going 0.1 m/s. In Figure 3.2.26 the two top graphs show the calculated velocity profiles in southerly and northerly directions respectively. The middle graph shows the calculated residual transports (averaged over the tidal cycle). The bed-load transport is almost zero for these conditions. The graph at the bottom of the figure shows the computed profile at 7 April. To give an indication of the backfilling during the considered period, the measured profile at 15 April is also shown. As can be seen no significant sedimentation has occurred during the calm sea conditions (period 22 March to 7 April). No differences can be observered between the initial and final profile.

Backfilling during storm conditions (RUN D09)

Storm conditions were present from 7 April to 13 April. The corresponding wave condition is listed in Table 3.2.4. The model parameters as shown in Table 3.2.5 are applied in the model. Unfortunately no velocity measurements were made during this period. The velocity measurements are made from 7 May 1982 to 8 November 1982. During this period one storm comparable to the simulated period was present. During this storm, which occurred at 13 June 1982, maximum windspeeds were 17 m/s and a maximum significant wave height of 2.3 m was observed. The storm of 7-13 April had a maximum windspeed of 20 m/s and a maximum wave height of 4.5 m. During the June-storm only south-going velocities were measured; a maximum velocity of 0.40 m/s was measured. The other peak in the tidal period was in the order of 0.30 m/s. Since the April storm was of a longer duration with higher windspeeds and wave heights, the applied (depth averaged) currents were
subjectively set to 0.50 and 0.40 m/s respectively (both in southerly direction as in the June-
storm).

It can be seen in Figure 3.2.27 that the trench experiences considerable backfilling during
the extreme condition. The migration of the trench is however underestimated with
approximately 20 m. The slopes are too steep on both sides, especially the northern slope
does not correspond well with the observed profile. The measured erosion north of the
trench is not predicted and also the erosion on the southern slope is underestimated. The
total amount of predicted backfilling is about 100 m$^3$/m. If it is assumed that the backfilling
mainly has taken place during the six days storm and all sediment would be trapped in the
trench (100% trapping efficiency), a residual transport of approximately 0.4 kg/m/s results.
In the simulation a residual transport of 1.6 kg/m/s was found. The trapping efficiency is in
the order of 25 to 50% (see Figure 3.2.27 middle graph) so the residual transports seem to
be of the right order of magnitude. The deviations between the predicted and observed
morphological behaviour of the trench are probably mainly caused by the rather crude tidal
schematisation.

**Simulation with decreased velocities (RUN D10)**

In RUN D10 the tidal velocities were set to 0.4 and 0.3 m/s respectively; both in southerly
directions. All other model settings are identical to RUN D09. The residual transports are
now in the order of 0.7 kg/m/s which is still too high. As can be seen in Figure 3.2.28
backfilling is underestimated considerably. If results of RUN D09 and D10 are compared,
the results of the former simulation are corresponding better with the measured profile
development. In the next simulation the effects of a smaller grain size are investigated.

**Simulation with a smaller grain size (RUN D11)**

In this simulation the grain size $d_{50}$ is set to 150 $\mu$m, $d_{90}$ is set to 200 $\mu$m. The
corresponding sediment fall velocity is then 0.014 m/s. In Figure 3.2.29 the resulting profile
is shown. It is clear that the backfilling is overestimated significantly. The migration of the
trench is however still underestimated considerably.

**3.2.6 Conclusions**

From the two investigated cases it can be concluded that SUTRENCH gives fairly reliable
results. The results for the basin experiment show good agreement with the measurements.
The migration of the trench is modelled accurately, the morphological development of the
trench slopes is not modelled satisfactory. The upstream slope is generally predicted too
steep whereas the erosion of the downstream slope is underpredicted. From the basin
experiment it is found that the trench slope development is influenced by varying the
roughness heights (cf. Figures 3.2.13 and 3.2.14). Varying the sediment fall velocity mainly
influences the sedimentation (and migration) of the trench. An optimal fit to the upstream
boundary velocity and concentration profiles (Figures 3.2.20 to 3.2.23) resulted in a good
representation of the measured trench development. Hindcast of the Danish field
experiment at CH1700 (cf. Figure 3.2.9) was hampered by the fact that no current data was
available for the simulated period. The migration and sedimentation of the trench over a
period of 3 weeks showed reasonable agreement with the observed morphological
developments by using reasonable estimates for the tide-and wind-induced currents. Also
for this case SUTRENCH predicts a too steep upstream slope and an under-prediction of the
erosion of the downstream slope.
The main conclusion from the results of these simulation runs is that the sedimentation in trenches can be quite well simulated, provided that representative incoming transport rates are known. The evolution of the downstream trench slope can only be accurately modelled if the acceleration effects on the velocity profile are taken into account (non-logarithmic profiles). Neglecting these effects, the computed erosion and migration rate of the slope are too small.

### 3.3 Sensitivity computations of SUTRENCH-model

#### 3.3.1 Introduction

In the previous section SUTRENCH was used to simulate two cases of sedimentation in a trench for combined currents and waves. Especially from the basin experiment it could be found that SUTRENCH yields reliable results if the measured upstream boundary conditions are represented accurately. This is a good indication of the validity of SUTRENCH with respect to the sedimentation and erosion processes. The present section presents a number of sensitivity analyses for the approach channel to Rotterdam Harbour: the Euro-Maas Channel (see also Figure 3.3.3).

The sensitivity computations were sub-divided into three separate analyses. First the process and model parameters are investigated. Next, the effect of the tidal schematisation on morphological development is studied. In the last sensitivity analysis an attempt is made to quantify the effect of the initial geometry on the morphological development of a trench or pit. Results of two detailed SUTRENCH model studies were available which could be used as a reference (Walstra et al. 1997 and Hoitink 1997). In the next section the two models are evaluated based on which a so-called base run is defined. The settings of the base run are used in the sensitivity analyses. Before discussing the sensitivity runs, a number of evaluation criteria (output parameters) have been defined to characterise the results. Finally, the results are evaluated and some conclusions are drawn.

#### 3.3.2 Definition of Base run

Results of different model runs will be compared to the results of the base run. In the present study, this base run is defined on an idealised bottom profile, closely resembling the Euro-Maas channel profile (at a water depth of approximately 20 m), with ‘reasonable’ forcing conditions and parameter settings. To define such a base run, ‘reasonable’ forcing conditions and parameter settings need to be found from a SUTRENCH base run on field data, in this case the Euro-Maas channel.

In this section, the settings for this base run will be defined. To that end, the results of recent studies based on SUTRENCH simulations of the Euro-Maas channel will be reviewed first. Finally, the base run itself will be evaluated.

Review of SUTRENCH applications to the Euro-Maas channel

Recently two studies (Walstra et al., 1997 and Hoitink, 1997) have been carried out in which the SUTRENCH model was applied to the approach channel of Rotterdam Harbour, the Euro-Maas channel. Both studies concentrated on the morphodynamic behaviour of the
Euro-Maas channel over a period of five years. The SUTRENCH-model was calibrated against bathymetric surveys and available maintenance dredging volumes. In this section both approaches are discussed and compared, with respect to the schematisation of the tidal currents, the wave climate and the wind induced residual current, as well as the setting of model parameters.

**Comparison of applied forcing conditions and parameter settings**

**Schematisation of tidal currents**
For the calibration in Walstra et al. (1997) a morphological tide was obtained according to the method of Latteux (Latteux, 1995). With DELFT3D-MOR, a 2DH-morphodynamic modelling system, the residual sediment transport rates over a period from neap to spring tide were determined (period: 11 July 1988 to 20 July 1988) with the dedicated Maasvlakte-2 model (Steijn et al., 1996 and Walstra et al., 1997). Subsequently, the residual sediment transport rates resulting from time-averaging over the individual tides were evaluated against the transports averaged over the period from neap to spring tide. The tide which gave the best representation of the residual transport rates was selected as the applied morphological tide (selected tide: 3hr20min 18 July 1988 to 15hr40min 18 July 1988). In Figure 3.3.25 second graph the selected tide is displayed, in this graph also the tidal schematisation as applied in the SUTRENCH model is given. Below a brief description of the applied method of Latteux is given:

**method of Latteux**
The method of Latteux is aimed at obtaining a representative tidal schematisation with respect to the residual transport rates. Individual tides are evaluated by comparing the residual transport fields (on the complete computational grid of the DELFT3D-MOR model) of this evaluated tide and the residual transports of a longer period (ideally a complete neap-spring tidal cycle). Below the applied steps are listed pointwise:

- **Determine residual transports of long period**
  Simulate a period over which it is expected that the residual transport rates are representative for yearly averaged transport rates. The resulting transport rates are used as a reference for the evaluation of the individual tides present in the considered period.

- **Determine scaling factor**
  The ratio between the residual transports of the individual tide and the long period is calculated in each computational cell. The scaling factor is subsequently determined by averaging the ratios over the complete grid.

- **Scaling of the transports of the individual tide**
  The residual transports of the individual tide are then scaled by multiplying them with the scaling factor obtained in the previous step.

- **Determine standard deviation**
  As a last step the standard deviation of the difference between the scaled transports of the individual tide and the residual transport of the long period is determined.

The described method is applied on all the individual tides present in the long period. The tide with the smallest standard deviation is chosen as the morphological tide. For more information on the applied method the reader is referred to Walstra et al. (1997).

Hoitink followed a somewhat different approach. In this study the residual currents where calculated in one location just south of the Euro-Maas channel over a period from neap to spring tide. To represent the residual transport rates, two tides were selected around neap and spring tide respectively. The residual transport rates were represented by assigning weights to both tides.
Morphological wave climate

In both studies one wave condition was derived to give the best representation of the overall, yearly wave climate. In Walstra et al. (1997) this was done by applying the Latteux method to evaluate the residual transport rates. Hoitink (1997) selected a wave condition by comparing the resulting sedimentation-erosion rates after one year. Both climates are summarised in Table 3.3.1 below.

<table>
<thead>
<tr>
<th>Study</th>
<th>$H_s$ (m)</th>
<th>$T_s$ (s)</th>
<th>Direction ($°$ N)</th>
<th>Period (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Walstra et al. (1997)</td>
<td>2.25</td>
<td>6.6</td>
<td>315</td>
<td>84</td>
</tr>
<tr>
<td>Hoitink (1997)</td>
<td>2.25</td>
<td>5.0</td>
<td>-</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 3.3.1  Applied wave conditions

Wind induced residual currents

A general rule of thumb is that wind induced surface currents are in the order of 1 to 2.5 % of the wind velocities (see e.g. Allersma and Ribberink, 1992). Along the Dutch coast prevailing winds are from south-westerly directions which lead to northward directed residual currents. In SUTRENCH this current can be taken into account by superimposing it on the tidal discharges. In Walstra et al. (1997) a residual current of 0.04 m/s was applied, in Hoitink (1997) wind induced currents were neglected.

Settings of model parameters

In Walstra et al. (1997) the default parameter settings according Van Rijn and Tan (1985) were applied. The model settings according to Hoitink (1997) were obtained from a calibration against profile evolution data after 5 years. Both settings are summarised in the table below.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference level, $z_a$ (m)</td>
<td>0.05</td>
<td>0.024</td>
</tr>
<tr>
<td>Wave roughness height, $r_w$ (m)</td>
<td>0.01</td>
<td>0.001</td>
</tr>
<tr>
<td>Current roughness height, $r_c$ (m)</td>
<td>0.05</td>
<td>0.001</td>
</tr>
<tr>
<td>Fall velocity, $w_s$ (m/s)</td>
<td>0.028</td>
<td>0.0245</td>
</tr>
<tr>
<td>Median grainsize, $D_{50}$ ($\mu$m)</td>
<td>210</td>
<td>210</td>
</tr>
<tr>
<td>90% grainsize, $D_{90}$ ($\mu$m)</td>
<td>310</td>
<td>309</td>
</tr>
</tbody>
</table>

Table 3.3.2  Applied settings in Walstra et al. (1997) and Hoitink (1997)

Evaluation of both approaches

Differences between the Hoitink and Walstra et al. approach merely originate from a different schematisation of the tide. The Hoitink approach, where a spring tide and a neap tide are weighted, is rather artificial; moreover, the presence of spring tide during 79% of time is not realistic. Due to this rather crude schematisation of the tidal currents, unrealistic model parameter settings needed to be chosen in order to calibrate the model against profile evolution data. Moreover, the model parameters lie outside the range over which they are derived from measured velocity profiles under laboratory and field conditions (Van Rijn, 1993). The applied wave climates, finally, differ only in a minor way.

The residual yearly transport rates as predicted by both studies are comparable. Just south of the Euro-Maas channel Hoitink (1997) reports northward transport rates in the order of
140-100 m$^3$/m/year, while inside the channel the rate amounts 20-30 m$^3$/m/year. In Walstra et al. (1997) transport rates of about 130 m$^3$/m/year just south of the Euro-Maas channel, and about 30 m$^3$/m/year inside the channel were reported. Recent field measurements on the migration of an artificial sand dam have shown that the yearly-averaged transport rates lie in the order of 30 to 60 m$^3$/m/year (see Chapter 2 of this report and Woudenberg, 1996).

In both approaches the SUTRENCH model was calibrated to give an optimal representation of the observed developments of the Euro-Maas channel. This however, has resulted in residual transport rates that are significantly higher then those reported in Woudenberg (1996). There seems to be an inconsistency between the expected residual transport rates and the observed morphodynamic behaviour of the channel. The under-estimation of the erosion of the northern slope was also observed in the calibration computations for the basin experiment and the Danish field experiment (see Section 3.2). It is thought that this is mainly due to the fact that the influence of the accelerating velocities over the slope on the vertical velocity profile is not modelled. For the Euro-Maas channel case it is possible that three-dimensional or two-dimensional horizontal (2DH) effects also play a significant role in the development of the channel. More information on this subject can be found in the sub-section of the evaluation of results of the base run.

**Settings of base run**

Based on the considerations above, it was concluded to apply forcing conditions (waves, tide) as described in Walstra et al. (1997). Likewise, the default model parameter settings as suggested by Van Rijn and Tan (1985) are taken as a starting point. Initially, by means of some test calculations, an attempt will be made to obtain slightly modified parameter settings for a base run, resulting in decreased transport rates, in better accordance with rates found in field. As a consequence, the calculated morphological developments of the Euro-Maas channel are in less accordance with the observed sedimentation-erosion patterns.

The bottom profile is obtained from a 1990-field survey, at a distance of 4650 m from the light at the Noorderdam breakwater. At this location, the current field is approximately uniform, with streamlines crossing the Euro-Maas channel under an angle of 17°. The schematisation of the wave climate and the tide does not differ from Walstra et al. (1997), hence reference is made to their report. Given the inaccuracy regarding the estimate of the residual, yearly, wind-induced current, it was decided to ignore the yearly averaged wind induced residual current. In Walstra et al. (1997) this had been included by adding a constant current of 0.04 m/s to the tidal current velocities. For completeness sake, the settings of all model parameters and boundary conditions are summarised in the table below. The correction factor for the reference concentration was set to a value (0.7), that gave the right order of magnitude for the mean annual longshore transport rate (about 50 m$^3$/m/yr).
Model parameter / Boundary Condition | Setting
---|---
Representative wave height, $H_{sig}$ (m) | 2.25
Wave period, $T_{sig}$ (s) | 6.6
Wave direction (degrees, relative to north) | 315
Percentage of occurrence representative wave (%) | 84
Wave related roughness, $R_w$ (m) | 0.01
Current related roughness, $R_c$ (m) | 0.05
Reference level, $Z_a$ (m) | 0.05
Median grain size, $d_{50}$ ($\mu$m) | 210
90% Grain size, $d_{90}$ ($\mu$m) | 310
Sediment fall velocity, $w_s$ (m/s) | 0.0275
Computational step size, $dx$ (m) | 10
Number of computational points in vertical | 15
Number of time steps per tide | 19
Coefficient pseudo viscosity, $\alpha_{\text{max}}$ | 0.0005
Correction factor for $c_a$ | 0.7

Table 3.3.3  Settings of base run

Evaluation of results base run

Figure 3.3.2 shows the transport rates, morphological changes and sedimentation-erosion after three years, as obtained from simulations with the base run. Due to the modified model settings, transport rates have decreased with respect to the rates reported by Walstra et al. (1997): just south of the Euro-Maas channel, the residual, yearly transport rate amount 50 to 55 m$^3$/m/year, in the channel itself about 10 m$^3$/m/year (Figure 3.3.2, second graph). To the north of the channel, transport capacity increases again, yielding transport rates of about 40 m$^3$/m/year. As a result, about 10 to 15 m$^3$/m/year of sediment is trapped by the channel. These residual transport rates are in good correspondence with the rates of 30 to 60 m$^3$/m/year reported by Woudenberg (1996) and the rates about 60 m$^3$/m/year (see also Chapter 2 for more information), found by Allersma and Ribberink (1992) at a depth of 19 m.

Figure 3.3.2 also shows that the suspended sediment transports dominate the bottom transports by a factor 4. As the suspended transport rate strongly depends on flow velocities, this explains the rather strong decrease in transport rates in the channel itself, where suspended transport and bottom transport are of the same order of magnitude. The amount of sediment transported during flood exceeds the amount during ebb by a factor 2.

Finally, Figure 3.3.2 (fourth graph) shows the sedimentation-erosion after 3 years. From the integrated area under the graph, it can be seen that the sedimentation at the southern slope is about 40 m$^3$/m/year, the erosion of the northern slope about 35 m$^3$/m/year, causing displacements of the slopes of order 5-10 m/year. Clearly these gradients are too small to explain the displacement of the channel of about 25 m/year, as observed in the field (cf. Walstra et al., where gradients of order 80 m$^3$/m/year across the slopes were needed in order to reproduce the morphodynamic behaviour observed in the field). Apparently, additional transport mechanisms are active, apart from the cross-channel mechanisms as modelled with SUTRENCH. It is thought that this is mainly due to the fact that the influence of the accelerating velocities over the slope on the vertical velocity profile is not modelled. For the Euro-Maas channel case it is possible that three-dimensional or two-dimensional
horizontal (2DH) effects also play a significant role in the development of the channel. In this respect one can think of for instance spiral flows through the channel, stirring sediment from the slopes which settles at the floor of the channel. In addition, also the setting of the gravity terms in SUTRENCH might play a role. If the process of settling of suspended sediment grains is slowed down relative to the process of picking up sediment particles by the flow, residual transport rates in the channel decrease (can even become negative!) and hence gradients across the slopes increase without an overall increase of the sediment transport rates (see Sections 3.3.4 to 3.3.6).

From the considerations above, it can be concluded that the present model reasonably well reproduces the order of magnitude of the transport rates as observed at the Euro-Maas channel. The morphological changes as obtained from the base run underestimate the changes observed in the field. Based on these observations, it is decided to adopt the base run as a starting point for the sensitivity analysis as described in the following section.

**Base run with idealised bottom profile**

Finally, an idealised bottom schematisation has been used. The trench has the approximate same dimensions as the trench of the Euro-Maas channel but is symmetrical. Also the adjacent bottom is perfectly horizontal, whereas in prototype conditions a slope is present. In Figure 3.3.3 (first graph) of the base run also the 1990 measured bathymetry (as applied in the base run) is shown for comparison. It can be seen that for the constructed profile the upstream (southern) slope form the reference profile has been taken also has been applied on the downstream (northern) slope. In all sensitivity computations the schematised bottom profile is used as a basis. In Figure 3.3.3 also the residual transports and during flood and ebb occurring transport rates are shown in the second and third graph, respectively. The fourth (bottom) graph the sedimentation erosion after three years is shown. The definitions of the indicated trench characteristics will be discussed in the next section.

**3.3.3 Definition of output parameters for sensitivity analyses**

**General**

In order to make an objective comparison between the various applied model settings and investigated trench geometry’s it is necessary to formulate some characteristic features (e.g. trench dimension, volumes, slopes, etc.). It is important that the applied method also yields a reliable definition of the slopes, even after a trench has experienced considerable sedimentation and flattening of the downstream slope. In this section a definition method is described which will be applied on the resulting bottom profiles and sediment transports. The method defines the outer dimensions of the trench, some characteristic locations for determining e.g. the trapping efficiency, the trench volume, width and averaged depth and the subdivision of the trench in three characteristic regions.

**Definition of trench dimensions**

In this sub-section a robust method is described for the definition of the trench dimensions. These trench dimensions are used to define trench features such as the width, average depth and volume of the trench. Furthermore, these trench definitions are, among others, also used to define the trapping efficiency and the sedimentation and erosion in characteristic trench regions.
The angle of the trench slopes is here defined by the inflection points of the first derivative of the trench bottom profile. The trench slopes are thus defined by the inflection points of the slope angle. The first derivative of the bottom profile results in the gradients, the second derivative represents the slope gradients. The zero-crossings of the third derivative then give the inflection points of the slope gradients (i.e. first derivative). In Figure 3.3.4 this is shown graphically, the graphs show from top to bottom respectively the bottom profile, the bottom gradients (first derivative), the gradients of bottom angles (second derivative) and the third derivative. In all graphs the zero-crossings are indicated.

Based on the derived trench slope angles a logarithmic approximation of the trench slopes can be made. The lower inflection points, defining the transition between the trench bottom and slopes of the trench (see Figure 3.3.5, points $P_2$ and $P_3$), are used the starting point of the logarithmic approximation. Below the applied logarithmic function is described.

It is assumed that the trench slopes can be approximated by a logarithmic function of the following form:

$$h(x) = h_e \left(1 - e^{-\frac{x}{x_{tau}}}\right)$$  \hspace{1cm} (3.3.1)

where $h$ is the resulting slope profile as a function a horizontal co-ordinate $x$, $h_e$ is the reference level which acts as a asymptotic limit and $x_{tau}$ which can be interpreted as a horizontal length scale.

This function starts from the lower inflection point (points $P_2$ and $P_3$ in Figure 3.3.5). It is assumed that the trench slope is a good representative of the gradient of the function at the starting point (the line through Points $P_3$ and $P_4$ and Points $P_1$ and $P_2$ in Figure 3.3.5). With this assumption the logarithmic function has been defined as the crossing of the line through the inflection points with the reference line, giving $x_{tau}$. The $h_e$ factor is equal to the vertical distance from the reference line to the bottom inflection point ($P_2$ and $P_3$). The resulting logarithmic approximation is shown in Figure 3.3.5 as a dashed line. It can be clearly seen that this approximation is in good agreement with the calculated profile which gives a good indication of the validity of the applied method. The outer dimensions of the trench are now defined by the vertical line from this zero-crossing to the bottom profile (Points $P_5$ and $P_6$ in Figure 3.3.5).

With this simple method, the four points define the trench geometry objectively. These four points can now be used to define some characteristic transport locations, trench volume, width etc. and some characteristic regions of the trench or pit. The next sub-section will be devoted to those definitions.

**Definition of transport locations and trench characteristics**

In the sensitivity analysis it is important to study the resulting transports over a trench or pit. For this reason in total six locations will be considered largely based on the trench definition as described in the previous sub-section.

In Figure 3.3.5 second graph the six locations are shown. The points $S_1$ to $S_4$ are located on the top and bottom of the trench slopes. Also the transports at the up- and downstream
boundary are evaluated, these are shown as $S_{x=0}$ and $S_{x=n}$ respectively. Besides the comparison of the occurring transports in the six individual points also two types of sediment trapping efficiency are defined. The first is the trapping efficiency over the complete trench:

$$TE_{tot} = \frac{S1 - S4}{S1}$$ \hspace{1cm} (3.3.2)

The second is the trapping efficiency, which is mainly related to the area where sedimentation occurs and which is a good indication of the sediment to be dredged, in case a constant depth has to be maintained. This trapping efficiency is referred to as the dredging trapping efficiency and is written as:

$$TE_{dredge} = \frac{S1 - S3}{S1}$$ \hspace{1cm} (3.3.3)

To define the volume of a trench, it is necessary to use some definition of the trench. In morphodynamic SUTRENCH-simulations the total trench area does not change because of the conservation of mass. In this study the outer dimensions as defined in the previous subsection are also used to calculate the trench area. This is shown in Figure 3.3.5 (third graph), in this graph also the trench width and averaged depth are shown which will be used in the sensitivity analysis. Furthermore, the centre of gravity of the trench (based on the total bottom profile) is used to characterise the migration. In the bottom graph of Figure 3.3.5 four characteristic regions are shown on which the sedimentation or erosion is determined. These regions are based on the trench definition. Region I and III are the slopes of the trench, Region II is the bottom of the trench and Region IV is the total trench. The sedimentation-erosion in the defined areas is indicated by the symbols: SER1, SER2, SER3 and SER4.

Finally, the relative sedimentation of the total trench (i.e. volume change) is considered indicated by SER:

$$SER = \frac{Area_{new}}{Area_{old}}$$ \hspace{1cm} (3.3.4)

Table 3.3.4 contains the output parameters considered in the sensitivity analysis. It is noted that a selection of output parameters is made to investigate the effects of the input parameters studied. In Sections 3.3.4 to 3.3.6 the effects of process/model input parameters, hydrodynamic input parameters and trench geometry are investigated respectively. In the table below it is indicated which output parameters are applied.
### Table 3.3.4  Overview of output parameters in sensitivity analyses

**Relative comparison definitions**

In a sensitivity analysis it is necessary to intercompare the various results in order to give insight in the importance of an input parameter but also to give an indication of the sensitivity of the SUTRENCH-model to variations of the investigated input parameter. The relative comparison is done for the selected output parameters as they are listed in Table 3.3.4. This relative comparison is done against the results from the base run by multiplying the variation range of resulting output parameter considered with the variation range of the investigated input parameter (Eq. 3.3.5). For example, if the applied wave height is increased by a factor 1.5 (150%) and the resulting transport at the upper boundary would increase with a factor 2 the model can be characterised as very sensitive to variations in wave height. This so-called relative influence (RI) is written as:

\[
RI = \left( \frac{RS_{\text{new}} - RS_{\text{old}}}{RS_{\text{old}}} \right) \left( \frac{Inp_{\text{old}}}{Inp_{\text{new}} - Inp_{\text{old}}} \right)
\]  

(3.3.5)
In which $R_{s\text{new}}$ is the value of the considered output parameter, $R_{s\text{old}}$ is the value of the output parameter for the base run, $Inp_{\text{new}}$ is the value of the input parameter of the sensitivity run and $Inp_{\text{old}}$ is the value of the input parameter for the base run.

The relative influence ($RI$) can be characterised as follows:

<table>
<thead>
<tr>
<th>Values for $RI$</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>$RI &gt; 1$</td>
<td>Very sensitive</td>
</tr>
<tr>
<td>$0 &lt; RI &lt; 1$</td>
<td>Weakly sensitive</td>
</tr>
<tr>
<td>$RI = 0$</td>
<td>Insensitive</td>
</tr>
<tr>
<td>$-1 &lt; RI &lt; 0$</td>
<td>Weakly inverse sensitive</td>
</tr>
<tr>
<td>$RI &lt; -1$</td>
<td>Strongly inverse sensitive</td>
</tr>
</tbody>
</table>

Table 3.3.5  Definition of relative influence, $RI$, classes

### 3.3.4 Effect of process and model parameters

**General**

To investigate the effects of variations in process and model input parameters, in total eight (combinations) of input parameters were selected. In all sensitivity simulations the base run is used as the reference against which the results are evaluated and interpreted. The investigated input parameters were given a high and low value with the settings from the base run as the reference. For each setting a morphodynamic SUTRENCH-simulation over a period of three years was made. In Table 3.3.4 the used output parameters are listed. All transport related output parameters are based on the initial residual transports (i.e. sediment transports as they are calculated on the initial (starting) bottom profile). The four defined sedimentation-erosion regions (see Figure 3.3.5, bottom graph) are derived from the initial profile. The sedimentation-erosion values refer to a period of three years.

The various simulations are first discussed individually where a comparison is made with the results from the base run. This will be followed by a section where a relative intercomparison is made to give a clear insight in the relative importance of the investigated process and model parameters.

The process and model parameters that have been investigated are listed in Table 3.3.6 below.
### Input parameter(s) | Symbol(s) | Decreased Value | Base Run | Increased Value
--- | --- | --- | --- | ---
Thickness of wave mixing layer [m] | $D_s$ (DS) | 0.15 (*0.5) | 0.30 | 0.45 (*1.5)
Wave related mixing coeff. [m$^{-2}$/s] | $E_w,bed$ & $E_w,max$ (EMP) | 0.002 & 0.0175 (*0.5) | 0.004 & 0.035 | 0.006 & 0.0525 (*1.5)
Wave height [m] | $H_s$ (HS) | 1.125 (*0.5) | 2.25 | 3.375 (*1.5)
Wave period [s] | $T_p$ (TP) | 4.4 (*0.67) | 6.6 | 9.9 (*1.5)
Wave roughness height [m] | $R_w$ (RW) | 0.005 (*0.5) | 0.01 | 0.05 (*5)
Current roughness height [m] | $R_c$ (RC) | 0.025 (*0.5) | 0.05 | 0.075 (*1.5)
Sediment characteristics [$\mu$m and m/s] | $d_{50,90}, W_s$ (SED) | 100, 200 & 0.01 (*0.5) | 200, 300 & 0.028 | 300, 450 & 0.04 (*1.5)
Reference level [m] | $Z_a$ (ZA) | 0.025 (*0.5) | 0.05 | 0.075 (*1.5)

**Table 3.3.6** Overview of investigated process and model input parameters

The keyword in brackets in the second column are used as a reference in various figures and tables.

In Tables 3.3.7a and 3.3.7b the absolute values of the selected output parameters are listed. They will be used in the description of the individual simulations that follow next.

<table>
<thead>
<tr>
<th>Run-ID</th>
<th>$S_{1-g}$ [m$^3$/m/yr]</th>
<th>$S_{2-g}$ [m$^3$/m/yr]</th>
<th>$S_{min}$ [m$^3$/m/yr]</th>
<th>$S_{1}$ [m$^3$/m/yr]</th>
<th>$S_{2}$ [m$^3$/m/yr]</th>
<th>$S_{3}$ [m$^3$/m/yr]</th>
<th>$S_{4}$ [m$^3$/m/yr]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base</td>
<td>51.60</td>
<td>53.41</td>
<td>3.67</td>
<td>59.68</td>
<td>24.36</td>
<td>3.88</td>
<td>27.16</td>
</tr>
<tr>
<td>DS-HIGH</td>
<td>335.48</td>
<td>327.82</td>
<td>8.19</td>
<td>400.73</td>
<td>162.43</td>
<td>11.96</td>
<td>138.47</td>
</tr>
<tr>
<td>DS-LOW</td>
<td>127.01</td>
<td>126.40</td>
<td>7.57</td>
<td>144.51</td>
<td>54.69</td>
<td>7.70</td>
<td>64.70</td>
</tr>
<tr>
<td>EMP-HIGH</td>
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<td>120.50</td>
<td>-11.38</td>
<td>159.04</td>
<td>94.27</td>
<td>-3.72</td>
<td>18.37</td>
</tr>
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<td>EMP-LOW</td>
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<td>28.09</td>
<td>4.25</td>
<td>27.63</td>
<td>6.64</td>
<td>4.40</td>
<td>23.53</td>
</tr>
<tr>
<td>HS-HIGH</td>
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<td>121.88</td>
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<td>173.42</td>
<td>107.19</td>
<td>-11.78</td>
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<td>HS-LOW</td>
<td>33.34</td>
<td>32.48</td>
<td>5.01</td>
<td>31.72</td>
<td>7.43</td>
<td>5.17</td>
<td>27.70</td>
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<tr>
<td>RC-HIGH</td>
<td>56.34</td>
<td>58.79</td>
<td>3.74</td>
<td>66.20</td>
<td>27.99</td>
<td>4.13</td>
<td>28.47</td>
</tr>
<tr>
<td>RC-LOW</td>
<td>46.80</td>
<td>47.77</td>
<td>3.26</td>
<td>53.04</td>
<td>20.23</td>
<td>3.35</td>
<td>25.57</td>
</tr>
<tr>
<td>RW-HIGH</td>
<td>65.32</td>
<td>68.09</td>
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<td>83.19</td>
<td>37.13</td>
<td>1.25</td>
<td>25.51</td>
</tr>
<tr>
<td>RW-LOW</td>
<td>49.07</td>
<td>50.71</td>
<td>4.10</td>
<td>55.77</td>
<td>22.34</td>
<td>4.18</td>
<td>27.05</td>
</tr>
<tr>
<td>SED-HIGH</td>
<td>35.19</td>
<td>34.06</td>
<td>4.75</td>
<td>33.81</td>
<td>8.24</td>
<td>4.89</td>
<td>27.98</td>
</tr>
<tr>
<td>SED-LOW</td>
<td>641.25</td>
<td>495.26</td>
<td>79.46</td>
<td>877.79</td>
<td>743.47</td>
<td>167.69</td>
<td>113.58</td>
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<tr>
<td>TP-HIGH</td>
<td>64.31</td>
<td>65.73</td>
<td>5.40</td>
<td>72.55</td>
<td>25.83</td>
<td>5.45</td>
<td>37.13</td>
</tr>
<tr>
<td>TP-LOW</td>
<td>50.64</td>
<td>50.97</td>
<td>7.11</td>
<td>54.52</td>
<td>24.29</td>
<td>7.12</td>
<td>29.89</td>
</tr>
<tr>
<td>ZA-HIGH</td>
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<td>51.89</td>
<td>3.50</td>
<td>58.68</td>
<td>25.58</td>
<td>3.91</td>
<td>25.21</td>
</tr>
<tr>
<td>ZA-LOW</td>
<td>66.08</td>
<td>67.87</td>
<td>3.92</td>
<td>75.42</td>
<td>28.65</td>
<td>4.07</td>
<td>35.06</td>
</tr>
</tbody>
</table>

**Table 3.3.7a** Overview of absolute values for selected output parameters
Table 3.3.7b  Overview of absolute values for selected output parameters

The results of the various sensitivity simulations is presented in figures with a standard layout. The figures show from top to bottom the profile development after three years including the slope definition points and the centres of gravity, the residual yearly transports (total, suspended and bottom transports), a comparison with the base run transports and the sedimentation-erosion after three years respectively.

**Thickness of wave mixing layer, Ds**

The results of simulations with an increased and decreased $Ds$ are shown in Figures 3.3.6 and 3.3.7. If the transport rates are compared with the base run it can be noticed that in both simulations the transport rates have increased. This can be explained by the fact that the value of the wave related mixing parameter increased within the range of the applied variations of $Ds$. It can also be seen in the relative transport graph that the model is fairly sensitive to variations of this parameter. The relative transports show a distinct peak near the bottom of the downstream slope. This is due to the fact that the southerly directed ebb-currents are increasing or decreasing. Since the residual transport at that location is relatively small, a fairly large variation in the relative transports can be observed. In the interpretation of the results this area is usually neglected as it originates from the tidal schematisation which is not the main subject of interest in this section. In accordance with the calculated transports the sedimentation and migration of the trench is also much larger compared to the base run; in case of an increased $Ds$ a significant migration of the centre of gravity can be observed. In Tables 3.3.7a and 3.3.7b the absolute values of the increased and decreased run are listed for the selected output parameters (cf. DS-HIGH and DS-LOW). From the table it can be seen that the trapping efficiency is not significantly affected by the variation. The sedimentation-erosion rates in the trench show a considerable effect as was already concluded from Figures 3.3.6 and 3.3.7.
It can be concluded that variation of the wave related mixing in the boundary layer has a considerable effect on the transports and the morphological behaviour of the trench. Also the effect of variation is uncertain; in the range that has been applied in the present study, even in the case of a decreased $Ds$ a significant increase in transport rates was found.

**Wave related mixing coefficients ($E_{w,\text{bed}}$ & $E_{w,\text{max}}$)**

As is shown in Figure 3.3.8 these two parameters, in combination with $Ds$, define the shape function of the wave related sediment mixing. The results of the increased and decreased settings both show the expected results (Figures 3.3.9 and 3.3.10); an increased sediment mixing results in an increased suspended transport load whereas with the decreased values the opposite effect can be observed. The bottom transport is not affected by variations in this parameters. The trapping efficiency is influenced significantly (see Tables 3.3.7a and 3.3.7b). An increased wave related mixing results in an increased sediment trapping. In case of decreased wave mixing especially the total sediment trapping efficiency decreases significantly. The sign changes of the transports at the toe of the upstream (northern) slope is due to the fact that the ebb (southern) transports have increased and are dominant over the northern (flood) transports in this particular region of the trench.

After three years significant sedimentation has occurred in the trench in case of increased wave mixing. The migration of the centre of gravity is however limited.

![Figure 3.3.8 Wave-related sediment mixing coefficient](image)

**Wave height ($H_s$)**

In Figures 3.3.11 and 3.3.12 the results are shown for simulation with an increased and decreased wave height. The influence of the increased wave height can clearly be seen; the residual transports have increased considerably and also has the subsequent sedimentation of the trench. At the bottom of the downstream slope negative (southward) transports occur which are caused by an increase of transports in ebb-direction. This is caused by the fact that due to the increased wave height more sediment is stays in suspension which results in a larger lag in the settling of suspended sediment causing larger transport at the bottom of the slopes. Trapping efficiency is also affected significantly; in case of an increased wave height the total trapping is 98% (see Tables 3.3.7a and 3.3.7b) and the dredge trapping is 100%. The decreased wave heights cause a significant decrease of the sediment trapping which is caused by the lower transports due to ebb-currents on the downstream (northern) slope. This is due to the opposite effect as described for the increased wave heights: lower suspended sediment concentration resulting in a smaller lag.
From the relative bottom transports plots in Figures 3.3.11 and 3.3.12 it can be seen that the bottom transport reacts inversely to changes in wave height. This is due to the fact that the current velocity in the bottom boundary layer decreases in case of increasing wave heights.

Variations in wave height affect the resulting morphological behaviour of the trench clearly. The variations in the results are of the same order as the applied changes in wave height.

**Wave period ($T_p$)**

The influence of variations in the wave period is small as can be seen in Figures 3.3.13 and 3.3.14. Residual transports show only a weak reaction to variations in the wave period. Also the migration of the centres of gravity are almost similar to the base run (22 and 14 m compared to 16 m, see Tables 3.3.7a and 3.3.7b).

**Sediment characteristics ($d_{50}$, $d_{90}$ and $W_s$)**

Variation of the sediment characteristics has a significant influence on the results from the SUTRENCH simulation. Decreasing the values for $d_{50}$, $d_{90}$ with a factor 0.5 (the fall velocity, $W_s$, is modified accordingly) results in transports which are about ten times higher (see Figure 3.3.15). The trench shows considerable sedimentation. It is noted that the results may not be realistic, as the model has not been calibrated for very fine sediments. An increased value of the sediment characteristics has a less dramatic effect as can be seen in Figure 3.3.16).

Varying the sediment characteristics can have a significant influence on the final results, especially an under-estimation of the sediment grain sizes can lead to relatively large errors.

**Wave roughness height ($R_w$)**

A decreased wave roughness height has no significant influence, an increased value (of 500%) also has a relative small influence on the final results (see Figures 3.3.17 and 3.3.18).

**Current roughness height ($R_c$)**

The current roughness height has a relative small effect on the sediment transports and trench development (Figures 3.3.19 and 3.3.20).

**Reference level ($Z_a$)**

Varying the reference level shows the expected results. This parameters is inverse proportional to the equilibrium bed concentration which also leads to an inverse effect on the residual transports. Its effect is however small both in case of an increase or decrease (see Figures 3.3.21 and 3.3.22).
Relative intercomparison of the varied parameters

In the previous sub-sections the individual simulations have been discussed. This section is devoted to give a (relative) intercomparison of all the simulations. This is done by applying Equation 3.3.4 on the selected output parameters in Table 3.3.4. The absolute values from all the runs are listed in Tables 3.3.7a and 3.3.7b.

In the Figures 3.3.23 and 3.3.24 the relative influence of the investigated process and model input parameters are shown. Figure 3.3.23 shows the influence in case of decreased parameters and Figure 3.3.24 for the increased parameters. For an explanation of the selected output parameters is referred to Sub-section 3.3.3. In case of a positive relative influence (RI) the output parameters are positively related to variations of the parameter, a negative value indicates an inverse relationship. For an explanation of the values is referred to Table 3.3.5.

The results in Figure 3.3.23, for the decreased parameters, shows that the SUTRENCH model is especially sensitive to variations in the sediment characteristics (SED) and near-bed mixing layer (Ds). The negative values indicate an inverse relationship. It can also be seen that the model is fairly sensitive to variations in wave height (HS) and wave-related mixing coefficients (EMP). For most other investigated parameters the model is only weakly sensitive (values of RI are between 1 and -1). For the increased parameters more or less the same patterns can be seen. What can be noticed however is that increased values for HS and EMP lead to a more sensitive reaction of SUTRENCH. This reveals an important limitation of the present sensitivity analysis. To get a complete indication of the sensitivity of the SUTRENCH-model, it should in fact be necessary to apply a whole range of values for most of the parameters because the model shows a “non-linear” reaction to variation of these input parameters. This lies however outside the scope of the present study. The investigated input parameters have been varied within physically realistic boundaries with the aim to get an indication of the input parameters that exert a strong reaction of the SUTRENCH-model.

As expected, the transport and sedimentation-erosion output parameters show more or less the same sensitivity. The migration of the trench, indicated by the horizontal shift of the centre of gravity, also shows the same level of sensitivity to variation of the input parameters. The trapping efficiencies are also fairly sensitive to variations although to a lesser extent as the other output parameters.

From the two figures it can be concluded that variation in the model parameters prescribing the wave related mixing over the vertical (DS and EMP) and the input parameters of the wave height and sediment characteristics result in considerable variations of calculated transports and morphological behaviour of the trench. The model is relative insensitive to variation of the roughness related parameters (Rw and Rc) and reference level (Za).

In engineering applications it is therefore necessary to make an accurate schematisation of the wave climate. Also the applied sediment characteristics should be determined with great care. Further calibration of the model for fine sediment is necessary. The sensitivity of the model to the parameters describing the wave-related mixing over the vertical indicates that in further development these parameters should be considered (e.g. validation against measurements under various conditions).
3.3.5 Effect of hydrodynamic input parameters

General

The effect of hydrodynamic input parameters has been investigated by modifying the number of steps in which the tidal currents and water level variations are approximated (referred to as tidal schematisation). The alternative schematisations are determined arbitrarily. This is not the most optimal schematisation. It would have been better to adopt a schematisation, based on a constant tide-integrated water volume or sand load.

As explained in Section 3.3.2 where the base run is discussed, the base run uses a schematisation of 19 steps. Two other alternatives have been constructed in which the tidal schematisation was in 7 and 2 steps respectively. The applied schematisations have been visualised in Figure 3.3.25 where the top graph shows the occurring water level variation...
and the second, third and fourth graph the tidal schematisation of the base run (19 intervals), the tidal schematisation in 7 intervals and 2 intervals respectively.

It can be seen in Figure 3.3.25 that the base run gives an accurate description of the current velocities whereas especially in case of a 2 step schematisation significant deviations from original velocities occur. The 7 step schematisation has an apparent shift of the velocities during flood. This is due to the fact that all the negligible currents have been combined in the first schematised step to reduce simulation time.

**Results**

The results of the three schematisations are depicted in Figure 3.3.26. In the top graph the resulting trenches after three years are shown. In the second and third graph the residual transports of the two investigated schematisations are compared against the transports of the base run. In the fourth (bottom) graph the resulting sedimentation-erosion of the three runs are shown. In Tables 3.3.8a and 3.3.8b the values for the selected output parameters are listed.

<table>
<thead>
<tr>
<th>Run-ID</th>
<th>$S_{x=0}$ [m$^3$/m/yr]</th>
<th>$S_{x=n}$ [m$^3$/m/yr]</th>
<th>$S_{min}$ [m$^3$/m/yr]</th>
<th>$S1$ [m$^3$/m/yr]</th>
<th>$S2$ [m$^3$/m/yr]</th>
<th>$S3$ [m$^3$/m/yr]</th>
<th>$S4$ [m$^3$/m/yr]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base run</td>
<td>51.60</td>
<td>53.41</td>
<td>3.67</td>
<td>59.68</td>
<td>24.36</td>
<td>3.88</td>
<td>27.16</td>
</tr>
<tr>
<td>2 steps</td>
<td>13.65</td>
<td>13.58</td>
<td>0.60</td>
<td>13.20</td>
<td>9.52</td>
<td>0.68</td>
<td>10.28</td>
</tr>
<tr>
<td>7 steps</td>
<td>42.21</td>
<td>46.52</td>
<td>-11.45</td>
<td>71.75</td>
<td>72.83</td>
<td>-6.57</td>
<td>1.89</td>
</tr>
</tbody>
</table>

**Table 3.3.8a** Overview of absolute values for selected output parameters

<table>
<thead>
<tr>
<th>Run-ID</th>
<th>$TE_{tot}$ [%]</th>
<th>$TE_{dredge}$ [%]</th>
<th>$\Delta cog$ [m]</th>
<th>$SER1$ [m$^3$/m]</th>
<th>$SER2$ [m$^3$/m]</th>
<th>$SER3$ [m$^3$/m]</th>
<th>$SER4$ [m$^3$/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base run</td>
<td>0.55</td>
<td>0.94</td>
<td>16.64</td>
<td>101.38</td>
<td>62.57</td>
<td>-58.80</td>
<td>105.15</td>
</tr>
<tr>
<td>2 steps</td>
<td>0.22</td>
<td>0.95</td>
<td>5.34</td>
<td>9.63</td>
<td>28.21</td>
<td>-27.59</td>
<td>10.25</td>
</tr>
<tr>
<td>7 steps</td>
<td>0.97</td>
<td>1.09</td>
<td>13.62</td>
<td>-0.35</td>
<td>217.82</td>
<td>-21.78</td>
<td>195.70</td>
</tr>
</tbody>
</table>

**Table 3.3.8b** Overview of absolute values for selected output parameters

The transports of the three schematisations show significant differences. The 7 steps schematisation approximately yields the same transports outside the trench but in the trench marked differences can be observed. At the toe of the northern slope negative transports in the order of 15 m$^2$/yr occur whereas in the base run transports are still positive (see also Tables 3.3.8a and 3.3.8b). Also at the top of the southern slope the 7 step transports deviate considerably from the base run transports. The negative transport at the toe of the northern slope is due to the fact that ebb currents are more dominant and the flood currents are less important. At the top of the southern slope the distinct maximum is due to the fact that the ebb transports have locally decreased due to a greater lag effect.

Finally, it is concluded that the schematisation of the horizontal tide (current velocities) have a significant effect. Small variations of the current velocity have a relatively large effect, because the transport rates in deep water are relatively small (close to the initiation of motion).
3.3.6 Effect of pit geometry

General

In this section the effects of the trench/pit geometry are investigated. As given in the previous section, morphodynamical simulations over a period of three years are made. In total six pit geometry’s are considered in which a geometrical feature is doubled or halved. These six alternatives and the base run are listed in Table 3.3.9 below:

<table>
<thead>
<tr>
<th>Run ID</th>
<th>Geometrical feature</th>
<th>Width [m]</th>
<th>Depth [m]</th>
<th>Slope angles [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Run</td>
<td>not applicable</td>
<td>600</td>
<td>7</td>
<td>4</td>
</tr>
<tr>
<td>Wide</td>
<td>Doubled width</td>
<td>1200</td>
<td>7</td>
<td>4</td>
</tr>
<tr>
<td>Narrow</td>
<td>Halved width</td>
<td>300</td>
<td>7</td>
<td>4</td>
</tr>
<tr>
<td>Deep</td>
<td>Doubled depth</td>
<td>600</td>
<td>14</td>
<td>4</td>
</tr>
<tr>
<td>Shallow</td>
<td>Halved depth</td>
<td>600</td>
<td>3.5</td>
<td>4</td>
</tr>
<tr>
<td>Steep</td>
<td>Doubled slope angles</td>
<td>600</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>Flat</td>
<td>Halved slope angles</td>
<td>600</td>
<td>7</td>
<td>2</td>
</tr>
</tbody>
</table>

Table 3.3.9  Overview investigated pit geometry's.

In this sensitivity analysis only the output parameters that are strongly related to the pit geometry, are considered; the selected output parameters are shown in Table 3.3.4. The investigated alternatives will mainly be evaluated in terms of the morphodynamic behaviour such as the migration of the trench, development of the slopes, etc.. The setup of this section is similar to the previous sections; first the simulations will be discussed individually after which a relative comparison is made. In Table 3.3.10 the absolute values of the criteria are listed for the considered alternatives. The output parameters listed in the rightmost four columns are after 3 years of simulation.

<table>
<thead>
<tr>
<th>Run-ID</th>
<th>$T_{E_{total}}$ T=0 yr</th>
<th>$T_{E_{total}}$ T=3 yr</th>
<th>$T_{E_{dredge}}$ T=0 yr</th>
<th>$T_{E_{dredge}}$ T=3 yr</th>
<th>$\Delta c_{og}$ [m]</th>
<th>SER [-]</th>
<th>L [-]</th>
<th>D [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base run</td>
<td>0.55</td>
<td>0.55</td>
<td>0.94</td>
<td>0.92</td>
<td>16.67</td>
<td>0.02</td>
<td>1.00</td>
<td>0.98</td>
</tr>
<tr>
<td>Width = 1200 m</td>
<td>0.58</td>
<td>0.58</td>
<td>0.98</td>
<td>0.97</td>
<td>17.66</td>
<td>0.01</td>
<td>1.00</td>
<td>0.99</td>
</tr>
<tr>
<td>Width = 300 m</td>
<td>0.51</td>
<td>0.51</td>
<td>0.87</td>
<td>0.85</td>
<td>17.33</td>
<td>0.03</td>
<td>1.00</td>
<td>0.97</td>
</tr>
<tr>
<td>Depth = 3.5 m</td>
<td>0.44</td>
<td>0.42</td>
<td>0.76</td>
<td>0.74</td>
<td>25.37</td>
<td>0.03</td>
<td>1.02</td>
<td>0.96</td>
</tr>
<tr>
<td>Depth = 14 m</td>
<td>0.65</td>
<td>0.67</td>
<td>1.01</td>
<td>1.00</td>
<td>10.12</td>
<td>0.01</td>
<td>1.00</td>
<td>0.99</td>
</tr>
<tr>
<td>Slopes = 2 %</td>
<td>0.49</td>
<td>0.48</td>
<td>0.92</td>
<td>0.91</td>
<td>18.72</td>
<td>0.01</td>
<td>1.01</td>
<td>0.98</td>
</tr>
<tr>
<td>Slopes = 8 %</td>
<td>0.59</td>
<td>0.60</td>
<td>0.95</td>
<td>0.93</td>
<td>16.52</td>
<td>0.02</td>
<td>1.01</td>
<td>0.97</td>
</tr>
</tbody>
</table>

Table 3.3.10  Absolute values of criteria to investigate geometrical effects (see also Table 3.3.4)

Influence of trench width

The results of the simulations with a wide (width = 1200m) and narrow trench (width = 300m) are shown in Figures 3.3.27 and 3.3.28 respectively. In the top graph it can be seen that compared with the base run the morphological behaviour of the trenches, over a period of three years, is not influenced significantly. If the residual transports of the narrow and
wide trench are compared (second graph in the respective figures) it can be seen that in case of the wide trench, almost all incoming sediment is trapped. This can also be seen in the trapping efficiencies listed in Table 3.3.10. The wide trench has a dredging trapping efficiency of 98% whereas for the narrow trench this is about 10% lower. The sedimentation or erosion of the wide trench mainly occurs on the trench slopes whereas for the narrow trench also some sedimentation of the trench bottom can be observed. The horizontal migration of the centres of gravity’s however are more or less comparable; in both cases a migration of approximately 17 m is calculated after 3 years.

**Influence of trench depth**

In the Figures 3.3.29 and 3.3.30 the results for the shallow and deep trench are shown respectively. If the residual transports of the two simulations are compared, it can be seen that the deep trench traps almost all the incoming sediment. The trapping efficiency ($TE_{dredge}$) for the deep trench is 100% and for the shallow trench only 75% (see Table 3.3.10). This can also be seen in the ebb and flood transports (third graph). Especially the dominant flood (northgoing) transports are negligible at the toe of the northern slope in case of the deep trench, whereas for the shallow trench the flood transports are still some 40% of the undisturbed transports. Comparison of the sedimentation-erosion (bottom graphs) shows that the slopes of the deep trench are subject to more sedimentation (southern slope) and erosion (northern slope) then the shallow trench.

**Influence of trench slope angle**

In Figures 3.3.31 and 3.3.32 the results for the trench with steep and flat slopes are shown respectively. From the sedimentation-erosion patterns (bottom graphs) it can be seen that in case of flat slopes most of the morphological adjustments occur on the slopes. This is mainly due to the fact that the slope length has increased because of which most sediment settles on the slopes. Also in case of steep slopes most sedimentation and erosion occurs on the slopes but it can be seen that relative more sedimentation has occurred at the bottom of the trench. From the results it can be concluded that most of the differences between both runs mainly originate from the fact that the total trench width at reference level (depth = 0 m) differs considerably, because of which the upstream and downstream slopes show more interaction in case of a trench with steep slopes. The opposite effect is visible for the trench with flat slopes. From the simulation it can be concluded that for the considered slope angles, no slope effects as such could be found. All the differences between the investigated geometry's mainly originate from different widths of the trenches.

**Relative intercomparison of alternatives**

From the considered geometry’s it can be concluded that the morphological behaviour of the trench is largely determined by the width and depth of the flume. The deep and wide trench have a trapping efficiency (dredging) which is approximately 100%, whereas the trapping for the narrow and shallow trenches are significantly lower (87 and 76% respectively, see Table 3.3.10). The behaviour of the northern (upstream) trench slope is strongly correlated to the transport gradient from toe to top of the slope, which clearly is affected by the absolute value of the transport at the toe of the upstream slope. From a comparison of the trenches with steep and flat slopes it can be concluded that especially the depth of the trench has a significant influence on the total morphological behaviour of the trench: in both simulations most sedimentation or erosion occurs on the slopes. Wave induced stirring of sediment is relatively sensitive to the water depth and also the additional
effect of reduced currents in the trench play a major role. In case of steep slopes this results in relative high sedimentation and erosion on the slopes whereas for the simulation with the flat slopes the maximum values are approximately halved (compare bottom graphs in Figure 3.3.31 and 3.3.32). The total sedimentation and erosion on both slopes is, in the considered period of three years, almost identical. The width has only an influence within a limited range which is determined by the length scale of the lag effect of the settling of suspended sediment. If a trench with a width of 2400 m (4 times the width of the trench of the base run) is considered no marked differences compared to the trench with a width of 1200 m would be observed for the considered period of three years, as for this geometry already most sediment has settled out at the toe of the upstream slope (see Figure 3.3.27). In Chapter 4 a trench with a width of 2400 m is actually investigated, for more information on this geometry the reader is referred to this chapter. There it was found that on larger time scales differences between the two runs were noticeable.

In the next chapter long term morphodynamic simulations will be discussed in which, among others, also the in this section considered geometry’s are included.

### 3.3.7 Conclusions

From the sensitivity analysis of the process and model input parameters it can be concluded that variation in the model parameters prescribing the wave related mixing over the vertical (DS and EMP) and the input parameters of the wave height and sediment characteristics result in considerable variations of calculated transports and morphological behaviour of the trench. The model is relative insensitive to variation of the roughness related parameters ($R_w$ and $R_c$) and reference level ($Z_a$).

In engineering applications it is therefore necessary to make an accurate schematisation of the wave climate. Also the applied sediment characteristics should be determined with great care. Further calibration of the model for fine sediment is necessary. The sensitivity of the model to the parameters describing the wave-related mixing over the vertical indicates that in further development these parameters should be considered (e.g. validation against measurements under various conditions).

The investigated effects of the hydrodynamic input parameters was found to be significant. As stated in Section 3.3.5, however, the applied tidal schematisations could be improved by adopting a constant tide-integrated water or sand load.

The effects of the pit geometry can be considerable. Especially the depth and width of the trench determine the magnitude of disturbance and hence the backfilling rates. The investigated slope angles as such did not show a significant influence.

The applied trench definitions resulted in a consistent and objective set of output parameters which provided a good basis for interpretation and comparison of the various results.

### 3.4 Summary and conclusions of model evaluation

From the two considered simulation cases (laboratory basin experiment and Danish field experiment) it can be concluded that SUTRENCH gives fairly reliable results. The results for the basin experiment show good agreement with the measurements. The migration of the
trench is modelled accurately, the morphological development of the trench slopes, however, is not modelled satisfactory. The upstream slope is generally predicted too steep whereas the erosion of the downstream slope is underpredicted. From the basin experiment it is found that the trench slope development is influenced by varying the roughness heights (cf. Figures 3.2.13 and 3.2.14). Varying the sediment fall velocity mainly influences the sedimentation (and migration) of the trench. An optimal fit to the upstream boundary velocity and concentration profiles (Figures 3.2.20 to 3.2.23) resulted in a good representation of the measured trench development. Hindcast of the Danish field experiment at CH1700 (cf. Figure 3.2.9) was hampered by the fact that no current data was available for the simulated period. The migration and sedimentation of the trench over a period of 3 weeks showed reasonable agreement with the observed morphological developments by using reasonable estimates for the tide-and wind-induced currents.

Evaluation of two recent studies (Walstra et al., 1997 and Hoitink, 1997) showed that there is an apparent inconsistency between the observed transports, which lie in the range of 20 to 50 m³/m/yr and the measured morphological development of the Euro-Maas channel. To reproduce the morphological development of the trench unrealistic high transports (in the order of 130 m³/m/yr) have to be imposed in the SUTRENCH-model. The base run was calibrated on the occurring transports rather then on the morphological behaviour of the trench.

From the sensitivity analysis of the process and model input parameters it was found that an accurate schematisation is imperative in yielding reliable results. Furthermore the model was found to be very sensitive to variations of the sediment characteristics, validation of the transport relations for fine sediment is necessary. The sensitivity of SUTRENCH to variations in the wave-related mixing parameters illustrates the need to validate the model for deep water conditions as the referred parameters have only been calibrated for shallow water conditions.

From the analysis of the effects of the hydrodynamic input parameters it could be concluded that the schematisation of the horizontal tide (current velocities) have a significant effect. Small variations of the current velocity have a relatively large effect, because the transport rates in deep water are relatively small (close to the initiation of motion).

The effects of the pit geometry can be considerable. Especially the depth and width of the trench determine the magnitude of disturbance and hence the backfilling rates. The investigated slope angles as such did not show a significant influence.

The applied trench definitions resulted in a consistent and objective set of output parameters which provided a good basis for interpretation and comparison of the various results.
4 Morphological development of a sand pit based on SUTRENCH model

4.1 Introduction

This chapter is aimed at coastal zone managers to provide them with some guidelines based on which they can get a quick impression of the impacts that proposed sandmining alternatives will have on the coastal system. For this purpose twelve long term simulations have been made with varying geometry’s. They are evaluated by intercomparing various output parameters such as e.g. the migration of the trench, sedimentation rates and sediment trapping efficiencies (for a definition is referred to Chapter 3).

The investigated alternatives have been defined in such a way that they cover a wide range of sand mining geometry’s that are under investigation.

In the next section the various simulation results will be discussed. In the intercomparison of the various simulations the output parameters as defined in Section 3.3.3 will be applied. For the evaluation of the results is focused on the development of the trench in time in terms of volume, shape and migration. In Section 4.3 conclusions will be drawn.

4.2 Sand pit development

General

In total twelve long term simulations have been made. The pit geometry’s that were investigated in Section 3.3.6, were also used in the long term simulations. In total five additional geometry’s were defined: two geometry’s in which the trench depth was set to 2 and 10 m, respectively and an extra wide trench with a width of 2400 m. To investigate the effect of water depth variation, two simulation were made in which the water depth was increased and decreased with 5 m.

First the individual simulations are described by evaluating the output parameters as defined in Chapter 3. In Table 4.2.1 the values for the output parameters are shown. The results in this table are used in the description of the individual runs. In the first four columns the sediment trapping efficiencies on the initial profile and after 50 years are shown. In the last four columns the values for respectively: the migration of trench ($\Delta_{\text{cog}}$), the relative sedimentation (SER), the relative trench width and depth are shown after 50 years.

As indicated in the introduction, attention is focused on the trench characteristics volume, shape and migration. The volume change is indicated by the relative sedimentation (SER), the shape of the trench is characterised by the width and depth of the trench.
Table 4.2.1 Absolute values of output parameters for long term simulations (see also Table 3.3.4)

<table>
<thead>
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<th>Run-ID</th>
<th>$T_{E_{\text{Total}}}$ t=0 yr</th>
<th>$T_{E_{\text{Total}}}$ t=50 yr</th>
<th>$T_{E_{\text{dredge}}}$ t=0 yr</th>
<th>$T_{E_{\text{dredge}}}$ t=50 yr</th>
<th>$\Delta_{\text{cog}}$ [m]</th>
<th>$\text{SER}$ [-]</th>
<th>Width [-]</th>
<th>Depth [-]</th>
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<td>0.56 0.54 0.94 0.77</td>
<td>432.49 0.23 1.07 0.72</td>
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<td>361.55 0.08 1.02 0.91</td>
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<td>372.85 0.14 1.03 0.83</td>
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<tr>
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<td>482.53 0.31 1.21 0.57</td>
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<tr>
<td>depth = 14 m</td>
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<td>248.06 0.14 0.88 0.97</td>
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<tr>
<td>depth = 10 m</td>
<td>0.57 0.67 0.98 0.87</td>
<td>329.98 0.19 0.94 0.86</td>
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</tr>
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<td>623.53 0.23 1.7 0.46</td>
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<td></td>
</tr>
<tr>
<td>depth = 2 m</td>
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<td>715.6 0.09 3.01 0.3</td>
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</tr>
<tr>
<td>slope = 8 %</td>
<td>0.57 0.54 0.94 0.75</td>
<td>438.21 0.26 1.15 0.64</td>
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<tr>
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<td>771.7 0.36 1.3 0.49</td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>water depth = 25 m</td>
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<td>257.24 0.13 1.06 0.83</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

$T_{E_{\text{Total}}}$ is the sediment trapping efficiency over the complete trench and $T_{E_{\text{dredge}}}$ is the percentage of sediment that has settled between the top of the upstream slope and the toe of the downstream slope. For more information on the definitions is referred to Chapter 3.

If the $T_{E_{\text{Total}}}$ after 50 years and at the beginning are compared it can be seen that for a number of alternatives the total dredging efficiencies after 50 years are significantly higher then at the beginning of the simulation. This has no physical meaning and originates from the applied trench definitions. Because of this $T_{E_{\text{Total}}}$ is not used in the evaluation of the results which is presented below.

At the end of each of the following sub-sections a summary of the trench characteristics (volume, shape, migration and sediment trapping efficiency) is given.

**Base run**

In Figure 4.2.1 the results for the base run are shown. The layout of the graphs is identical to the previously presented results. The top graph shows the morphological development of the trench after 50 years. It can be seen that the trench has experienced considerable sedimentation during the simulated period. The upstream (southern) slope has kept the same approximate shape whereas the downstream slope has flattened considerably.

It can be seen that the trench has experienced considerable sedimentation over the complete trench. The original bottom of the trench, which is defined by the points P2 and P3 in Figure 3.3.5, has accreted over the total length. Also the toe’s of the slopes (same points) have accreted. Since the residual transports are northward directed the sediment that has accreted in the trench is mainly originating from south of the trench. This implies that if the toe of the northern slope (P3 in Figure 3.3.5) is accreting, there is a “morphological interaction” between the northern and southern slope. The morphological developments of one slope is influenced by the developments of the other. This can also be seen in the residual transport rates (second graph in Figure 4.2.1); there is still a considerable gradient...
visible across the bottom of the trench. In the next sub-section this morphological interaction is further investigated by varying the width of the trench.

**Summary of trench characteristics**

**Volume**
The trench volume, according to the definition in Figure 3.3.5, has decreased with approximately 25% (Table 4.2.1, column: SER).

**Migration**
The horizontal displacement of the centre of gravity is 430 m northwards (see Table 4.2.1, column: \( \Delta_{cog} \)).

**Shape**
The total width has not changed significantly but the averaged depth of the trench has decreased considerably.

**Trapping efficiency (dredging)**
The dredging trapping efficiency has decreased with some 15%.

**Variation of trench width**
The results for trenches with a width of 2400 m, 1200 m and 300 m are shown in Figures 4.2.2 and 4.2.4, respectively. It can be seen that in case of the 2400 m wide trench (Figure 4.2.2) there is no morphological interaction of the slopes. On the bottom of the trench there is a relative large area where no sedimentation has occurred. This implies that, because the morphological development of the two slopes is independent, the development of both trench slopes can be considered as autonomic behaviour. The morphological development of the two slopes differ considerably due to the hydrodynamic boundary conditions. Because of the northern directed residual transports the southern slope experiences considerable sedimentation. Both the top of the slope (P5 in Figure 3.3.5) and the toe (P2 in the same figure) show a considerable northward migration. The trench slope, however, remains more or less constant. The transition between the trench slope and bottom is much more gradual. This is due to the gradual settling of sediment in this region. The northern slope shows a relative small migration, the region around the toe of the slope (P3 in Figure 3.3.5) does not accrete and the location of the toe remains more or less constant. The shape of northern slope does, however, change considerably. The trench slope is flattening and the transition between the slope and the original sea floor is much more gradual. The morphological development of the northern slope is mainly governed by the positive gradient of the (residual) sediment transports over the slope. The erosion north of the trench is caused by the small gradient in the residual transports in this area, which is caused by the lag in picking up the sediment.

If, for decreasing trench width, the Figures 4.2.2, 4.2.3, 4.2.1 and 4.2.4 are compared the increased morphological interaction between the two slopes is clearly visible.

On the time scale considered, the migration of the trench seems to be correlated in an almost linear inverse way to the width. A doubling of the width results in a 60 m reduction of the migration whereas halving the width results in a 50 m increase of the migration (see also Table 4.2.1). The alternative of a trench width of 2400 m however only shows a relatively small decrease. This is probably caused by the fact that for this alternative the slopes react independently, resulting in a relatively small decrease in the migration of the
centre of gravity. The trench volume has decreased considerably (30%) for the 300 m alternative, the trench volumes of the 1200 m and 2400 m alternatives have only reduced by 15% and 8%, respectively after 50 years.

In Figures 4.2.5 to 4.2.7 the development in time of the selected output parameters is shown with a time step of 10 years, for the 2400 m, 1200 m and 300 m alternatives. In each figure the results for the base run are also included as a reference. In general it can be concluded from these figures that the output parameters show a more or less linear development. For the 2400 m alternative (Figure 4.2.5) it can be seen that most parameters show a similar development as for the base run. Only the relative trench depth and sedimentation show some deviation which can be explained by the fact that the total volume of the trench is larger. This can be seen if the sediment trapping efficiencies (which are more or less similar) are compared. In Figure 4.2.6 the results for the 1200 m trench are shown. The results are comparable to those of the 2400 m trench. In Figure 4.2.7 the results for the 300 m alternative are shown. In this figure it can be seen that the trench definition has some influence, if the development of the trench width is considered it can be seen that it shows a significant increase after 40 years. This also has an effect on the trapping efficiencies as these are derived from the residual transports at the trench slope boundaries (see also Chapter 3).

**Summary of trench characteristics**

**Volume**
The trench volume has decreased considerably (30%) for the 300 m alternative, the trench volumes of the 1200 m and 2400 m alternatives have only reduced by 15% and 8%, respectively after 50 years (Table 4.2.1, column: SER). As for the 2400 m and 1200 m alternatives the trapping efficiencies are more or less comparable to those of the base run the sedimentation rates of these trenches are not influenced. The relative sedimentation however decreases with increasing initial volume of the trench (see Eq. 3.3.4).

**Migration**
On the time scale considered, the migration of the trench seems to be correlated in an almost linear inverse way to the width. A doubling of the width results in a 60 m reduction of the migration whereas halving the width results in a 50 m increase of the migration (see also Table 4.2.1, column: \(\Delta_{mig}\)). The alternative of a trench width of 2400 m however only shows a relatively small decrease. This is probably caused by the fact that for this alternative the slopes react independently, resulting in a relatively small decrease in the migration of the centre of gravity.

**Shape**
The total width is relatively insensitive to variations in the trench width. The relative width change of the 2400 m and 1200 m alternative is 2% and 3%, respectively (Base run 7%). The width of the 300 m alternative increases with 21% after 50 years of simulation. If the development in time for this alternative is observed (Figure 4.2.7) it can be seen that after 40 years suddenly a significant increase of the output variable occurs. This is due to the applied trench definitions.

**Trapping efficiency (dredging)**
The dredging trapping efficiency is relative insensitive for the investigated alternatives. The development in time is approximately linear. For the wider trenches the dredging trapping efficiency is higher and decreases slower. For the 300 m alternative it is lower and decreases faster.
Variation of trench depth

In total four geometry’s have been constructed; two with increased depths (14 and 10 m) and two with decreased depths (3.5 and 2 m). The results of the four simulations are shown in the Figures 4.2.8 to 4.2.11. The sedimentation-erosion on the slopes of the 14 m alternative are, similar to the simulation with a trench width of 2400 m (and to some extent the 1200 m alternative) more or less independent. On the considered time scale the morphological developments mainly take place on the trench slopes. The settling of sediment occurs mainly on the upstream (southern) slope. The toe of the downstream slope only shows a minor shift. In case of a 10 m deep trench the same effects can be observed but to a lesser extent. If the other alternatives (trench depths of 3.5 m and 2 m, respectively) are compared it can be seen that the toe of the downstream slope shows a gradually accretion and horizontal shift if the depth is decreasing. The downstream slope also shows an increasing flattening if the trench depth decreases.

The morphological development of the upstream slope also changes in case of a decreasing trench depth. The 14 and 10 m alternatives show mainly a migration and steepening of the upstream slope. In the shallow trenches (3.5 and 2 m) the upstream slopes also show an increased migration. In contrast to the deeper trenches, however, the slopes have flattened considerably. This is an indication of the increasing morphological interaction between the upstream and downstream slopes with a decreasing trench depth.

The relative sedimentation of the various geometry’s after 50 years shows a remarkable agreement. The volume loss of the investigated trenches lies in the range of 15 to 22 %. This is mainly due to the fact that for the shallow trenches the total width has approximately doubled after 50 years which results in an increased trench volume. Indirectly this is also caused by the applied trench definitions. This is illustrated in Figure 4.2.11 where it is obvious that a objective indication of the trench slopes is difficult. As expected the migration of the centre of gravity increases with a decreasing trench depth (see also Table 4.2.1).

In Figures 4.2.12 to 4.2.15 the development in time of the selected output parameters is shown. The dredging trapping efficiencies decrease with an increasing depth and visa versa. In time the dredging efficiency also shows the expected development, for the deep trenches the decrease in time is less then that of the base run, for the shallow trenches the decrease is faster.

The relative trench width decreases in case of the deep alternatives (trench depth is 14 m and 10 m). This is mainly caused by the migration of the upstream slope. For the shallow alternatives (trench depth 2 m and 3.5 m), it can be seen that the width is increasing progressively in time. The relative sedimentation of the trench for the shallow alternatives shows an interesting development. Especially for the alternative with a trench depth of 2 m (Figure 4.2.15) the relative sedimentation decreases after 30 years. This is clearly influenced by the trench definitions. Due to the flattening of the slopes the trench volume increases according to the applied definition. This is also visible in the relative trench width which is also increasing significantly.

Summary of trench characteristics

Volume

The relative sedimentation of the various geometry’s after 50 years shows a remarkable agreement. The volume loss of the investigated trenches lies in the range of 15 to 22 %.
This is mainly due to the fact that for the shallow trenches the total width has approximately doubled after 50 years which results in an increased trench volume. It is important to realise that in fact the total volume of the trench cannot change. In fact, any decrease in of the volume is due to the trench definition. The development in time for the 2m alternative is clearly influenced by the trench definitions as after 30 years the volume starts to increase again (see Figure 4.2.15).

Migration
The migration of the trenches shows the expected development. For the deeper alternatives the migration is slower then that of the base run whereas for the shallow alternatives an increased migration can be observed.

Shape
The sedimentation-erosion on the slopes of the 14 m alternative are more or less independent. On the considered time scale the morphological developments mainly take place on the trench slopes. The settling of sediment occurs mainly on the upstream (southern) slope. The toe of the downstream slope only shows a minor shift. In case of a 10 m deep trench the same effects can be observed but to a lesser extent. If the other alternatives (trench depths of 3.5 m and 2 m, respectively) are compared it can be seen that the toe of the downstream slope shows a gradually accretion and horizontal shift if the depth is decreasing. The downstream slope also shows an increasing flattening if the trench depth decreases.

Trapping efficiency (dredging)
The dredging trapping efficiencies decrease with an increasing depth and visa versa. In time the dredging efficiency also shows the expected development, for the deep trenches the decrease in time is less then that of the base run, for the shallow trenches the decrease is faster.

Variation of trench slopes
Two alternatives have been investigated with increased (slope=8%) and decreased (slope=2%) slope angles, respectively (slope angles of base run are 4%). The results of the simulations are listed in Figures 4.2.16 and 4.2.17. If the two resulting profiles are compared it can be seen that in case of steep slopes more interaction between the two slopes is present. This is mainly due to the fact that the total width of the trench has increased in case of the flat slopes which results in lower sediment transports at the toe of the upstream slope due to the lag effect.

The 8% alternative has lost 26% of its original volume due to accretion, whereas the 2% alternative has accreted with 20%. This can be explained from the fact that the dredging trapping efficiency is somewhat higher for the 8% alternative. This, in combination with the smaller trench area, leads to a significant increase of the sedimentation in the trench.

In Figures 4.2.18 and 4.2.19 the development in time of the selected output parameters is shown. It can be seen that the dredging trapping efficiency is not affected by variation of the trench slopes. Also the development of the total trapping efficiencies shows a comparable development in time. For the alternative with the slopes of 2% it can be seen that the trench width decreases for the first 40 years. After 50 years the trench width has increased slightly. The migration of the top of the upstream slope is for the first 40 years larger than the migration of the top of the downstream slope. Probably the migration rate of the top of the downstream slope increases due to the increased interaction between the two slopes which
initially will result in accretion at the toe of the slope and subsequently a higher migration rate of the top of this trench slope.

**Summary of trench characteristics**

**Volume**
The 8% alternative has lost 26% of its original volume due to accretion, whereas the 2% alternative has accreted with 20%. This can be explained from the fact that the dredging trapping efficiency is somewhat higher for the 8% alternative. This, in combination with the smaller trench area, leads to a significant increase of the sedimentation in the trench.

**Migration**
The migration is not affected by the variation of the trench slopes as can been in Figures 4.2.18 and 4.2.19.

**Shape**
The shape of the trenches is not influenced significantly. It can be seen however that for the 2% alternative the trench slopes only show a limited morphological interaction which results in an initial decrease of the trench width, after 40 years a slight increase can be observed which indicates an increased interaction between the slopes.

**Trapping efficiency (dredging)**
The dredging trapping efficiencies are not significantly affected by the variation of the trench slopes.

**Variation of the water depth**

To investigate the variation of the water depth, two alternatives have been defined in which the water depth was increased and decreased with 5 m respectively. The tidal velocities were identical to the velocities applied in the base run, the wave condition was also kept the same. The results of the simulations are shown in Figures 4.2.20 and 4.2.21. It can be seen that for the decreased water depth the residual transports have increased considerably, for the increased water depth a decrease can be observed. In case of a decreased water depth it can be seen that the resulting trench after 50 years has experienced considerable sedimentation. The resulting trench for the increased water depth shows, compared to the base run, reduced sedimentation.

In Figures 4.2.22 and 4.2.23 the development of the selected output parameters in time is shown. It can be seen that in case of a reduced water depth that the response of the trench is much quicker than that of the base run. Both trapping efficiencies decrease at a much higher rate than that of the base run. For the other output parameters the same pattern can be observed. Also in this case the influence of the definition of the trench dimensions is visible in the relative trench width after 5 years. If the resulting profile is taken into consideration it can be seen that it is, also in this case, difficult to define the trench slopes. In case of an increased water depth the effects are less pronounced. It can be concluded for this case that the developments are slower than that of the base run.

**Summary of trench characteristics**

**Volume**
The alternative with 15m water depth has lost 36% of its volume after 50 year whereas the alternative with 25m water depth has lost only 13% (base run 23%). Hence, the
morphological development of the trench is fairly sensitive to variations in the total water depth. This is caused by the sensitivity of the calculated transports to variation in the water depth.

Migration
The migration of the trench increases in shallower water whereas for increased water depths the migration rate decreases.

Shape
For the 15m alternative it can be seen in Figure 4.2.22 that after 40 years a sudden increase in the trench width occurs. This again, is due to the applied trench definition. It can be seen however in Figure 2.2.20 the trench width does increase significantly. Variation in water depth have a significant effect on the shape of the trench.

Trapping efficiency (dredging)
For the 15m alternative it can be seen that the trapping efficiency is initially more or less comparable but that it decreases faster then in the base run. For the 25m alternative the opposite development can be observed.

Intercomparison of investigated geometry’s

The investigated alternatives are compared by determining the relative influence (RI) according to Equation 3.3.5. The parameter modification which appears in the left term in the equation is now determined by the factor with which the shape parameters have been modified (e.g. width is doubled). The seven output parameters that have been used in the previous sub-sections are also considered here. First however, the absolute values as they are listed in Table 4.2.1 are discussed. The comparison is sub-divided into: volume, migration, shape and dredging trapping efficiency.

Volume
In Figure 4.2.24 the relative sedimentation after 50 years is shown, the relative sedimentation is determined dividing the initial and final trench area. Compared to the output parameters which have been discussed so far, the relative sedimentation of the geometry's is fairly insensitive to variations of shape parameters. If the resulting profiles after 50 years of the trenches are considered it is clear that the method of defining the trench area has a significant influence. This is especially the case for trenches which experience significant sedimentation such as the shallow trenches (cf. Figures 4.2.6 and 4.2.7). All other alternatives show the expected results.

From the results below it can be concluded that wide trenches have a relative short morphological time scale whereas deep trenches have relative long morphological time scales but also a relative small migration rate. It is thought that a minimum migration rate is one of the most important parameters as it determines to a large extent the area in which the effects of sand mining can be distinguished. Especially, to limit the effects on the coast a low migration rate in cross shore direction is important.
Figure 4.2.24  Relative sedimentation after 50 years

Migration
In Figure 4.2.25 below the migration of the trenches is visualised by the displacement of the centres of gravity. It can be seen that all alternatives with a decreased width or depth show an (expected) increased migration rate.

Figure 4.2.25  Migration of centre of gravity of trenches

Shape
In Figure 4.2.26 the relative width and depth changes after 50 years are shown, the relative changes are determined by dividing the initial and final width or depth. As can be seen in Figure 4.2.26 the relative width is not significantly affected by variations in trench width. A decrease of the trench depth however results in a significant increase of the width and decrease of the depth. The deeper trenches do not significantly affect the relative shape changes of the trench after 50 years.
Figure 4.2.26  Relative changes of trench width and depth after 50 years

Trapping efficiency (dredging)
In Figure 4.2.27 below the trapping efficiencies after 0 and 50 years are compared. As stated earlier the total trapping efficiency is not significantly affected by the applied trench definitions and is therefore not really suitable for an intercomparison of the alternatives and is therefore not considered here.

The first and second bar represent the dredging trapping efficiency at the start and end (50 years) of the simulation, respectively. It can be seen that the $T_E^{dredge}$ is not significantly affected by if the trench width is increased, a decreased with results in a significant reduction of the trapping efficiency. For the shallow trenches $T_E^{dredge}$ reduces significantly. The trapping efficiency is increases if the water level is decreased and vice versa. The dredging trapping efficiency decreases after 50 years in all cases. The alternatives with reduced width and depth show a relative large decrease in $T_E^{dredge}$ which indicates that the morphological time scales of these geometry's are significantly shorter then the alternatives with an increased depth or width. It can be seen that the dredging trapping efficiency shows a significant reduction after 50 years if the water level is reduced with 5 m. In case of an increased water level of 5 m a relative small decrease of the dredging trapping efficiency can be observed.

Figure 4.2.27  Comparison of initial and final trapping efficiencies

Morphological time scale
From the discussed results so far, it can be concluded that the sediment trapping efficiency is a good indication of morphological developments of a trench. For the derivation of the morphological time scales it is suggested here, to use the dredging trapping efficiency. An extrapolation from the developments in time of this output parameter until it is zero, seems to be sensible criteria for the morphological time scale. Since it shows an approximate linear decrease for all the considered cases a first order estimate of the morphological time scales can be derived from the dredging trapping efficiencies in listed Table 4.2.1. In Table 4.2.2 the resulting morphological time scales are shown.

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<th>Time scale [yr.]</th>
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<td>depth = 2 m</td>
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</tr>
<tr>
<td>slope = 2 %</td>
<td>400</td>
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<tr>
<td>water depth = 15 m</td>
<td>150</td>
</tr>
<tr>
<td>water depth = 25 m</td>
<td>450</td>
</tr>
</tbody>
</table>

Table 4.2.2 First order estimate of morphological time scales

From the table above it can be seen that the morphological time scales are in the order of centuries (complete filling of trench). The alternative with a trench depth of 14 m has the largest time scale. According to this definition of the morphological time scale it can be concluded that deep trenches have relative long time scales. Time scales of wide trenches is relatively shorter. If the total volume of the trench is also taken into account it can be concluded that wide trenches are favourable to deep trenches if a minimal time scale is the main criteria. It has to be noted that the listed time scales should be considered as a lower limit as the backfilling rate will probably decrease in time when the trench accretes.

Relative influence

The relative influence for the various parameters are shown in Figures 4.2.28 and 4.2.29 which represent the geometry’s with shape parameters increased and decreased respectively. Apart from the simulation with an increased water depth, the increased shape parameters have no influence on the total trapping efficiency, only after 50 years some changes can be seen. The two deep trenches (10 and 14 m) have a relative higher total trapping efficiency after 50 years. The same can be concluded for the relative influence of the input parameters on the dredging trapping efficiency. For the decreased parameters there is also a relative limited influence of the shape parameters on the trapping efficiencies. The decrease of the water depth with 5 m has a relative large effect. If the relative influences concerning the trapping efficiencies for the increased and decreased shape parameters are compared it can be seen that especially in the case of decreased depths (2 and 3.5 m) a larger influence is visible. The relative change in width and depth of the trenches is small if the change in shape parameters is concerned. Also the influence on the migration of the trench is relatively small for all the considered geometry’s.
From the Figures 4.2.28 and 4.2.29 it can be concluded that especially variation of the water depth has the largest effect. Varying the width of the trench or the slopes of the trench only has a very limited effect on the resulting output parameters. The shallow trenches have a relative short morphological time scale and subsequently the relative influence on the output parameters is relatively large.

**Figure 4.2.28** Relative influence of alternatives with increased shape parameters

**Figure 4.2.29** Relative influence of alternatives with decreased shape parameters

### 4.3 Conclusions

**Intercomparison of investigated alternatives**

From the long term simulations it can be concluded that the relative influence of the investigated geometry’s is limited. Varying the water depth has the largest influence on the considered output parameters.
The maximum initial total trapping efficiency is 77% for the simulation with a reduced water depth. The minimum trapping efficiency is 36% for the shallow trench with a depth of 2 m. The maximum migration of the trench is 800 m for the decreased water depth case. The minimum migration is 260 m for the increased water depth case. The relative volume decrease of the trench after 50 years was the highest for the reduced water depth case, minimum relative sedimentation occurred for the trench with a width of 2400 m. It has to be noted that for the shallow trenches (trench depths of 2 and 3.5 m), the applied trench definitions resulted in unrealistic trench boundaries. This is mainly due to the fact that these trenches have experienced considerable sedimentation in the simulated period which makes an objective definition of the trench boundaries almost impossible.

The suggested definition of the morphological time scales by extrapolating the dredging efficiencies until it is zero seems to be a give reliable results (see Table 4.2.2). From the time scales it can be concluded that wide trenches are preferable over deep trenches if the morphological time scale is an important criteria. A deep trench of 14 m (twice as deep as base run) has a time scale of 800 years whereas a wide trench of 2400 m (four times as wide as base run) only has a time scale of about 500 to 600 years (Table 4.2.2). It has to be noted however that these are first order estimates of the morphological time scales. For a more reliable prediction simulations in the order of 500 years have to be made. This lies however outside the scope of this study.

The morphological development of all the investigated geometry's is mainly influenced by the lag of settling and picking up of the suspended sediment. If most of the sediment has settled on the trench bottom before the toe of the upstream slope, both slopes will develop almost independently from each other. For shallow and narrow trenches there is a relative strong morphological interaction. In case of reduced width or depth an increasing interaction between both slopes results in an increasing sedimentation at the toe of the upstream slope. This is clearly illustrated when the five simulations in which the depth was modified are compared. An increased sedimentation at the toe of the upstream slope also results in considerable flattening of that slope. It can be concluded that wide trenches have a relative short morphological time scale whereas deep trenches have relative long morphological time scales but also a relative small migration rate. It is thought that a minimum migration rate is one of the most important parameters as it determines to a large extent the area in which the effects of sand mining can be distinguished. Especially, to limit the effects on the coast a low migration rate in cross-shore direction is important.

**Cross shore versus long shore**
All the presented results have been derived for the longshore directions. In the cross-shore directions tidal currents are of a smaller order. The transport capacity of waves however increases if the water depth decreases. This has been illustrated by performing two simulations in which the water depth was increased (water depth is 25 m) and decreased with 5 m (water depth is 15 m) respectively. All hydrodynamic parameters have been kept similar to the those of the base run. It could be seen that the results after 50 years of simulation are seriously affected by these variations. A decrease of 25 % in water depth results in an increase of the migration rate of almost a factor 2. The morphological time scale (first order estimate of when the dredging trapping efficiency reaches zero) is also halved. It illustrates that the dynamic behaviour of a trench or a mined area changes dramatically if it is located or migrates to shallower waters. The results that have been presented in this study cannot directly be translated to the cross-shore direction. Additional research is needed to give a reliable indication of the dynamic development in cross-shore directions. The results of this study can be seen as an upper limit for the cross-shore case.

In Van Rijn et al. (1995) both cross-shore and longshore residual transport rates have been predicted. The ratio between longshore and cross-shore transports along the Dutch coast vary between 5 and 20. As a first estimate this ratio could be applied to the presented results.

**Applying the results to other locations along the Dutch coast**

The results of the various simulations may be used to get a first impression of the development of trenches or mined areas at different locations then at the Euro-Maas channel. The results presented in this chapter and Chapter 3, in which most important model and run parameters were varied, can be used for this purpose.

It is advised to determine the residual transports at the location of interest (see e.g. Van Rijn et al. 1995). If it is assumed that the wave conditions are more or less constant, the tidal current (duration, peak velocities, etc.) and sediment characteristics are the most important parameters. The resulting residual transports can then be used to derive an estimate for e.g. the migration rate of a trench from Tables 3.3.7a and 3.3.7b. Sub-sequently, Table 4.2.1 can be used to get an impression of the “geometrical” influence.

It is noted that extrapolation of the presented results can only be performed for locations where no significant 3-Dimensional currents are present (e.g. ebb tidal delta’s of the Wadden Sea and the southern delta coast). This limits the applicability of the model results to areas along the closed Dutch coast from Hoek van Holland to Den Helder.

In Van Rijn et al. (1995) the residual transports at 20 m water depth near Callantsoog are reported to be in the order of 75 m³/m/year. For a trench with approximately the same geometry as the Euro-Maas channel this would result in a migration of the trench in 50 years in the range of 500 to 700 m (Table 4.2.1).
5 Sand transport measurements on shoreface: test and calibration of OBS-transportmeter for near-bed layer

5.1 Introduction

Proper validation of mathematical models (like SUTRENCH and UNIBEST) for shoreface conditions requires field data of sedimentation and erosion in trenches/pits. Furthermore, the incoming sand transport should be known to serve as input data.

Measurements of sand transport rates on the shoreface in rough weather conditions are, however, problematic because self-registrating instruments operated from stand-alone tripods are required. Generally accepted instruments for measuring sand transport near the bed in oscillatory flow (short waves) plus tide/wind-driven currents in field conditions are, however, not yet available.

An attempt based on the application of a bag-type sampler (Van der Lee, 1994) was not successful, because of excessive scour around the sampler at relatively high velocities. Furthermore, this type of sampler cannot be operated from a stand-alone tripod in deep water.

Van Rijn (1997; in De Boer et al., 1997) tried to use a pump-type mechanical bed load transport meter (PBLT). This instrument was roughly tested in the large oscillating water tunnel of WL | DELFT HYDRAULICS. The net transport rates in the wave tunnel are known, which facilitates the calibration of the instrument. The PBLT consisted of two tube-type intake nozzles placed in opposite direction on a metal footplate (length of about 0.1 m, width of about 0.05 m). Each nozzle (internal diameter of 8 mm) was connected to a plastic hose for pumping of water and sediment. The nozzles are opened and closed alternately by circular metal valves through the action of oscillatory fluid drag on a metal pivoting plate connected by thin steel rods to the valves. The instrument basically measured the near-bed sand concentrations (integrated values) during the onshore and offshore phase of the wave cycle. These values were converted to transport rates, using measured near-bed velocities and calibration factors. The operation and calibration of the PBLT was found to be successful; scour around the instrument was minimum even at peak velocities of 1.7 m/s; the handling of the PBLT was however laborious; pumps and containers for collection of water/sand samples, etc. were involved.

For field conditions it is more attractive to measure the sand concentrations in the near-bed layer by an optical probe (OBS) in stead of a pump sampler. The field instrument should be equipped with a velocity meter to measure the velocities close to the bed (say 0.02 to 0.03 m above bed). The operational range of the OBS is between 1 and 100 kg/m², covering the concentration range in the near-bed layer. This OBS-transport meter will be tested in the present study.
The objective of the present tests is to get a rough calibration of the OBS-transport meter in the large oscillating water tunnel of WL | DELFT HYDRAULICS (Figs. 5.1.1 and 5.1.2), which has an operational range comparable to field conditions. The net transport rates in the wave tunnel are known (derived from measured bed level changes; Ribberink and Al-Salem, 1993), so that the transport rates measured with the instrument can be compared to the transport rates in the wave tunnel.

The test results were analysed and reported by Mr. B. Grasmeijer of the Dep. of Phys. Geography of the Univ. of Utrecht.

5.2 Instruments

5.2.1 Description of instruments

OBS-transport meter

The OBS-transport meter consists of three OBS-concentration sensors and one EMF-velocity sensor (Fig. 5.2.8) attached to a vertical rod on a footplate. The footplate is resting on the bed and can move downward (due to the movable instrument arrangement) when the bed surface is eroding. Using this arrangement, an approximately constant distance can be maintained between the bed and the measurement elevations of both EMF and OBS-sensors during a short measurement period of say 15 to 30 minutes. The EMF-sensor is placed at 0.05 m above the upper side of the footplate. The OBS-sensors are placed at distances of 0.03, 0.05 and 0.1 m above the upper side of the footplate. The effective measurement elevations of the OBS-sensors with respect to the surrounding bed surface are approximately 0.025, 0.045 and 0.095 m, as the instrument will sink down into the bed over about 0.005 m (due to local erosion). The OBS and EMF sensors are described below.

Optical Backscatterance Sensor

The heart of the OBS monitors is an optical sensor for measuring turbidity and suspended solids concentrations by detecting infrared (IR) radiation scattered from suspended matter. The response of the OBS sensors depends on the size, composition and shape of the suspended particles. For this reason, OBS sensors must be calibrated with the suspended solids from the waters to be monitored (using pump sampler).

OBS sensors consist of a high-intensity infrared emitting diode (IRED), a detector (four photodiodes), and a linear, solid state temperature transducer; mechanical dimensions are shown in Figure 5.2.2. The IRED produces a beam with half power points at 50° in the axial plane of the sensor and 30° in the radial plane (Figure 5.2.2). The detector integrates IR scattered between 140° and 160°. Visible light incident on the sensor is absorbed by a filter. Sensor components are potted in glass-filled polycarbonate with optical-grade epoxy. The sensor gain of the OBS has to be adjusted in order to match the highest output voltage expected from the OBS during the measurements with the input span of the data logger. One of two undesirable results will be obtained if the gain is not correctly adjusted. When the gain is too high, data will be lost because the sensor output is limited by the supply voltage and will “saturate” before peaks in sediment concentration are detected. If the gain is too low, the full resolution of your data logger will not be utilised. Experiments have
shown that the sensor gain varies with particle size. Ranging from mud (< 10 µm) to sand (> 200 µm) the gain decreases approximately by a factor 10. Knowledge of the particle size is therefore critical when monitoring suspended sediments. As regards the present experiments the gain of the OBS sensors was set to a value related to sand. In addition to this, the influence of the variation in particle size within the sand range was incorporated in the calibration procedure.

The OBS sensors have about the same size (or larger) as the length of gradients in the sand concentration being measured. This may cause hydrodynamic noise in the output signal because the turbulent flow around the sensor redistributes the particles in the water and increases the variation of sediment concentration above natural levels. Furthermore, the volume sampled by the OBS sensors depends on how far the IR beam penetrates into the water. This decreases as sediment concentration increases and so the sample volume is constantly varying with concentration which may also cause random noise in the output signal. From limited tests performed by the manufacturer it appeared unlikely that the random noise would exceed 30% of the mean signal in situations with high concentrations of coarse sediment. The manufacturer recommends post processing of the data with a low-pass filter to reduce the random noise in the output signal.

Other noise in the output signal may be caused by electronic noise or environmental conditions. According to specifications the electronic noise is insignificant for most applications. Some causes for environmental noise are: biofouling, excess in suspended sediment resulting from scour around instrument structures, and mooring cable, line or fish moving in front of the OBS sensor with the currents.

Some test were done in the present study to determine the penetration length of the OBS sensor in clear water. An aluminium plate was placed at various distances from the sensor and the output signal (scattered light from the plate) was recorded. The signal was minimum at distances larger than 0.1 m. Thus, the maximum length of the measuring volume will be about 0.1 m in conditions with low concentrations. In conditions with high concentrations the penetration length will be much smaller (0.01 to 0.03 m). No obstacles should be present within 0.1 m from the sensor.

**Electromagnetic velocity meter**

Horizontal orbital velocities were measured using an electromagnetic velocity meter (EMF) consisting of a 2-axis, 4 cm diameter, ellipsoid probe with an inaccuracy of 0.01 m/s ± 1% of the measured value and a zero stability of less than 5 mm/s.

The EMF generates an electromagnetic field under the ellipsoid probe, which when water flows along it generates an electric current proportional to the speed of the water flow. The EMF measures the water flow along two perpendicular horizontal axes. In the present experiments the x-axis of the EMF was oriented parallel to the wave tunnel. The EMF was placed at approximately 5 cm above the sand bottom.

**Laser Doppler Anemometer**

A forward scatter laser doppler anemometer (LDA) was used for the measurement of the horizontal and vertical velocity components of the water particles at z = 0.10 m or 0.20 m, dependent on the suspended sediment concentration. Below this level, it turned out to be impossible to measure velocities by LDA, because the suspended sediment concentration...
was so large that the particles blocked the laser beams and disturbed the measurements. The LDA was positioned on a measurement carriage standing over the tunnel, such that vibrations of the tunnel did not disturb the LDA (see Janssen et al, 1996).

**Pump sampler**

The pump sampler consisted of a vertical array of 10 intake tubes of 3 mm internal diameter connected to the pumps by plastic hoses. The lowest intake tubes were placed at about 0.01 m above the bed, with the intake openings placed in a direction transverse to the plane of orbital motion. The intake velocity was about 1 m/s, satisfying sampling requirements. The 10 liter samples were collected in calibrated buckets. The pump sampler was opened for 15 minutes giving an average concentration over the measuring time. In case of fast bed level variations due to large transport rates the operation time of the pump sampler was reduced to 8 minutes giving 5 liter samples. Figure 5.2.1 shows an outline of the pump sampler.

**5.2.2 Calibration of OBS concentration meters**

**Calibration of OBS**

A detailed description of the calibration of the OBS’s is given by Van de Meene (1994). The OBS’s were calibrated in a calibration tank of the Physical Geographic Laboratory at Utrecht University. An outline of the calibration tank is presented in Figure 5.2.3. Water is circulated in a closed circuit by a strong slurry pump. The sediment is added from above in a large perspex cylinder. The circulating water-sediment mixture is jetted into the cylinder, where the flow expands and decelerates. A flow straightener is present to make the flow as smooth as possible. The water sediment mixture flows undisturbed along the sensors with a velocity of approximately 0.25 m/s, which is large enough to suppress inhomogeneities due to settling and small enough to prevent inhomogeneities due to turbulence. Two OBS’s can be calibrated simultaneously. A suction tube is present near the sensors to draw concentration samples. A vacuum pump was mounted on top of the perspex cylinder, giving a small underpressure to prevent air bubbles flowing along the sensors.

Approximately 30 seconds is needed to distribute a sediment sample homogeneously through the tank after its injection. To maintain a certain margin, calibration measurements were always done two minutes after the sediment was added to the tank. The calculated input concentrations ($c_{input}$) were compared to the concentrations sampled with the suction tube ($c_{output}$) by Van de Meene (1994). He found that the ratio between the two varied between 0.7 and 0.9. The deviation from unity is probably caused by undersampling of the suction tube. Therefore, the calibrations were carried out using $c_{input}$ as actual concentration. According to Van de Meene (1994) the sediment distribution across the horizontal plane in the measurement region appeared reasonably homogeneous. Variations were in the order of 5 to 10% of the mean concentration.

The OBS’s were calibrated using the bed material of the wave tunnel experiments ($D_{50} = 0.12$ mm and 0.19 mm). The results of the calibrations are presented in Figure 5.2.4. The different response of the OBS’s to the two different grain sizes is reflected by the different slopes of the calibration curves. Figure 5.2.5 shows this influence of the grain size on the calibration factor (slope of calibration curve).
To take into account the effect of a varying grain size with height above the bed in the wave tunnel experiment, sediment samples obtained with the pump sampler were used to determine the grain size of the suspended sediment at different elevation above the bed. Subsequently, the calibration factor was determined using Figure 5.2.5. Vertical sorting of sediment is discussed further in Section 5.4.5 of this report.

**Calibration of EMF**

Before and after the wave tunnel experiments the EMF was calibrated at Delft Hydraulics by towing the EMF at different constant speeds through a tank. The calibration was performed in the range 0-2.5 m/s and both axes were calibrated independently. The calibration curves are linear with a high correlation coefficient ($r^2 \approx 0.99$). Comparison of the calibration performed before and after the wave tunnel experiments did not show significant changes in calibration. Figure 5.2.6 shows the calibration curves of the EMF sensor.

During the wave tunnel experiments the EMF velocity signals were compared with the velocity signals from the Laser Doppler velocity meter. An example of such a comparison is presented in Figure 5.2.7 (upper figure). In this test case both instruments were positioned at the same height above the bed (± 0.10 m). It can be observed from Figure 5.2.7 (upper figure) that the general trend of the EMF signal follows the Laser Doppler signal fairly well except for high-frequency peaks in the EMF signal, occasionally exceeding the Laser Doppler signal with more than 0.5 m/s.

From visual observations it appeared that in case of relatively large orbital velocities (> 0.7 m/s) the EMF attached to the vertical rod of the OBS-transport meter moved in a short shock-wise motion, in most cases at the moment of maximum orbital velocity. This shock-wise motion is inherent to the movable (in vertical direction) instrument arrangement. In order to test the influence of this phenomenon on the measured velocity signal the EMF was firmly fixed to the wave tunnel to prevent any motion of the instrument at all. It was found that in this case the high-frequency peaks did not occur in the measured velocity signal.

In case of test conditions with relatively large orbital velocities in combination with a net current the high-frequency peaks even exceeded twice the imposed orbital velocities (see Figure 5.2.7, lower figure). However, in most cases only part of the EMF signal showed these disturbances. Part of the EMF signal which did not show the high-frequency effects was selected for further analysis. Furthermore, the selected part of the EMF signals was low-pass filtered in order to eliminate possible remaining disturbances. Tests B00, B2, B7 and B8 (Table 5.3.1) were low-pass filtered at a frequency of 1.0 Hz. Tests J1, E2, E4, H2, H5 and H8 were low-pass filtered at a frequency of 0.28 Hz. The occurrence of high-frequency peaks related to the movability of the instrument arrangement should always be checked during analysis of the data.

**5.3 Test program**

The OBS transport meter was tested in the oscillating water tunnel (Ribberink and Al-Salem, 1991, 1992) of Delft Hydraulics. The tunnel has a width of 0.3 m and a height above the sand bed of 0.8 m. Two different test series were performed using two different sediment sizes with a median diameter of 0.19 mm and 0.12 mm respectively. Regular asymmetric wave motion (second order Stokes) and irregular wave motion were generated. The test program is given in Table 5.3.1. Orbital velocities were measured by means of a
Laser Doppler velocity meter at about 0.1 m and 0.2 m above the bed. Most tests were repeated twice to determine the variation related to small differences in bed arrangement (refilling of sand in the tunnel). The net sand transport rate during each test was not measured (by profiling of the bed), but it was taken from earlier tests results performed under similar conditions. This was considered to be sufficiently accurate to meet the objective of the present testing of the OBS transport meter (rough calibration).

The test procedure was as follows:

- start the datalogger
- positioning of OBS transport meter on bed; footplate flush with bed surface;
- positioning of vertical array of intake tubes with lowest intake tube at 0.01 m above bed;
- generation of oscillatory flow during 10 to 20 min.;
- establishment of equilibrium bed conditions during about 10 min.;
- sampling of water-sediment mixture through each intake tube after establishment of equilibrium conditions;
- separation of water and sand;
- determination of wet and dry sand volumes of samples.

5.4 Test results

5.4.1 Scour around OBS-transport meter

Scour around the footplate could be observed visually through the glass window of the tunnel. Initially, the OBS-transport meter had a relatively small footplate of 0.05 x 0.07 m$^2$. Preliminary test were done, which showed unacceptable scour around the footplate at high orbital velocities (scour depths of maximum 0.02 m). The scour depth around the footplate could be reduced to about 0.005 m by enlarging the footplate to 0.075 x 0.10 m$^2$. Scour observations during the tests with high orbital velocities (Test E2 and H5) showed that a small scour hole was generated with maximum dimensions: length = 0.1 m, width = 0.05 m, depth = 0.005 to 0.01 m. Considerably less scour (depths of 0.001 to 0.003 m) was observed in the test with relatively low orbital velocities. As a result of the scouring process the footplate sinks into the bed over a small distance (maximum of 0.005 to 0.01 m). Hence, the effective measurement elevation will be about 0.005 m (average over measurement period of about 15 min) lower than the nominal values. For field conditions it is advised to take an average lowering of 0.01 m (due to local scour), yielding the following effective measurement elevations: 0.02, 0.04 and 0.09 m (with an error of +/- 0.005 m). This should be checked in field conditions by (visual) monitoring of the scour process around the footplate in the swash zone (water depth of about 0.2 m).

5.4.2 Near-bed velocities

Three parameters were computed from the selected part of the EMF signal and from the laser doppler signal:

$u_{\text{mean}}$: mean velocity
$u_{\text{on}}$: onshore peak value of the oscillatory component ($u_{1/3,\text{on}}$ in case of irregular waves)
$u_{\text{off}}$: offshore peak value of the oscillatory component ($u_{1/3,\text{off}}$ in case of irregular waves)

Velocities were measured at two positions above the bed (EMF at $z = 0.05$ m and laser doppler at $z = 0.10$ or 0.20 m). Mean velocities at additional positions were computed assuming a theoretical logarithmic velocity distribution.
The data of bed-shear velocity and bed roughness derived from the velocity profiles are given in Table 5.4.1. The data give no more than a rough estimate of the bed-shear velocity and apparent roughness, because velocity measurements have only been taken at two elevations above the bed. Accurate determination of these parameters require velocity measurements in at least five points. The apparent bed roughness due to wave-current interaction was found to increase with increasing strength of the wave motion compared to strength of the current (test E2 and H5).

The data of velocity, concentration and transport rates are given in Tables 5.4.2 - 5.4.13. Velocity signals are presented in Figures 5.4.1 - 5.4.10.

For tests B00, B2, B7 and B8 (asymmetric wave motion without imposed current) the mean velocity at z = 0.05 m is negligible. The mean velocities at z = 0.10 m are relatively small (< 0.02 m/s) and onshore directed in most cases. Ribberink and Al-Salem (1993) found similar results in case of fine sand (D_{50} = 0.13 mm). The net flow is due to the asymmetric wave motion.

For tests J1, E2, E4, H2, H5 and H8 (symmetric wave motion superimposed on a current) the onshore directed mean velocity increases with height above the bed. Janssen et al (1996) found similar results in case of fine sand (D_{50} = 0.13 mm) and similar test conditions.

In all test cases the oscillatory velocities are slightly smaller (less than 10%) at the lower measuring position (z = 0.05 m) compared to the higher position (z = 0.10 or 0.20 m). Janssen et al (1996) found the oscillatory velocities to be almost constant for decreasing elevations.

### 5.4.3 Near-bed sand concentrations

Time-averaged concentrations were determined from the pump-measurements and from the OBS-measurements. The data are given in Tables 5.4.2 - 5.4.12. Instantaneous velocities and sand concentrations at the two lower elevations are given in Figures 5.4.2 to 5.4.10. The sand concentration peaks are largest (30 kg/m³) in test E2 and H5.

Time-averaged concentration profiles for all tests are presented in Figures 5.4.11 – 5.4.15. The concentrations measured with the pump sampler are compared to the concentrations measured with the OBS sensors. In case of relatively coarse sand (D_{50} = 0.19 mm) the upper OBS sensor produced time-averaged concentrations which were much larger than the time-averaged concentrations measured with the pump sampler. The concentrations at this upper level (0.1 m) are so small that they can not be accurately detected by the optical OBS-sensor; the signal in that case represents the background concentration of very fine silt present in the tunnel. Therefore, in case of coarse sand, the concentrations determined from the upper OBS sensor were not used for further analysis. In case of relatively fine sand (D_{50} = 0.12 mm) all three OBS sensors were used in the analysis. The concentrations at the upper level (about 0.1 m) were large enough to be detected by the OBS-sensor.

It can be observed from Figures 5.4.11 – 5.4.15 that the concentrations measured with the OBS and the concentrations measured with the pump sampler are of the same order of magnitude. On average, the OBS sensors gave values that were 15% larger (two largest deviations are 250% and -70%) in case of coarse sand and 30% larger (two largest
deviations are 150% and -50%) in case of fine sand than the values determined with the pump sampler. The largest differences were found for test B2 and H2 in which the concentration gradient is relatively large (fine sand, small orbital velocity and weak current compared to other tests).

### 5.4.4 Near-bed sand transport rates

Instantaneous suspended transport rates and mean suspended transport rates determined from EMF and OBS measurements are presented in Tables 5.4.2 – 5.4.12. The instantaneous transport is defined as the time-averaged value (over measurement period) of the product of the instantaneous velocity and concentration. This value represents the net transport. The mean transport is defined as the product of the time-averaged velocity and time-averaged concentration. Onshore and offshore-directed transport rates are distinguished: onshore transport is defined to be the transport in the direction of wave propagation (largest peak velocity for asymmetric waves). When a current is present, it is in the direction of wave propagation (in direction of largest peak velocity). The total net transport rates are based on data from Ribberink and Al-Salem (1993) and are given in Table 5.3.1.

Conclusions are summarised as follows:

**Tests B2, B7 and B8** (D$_{50}$ = 0.19 mm; regular asymmetric waves; no current):
- The instantaneous transport rates are offshore directed (against wave propagation direction) at the lower and middle elevations (about 0.025 and 0.045 m);
- The instantaneous transport rates are found to increase with increasing orbital velocities.
- The mean transport rates are negligible.

**Tests J1 and E2** (D$_{50}$ = 0.19 mm; regular symmetric waves; weak current in onshore direction):
- The instantaneous transport rate is onshore directed at the lowest elevation and offshore directed at the middle elevation.
- The magnitude of the instantaneous transport rate at both measuring points (although oppositely directed) increases when increasing the maximum orbital velocity from 1.1 m/s to 1.5 m/s.
- The mean transport component is relatively large, onshore directed, increasing with increasing orbital velocities and increasing for lower elevations.
- Instantaneous transport in offshore direction and mean transport in onshore direction implies oscillatory transport in offshore direction against the current direction.

**Test E4** (D$_{50}$ = 0.19 mm; regular symmetric waves; strong current in onshore direction):
- The instantaneous transport rate is onshore directed at both elevations above the bed.
- The mean transport component is relatively large, onshore directed and increasing for lower elevations.

**Test H2 and H5** (D$_{50}$ = 0.13 mm; regular symmetric waves; weak current in onshore direction):
- The instantaneous transport rate is onshore directed at the upper elevation (about 0.1 m) and tends to be offshore directed at lower elevations, especially in case of relatively large orbital velocities.
- The mean transport component is relatively large, onshore directed and increasing for lower elevations.
Test H8 ($D_{50} = 0.13$ mm; regular symmetric waves; strong current in onshore direction):

- The instantaneous transport rate is onshore directed and nearly constant for lower elevations.
- The mean transport component is much larger than the instantaneous transport rate and increasing for lower elevations; hence, the oscillatory transport is offshore directed.

In all tests the net bed load transport in the zone below the lowest measurement point (<0.025 m) is dominant (90% and larger), even in the test with a relatively strong current of 0.4 m/s (Test E4 and H8, see Table 5.5.1). The net bed load transport represents the difference between the total net transport rate (measured earlier by Ribberink and Al-Salem, 1993) and the suspended transport measured in the present study. The relative importance of bed-load transport should be checked in field conditions.

### 5.4.5 Vertical sorting

Sediment samples obtained with the pump sampler were used to determine the grain size of the suspended sediment at different elevation above the bed. In order to determine the grain sizes at approximately the same height as the height of the OBS sensors, sediment samples from the following suction tubes were used (see Figure 5.2.1), of which some were put together:

- 2 and 3: $z \approx 0.025$ m
- 4: $z = 0.045$ m
- 5, 6 and 7: $z = 0.095$ m

A grain size analysis was performed using the Visual Accumulation Tube (VAT) of Delft Hydraulics. For each sample a (cumulative) probability distribution was made of the settling velocities. From the 10%, 50% and 90% exceedance values, the values for $D_{10}$, $D_{50}$ and $D_{90}$ were determined. The vertical distribution of these grain sizes is presented in Table 5.4.13.

It appears that the finer size fractions are more easily suspended to higher elevations above the bed, while the coarser grain sizes stay closer to the bed, especially in case of relatively coarse bed material ($D_{50} = 0.19$ mm). At height $z = 0.025$ m and $z = 0.095$ m above the bed the median diameter of the suspended sediment is approximately 20%, respectively 30% smaller than the median grain size of the bed material.

In case of relatively fine bed material ($D_{50} = 0.12$ mm) the vertical sorting is less pronounced. For nearly all tests with fine sand and at all three heights above the bed the median diameter of the suspended sediment is approximately 10% smaller than the median grain size of the bed material.

Ribberink and Al-Salem (1992) also found a considerable sorting of sand sizes under similar conditions as in the present experiments and bed material with a median grain size of 0.21 mm. The coarser fractions have a preference to be transported close to the bed in the direction of wave propagation (asymmetry effect), while the finer fractions have a preference to be transported higher in suspension against the direction of wave propagation due to the presence of mean offshore-directed flow above the boundary layer (Ribberink and Al-Salem, 1993).
5.5 Comparison of measured and computed transport rates

5.5.1 Definition of bed load and suspended load transport

In this report the total load of moving sediment (q_total) is defined to consist of two parts:
- bed load transport (q_near-bed) in the near-bed layer below the lowest measurement point (<0.025 m) and,
- suspended transport (q_suspension) in the suspension layer (>0.025 m).

The total load transport rate was determined during previous experiments in the wave tunnel using a mass conservation technique (Ribberink and Al-Salem, 1993; Janssen et al., 1997). The present measurements with the OBS-transport meter are related to the suspension layer between z= 0.025 and 0.095 m (sand concentrations above z= 0.095 m are negligibly small). This facilitates the determination of the transport rate in the unmeasured zone near the bed (<0.025 m):

\[ q_{\text{near-bed}} = q_{\text{total}} - q_{\text{suspension}} \]

5.5.2 Bed load transport

The bed load transport in the near-bed layer can not be measured directly in field conditions, because instruments are not yet available. In this section it is tried to relate the near-bed transport rate to a simple transport formulation of the form \( q = q(\theta) \), in which \( q \) is the instantaneous transport rate during the wave cycle in sediment volume per unit width and time and \( \theta \) is the instantaneous dimensionless bed shear stress. The formula will be calibrated, using transport data that have been measured (indirectly) in the wave tunnel.

The instantaneous near-bed transport rate is expressed as (see Van Rijn et al., 1995):

\[
\frac{<q_{\text{near-bed}}^{>}}{\sqrt{(s-1)gD_{50}}} = m <\left| \theta^{'} - \theta_{\text{cr,shields}}^{'} \right|^{n} > \theta^{'} \quad (5.1)
\]

in which:
- \(<...> = time-averaging\)
- \(q_{\text{near-bed}} = \) instantaneous transport rate near the bed during the wave cycle [m³/m/s]
- \(s = \) relative density of sediment \((\rho_s/\rho)\) [-]
- \(g = \) acceleration of gravity [m/s²]
- \(D_{50} = \) median grain size of sediment [m]
- \(m, n = \) coefficients [-]
- \(\theta = \) instantaneous dimensionless bed shear stress [-]
- \(\theta_{\text{cr,shields}} = \) critical dimensionless bed shear stress [-]

The instantaneous dimensionless bed shear stress \( \theta' \) is calculated as described below:

\[
\theta' = \frac{\tau'}{(\rho_s - \rho)gD_{50}} \quad (5.2)
\]

An approach as suggested by Grant and Madsen (1979) is used by assuming that the bed shear stress can be expressed as a quadratic function of the combined wave-current velocity \( u \) at some height \( z \) above the bed (above the wave boundary layer):

\[
\tau' = \frac{1}{2} \rho f_{\text{crit}} u^{2} \quad (5.3)
\]

The quadratic friction law is used together with a weighted friction coefficient for currents and waves. Following van Rijn (1993):
\[ f_{cw} = \alpha f_c + (1 - \alpha) f_w \]
with:
\[ \alpha = \frac{u_{\text{mean}}}{u_{\text{mean}} + \bar{U}_\delta} \]
\[ \bar{U}_\delta = \frac{1}{2} (u_{1/3,\text{on}} + u_{1/3,\text{off}}) \quad \text{in case of irregular waves} \]
\[ \bar{U}_\delta = \bar{u} \quad \text{in case of regular waves} \]

In the present study the current-related friction coefficient is used, as follows:
\[ f_c = 0.24 \left( \log \left( \frac{12h}{k_{s,c}} \right) \right)^{-2} \]

The wave-related friction factor is calculated according to Swart (1974):
\[ f_w = \exp \left( -6 + 5.2 \left( \frac{\hat{A}_\delta}{k_{s,w}} \right)^{-0.19} \right) \]
\[ f_{w,\text{max}} = 0.3 \]
with:
\[ \hat{A}_\delta = \frac{2\pi}{T_p} \bar{U}_\delta \]
\[ k_{s,c} = k_{s,w} = 3D_{90} \]

with:
\[ \hat{A}_\delta = \text{near-bed peak orbital excursion} \quad [\text{m}] \]
\[ f_c = \text{current-related friction factor} \quad [-] \]
\[ f_w = \text{wave-related friction factor} \quad [-] \]
\[ f_{cw} = \text{friction factor in case of current + waves} \quad [-] \]
\[ g = \text{acceleration of gravity} \quad [\text{m}^2/\text{s}] \]
\[ k_{s,c} = \text{current-related bed roughness height} \quad [\text{m}] \]
\[ k_{s,w} = \text{wave-related bed roughness height} \quad [\text{m}] \]
\[ s = \text{relative density of sediment} \quad (\rho_s/\rho) \quad [-] \]
\[ T = \text{wave period} \quad [\text{s}] \]
\[ u = \text{horizontal velocity} \quad [\text{m/s}] \]
\[ u_{\text{mean}} = \text{time-averaged horizontal velocity} \quad [\text{m/s}] \]
\[ u_{\text{on}} = \text{onshore directed peak orbital velocity} \quad [\text{m/s}] \]
\[ u_{\text{off}} = \text{offshore directed peak orbital velocity} \quad [\text{m/s}] \]
\[ \bar{u} = \text{amplitude of horizontal (sinusoidal) velocity} \quad [\text{m/s}] \]
\[ z = \text{height above sand bed} \quad [\text{m}] \]
\[ z_0 = \text{zero-velocity level} \quad [\text{m/s}] \]
\[ \alpha = \text{coefficient} \quad [-] \]
\[ \theta = \text{instantaneous dimensionless bed shear stress} \quad [-] \]
\[ \theta_{\text{cr,shields}} = \text{critical dimensionless bed shear stress} \quad [-] \]
\[ \rho = \text{density of water} \quad [\text{kg/m}^3] \]
\[ \rho_s = \text{density of sediment} \quad [\text{kg/m}^3] \]
\[ \tau = \text{instantaneous bed shear stress} \quad [\text{N/m}^2] \]

In the present study the mean bed shear stress is not calculated interatively. This is in contrast to Van Rijn et al (1995). They proposed the mean bed shear stress to be first
approximated with the input of the grain roughness $k_s = 3D_{90}$. With this bed shear stress they obtained a new approximation using $k_s = 3D_{90}$. The computation was repeated until the solution converged and changed less than 1% during the last iteration. In the present study the first approximation $k_s = 3D_{90}$ appeared to be quite accurate. The threshold value $\theta_{cr, shields}$ is calculated according to the classical shields curve, as modelled by Brownlie (1981; in Chen, 1995):

$$\theta_{cr, shields} = 0.22 Y + 0.06 \times 10^{-7.7} Y$$

in which:

$$Y = \frac{\sqrt{g(s-1)D_{50}^3}}{\nu}$$

Van Rijn et al (1995) found $m = 9.1$ and $n = 1.78$ using datasets from various researchers. The validity range expressed in terms of the Shields parameters and the grain sizes was given as: $\theta' = 0.1 - 7$ and $D_{50} = 0.2 – 3.8$ mm. However, for steady flows the bed-load formula showed a tendency of increasing $m$ for increasing Shields number.

In the present study an alternative current-related friction coefficient is used compared to that proposed by Van Rijn et al (1995). Therefore, different values for the coefficients $m$ and $n$ may be expected. As a first approximation the following values were used: $m = 5$, $n = 1.65$.

5.5.3 Measured and computed transport rates

Measured and calculated transport rates are presented in Table 5.5.1. It can be observed from Table 5.5.1 that the transport rates based on EMF and OBS data ($q_{suspension}$) are relatively small. The absolute values of $q_{suspension}$ make less than 10% (on an average: 4%) of the total net transport rate $q_{total}$. This indicates that more than 90% of the total transport rate takes place in the unmeasured zone near the bed ($z < 0.025$ m). This is consistent with findings by Ribberink and Al-Salem (1991, 1992) in the wave tunnel and findings by Vincent and Green (1990) in the field.

It can also be observed from Table 5.5.1 that the near-bed transport rates are rather accurately predicted using the method described above. On average, the calculated transport rates are 10% smaller than the measured transport rates. The standard error is less than 20%. Largest differences are found for test B8 (large asymmetrical orbital velocities; no current). The calculated near-bed transport rate is 45% smaller than the measured near-bed transport rate in this case. Slightly better results are obtained using $m = 5.35$ and $n = 1.65$. Using $m = 5$ and $n = 1.78$ (same value for $n$ as determined by Van Rijn et al, 1995) also improves the results. The calculated transport rates are 3% larger (on average) than the measured values in this case. The standard error is less than 20%. The largest difference is found for test B8 in which the calculated transport rate is 36% smaller than the measured transport rate. A comparison between calculated and measured transport rates is presented in Figure 5.4.16.

5.6 Conclusions and recommendations

The main results and conclusions of the present experiments are summarised as follows:
Velocities:
- The general trend of the EMF signal follows the Laser Doppler signal fairly well except for high-frequency peaks in the EMF signal; the latter being caused by the movable instrument arrangement.
- Part of the EMF signal which did not show the high-frequency effects was selected for further computation. The selected part of the EMF signal was low-pass filtered in order to eliminate possible remaining disturbances.
- Results from the present study are consistent with findings from previous studies in the wave tunnel by Ribberink and Al-Salem (1993) and Janssen et al (1996).

Concentrations:
- Time-averaged concentration can be rather accurately determined using the OBS transport meter. On average, the OBS sensors gave values that were 15% larger in case of coarse sand and 30% larger in case of fine sand than the values determined with the pump sampler.

Vertical sorting:
- In case of relatively coarse bed material ($D_{50} = 0.19$ mm) the median grain size of the suspended sediment was 20% to 30% smaller than the median grain size of the bed material.
- In case of relatively fine bed material ($D_{50} = 0.13$ mm) the vertical sorting was less pronounced. The median grain size of the suspended sediment was 10% smaller than the median grain size of the bed material.

Transport rates:
- The near-bed transport rate ($z < 0.025$ m) was relatively large and accounted for more than 90% of the total net transport rate.
- The near-bed transport rate can be rather accurately predicted (standard error less than 20%) using a simplified version of the transport formulation by Van Rijn et al (1995).
- Best results (calibration) were obtained using $m = 5.35$ and $n = 1.65$, or, $m = 5$ and $n = 1.78$.
- The calibrated formula will be used to estimate the sand transport in the unmeasured zone near the bed in field conditions with the measured instantaneous near-bed velocities as input data; the sand transport above this zone will be derived from measured sand concentrations and velocities (using optical, acoustical and electromagnetic sensors).

As regards the use of the OBS transport meter, the following recommendations are made:
- Special attention should be paid to the vertical sorting of sediment because the OBS sensor is more sensitive for the finer fractions, thus influencing the calibration factor.
- Special attention should be paid to filtering off the high-frequency disturbances in the EMF signal caused by short shock-wise motions of the OBS transport meter. Low-pass filtering at a frequency of at most 1.0 Hz is recommended.
6 Evaluation of process knowledge and proposal for large-scale laboratory tests

6.1 Introduction

The long-term effects of large-scale mining of sand in deeper water on the nearshore morphology (shoreline position and shoreface) can only be evaluated if the net annual sand transport rates in longshore and in cross-shore direction are known with sufficient accuracy.

Research in 1995 (Van Rijn, 1995) has resulted in estimates (and variation ranges) of the net annual transport rates in longshore and cross-shore direction, based on state of the art mathematical computations (see Section 2.2). The computational results show that the variation ranges of the cross-shore transport rates are considerably larger than those of the longshore transport rates, because the cross-shore transport rate is a delicate balance of onshore and offshore-directed transport processes. At present state of research the net annual transport direction can not be determined along the cross-shore profile, which is an unacceptable situation with respect to coastal zone management.

The SUTRENCH-sensitivity computations (present study results) show that the model is rather sensitive to the wave-related sediment mixing parameters, which are of importance for the vertical distribution of the sediment concentrations. The uncertainty ranges of these parameters are too wide; more process data are required to improve the mixing-related parameters. Furthermore, the SUTRENCH-model appears to be quite sensitive to the suspended sediment size, which is often relatively small on the shoreface (<0.2 mm). On the one hand this means that the particle size data should be based on a relatively large amount of bed samples to obtain reliable average values; on the other hand the winnowing of finer sand fractions in relation to wave conditions should be studied in more detail (experimentally). This may lead to an estimate of the suspended size as a function of wave conditions. If necessary, a multi-fraction method can be implemented to compute the sand transport rates.

Another problem is the application of logarithmic velocity profiles. This leads to underestimation of near-bed velocities and bed-shear stresses and hence the transport gradients in the acceleration (downstream) zone of the trench/pit. As a result the scour of the bed in this zone of the trench/pit (migration of the downstream slope) is underestimated.

The SUTRENCH-simulation computations show that the morphological behaviour of longshore trenches (pits) can be simulated reasonably well, provided that the net longshore transport approaching the trench is known with sufficient accuracy. Information of the net longshore transport rates have been obtained from morphological field data (trial dredge trenches, sand dumpings, etc., see Chapter 2), yielding values of comparable order as previously computed values (Van Rijn, 1995), which is a rather encouraging result.

The long-term (50 years) SUTRENCH-simulations of trenches in longshore direction show that the downstream end of the trench (taking a downstream depth of 0.5 m as a characteristic value) can migrate over a maximum distance of about 2 km over 50 years in a
depth of 20 m. Longshore trench migration is of interest with respect to pipelines and cables buried in the bottom. In cross-shore direction the trench migration will be much smaller, because the tidal currents and the associated net transport rates are much smaller (factor 5). Sensitivity computations show that the trench migration is strongly dependent on depth. Using the SUTRENCH-model, the migration of sand mining pits in cross-shore direction can be determined as a function of depth. From these results the limiting depth (without noticeable effect on the coast on a certain time scale) for sand mining can be obtained. Most likely, this depth will be somewhere between the 10 m and 20 m depth contours. Crucial in such an evaluation is however the accuracy of the applied model and hence the accuracy of the underlying process knowledge.

Compared to the available knowledge of the longshore transport rates, the cross-shore transport rates are not known with sufficient accuracy. Furthermore, field data of cross-shore transport rates and morphological features are missing, so that the available models can not be calibrated.

The sand transport measurements in the wave tunnel (Chapter 5) show that the sand transport rates in a thin layer (order of 2 to 3 cm) near the bed are relatively large and may be dominant. It should be realized, however, that the suspended transport in the wave tunnel may be underestimated considerably, because vertical velocities are absent (no real waves) in the wave tunnel. Thus, the relative importance of the near-bed sand transport can not yet be evaluated. This information can not come from field data because instruments to measure the transport in the near-bed layer are not operational.

Given these facts, the available mathematical models can not yet be applied to accurately estimate the cross-shore behaviour of sand mining pits/trenches.

Therefore, the research efforts should be focused on improvement of cross-shore sand transport processes and morphology.

The best strategy to extend the knowledge of cross-shore sand transport processes and morphology in deeper water would be to perform a detailed field study of these processes in and near a test pit (process measurements and monitoring of morphological behaviour) in combination with model simulations. Accurate sand transport measurements in deeper water under storm wave conditions are, however, not yet feasible, because self-registrating in-situ instruments are not available. The test results of the OBS-transport meter in the wave tunnel (Chapter 5) show however promising results for storm conditions. This method should be further tested in field conditions. Accurate (remotely controlled) determination of the measurement elevations above the bed is the most pressing problem to be solved. A practical disadvantage of a trial pit/trench in nature is that the results will not readily be available, as the monitoring of the morphological behaviour of the pit/trench will take some years. The cost involved will also be relatively high, as the dredging of test pits/trenches is necessary.

Based on this, it may be more cost-effective to perform large-scale experiments in the Delta-flume of WL | DELFT HYDRAULICS with the objective of improving the process knowledge of cross-shore sand transport and morphology. This can be achieved by studying the morphological behaviour of a trench and a bar under conditions of non- or weakly breaking storm waves combined with a weak current. The hydrodynamic conditions should be such that the filling and onshore migration of a trench will take place, simulating the behaviour of a sand mining pit in water with depths between 10 and 20 m migrating to the shore. It is also proposed to study the morphological behaviour of a sand bar under shoaling
wave conditions, creating more extreme conditions (better measurable) with respect to the oscillatory suspended sand transport. The results will be used to improve the existing mathematical models of cross-shore sand transport and morphology.

Summarising, the objectives of large-scale experiments in the Delta-flume are:

- measurement of sand concentration profiles to derive the wave-related mixing parameters under non-breaking waves in relatively deep water outside the surf zone; based on this the validity range of the process models can be extended to deeper water (the present parameters are only valid for shallow water in the surf zone);
- measurement of velocity profiles along the trench/bar profile to improve the mathematical model in the deceleration and acceleration zones;
- measurement of bed and suspended sand composition to study the effect of winnowing processes on the computed concentration profiles, sand transport rates and bed evolution and to evaluate whether the sand transport rates should be computed by using the single fraction method or the multi-fraction method;
- measurement of near-bed orbital velocities and mean current velocities and associated oscillatory and mean transport components to determine their relative importance to the net transport rates; these parameters can be measured at close spacing along the flume so that the cross-shore gradients can be determined accurately; based on this the measurement locations during future field measurements can be better determined;
- measurements of cross-shore trench morphology and associated net transport rates to improve process models and behaviour-related models, so that the cross-shore behaviour of sand mining trenches and pits in the nearshore zone (especially between 10 and 20 m depth contours) and their influence on the shoreline can be described;
- measurements of cross-shore bar morphology and associated net transport rates to improve process models and behaviour-related models, so that the cross-shore behaviour of natural and artificial bars (nearshore dumpings) in the outer surf zone and their influence on the shoreline can be described; similarly the behaviour of ebb delta bars in front of inlets can be better described, if cross-shore transport processes are better known;
- indirect evaluation of the relative importance of the bed load transport (verification of wave tunnel results), using infill rates of trench and measured suspended transport upstream of trench (diagnostic modelling using data and models);
- the net transport rates derived from the measured data and process models can be directly used to feed the available behaviour-related models, which fully depend on net transport rates formulated in terms of wave-current conditions.

Hereafter, a proposal is given for the experimental setting in the Delta-flume.

6.2 Description of Delta-flume

The flume has a length of 230 m, a width of 5 m and a depth of 7 m (inside the hall the depth is 9 m). The additional depth in the hall of the flume makes this facility very suitable for combined hydrodynamic and geotechnical research. The flume has a wave generator able to generate waves with a maximum height of 2.5 m (regular waves) or a maximum significant wave height of $H_s = 1.75$ m for irregular waves. The wave generator is provided with patented devices with which wave reflection against the board can be avoided. The flume is provided with a controlled towing carriage, maximum carriage velocity of 1 m/s.
6.3 Design of experiments

6.3.1 Scale relationships

The basic features of sand transport and morphology in coastal seas (current and waves) can be characterised by a series of dimensionless numbers being: the relative wave height, the Froude number, the suspension number and the particle mobility number.

Simulation of field conditions in a laboratory flume or basin requires that these characteristic numbers have approximately the same values in the field and in the laboratory.

The wave heights and near-bed velocities in the Delta-flume are smaller than those in nature. The above-mentioned characteristic numbers will remain approximately constant, if the sand size can be scaled down over a sufficiently large range. There are, however, practical constraints with respect to the availability of fine and reasonably clean (percentage of silt and clay<3%) sand in sufficiently large quantities (500 to 1000 m³). Commercially available at a reasonable price is sand with a median diameter of about 0.15 mm.

The basic characteristic numbers are:

Relative wave height: \( H_s / h \)

Froude number: \( u / (gh)^{0.5} \)

Suspension number: \( U_w / w_s \)

Particle mobility number: \( (U_w)^2 / (s-1)gd_{50} \)

with: \( u \) = depth-mean velocity, \( H_s \) = significant wave height, \( w_s \) = particle fall velocity, \( T \) = wave period, \( U_w \) = peak orbital velocity near the bed, \( d_{50} \) = median particle diameter, \( s = \rho / \rho_w \) = relative density.

The relative wave height should be smaller than about 0.5 to prevent strong breaking of waves in deeper water.

The suspension number determines the ratio of suspended load transport and bed-load transport.

The particle mobility number describes the type of bed forms present along the sea bed. During calm and moderate wave-energy conditions, ripple-type bed forms are generally present in nature, whereas during storm conditions the ripples are washed out, resulting in plane bed with sheet flow transport. The transition occurs at a particle mobility number of about 250. It is of essential importance that sheet flow occurs in the Delta-flume during simulation of storm conditions.

6.3.2 Wave, current and sea bed conditions North Sea and Delta-flume

The hydrodynamic conditions during storms in deeper water of the North sea can be represented as:
Sand transport at the middle and lower shoreface of the Dutch coast

May 1998

water depths: 10 to 20 m;
wave heights: 4 to 8 m;
wave periods: 6 to 10 s;
mean current cross-shore: 0.05 to 0.1 m/s
median sand size: 0.2 to 0.25 mm;
fall velocity (te= 15 °C): 0.023 to 0.031 m/s

Using these values, the characteristic numbers for field conditions roughly are:
Hs/h= 0.4;
u/(gh)0.5= 0.005 to 0.01;
Uw/ws= 50 to 80;
(Uw)²/((s-1)gd₅₀)= 400 to 1000.

Experiments in the Delta-flume impose restrictions with respect to the selection of the length scales and sand scales involved. The minimum sand size commercially available in large quantities (500 to 1000 m³) is about 0.15 mm. The maximum water depth (above a sand bed with thickness of about 1 m) and the maximum significant wave height (irregular waves) in the Delta-flume are about 3 and 1.5 m respectively. Thus,

maximum water depth: 3 m;
maximum wave height: 1.5 m;
minimum sand size: 0.15 mm;
fall velocity (te= 15 °C): 0.016 m/s.

Using these values and a wave period of Tₚ= 5 s, the maximum near-bed peak orbital velocity is about Uw= 1.15 m/s and the dimensionless numbers roughly are:
Hs/h= 0.5;
Uw/ws= 70;
(Uw)²/((s-1)gd₅₀)= 550.

The minimum wave height will be about Hs= 1 m, yielding Uw= 0.8 m/s and:
Hs/h= 0.33;
Uw/ws= 50;
(Uw)²/((s-1)gd₅₀)= 260.

Thus, using a water depth of about h= 3 m, a wave height between Hs= 1 and 1.5 m (period Tₚ= 5 s), sand of 0.15 mm, it is possible to simulate storm conditions (non-breaking waves) with sheet flow transport (no ripples) and the right distribution of bed load and suspended load transport in the Delta-flume. It is proposed to use a cross-shore mean current velocity of about 0.05 to 0.1m/s in the flume to represent the peak tidal current (discharge Q of about 1 m³/s), yielding a Froude number of the right order of magnitude (about 0.01). Special arrangements are necessary to generate a current in the Delta flume.

6.3.3 Dimensions of trench and bar

Simulating a sand mining pit with a depth of 3 to 5 m (in nature), the trench depth in the Delta flume (water depth of 3 m) will be about 1 m. A practical length will be about 30 to 50 m with side slopes of 1 to 10, see Figure 6.3.1.
The sand bar is proposed to have similar dimensions: crest height of 1 m, length of 30 to 50 m and side slopes of 1 to 10.

### 6.3.4 Expected morphological changes (SUTRENCH results)

SUTRENCH was used to make a preliminary run over 200 hours to simulate the migration of a trench (depth = 1 m, length = 30 m, slopes = 1 to 10) in the Delta-flume. The mean velocity was set to 0.075 m/s. The water depth was 3 m. The significant wave height was 1.25 m (period of 5 s). Other data are \(d_{50} = 0.00015\) m, \(d_{90} = 0.0002\) m, \(k_{x,c} = k_{x,w} = z_a = 0.01\) m. The bed evolution is given in Fig. 6.3.2.
6.4 Test program and instruments

6.4.1 Hydrodynamic conditions

The hydrodynamic conditions are given in the following Table 6.4.1.

<table>
<thead>
<tr>
<th>Morphological feature</th>
<th>Wave height (m)</th>
<th>Mean current (m/s)</th>
<th>Type of measurements</th>
<th>Duration of test (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trench</td>
<td>1, 1.25, 1.25</td>
<td>0.05, 0.05, 0.05</td>
<td>sand transport, sand transport, bed evolution</td>
<td>1, 1, 10-15</td>
</tr>
<tr>
<td>Bar</td>
<td>0.8, 1, 1.25, 1</td>
<td>0, 0, 0, 0</td>
<td>sand transport, sand transport, sand transport, bed evolution</td>
<td>1, 1, 10-15</td>
</tr>
</tbody>
</table>

Table 6.4.1 Test program

6.4.2 Parameters to be measured and available instruments

The parameters to be measured are:

- bed levels of trench and bar at several times during the test (longitudinal bed profiles),
- wave height at various locations along the flume at several times,
- instantaneous velocity and sand concentration profiles updrift, in and downdrift of trench and bar at several times,
- time-averaged sand concentrations at same locations at several times,
- water pressure near bed at same locations at several times,
- bed forms dimensions at same locations,
- bed material samples at same locations,
- suspended sediment samples for determination of fall velocity at same locations,
- water temperature (daily).

The instruments required are:
- ASTM (5 levels) and OBS (3 levels) transport meters and EMF (3 levels) velocity meter in a movable frame (tripod; UU arrangement),
- pump sampler at 7 levels in the vertical (2 OBS and 5 ASTM levels) with nozzles close to ASTM and OBS sensors (UU),
- pressure sensor (1) attached to frame (UU),
- ripple profiler attached to frame (UU),
- underwater video camera (RWS) attached to frame,
- wave height recorders (2 fixed locations and 1 movable at frame location, DH),
- EMF velocity meters (3) attached to flume wall upstream of trench (DH),
- echo-sounding equipment attached to movable carriage on top of flume (5 longitudinal profiles, DH),
- thermometer,
- bed sample grab.

A frame (tripod) with instruments will be moved along the flume by a mobile crane. Pump system will be installed to generate discharge of maximum 1 m³/s.

### 6.4.3 Required data analyses

The following parameters will be derived from the data:

**waves (both for high frequency<10 s and low-frequency motions>10**

- wave height (m) and periods (s),
  - counting: $H_{rms}$, $H_{1/3}$, $H_{1/10}$, $T_{rms}$, $T_{1/3}$, and $T_z$
  - spectral: $H_{m,0}$ and $T_p$
  (both high and low-frequency)
  (1 value per hour over available burst length)

- fraction of broken waves;

**velocity (m/s)**

- time-averaged velocity at 8 levels above bed,
- asymmetry parameters: $U_{peak,1/3,on}$ and $U_{peak,1/3,off}$ and $U_{rms}$ of cross-shore near-bed velocity component (h.f. and l.f. motions); 1 value per hour over available burst length;

**sediment**

- time-averaged sand concentration profiles (in kg/m³);
- cross-shore sand transport (in kg/s/m²): net, mean and oscillating components ($q_{s,net}$, $q_{s,c}$, $q_{s,w,hf}$, $q_{s,w,lf}$, $q_{s,w,1}$) at various levels and depth-integrated values;

$$
q_{s,net} = <VC>
$$
$$
q_{s,c} = vc
$$
\[
q_{w,\text{hf}} = <(V_{\text{hf}} - v)(C_{\text{hf}} - c)>
\]
\[
q_{w,\text{lf}} = <(V_{\text{lf}} - v)(C_{\text{lf}} - c)>
\]

in which: \(q_{w,c}\) = time-averaged current-related suspended sediment transport rate and \(q_{w,w}\) = time-averaged wave-related suspended sediment transport rate (oscillating component), \(v\) = time-averaged cross-shore velocity components, \(V\) = instantaneous cross-shore velocity components, \(C\) = instantaneous concentration and \(c\) = time-averaged concentration; < > averaging over time; hf=high frequency<10s; lf=low frequency>10 s.

- composition (\(d_{10}, d_{50}, d_{90}\) in m) and fall velocity (\(w_{s,50}\) in m/s) of suspended sand (bulk sample);
- composition (\(d_{10}, d_{16}, d_{35}, d_{50}, d_{65}, d_{84}, d_{90}\)) and fall velocity (\(w_{s,50}\)) of sand fraction of bed material samples and percentage of silt (<0.063 mm);
- bed form type and dimensions (height and length in m);

**bed levels along sand bed**

- 5 longitudinal profiles

### 6.4.4 Expected results/products

The products are:

- Validation data sets of bed level evolution of trench and bar;
- data sets of depth-integrated transport rates;
- improved model of oscillatory suspended sand transport to be implemented in SURENCH- and UNIBEST-models;
- validated SURENCH-model for simulation of cross-shore migration of sand mining pits.

### 6.5 Conclusions and recommendations

It is proposed to study the morphological behaviour of a trench under conditions of non- or weakly breaking storm waves combined with a weak current. The hydrodynamic conditions should be such that filling and onshore migration of a trench will take place, simulating the behaviour of a sand mining pit in water with depths between 10 and 20 m migrating to the shore. It is also proposed to study the morphological behaviour of a sand bar under shoaling wave conditions, creating more extreme conditions (better measurable) with respect to the oscillatory suspended sand transport. The results will be used to improve the existing mathematical models of cross-shore sand transport and morphology.

Using a water depth of about \(h = 3\) m, a wave height between \(H_s = 1\) and 1.5 m (period \(T_p = 5\) s), sand of 0.15 mm, it is possible to simulate storm conditions (non-breaking waves) with sheet flow transport (no ripples) and the right distribution of bed load and suspended load transport in the Delta-flume. It is proposed to use a cross-shore mean current velocity of about 0.05 to 0.1 m/s in the flume to represent the peak tidal current (discharge Q of about 1
m$^3$/s), yielding a Froude number of the right order of magnitude (about 0.01). Special arrangements are necessary to generate a current in the Delta-flume.

Simulating a sand mining pit with a depth of 3 to 5 m (in nature), the trench depth in the Delta-flume (water depth of 3 m) will be about 1 m. A practical length will be about 30 to 50 m with side slopes of 1 to 10. The sand bar is proposed to have similar dimensions: crest height of 1 m, length of 30 to 50 m and side slopes of 1 to 10.
7 Summary, conclusions and recommendations

7.1 Summary and conclusions

The National Institute for Coastal Management of Rijkswaterstaat commissioned WL | DELFT HYDRAULICS to evaluate the knowledge of sand transport processes in deeper water of the North Sea, to evaluate the mathematical models available for simulation of these processes and associated morphology and to make a proposal for improvement of the knowledge of sand transport processes through large-scale experiments and implementation of this knowledge in models. These latter models will be used to give advice on the morphological effects of large-scale mining of sand in the coastal waters of the North Sea.

The central theme in the present study is the knowledge and modelling of sand transport processes on the lower shoreface with the aim of predicting the morphological behaviour of large-scale sand mining pits. The available knowledge of sand transport processes on the lower shoreface is summarised in Chapter 2. The process-models available for simulation of sand transport processes and associated bed level changes along dredged pits and trenches are: SUTRENCHED model, UNIBEST-TC 2.0 model and DELFT-2D/3D model.

At present stage of research the SUTRENCHED model is most suited for cross-currents over relatively long trenches, because of the two-dimensional vertical approach, modelling the turbulence- and wave-induced mixing and the gravity-induced settling of the sand particles. The DELFT-2D/3D model can also model the transport processes over the depth, based on a depth-integrated approach. The UNIBEST-TC 2.0 model is based on a local equilibrium approach. Given the superiority of the SUTRENCHED model for small-scale sedimentation and erosion processes in wave-current conditions, the attention has been focused on this model in the present study.

Data that has become available since previous studies, has been analysed to further validate the model for wave-dominated conditions. Sensitivity computations have been carried out to identify the most relevant process and input parameters. The model has also been used to explore the effect of trench geometry and dimensions on the long-term trench behaviour. This information can be used to determine practical trench/pit dimensions and to design future field experiments of trial pits/trenches.

At present stage of research the cost-effectiveness of field experiments related to sand mining pits in relatively deep water is not optimum, because self-registrating in-situ process instruments are not yet available and results will not readily be available, as monitoring of morphological evolution over several years is involved. To explore the possibility of optical sand concentration instruments for field measurements in the near-bed region, a commercially available OBS-transport meter has been tested in the wave tunnel in the present study. To overcome the problems of field and laboratory experiments, a proposal has been made for large-scale experiments in the Delta-flume, based on an evaluation of available process knowledge.
Sand transport processes on the lower shoreface

The shoreface is herein defined, as follows:
- upper shoreface landward of -8 depth contour;
- middle shoreface between -8 and -20 m depth contours;
- lower shoreface seaward of -20 m contour.

The fluid in the shoreface zone may be homogeneous (well-mixed) or stratified with a surface layer consisting of relatively low fluid density (fresh warmer water) and a bottom layer of relatively high density (saline colder water). Strong horizontal density-related pressure gradients may occur in regions close to the river mouth of the Rhine-Meuse Estuary. In meso-tidal environments like the North Sea both tide- and wind-induced currents are important.

The semidiurnal tide along the Holland coast of the North Sea propagates northwards and the tidal range roughly varies between 1 and 2 m (meso-tidal). The horizontal tide becomes more asymmetric in northern directions; the peak flood and peak ebb depth-averaged velocity are about 0.6 and 0.5 m/s in depth of 20 m near Hoek van Holland and about 0.75 and 0.45 m/s near den Helder. The flood duration is about 5 hours and the ebb duration is about 7 hours along the Holland coast (Van Rijn, 1995); these values are reasonably constant along the coast.

The wave climate is rather constant along the Holland coast; the dominant wave direction is south-west. Some values of the probability of occurrence (duration in % of time) for waves in deep water are:
- south-west (180°-270°): 15% waves of 1-2 m, 4-5% between 2-3 m, 1-2% between 3-5 m;
- north-west (270°-360°): 10% between 1-2 m, 4-5% between 2-3 m, 1-2% between 3-5 m.

The sediment size of the bed material (Van Rijn, 1995) on the lower shoreface (20 m depth) varies between 0.15 mm (near den Helder) and 0.25 mm (near Hoek van Holland). For the upper shoreface (depth of 8 to 10 m) these values vary between 0.15 mm (Noordwijk) and 0.2 mm (Egmond).

Sand can be transported by wind-, wave-, tide- and density-driven currents (current-related transport), or by the oscillatory water motion itself (wave-related transport). The waves generally act as a sediment stirring agent, whereas the sediments are transported by the mean current. Wave-related transport may be caused by the deformation of short waves under the influence of decreasing water depth (wave asymmetry). Low-frequency waves interacting with short waves may also contribute to the sediment transport process (wave-related transport), especially in shallow water in the surf zone.

In friction-dominated deeper water on the lower shoreface zone the transport process generally is concentrated in a layer close to the sea bed and mainly takes place as bed-load transport in close interaction with small bed forms (ripples). Bed-load transport is dominant in areas where the mean currents are relatively weak compared to the wave motion (small ratio of depth-averaged velocity and peak orbital velocity). Net sediment transport by the oscillatory motion is relatively small in depths larger than 15 m (Van Rijn, 1995, 1997), because the wave motion tends to be more symmetrical in deeper water.

Suspension of sediments on the lower shoreface can be generated by ripple-related vortices. Suspended load transport will become increasingly important with increasing strength of the tide- and wind-driven mean currents due to the turbulence-related mixing capacity of the mean
current (shearing in boundary layer) in combination with wave-induced mixing. By this mechanisms the sediments will be mixed up from the bed-load layer to the upper layers of the water column.

Estimates (based on model computations) of net annual longshore and cross-shore sand transport rates (at a depth of 20 m) in various stations (sand size of $d_{50} = 0.25$ mm) along the coast of Holland have been presented by Van Rijn (1995, 1997). The net annual longshore transport rates at a depth of about 20 m vary in the range between 25 and 75 m$^3$/m/yr, depending on location along the coast (Den Helder to Hoek van Holland). The net annual cross-shore transport rates at a depth of 20 m are onshore-directed and vary in the range between 0 and 15 m$^3$/m/yr, depending on location along the coast. Net annual longshore transport rates derived from sand dumpings near Hoek van Holland (Rijkswaterstaat, 1996) and near Wijk aan Zee (Rijkswaterstaat, 1992) are in the range of 30 to 100 m$^3$/m/yr for depths between 10 and 20 m. The results obtained from model computations and from field morphological data are of the same order of magnitude, which is a rather encouraging result.

**Description and comparison of models**

The SUTRENCH, UNIBEST-TC 2.0 and DELFT-2D/3D models are all process-based models, simulating bed-load and suspended load transport and associated bed level changes.

The SUTRENCH-2DV quasi-steady state model is based on detailed modelling of the sand transport processes, which are represented by expressions modelling diffusive, advective and settling processes (lag effects). The bed-load transport takes all transport components into account. The wave-related (oscillatory) suspended load transport is not yet modelled. The hydrodynamics are not modelled, but have to be given as input data (wave height along domain, discharge, width of streamtube). The model is generally applicable to erosion and sedimentation problems, provided that the hydrodynamic data in relevant streamtubes are known (input data).

The UNIBEST-TC 2.0 model is a 2DV model, which gives a detailed description of the hydrodynamic processes (waves and currents) in cross-shore direction, assuming invariant conditions alongshore. The bed-load and suspended load transport processes are represented by assuming local equilibrium conditions (no lag effects). The bed-load transport takes all transport into account. The wave-related (oscillatory) suspended load transport is not yet modelled. The model is only applicable for simulation of cross-shore profile evolution.

The DELFT-2D/3D non-steady model is a universal field model, simulating waves and currents on a 2D or 3D grid; the bed-load and suspended load transport can be simulated by using a local equilibrium or non-equilibrium (lag effects) approach. The wave-related (oscillatory) bed-load and suspended load transport components are not yet modelled.

At present stage of research the SUTRENCH-model is most suited for cross-currents over relatively long trenches, because of the two-dimensional vertical approach, modelling the turbulence- and wave-induced mixing and the gravity-induced settling of the sand particles. Therefore, this model has been used for further analysis.

**Simulation results of SUTRENCH (Laboratory and field experiments)**
From the two investigated cases it can be concluded that SUTRENCH gives fairly reliable results. The results for the laboratory basin experiment show good agreement with the measurements. The migration of the trench is modelled accurately, the morphological development of the trench slopes is not modelled satisfactory. The upstream slope is generally predicted too steep whereas the erosion of the downstream slope is underpredicted. From the basin experiment it is found that the trench slope development is influenced by varying the roughness heights. Varying the sediment fall velocity mainly influences the sedimentation (and migration) of the trench. An optimal fit to the upstream boundary velocity and concentration profiles resulted in a good representation of the measured trench development. Hindcast of the Danish field experiment at CH1700 was hampered by the fact that no current data was available for the simulated period. The migration and sedimentation of the trench over a period of 3 weeks showed reasonable agreement with the observed morphological developments by using reasonable estimates for the tide-and wind-induced currents.

**Sensitivity computation results of SUTRENCH**

For the sensitivity analyses a base run had to be defined for the Euro-Maas channel. Evaluation of two recent studies (Walstra et al., 1997 and Hoitink, 1997) showed that there is an apparent inconsistency between the observed transports, which lie in the range of 20 to 50 m$^3$/m/yr and the measured morphological development of the Euro-Maas channel. To reproduce the morphological development of the trench unrealistic high transports (in the order of 130 m$^3$/m/yr) have to be imposed in the SUTRENCH-model. The base run was calibrated on the occurring transports rather than on the morphological behaviour of the trench.

From the sensitivity analysis of the process and model input parameters it was found that an accurate tide and wave schematisation is imperative in yielding reliable results. Furthermore the model was found to be very sensitive to variations of the sediment characteristics, validation of the transport relations for fine sediment is necessary. The sensitivity of SUTRENCH to variations in the wave-related mixing parameters illustrates the need to validate the model for deep water conditions as the referred parameters have only been validated for shallow water conditions.

From the analysis of the effects of the hydrodynamic input parameters it could be concluded that the schematisation of the horizontal tide (current velocities) has a significant effect. Small variations of the current velocity have a relatively large effect, because the transport rates in deep water are relatively small (close to the initiation of motion).

**Morphological development of sand pit using SUTRENCH**

*Simulation results*

From the long term simulations it can be concluded that the relative influence of the investigated geometry's is limited. Varying the water depth has the largest influence on the considered output parameters.

The maximum initial total trapping efficiency is 77% for the simulation with a reduced water depth. The minimum trapping efficiency is 36% for the shallow trench with a depth of 2 m. The maximum migration of the trench is 800 m for the decreased water depth case. The minimum migration is 260 m for the increased water depth case. The relative volume decrease of the trench after 50 years was the highest for the reduced water depth case,
Sand transport at the middle and lower shoreface of the Dutch coast

minimum relative sedimentation occurred for the trench with a width of 2400 m. It has to be noted that for the shallow trenches (trench depths of 2 and 3.5 m), the applied trench definitions resulted in unrealistic trench boundaries. This is mainly due to the fact that these trenches have experienced considerable sedimentation in the simulated period which makes an objective definition of the trench boundaries almost impossible.

The suggested definition of the morphological time scales by extrapolating the dredging efficiencies until it is zero seems to be a give reliable results. From the time scales it can be concluded that wide trenches are preferable over deep trenches if the morphological time scale is an important criteria. A deep trench of 14 m (twice as deep as base run) has a time scale of 800 years whereas a wide trench of 2400 m (four times as wide as base run) only has a time scale of about 500 to 600 years. It has to be noted however that these are first order estimates of the morphological time scales. For a more reliable prediction simulations in the order of 500 years have to be made. This lies however outside the scope of this study.

The morphological development of all the investigated geometry's is mainly influenced by the lag of the settling and picking up of the suspended sediment. If most of the sediment has settled on the trench bottom before the toe of the upstream slope, both slopes will develop almost independently from each other. This so-called morphological interaction between the trench slopes mainly occurs for the wide and deep trenches. In case of reduced width or depth an increasing interaction between both slopes can be seen which results in an increasing sedimentation at the toe of the upstream slope. This is clearly illustrated when the five simulations in which the depth was modified are compared. An increased sedimentation at the toe of the upstream slope also results in considerable flattening of that slope.

It can be concluded that wide trenches have a relative short morphological time scale whereas deep trenches have relative long morphological time scales but also a relative small migration rate. It is thought that a minimum migration rate is one of the most important parameters as it determines to a large extent the area in which the effects of sand mining can be distinguished. Especially, to limit the effects on the coast a low migration rate in cross shore direction is important.

Cross-shore versus longshore

All the presented results have been derived for the long shore directions. In the cross shore directions tidal currents are of a smaller order. The transport capacity of waves however increases if the water depth decreases. This has been illustrated by performing two simulations in which the water depth was increased (water depth is 25 m) and decreased with 5 m (water depth is 15 m) respectively. All hydrodynamic parameters have been kept similar to the those of the base run. It could be seen that the results after 50 years of simulation are seriously affected by these variations. A decrease of 25 % in water depth results in an increase of the migration rate of almost a factor 2. The morphological time scale (first order estimate of when the dredging trapping efficiency reaches zero) is also halved. It illustrates that the dynamic behaviour of a trench or a mined area changes dramatically if it is located or migrates to shallower waters. The results that have been presented in this study cannot directly be translated to the cross-shore direction. Additional research is needed to give a reliable indication of the dynamic development in cross-shore directions. The results of this study can be seen as an upper limit for the cross-shore case.

In Van Rijn et al. (1995) both cross shore and longshore residual transport rates have been predicted. The ratio between long shore and cross shore transports along the Dutch coast
vary between 5 and 20. As a first estimate this ratio could be applied to the presented results.

**Applicability of results**

The results of the various simulations may be used to get a first impression of the development of trenches or mined areas at different locations then at the Euro-Maas channel.

It is advised to determine the residual transports at the location of interest (see e.g. Van Rijn et al. 1995). If it is assumed that the wave conditions are more or less constant, the tidal current (duration, peak velocities, etc.) and sediment characteristics are the most important parameters. The resulting residual transports can then be used to derive an estimate for e.g. the migration rate of a trench from Tables 3.3.7a and 3.3.7b. Subsequently, Table 4.2.1 can be used to get an impression of the “geometrical” influence.

It is noted that extrapolation of the presented results can only be performed for locations where no significant 3-Dimensional currents are present (e.g. ebb tidal delta’s of the Wadden Sea and the southern delta coast). This limits the applicability of the model results to areas along the closed Dutch coast from Hoek van Holland to Den Helder.

In Van Rijn et al. (1995) the residual transports at 20 m water depth near Callantsoog are reported to be in the order of 75 m$^3$/m/year. For a trench with approximately the same geometry as the Euro-Maas channel this would result in a migration of the trench in 50 years in the range of 500 to 700 m (Table 4.2.1).

**Calibration of OBS transport meter for near-bed layer**

Proper validation of mathematical models (like SUTRENCH and UNIBEST) for shoreface conditions requires field data of sedimentation and erosion in trenches/pits. Furthermore, the incoming sand transport should be known to serve as input data.

Measurements of sand transport rates on the shoreface in rough weather conditions are, however, problematic because self-registrating instruments operated from stand-alone tripods are required. Generally accepted instruments for measuring sand transport near the bed in oscillatory flow (short waves) plus tide/wind-driven currents in field conditions are, however, not yet available.

For field conditions it is most attractive to measure the sand concentrations in the near-bed layer by an optical probe (OBS). The field instrument should be equipped with a velocity meter to measure the velocities close to the bed (say 0.02 to 0.03 m above bed). The operational range of the OBS is between 1 and 100 kg/m$^3$, covering the concentration range in the near-bed layer. This OBS-transport meter will be tested in the present study.

The objective of the present tests is to get a rough calibration of the OBS-transport meter in the large oscillating water tunnel of Delft Hydraulics, which has an operational range comparable to field conditions. The net transport rates in the wave tunnel are known (derived from measured bed level changes; Ribberink and Al-Salem, 1993), so that the transport rates measured with the instrument can be compared to the transport rates in the wave tunnel.
The conclusions of the experimental study are:

**Velocities**
The general trend of the EMF signal follows the Laser Doppler signal fairly well except for high-frequency peaks in the EMF signal; the latter being caused by the movable instrument arrangement. Results from the present study are consistent with findings from previous studies in the wave tunnel by Ribberink and Al-Salem (1993) and Janssen et al. (1996).

**Concentrations**
Time-averaged concentration can be rather accurately determined using the OBS transport meter. On average, the OBS sensors gave values that were 15% larger in case of coarse sand and 30% larger in case of fine sand than the values determined with the pump sampler.

**Vertical sorting**
In case of relatively coarse bed material ($D_{50} = 0.19$ mm) the median grain size of the suspended sediment was 20% to 30% smaller than the median grain size of the bed material. In case of relatively fine bed material ($D_{50} = 0.13$ mm) the vertical sorting was less pronounced. The median grain size of the suspended sediment was 10% smaller than the median grain size of the bed material.

**Transport rate**
The near-bed transport rate ($z < 0.025$ m) was relatively large and accounted for more than 90% of the total net transport rate. The near-bed transport rate can be rather accurately predicted (standard error less than 20%) using a simplified version of the transport formulation by Van Rijn et al. (1995).

**Proposal for large-scale experiments in Delta flume**
The results of the SUTRENCH-sensitivity computations and the long-term trench simulations show several model deficiencies (wave-related mixing, fine sediment effects, velocity profiles in acceleration zone of trench). Furthermore, there is insufficient knowledge of cross-shore transport processes, so that net transport rates at the model boundary can not be supplied.

At present stage of research the SUTRENCH model can not be used with sufficient accuracy to evaluate the migrational effect of sand mining trenches/pits on pipelines, cables and on the behaviour of the shoreface/shoreline.

It is recommended to focus new research efforts on the improvement of cross-shore sand transport processes and morphology.

The best strategy to extend the knowledge of cross-shore sand transport processes and morphology in deeper water would be to perform a detailed field study of these processes in and near a test trench/pit (process measurements and monitoring of morphological behaviour) in combination with model simulations. Accurate sand transport measurements in deeper water under storm wave conditions are, however, not yet feasible, because self-registrating in-situ instruments are not available.

A practical disadvantage of a trial pit/trench in nature is that the results will not readily be available, as the monitoring of the morphological behaviour of the pit/trench will take some years. The cost involved will also be relatively high, as the dredging of test pits/trenches is necessary.
Based on this, it may be more cost-effective to perform large-scale experiments in the Delta-flume of WL | DELFT HYDRAULICS with the objective of improving the process knowledge of cross-shore sand transport and morphology.

It is proposed to study the morphological behaviour of a trench and a bar under conditions of non- or weakly breaking storm waves combined with a weak current. The hydrodynamic conditions should be such that filling and onshore migration of a trench will take place, simulating the behaviour of a sand mining pit in water with depths between 10 and 20 m migrating to the shore. It is also proposed to study the morphological behaviour of a sand bar under shoaling wave conditions, creating more extreme conditions (better measurable) with respect to the oscillatory suspended sand transport. The results can be used to improve the existing mathematical (process-oriented and behaviour-oriented) models of cross-shore sand transport and morphology.

Using a water depth of about \( h = 3 \) m, a wave height between \( H_s = 1 \) and 1.5 m (period \( T_p = 5 \) s), sand of 0.15 mm, it is possible to simulate storm conditions (non-breaking waves) with sheet flow transport (no ripples) and the right distribution of bed load and suspended load transport in the Delta-flume. It is proposed to use a cross-shore mean current velocity of about 0.05 to 0.1 m/s in the flume to represent the peak tidal current (discharge \( Q \) of about 1 m\(^3\)/s), yielding a Froude number of the right order of magnitude (about 0.01). Special arrangements are necessary to generate a current in the Delta-flume.

Simulating a sand mining pit with a depth of 3 to 5 m (in nature), the trench depth in the Delta-flume (water depth of 3 m) will be about 1 m. A practical length will be about 30 to 50 m with side slopes of 1 to 10. The sand bar is proposed to have similar dimensions: crest height of 1 m, length of 30 to 50 m and side slopes of 1 to 10.

### 7.2 Recommendations

New research is required to improve the modelling of cross-shore sand transport processes and associated morphology, so that the onshore migration of sand mining pits and their effects on the shoreline can be better predicted. It is proposed to study the morphological behaviour of a trench and a bar under conditions of non- or weakly breaking storm waves combined with a weak current in the Delta-flume of WL | DELFT HYDRAULICS.

The results of large-scale experiments in the Delta-flume will be:

- data of sand concentration profiles to derive the wave-related mixing parameters under non-breaking waves in relatively deep water outside the surf zone; based on this the validity range of the process models can be extended to deeper water (the present parameters are only valid for shallow water in the surf zone);
- data of velocity profiles along the trench/bar profile to improve the mathematical model in the deceleration and acceleration zones;
- data of bed and suspended sand composition to study the effect of winnowing processes on the computed concentration profiles, sand transport rates and bed evolution and to evaluate whether the sand transport rates should be computed by using the single fraction method or the multi-fraction method;
- data of near-bed orbital velocities and mean current velocities and associated oscillatory and mean transport components to determine their relative importance to the net transport rates; these parameters can be measured at close spacing along the flume so that the cross-shore gradients can be determined accurately; based on this the measurement locations during future field measurements can be better determined;
• data of cross-shore trench morphology and associated net transport rates to improve process models and behaviour-related models, so that the cross-shore behaviour of sand mining trenches and pits in the nearshore zone (especially between 10 and 20 m depth contours) and their influence on the shoreline can be described;

• data of cross-shore bar morphology and associated net transport rates to improve process models and behaviour-related models, so that the cross-shore behaviour of natural and artificial bars (nearshore dumpings) in the outer surf zone and their influence on the shoreline can be described; similarly the behaviour of ebb delta bars in front of inlets can be better described, if cross-shore transport processes are better known;

• indirect evaluation of the relative importance of the bed load transport (verification of wave tunnel results), using infill rates of trench and measured suspended transport upstream of trench (diagnostic modelling using data and models);

• the net transport rates derived from the measured data and process models can be directly used to feed the available behaviour-related models, which fully depend on net transport rates formulated in terms of wave-current conditions.

The results should be used to upgrade the existing process-based SUTRENCH and UNIBEST models, so that tools become available that can be applied to simulate the cross-shore behaviour of sand mining trenches/pits and shoreface nourishments.

The upgraded models should be used to determine the net annual longshore and cross-shore transport rates (and variation ranges) at various depth contours in a number of profiles along the Dutch coast. Furthermore, the upgraded models should be used to determine the cross-shore behaviour of trenches (sand mining) and shoreface nourishments (artificial bars) and their effect on the coast.
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A Formulations of Sutrench-model

BASIC EQUATIONS

1. Sediment continuity equation

The sand concentrations are described by the mass continuity equation for a control volume assuming steady conditions:

\[ \partial \frac{buc}{\partial x} + \partial \frac{[b(w-w_s)c]}{\partial z} - \partial \frac{b\varepsilon_{s,z} \partial c}{\partial z}/\partial z = 0 \]

with: \( c = \) sand concentration, \( u = \) horizontal fluid velocity, \( w = \) vertical fluid velocity, \( w_s = \) particle fall velocity, \( \varepsilon_{s,z} = \) sediment mixing coefficient, \( b = \) width of streamtube, \( z = \) height above bed.

Sand transport in tidal conditions is represented, assuming a series of quasi-steady flow blocks over the tidal cycle.

2. horizontal fluid velocity, \( u \)

The horizontal fluid velocity is represented by a logarithmic velocity profile (see section 11 of Appendix D), taking the effects of wave-current interaction into account. The application of logarithmic profiles is only valid for gently varying non-uniform flow conditions.

3. vertical fluid velocity, \( w \)

The vertical fluid velocity is derived from the continuity equation for the fluid:

\[ (1/b) \partial \frac{bu}{\partial x} + \partial w/\partial z = 0 \]

4. sediment mixing coefficient

The sediment mixing coefficient due to currents and waves is described in section 12 of appendix D.

5. bed level changes

The bed level changes are derived from:

\[ \partial z_b/\partial t + (1-p)\rho_s \partial Q_t/\partial x = 0 \]

with: \( z_b = \) bed level above datum, \( Q_t = \) total depth- and width-integrated sand transport \((-Q_s+Q_b)\), \( Q_s = \) suspended sand transport, \( Q_b = \) bed load transport, \( \rho_s = \) sediment density, \( p = \) porosity factor.

\[ Q_s = b \int (uc)dz \]
Q_b = bed-load transport formula, see section 15 of Appendix D

6. boundary conditions

flow domain: bed level, water depth, tide level variation, streamtube width, bed-roughness,
inlet boundary: discharge, sand concentration profile (measured or computed equilibrium profile),
outlet boundary: none (horizontal diffusion is neglected),
water surface: net vertical sand transport is zero,
bed boundary: vertical fluid velocity w is zero,
sand concentration or gradient is specified as a function of bed-shear stress and sediment parameters (see section 13 of Appendix D).
B Formulations of bed-load and suspended load transport model for equilibrium conditions
1. Input

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

\[ \bar{v}_R = \text{depth-averaged velocity vector in main current direction, see Fig. 1} \] (m/s)

\[ \bar{u}_t = \text{time-averaged and depth-averaged return velocity below wave trough} \] (m/s)

\[ \bar{u}_b = \text{time-averaged near-bed velocity due to waves, wind or density-gradient} \] (m/s)

\[ H_s = \text{significant wave height} \] (m)

\[ T_p = \text{(absolute) wave period of peak of spectrum} \] (s)

\[ \phi = \text{angle between wave and main current direction (0-360°)} \] (-)

\[ d_{so} = \text{median diameter of bed material} \] (m)

\[ d_{so} = \text{90\% diameter of bed material} \] (m)

\[ d_s = \text{representative diameter of suspended material} \] (m)

\[ k_{xe} = \text{current-related bed roughness height (minimum } k_{xe} = 0.01 \text{ m)} \] (m)

\[ k_{sw} = \text{wave-related bed roughness height (minimum } k_{sw} = 0.01 \text{ m)} \] (m)

\[ T_{fe} = \text{fluid temperature} \] (°C)

\[ SA = \text{fluid salinity} \] (°e)

Figure 1 Schematic presentation of current and wave direction

Remarks:

A. The representative particle size (\(d_s\)) of the suspended sediment will be in the range of:

\[ d_s = (0.6 \text{ to } 1) \ d_{50,\text{bed}}, \text{ see Section 8.4.3.} \]

A reasonable estimate is \(d_s = 0.8 \ d_{50,\text{bed}}\).
B. The wave-related bed roughness height in the ripple regime will be in the range \(k_w = (1 \text{ to } 3) \Delta_s\) with values from 0.01 to 0.1 m.
The wave-related bed roughness height in the sheet flow regime will be: \(k_w = 0.01 \text{ m}\).The current-related bed roughness height will be in the range \(k_{\kappa} = 0.01 \text{ to } 1 \text{ m}\).

C. The constant of Von Karman is assumed to be \(\kappa = 0.4\). The sediment density is \(\rho_s = 2650 \text{ kg/m}^3\).

2. Compute general parameters

Chlorinity : \(\text{CL} = (\text{SA} - 0.03)/1.805\)
Fluid density : \(\rho = 1000 + 1.455 \text{ CL} - 0.0065 (\text{Te} - 4 + 0.4 \text{ CL})^2\)
Kinematic viscosity : \(\nu = (4/(20 + \text{Te})) \times 10^5\)
Fall velocity : see Equations (3.2.21), (3.2.22) and (3.2.23)

3. Compute sediment characteristics

Relative density : \(s = \rho_s/\rho\)
Particle parameter : \(D_p = d_50 \times (s-1)g/(\nu^2)^{1/3}\)
Shields parameter
\[
\begin{align*}
1 < D_s & \leq 4 : \theta_s = 0.24 D_s^{0.64} \\
4 < D_s & \leq 10 : \theta_s = 0.14 D_s^{0.1} \\
10 < D_s & \leq 20 : \theta_s = 0.04 D_s^{0.29} \\
20 < D_s & \leq 150 : \theta_s = 0.013 D_s^{0.29} \\
D_s > 150 : \theta_s = 0.055
\end{align*}
\]
Critical bed-shear stress : \(\tau_{\text{crit}} = (\rho_s - \rho) g d_50 \theta_s\)
Critical depth-averaged velocity : \(\bar{u}_{\text{crit}} = 5.75/(s-1) g d_50^{0.5} (\theta_{\text{crit}})^{0.5} \log(4h/d_{90})\)
Critical peak orbital velocity (Komar)
\[
\begin{align*}
d_{50} < 0.0005 \text{ m} : \hat{U}_{\text{crit}} &= [0.12(s-1)g(d_{50})^{0.5}(T_p)^{0.5}\pi]^{1/2} \\
d_{50} \geq 0.0005 \text{ m} : \hat{U}_{\text{crit}} &= [1.09(s-1)g(d_{50})^{0.75}(T_p)^{0.25}\pi^{0.57}]^{1/2}
\end{align*}
\]

4. Compute wave length

Wave length modified by currents : \[
\left[\frac{L'}{T_p} - \frac{\vec{v}_s \cos \phi}{2\pi}\right]^2 = \left[\frac{gL'}{2\pi} \tanh \left(\frac{2\pi h}{L'}\right)\right]^{1/2}
\]

5. Compute relative wave period

The relative wave period is : \(T'_p = \frac{T_p}{1 - (\vec{v}_s/T_p \cos \phi)/L'}\)

6. Compute wave parameters

Near-bed peak orbital excursion : \(\hat{A}_a = \frac{H_a}{2 \sinh(2\pi h/L')}\)
Near-bed peak orbital velocity : \(\hat{U}_a = \frac{\pi H_a}{T_p' \sinh(2\pi h/L')}\)
Wave-boundary layer thickness : \(\delta_w = 0.072 \hat{A}_a (\hat{A}_a/k_w)^{0.25}\)
Near-bed peak orbital velocity in forward direction

\[ h \geq 0.01 \, g \left( T_p^2 \right) : \quad \hat{U}_{b,f} = \hat{U}_b + \frac{3 \pi^2 (H_b)^2}{4 \left( T_p \right) \left( L' \right) \left( \sinh(2\pi h/L') \right)^4} \]

\[ h < 0.01 \, g \left( T_p^2 \right) : \quad \hat{U}_{b,f} = \alpha \hat{U}_b \]

\[ \alpha = 1 + 0.3 \left( H_b/h \right) \]

Near-bed orbital velocity in backward direction

\[ h \geq 0.01 \, g \left( T_p^2 \right) : \quad \hat{U}_{b,b} = \hat{U}_b - \frac{3 \pi^2 (H_b)^2}{4 \left( T_p \right) \left( L' \right) \left( \sinh(2\pi h/L') \right)^4} \]

\[ h < 0.01 \, g \left( T_p^2 \right) : \quad \hat{U}_{b,b} = (2-\alpha) \hat{U}_b \]

Return velocity mass transport

\[ \bar{u}_r = -\frac{0.125 \, g^{0.5} (H_b)^2}{h^{0.5} h_i} \]

\[ h_i = (0.95 - 0.35 \left( H_b/h \right)) \, h \]

Near-bed wave-induced velocity

\[ u_b = \left( 0.05 - (\alpha - 0.5) \right) \hat{U}_b \]

\[ \alpha_s = \frac{\hat{U}_{b,f}}{\left( \hat{U}_{b,f} + \hat{U}_{b,b} \right)} \]

7. Compute apparent bed roughness

\[ k_s = k_{sc} \exp \left[ \gamma \hat{U}_b / \left( \bar{v}_r \right)^2 + (\bar{v}_r)^{0.5} \right], \quad k_{s,\text{max}} = 10 \, k_{sc} \]

\[ \gamma = 0.8 + \beta - 0.3 \, \beta^2 \]

\[ \beta = \left( \frac{\phi}{360^\circ} \right) 2\pi \]

8. Compute friction factors

Current :  \[ C' = 18 \, \log(12h/3d_{90}) \]

\[ C = 18 \, \log(12h/k_{sc}) \]

\[ f'_c = 0.24 \, \log^{-2}(12h/3d_{90}) \]

\[ f_c = 0.24 \, \log^{-2}(12h/k_{sc}) \]

\[ f_s = 0.24 \, \log^{-2}(12h/k_s) \]

Waves :  \[ f'_w = \exp \left[ -6 + 5.2 \left( \hat{A}_w/3d_{90} \right)^{-0.19} \right] \]

\[ f_w = \exp \left[ -6 + 5.2 \left( \hat{A}_w/k_{sw} \right)^{-0.19} \right] \]

\[ f_{w,\text{max}} = 0.3 \]
9. Compute effective time-averaged bed-shear stresses

Efficiency factor current : \( \mu_c = \frac{f_c}{f} \)

Efficiency factor waves : \( \mu_w = \frac{f_w}{f} \)

\( \mu_w = 0.6/D \)

Wave-current interaction coefficient : \( \alpha_{cw} = \frac{[\ln(90\delta_w/k_s)]^2}{[\ln(90\delta_w/k_{sc})]^2} \left[ -1 + \ln(30h/k_{sc}) \right] \)

\( \alpha_{cw,max} = 1 \)

Bed-shear stress current : \( \tau_c = \frac{1}{8} \rho f_c \left( \vec{v}_c \right)^2 + \left( \vec{u}_c \right)^2 \)

Bed-shear stress waves : \( \tau_w = \frac{1}{4} \rho f_w (\vec{u}_w)^2 \)

Bed-shear stress current-waves : \( \tau_{cw} = \tau_c + \tau_w \)

Effective bed-shear velocity current : \( u_{*c} = \left[ (\alpha_{cw} \mu_c \tau_c) / \rho \right]^{0.5} \)

10. Compute bed-shear stress parameters

Dimensionless bed-shear stress for bed load transport : \( T = \frac{(\alpha_{cw} \mu_c \tau_c + \mu_w \tau_w) - \tau_{\alpha}}{\tau_{\alpha}} \)

Dimensionless bed-shear stress for reference concentration at \( z = a \) : \( T_s = \frac{(\alpha_{cw} \mu_c \tau_c + \mu_{w,a} \tau_w) - \tau_{\alpha}}{\tau_{\alpha}} \)

\( T = 0 \) if \( T < 0 \)

11. Compute velocity distribution over the depth

Outside wave-boundary layer, \( z \geq 3\delta_w \) : \( v_{R,Z} = \frac{\bar{v}_R \ln(30z/k_s)}{-1 + \ln(30h/k_s)} \)

Inside wave-boundary layer, \( z < 3\delta_w \) : \( v_{R,Z} = \frac{v_\delta \ln(30z/k_s)}{\ln(90\delta_w/k_{sc})} \)

\( v_\delta = \frac{\bar{v}_R \ln(90\delta_w/k_s)}{-1 + \ln(30h/k_s)} \)
12. Compute sediment mixing coefficient distribution over the depth

**Current,**

\[ z < 0.5 \ h \quad : \quad \varepsilon_{s,c} = \kappa \beta u_{s,c} z(1-z/h) \]
\[ z \geq 0.5 \ h \quad : \quad \varepsilon_{s,c} = 0.25 \kappa \beta u_{s,c} h \]
\[ u_{s,c} = \left( g^{0.5}/C \right) \left[ (\bar{v}_R)^2 + (\bar{u}_s)^2 \right]^{10.5} \]
\[ \beta = 1 + 2 \left( \frac{w_s}{u_{s,c}} \right)^2 \]
\[ \beta_{\text{max}} = 1.5 \]

**Waves,**

\[ z \leq \delta_s \quad : \quad \varepsilon_{s,w} = \varepsilon_{s,\text{bed}} = 0.004 D_s \delta_s \bar{U}_s \]
\[ z \geq 0.5 \ h \quad : \quad \varepsilon_{s,w} = \varepsilon_{s,\text{max}} = 0.035 h H_s/T_p \]
\[ \delta_s < z < 0.5 \ h \quad : \quad \varepsilon_{s,w} = \varepsilon_{s,\text{bed}} + \left[ \varepsilon_{s,\text{max}} - \varepsilon_{s,\text{bed}} \right] \left[ \frac{z - \delta_s}{0.5h - \delta_s} \right] \]
\[ \delta_s = 0.3 h (H_s/h)^{0.5} \]
\[ \delta_{s,\text{min}} = 0.05 \text{ m}, \quad \delta_{s,\text{max}} = 0.2 \text{ m} \]

Current and waves

\[ \varepsilon_{s,cw} = \left( \varepsilon_{s,c}^2 + \varepsilon_{s,w}^2 \right)^{0.5} \]

13. Compute concentration distribution over the depth by numerical integration

**Reference level**

\[ a = \text{maximum}(k_{s,c}, k_{s,w}) \]

**Concentration gradient**

\[ (z > a) \quad : \quad \frac{dc}{dz} = - \frac{(1-c)c w_s}{\varepsilon_{s,cw} \left[ 1 + (c/c_o)^{0.8} - 2(c/c_o)^{0.4} \right]} \]

**Bed concentration**

\[ (z \leq a) \quad : \quad c_s = 0.015 \frac{d_{so}}{a} \frac{T_s^{1.5}}{D_s^{0.3}} \]
\[ c_o = 0.65 = \text{maximum volume concentration} \]
\[ w_s = \text{fall velocity of suspended sediment} \]

14. Compute time-averaged suspended load transport rates

**Current direction**

\[ q_s = \rho_s \int_a^h v_R c \ dz \]

**Wave direction**

\[ q_s = \rho_s \int_a^h u_I c \ dz \]
15. Compute instantaneous and time-averaged bed-load transport

- x-axis along current velocity vector (see Fig. 2)
- y-axis normal to current velocity vector (see Fig. 2)

Current velocities at \( z = \delta \) above bed:
- \( \delta = \max(3\delta_w, k_{sc}) \)
- \( \nu_{R,\delta} = \frac{\bar{v}_R \ln(30\delta/k_s)}{-1 + \ln(30h/k_s)} \)
- \( u_{r,\delta} = \left( \frac{u_i}{\bar{v}_R} \right)^{\nu_{R,\delta}} \)

Orbital velocities (asymmetrical):
- \( U_{o,f} \) and \( U_{o,b} \)

Instantaneous velocity \( x \):
- \( \sum U_{o,x} = U_0 \cos \phi + \nu_{R,\delta} + (u_b + u_{r,b}) \cos \phi \)

Instantaneous velocity \( y \):
- \( \sum U_{o,y} = U_0 \sin \phi + (u_b + u_{r,b}) \sin \phi \)

Instantaneous velocity:
- \( U_{o,R} = \left[ (\sum U_{o,x})^2 + (\sum U_{o,y})^2 \right]^{0.5} \)

Instantaneous friction coefficient:
- \( \alpha = \frac{\nu_{R,\delta}}{\nu_{R,\delta} + \bar{U}_b} \)
- \( \beta = 0.25 \left[ \frac{-1 + \ln(30h/k_{sc})^2}{\ln(30\delta/k_{sc})} \right] \)
- \( f'_{cw} = \alpha \beta f'_{c} + (1 - \alpha) f'_w \)

Instantaneous bed-shear stress:
- \( \tau'_{b,cw} = 0.5 \rho f'_{cw} (U_{o,R})^2 \)

Instantaneous bed-load transport:
- \( \gamma = 1 - (H_s/h)^{0.5} \), \( \gamma_{min} = 0.3 \)
- \( q_b = 0.25 \gamma \rho_s d_{50} D_{s0}^{-0.5} \left[ \frac{U_{o,R}}{\rho} \right]^{0.5} \left[ \frac{\tau'_{b,cw} - \tau'_{b,cr}}{\tau'_{b,cr}} \right]^{1.5} \)
- \( q_{b,x} = (\sum U_{o,x}/U_{o,R}) q_b \)
- \( q_{b,y} = (\sum U_{o,y}/U_{o,R}) q_b \)

Time-averaged values are obtained by averaging over the wave period.

16. Compute bed form dimensions

Bed form dimensions are computed according to formulae given in Chapter 5.
Figure 2 Instantaneous velocity vector near bed \((z = \delta)\)
<table>
<thead>
<tr>
<th>Test</th>
<th>Condition</th>
<th>Mean current at 0.1 m to bed (m/s)</th>
<th>$&lt;U^3&gt;$ (m&lt;sup&gt;2&lt;/sup&gt;/s&lt;sup&gt;2&lt;/sup&gt;)</th>
<th>$U_{max_{cm}}/U_{1/3_{cm}}$</th>
<th>$U_{max_{off}}/U_{1/3_{off}}$</th>
<th>Period T (s)</th>
<th>Bed type and $D_{50}$</th>
<th>Sand transport</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2</td>
<td>Irregular Jonswap (st-jons.ced)</td>
<td>0</td>
<td>0.32</td>
<td>0.84</td>
<td>0.48</td>
<td>6.5</td>
<td>plane</td>
<td>0.009</td>
</tr>
<tr>
<td></td>
<td>(A=56%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.21</td>
<td></td>
</tr>
<tr>
<td>B7</td>
<td>Regular Asym. (st-r65.ced)</td>
<td>0</td>
<td>0.49</td>
<td>0.102</td>
<td>0.98</td>
<td>0.51</td>
<td>6.5</td>
<td>0.033</td>
</tr>
<tr>
<td></td>
<td>(A=39%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>plane</td>
<td>0.21</td>
</tr>
<tr>
<td>B8</td>
<td>Regular Asym. (st-r65.ced)</td>
<td>0</td>
<td>0.69</td>
<td>0.26</td>
<td>1.28</td>
<td>0.66</td>
<td>6.5</td>
<td>0.102</td>
</tr>
<tr>
<td></td>
<td>(A=54.7%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>plane</td>
<td>0.21</td>
</tr>
<tr>
<td>B0</td>
<td>Regular Asym. (st-r65.ced)</td>
<td>0</td>
<td>0.21</td>
<td>0.52</td>
<td>0.33</td>
<td>6.5</td>
<td>ripples H=3 cm L=8 cm</td>
<td>not measured</td>
</tr>
<tr>
<td></td>
<td>(A=15.6%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.21</td>
<td></td>
</tr>
<tr>
<td>J1</td>
<td>Regular Symm. (st72, A=49.2%)</td>
<td>0.24</td>
<td>-</td>
<td>-</td>
<td>1.06</td>
<td>1.06</td>
<td>7.2</td>
<td>0.104</td>
</tr>
<tr>
<td></td>
<td>(Q= 47 l/s)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>plane</td>
<td>0.21</td>
</tr>
<tr>
<td>E2</td>
<td>Regular Symm. (st72, A=66.5%)</td>
<td>0.23</td>
<td>-</td>
<td>-</td>
<td>1.47</td>
<td>1.47</td>
<td>7.2</td>
<td>?</td>
</tr>
<tr>
<td></td>
<td>(Q= 47 l/s)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>plane</td>
<td>0.21</td>
</tr>
<tr>
<td>E4</td>
<td>Regular Symm. (st72, A=42.2%)</td>
<td>0.44</td>
<td>-</td>
<td>-</td>
<td>0.95</td>
<td>0.95</td>
<td>7.2</td>
<td>0.188</td>
</tr>
<tr>
<td></td>
<td>(Q= 89 l/s)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>plane</td>
<td>0.21</td>
</tr>
<tr>
<td>H2</td>
<td>Regular Symm. (st72, A=30.4%)</td>
<td>0.23</td>
<td>-</td>
<td>-</td>
<td>0.68</td>
<td>0.68</td>
<td>7.2</td>
<td>0.042</td>
</tr>
<tr>
<td></td>
<td>(Q= 47 l/s)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>plane</td>
<td>0.13</td>
</tr>
<tr>
<td>H5</td>
<td>Regular Symm. (st72, A=58.8%)</td>
<td>0.23</td>
<td>-</td>
<td>-</td>
<td>1.3</td>
<td>1.3</td>
<td>7.2</td>
<td>0.11</td>
</tr>
<tr>
<td></td>
<td>(Q= 47 l/s)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>plane</td>
<td>0.13</td>
</tr>
<tr>
<td>H8</td>
<td>Regular Symm. (st72, A=30.4%)</td>
<td>0.44</td>
<td>-</td>
<td>-</td>
<td>0.67</td>
<td>0.67</td>
<td>7.2</td>
<td>0.105</td>
</tr>
<tr>
<td></td>
<td>(Q= 89 l/s)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>plane</td>
<td>0.13</td>
</tr>
</tbody>
</table>

Table 5.3.1 Test program OBS-transportmeter in wave tunnel
<table>
<thead>
<tr>
<th>Test</th>
<th>Shear stress velocity $U_*$ (m/s)</th>
<th>Zero elevation $z_0$ (m)</th>
<th>Apparent roughness $k_s$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>J1-1</td>
<td>0.022</td>
<td>0.001</td>
<td>0.030</td>
</tr>
<tr>
<td>J1-2</td>
<td>0.021</td>
<td>0.001</td>
<td>0.030</td>
</tr>
<tr>
<td>E2-1</td>
<td>0.036</td>
<td>0.005</td>
<td>0.150</td>
</tr>
<tr>
<td>E2-2</td>
<td>0.041</td>
<td>0.007</td>
<td>0.210</td>
</tr>
<tr>
<td>E4-1</td>
<td>0.030</td>
<td>0.000</td>
<td>0.003</td>
</tr>
<tr>
<td>E4-2</td>
<td>0.027</td>
<td>0.000</td>
<td>0.003</td>
</tr>
<tr>
<td>H2-1</td>
<td>0.014</td>
<td>0.000</td>
<td>0.003</td>
</tr>
<tr>
<td>H2-2</td>
<td>0.015</td>
<td>0.000</td>
<td>0.003</td>
</tr>
<tr>
<td>H2-3</td>
<td>0.014</td>
<td>0.000</td>
<td>0.003</td>
</tr>
<tr>
<td>H5-1</td>
<td>0.036</td>
<td>0.006</td>
<td>0.180</td>
</tr>
<tr>
<td>H5-2</td>
<td>0.033</td>
<td>0.005</td>
<td>0.150</td>
</tr>
<tr>
<td>H5-3</td>
<td>0.033</td>
<td>0.005</td>
<td>0.150</td>
</tr>
<tr>
<td>H8-1</td>
<td>0.024</td>
<td>0.000</td>
<td>0.003</td>
</tr>
<tr>
<td>H8-2</td>
<td>0.026</td>
<td>0.000</td>
<td>0.003</td>
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</tbody>
</table>

Table 5.4.1 Parameters based on theoretical logarithmic profile
### TEST B00-1 small footplate (98011214)

<table>
<thead>
<tr>
<th>Pump concentrations</th>
<th>OBS-concentrations</th>
<th>EMS-velocities</th>
<th>Sand transport between lowest and highest OBS</th>
</tr>
</thead>
<tbody>
<tr>
<td>z (m)</td>
<td>c (kg/m³)</td>
<td>z (m)</td>
<td>c (kg/m³)</td>
</tr>
<tr>
<td>0.010</td>
<td>5.71</td>
<td>0.025</td>
<td>0.96</td>
</tr>
<tr>
<td>0.020</td>
<td>2.04</td>
<td>0.045</td>
<td>0.56</td>
</tr>
<tr>
<td>0.030</td>
<td>1.26</td>
<td>0.095</td>
<td>0.63</td>
</tr>
<tr>
<td>0.045</td>
<td>0.50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.065</td>
<td>0.28</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.085</td>
<td>0.16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.115</td>
<td>0.11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.155</td>
<td>0.084</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.205</td>
<td>0.056</td>
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<td></td>
</tr>
</tbody>
</table>

### TEST B00-2 large footplate (98011411)

<table>
<thead>
<tr>
<th>Pump concentrations</th>
<th>OBS-concentrations</th>
<th>EMS-velocities</th>
<th>Sand transport between lowest and highest OBS</th>
</tr>
</thead>
<tbody>
<tr>
<td>z (m)</td>
<td>c (kg/m³)</td>
<td>z (m)</td>
<td>c (kg/m³)</td>
</tr>
<tr>
<td>0.006</td>
<td>13.80</td>
<td>0.026</td>
<td>0.56</td>
</tr>
<tr>
<td>0.016</td>
<td>1.81</td>
<td>0.046</td>
<td>0.45</td>
</tr>
<tr>
<td>0.026</td>
<td>1.16</td>
<td>0.096</td>
<td>0.51</td>
</tr>
<tr>
<td>0.041</td>
<td>0.592</td>
<td></td>
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</tr>
<tr>
<td>0.061</td>
<td>0.278</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.081</td>
<td>0.129</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.111</td>
<td>0.0587</td>
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</tr>
<tr>
<td>0.151</td>
<td>0.0408</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.201</td>
<td>0.0348</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.261</td>
<td>0.0204</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### TEST B00-3 large footplate (98011510)

<table>
<thead>
<tr>
<th>Pump concentrations</th>
<th>OBS-concentrations</th>
<th>EMS-velocities</th>
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### Table 5.4.2

Time-averaged concentrations and characteristic velocity parameters; Test B00-1 – B00-3
## Table 5.4.3
Time-averaged concentrations and characteristic velocity parameters; Test B2-1 – B2-3

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|          | Laser-velocities |          |          |          |          |          |          |          |          |          |          |
|          | \(u_{\text{on}}\) (m/s) | \(u_{\text{off}}\) (m/s) | \(S_{\text{intert}}\) (kg/sm) | \(S_{\text{mean}}\) kg/sm |
|          | 0.01 | 0.823 | 0.489 | | | | | | | | |

Integrated: \(-0.00084\) \(-0.00012\)
### Table 5.4.4  Time-averaged concentrations and characteristic velocity parameters; Test B2-4 – B2-5

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<td>c (kg/m³)</td>
<td>z (m) c (kg/m³) st.dev. (kg/m³)</td>
<td>z (m) u\text{max} (m/s) u_\text{m} (m/s) u_\text{off} (m/s) S_\text{max} (kg/sm) S_\text{mean} (kg/sm)</td>
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integrated: -0.00056 0.00002
### Table 5.4.5  Time-averaged concentrations and characteristic velocity parameters; Test B7-1 – B7-3

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<td>$c$ (kg/m$^3$)</td>
<td>$z$ (m)</td>
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<td>---------</td>
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| TEST B7-2 large footplate (98011413) |
|---------------------|---------|----------------|---------|----------------|-------------------------------|---------|----------------------|----------------------|----------------------|---------------------|----------------------|
| 0.010              | 1.95    | 0.027          | 0.56    | 0.57           | 0.047                | 0.006  | 0.874                | 0.463                | -0.047               | 0.003              |
| 0.020              | 0.647   | 0.027          | 0.38    | 0.21           | 0.047                | 0.006  | 0.874                | 0.463                | -0.047               | 0.003              |
| 0.030              | 0.348   | 0.027          | 0.41    | 0.18           | 0.047                | 0.020  | 0.949                | 0.497                | -0.024               | 0.002              |
| 0.045              | 0.164   | 0.027          | 0.37    | 0.21           | 0.047                | 0.020  | 0.949                | 0.497                | -0.024               | 0.002              |
| 0.065              | 0.0979  | 0.027          | 0.41    | 0.18           | 0.047                | 0.020  | 0.949                | 0.497                | -0.024               | 0.002              |
| 0.085              | 0.0634  | 0.027          | 0.41    | 0.18           | 0.047                | 0.020  | 0.949                | 0.497                | -0.024               | 0.002              |
| 0.115              | 0.0493  | 0.027          | 0.41    | 0.18           | 0.047                | 0.020  | 0.949                | 0.497                | -0.024               | 0.002              |
| 0.155              | 0.0297  | 0.027          | 0.41    | 0.18           | 0.047                | 0.020  | 0.949                | 0.497                | -0.024               | 0.002              |
|                     | integrated: | -0.00071 |                     |                     |                     |                     |                     |                     |                     |                     | 0.00005              |

<p>| TEST B7-3 large footplate (98011511) |
|---------------------|---------|----------------|---------|----------------|-------------------------------|---------|----------------------|----------------------|----------------------|---------------------|----------------------|
| 0.007              | 3.02    | 0.027          | 0.50    | 0.39           | 0.047                | 0.007  | 0.855                | 0.474                | -0.050               | 0.004              |
| 0.017              | 0.835   | 0.027          | 0.38    | 0.25           | 0.047                | 0.007  | 0.855                | 0.474                | -0.031               | 0.003              |
| 0.027              | 0.464   | 0.027          | 0.38    | 0.25           | 0.047                | 0.007  | 0.855                | 0.474                | -0.031               | 0.003              |
| 0.042              | 0.203   | 0.027          | 0.38    | 0.25           | 0.047                | 0.007  | 0.855                | 0.474                | -0.031               | 0.003              |
| 0.062              | 0.104   | 0.027          | 0.38    | 0.25           | 0.047                | 0.007  | 0.855                | 0.474                | -0.031               | 0.003              |
| 0.082              | 0.0690  | 0.027          | 0.38    | 0.25           | 0.047                | 0.007  | 0.855                | 0.474                | -0.031               | 0.003              |
| 0.112              | 0.0457  | 0.027          | 0.38    | 0.25           | 0.047                | 0.007  | 0.855                | 0.474                | -0.031               | 0.003              |
| 0.152              | 0.0317  | 0.027          | 0.38    | 0.25           | 0.047                | 0.007  | 0.855                | 0.474                | -0.031               | 0.003              |
| 0.202              | 0.0348  | 0.027          | 0.38    | 0.25           | 0.047                | 0.007  | 0.855                | 0.474                | -0.031               | 0.003              |
| 0.262              | 0.0223  | 0.027          | 0.38    | 0.25           | 0.047                | 0.007  | 0.855                | 0.474                | -0.031               | 0.003              |
|                     | integrated: | -0.00081 |                     |                     |                     |                     |                     |                     |                     |                     | 0.00007              |</p>
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<th>z (m)</th>
<th>c (kg/m³)</th>
<th>st.dev. (kg/m³)</th>
<th>z (m)</th>
<th>u_{max} (m/s)</th>
<th>u_{min} (m/s)</th>
<th>u_{off} (m/s)</th>
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<th>c (kg/m³)</th>
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Table 5.4.6  Time-averaged concentrations and characteristic velocity parameters; Test B8-1 – B8-2
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<td>Measured Laser-velocities</td>
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integrated: -0.00017 0.00450

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<td>Measured Laser-velocities</td>
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integrated: 0.00000 0.00511

Table 5.4.7  Time-averaged concentrations and characteristic velocity parameters; Test J1-1 – J1-2
### Table 5.4.8  Time-averaged concentrations and characteristic velocity parameters; Test E2-1 – E2-2
### TEST E4-1 (98011611)

<table>
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<tr>
<th>Pump concentrations</th>
<th>OBS-concentrations</th>
<th>Measured EMS-velocities</th>
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<td>c (kg/m³)</td>
<td>z (m)</td>
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<td>0.029</td>
</tr>
<tr>
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<td>0.099</td>
</tr>
<tr>
<td>0.113</td>
<td>0.314</td>
<td></td>
</tr>
<tr>
<td>0.153</td>
<td>0.218</td>
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<tr>
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integrated: 0.00291 0.00789

### TEST E4-2 (98011613)

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<th>Measured EMS-velocities</th>
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<tr>
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<td></td>
</tr>
<tr>
<td>0.155</td>
<td>0.118</td>
<td></td>
</tr>
<tr>
<td>0.205</td>
<td>0.110</td>
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</tr>
<tr>
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integrated: 0.00269 0.00854

**Table 5.4.9** Time-averaged concentrations and characteristic velocity parameters; Test E4-1 – E4-2
### TEST H2-1 (98012015)

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<th>z (m)</th>
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<th>z (m)</th>
<th>c (kg/m³)</th>
<th>st.dev. (kg/m³)</th>
<th>z (m)</th>
<th>( u_{\text{mean}} ) (m/s)</th>
<th>( u_{\text{in}} ) (m/s)</th>
<th>( u_{\text{off}} ) (m/s)</th>
<th>( S_{\text{instant}} ) (kg/sm)</th>
<th>( S_{\text{mean}} ) (kg/sm)</th>
</tr>
</thead>
<tbody>
<tr>
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<td></td>
<td>0.025</td>
<td>0.198</td>
<td>0.658</td>
<td>0.612</td>
<td>-0.024</td>
<td>0.224</td>
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<td></td>
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<td>0.025</td>
<td>0.198</td>
<td>0.658</td>
<td>0.612</td>
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<td>0.079</td>
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<td>0.045</td>
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<td>0.095</td>
<td>0.246</td>
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<td>0.672</td>
<td>0.061</td>
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<td>0.095</td>
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### TEST H2-2 (98012016)

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<th>z (m)</th>
<th>c (kg/m³)</th>
<th>st.dev. (kg/m³)</th>
<th>z (m)</th>
<th>( u_{\text{mean}} ) (m/s)</th>
<th>( u_{\text{in}} ) (m/s)</th>
<th>( u_{\text{off}} ) (m/s)</th>
<th>( S_{\text{instant}} ) (kg/sm)</th>
<th>( S_{\text{mean}} ) (kg/sm)</th>
</tr>
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<td></td>
<td>0.026</td>
<td>0.195</td>
<td>0.659</td>
<td>0.623</td>
<td>-0.007</td>
<td>0.199</td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td>0.046</td>
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<td>0.659</td>
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<td>0.096</td>
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<td>0.718</td>
<td>0.675</td>
<td>0.060</td>
<td>0.071</td>
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<td>0.63</td>
<td>0.60</td>
<td>0.096</td>
<td>0.245</td>
<td>0.718</td>
<td>0.675</td>
<td>0.060</td>
<td>0.071</td>
</tr>
<tr>
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<td>0.096</td>
<td>0.29</td>
<td>0.15</td>
<td>0.20</td>
<td>0.273</td>
<td>0.718</td>
<td>0.675</td>
<td>0.061</td>
<td>0.079</td>
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### TEST H2-3

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<th>z (m)</th>
<th>c (kg/m³)</th>
<th>st.dev. (kg/m³)</th>
<th>z (m)</th>
<th>( u_{\text{mean}} ) (m/s)</th>
<th>( u_{\text{in}} ) (m/s)</th>
<th>( u_{\text{off}} ) (m/s)</th>
<th>( S_{\text{instant}} ) (kg/sm)</th>
<th>( S_{\text{mean}} ) (kg/sm)</th>
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<td></td>
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<td>0.225</td>
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<td>0.096</td>
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<td>0.51</td>
<td>0.096</td>
<td>0.273</td>
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<td>0.669</td>
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<th>z (m)</th>
<th>c (kg/m³)</th>
<th>st.dev. (kg/m³)</th>
<th>z (m)</th>
<th>( u_{\text{mean}} ) (m/s)</th>
<th>( u_{\text{in}} ) (m/s)</th>
<th>( u_{\text{off}} ) (m/s)</th>
<th>( S_{\text{instant}} ) (kg/sm)</th>
<th>( S_{\text{mean}} ) (kg/sm)</th>
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<td>0.203</td>
<td>0.259</td>
<td>0.709</td>
<td>0.669</td>
<td>0.00140</td>
<td>0.00738</td>
</tr>
<tr>
<td>0.153</td>
<td>0.040</td>
<td>0.203</td>
<td>0.025</td>
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<td>0.00140</td>
<td>0.00738</td>
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<td>0.013</td>
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**Table 5.4.10** Time-averaged concentrations and characteristic velocity parameters; Test H2-1 – H2-3
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<th>z (m)</th>
<th>c (kg/m³)</th>
<th>st.dev. (kg/m³)</th>
<th>z (m)</th>
<th>( u_{\text{mean}} ) (m/s)</th>
<th>( u_{\text{tan}} ) (m/s)</th>
<th>( u_{\text{eff}} ) (m/s)</th>
<th>( S_{\text{mean}} ) (kg/sm)</th>
<th>( S_{\text{mean}} ) (kg/sm)</th>
</tr>
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<td>0.026</td>
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<td>7.35</td>
<td>0.046</td>
<td>0.188</td>
<td>1.278</td>
<td>1.298</td>
<td>-0.099</td>
<td>0.840</td>
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<tr>
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<td>4.67</td>
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<td>1.360</td>
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<td>0.320</td>
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**TEST H5-2 (98012111)**

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<th>z (m)</th>
<th>c (kg/m³)</th>
<th>st.dev. (kg/m³)</th>
<th>z (m)</th>
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<th>( u_{\text{tan}} ) (m/s)</th>
<th>( u_{\text{eff}} ) (m/s)</th>
<th>( S_{\text{mean}} ) (kg/sm)</th>
<th>( S_{\text{mean}} ) (kg/sm)</th>
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<td>1.297</td>
<td>-0.122</td>
<td>0.839</td>
</tr>
<tr>
<td>0.028</td>
<td>6.98</td>
<td></td>
<td></td>
<td></td>
<td>0.046</td>
<td>0.188</td>
<td>1.267</td>
<td>1.296</td>
<td>-0.377</td>
<td>0.513</td>
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<td>5.95</td>
<td>6.93</td>
<td>0.046</td>
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<td>1.296</td>
<td>-0.377</td>
<td>0.513</td>
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<tr>
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<td>2.73</td>
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**TEST H5-3 (98012211)**

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<th>z (m)</th>
<th>c (kg/m³)</th>
<th>st.dev. (kg/m³)</th>
<th>z (m)</th>
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<th>( u_{\text{tan}} ) (m/s)</th>
<th>( u_{\text{eff}} ) (m/s)</th>
<th>( S_{\text{mean}} ) (kg/sm)</th>
<th>( S_{\text{mean}} ) (kg/sm)</th>
</tr>
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<td>1.298</td>
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<td>0.840</td>
</tr>
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<td>9.16</td>
<td></td>
<td></td>
<td></td>
<td>0.026</td>
<td>0.144</td>
<td>1.278</td>
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<td>0.191</td>
<td>1.278</td>
<td>1.298</td>
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<td>0.280</td>
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<tr>
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<td>2.90</td>
<td>0.046</td>
<td>2.49</td>
<td>3.73</td>
<td>0.096</td>
<td>0.252</td>
<td>1.371</td>
<td>1.390</td>
<td>0.277</td>
<td>0.280</td>
</tr>
<tr>
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<td>1.371</td>
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integrated: -0.00486 0.03750

integrated: -0.00609 0.03370

integrated: -0.00509 0.03206

**Table 5.4.11**  Time-averaged concentrations and characteristic velocity parameters; Test H5-1 – H5-3
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<th>Pump concentrations</th>
<th>OBS-concentrations</th>
<th>Measured EMS-velocities</th>
<th>Sand transport between lowest and highest OBS</th>
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<td>z (m)</td>
<td>c (kg/m³)</td>
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<td>0.95</td>
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</table>

**Table 5.4.12** Time-averaged concentrations and characteristic velocity parameters; Test H8-1 – H8-3
## Results from Visual Accumulation Tube

**Method:** Van Rijn corrected  
**Water temperature:** ± 10° C

<table>
<thead>
<tr>
<th>Testnr.</th>
<th>levelnr.</th>
<th>fall velocity of sediment</th>
<th>grain diameter of sediment</th>
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<tbody>
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<td>$w_{10}$</td>
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<td>5, 6 and 7</td>
<td>5.92</td>
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<td>bed material 2</td>
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Table 5.4.13  Fall velocities and grain diameters of suspended sediment and bed material
<table>
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<tr>
<th>Test</th>
<th>Transport rate based on OBS and EMF data (kg/m/s)</th>
<th>Transport rate based on mass conservation (kg/m/s)</th>
<th>Transport rate in unmeasured zone near the bed: ( q_{\text{total}} = q_{\text{suspension}} ) (kg/m/s)</th>
<th>Calculated transport rate ( m = 5, n = 1.65 ) (kg/m/s)</th>
<th>Calculated transport rate ( m = 5, n = 1.78 ) (kg/m/s)</th>
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<td>0.0954</td>
<td>0.1019</td>
<td>0.1135</td>
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<td>0.0963</td>
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Table 5.5.1  Measured and calculated transport rates
Sand transport in Deeper water
Location of cross-shore profiles
14, 40, 76 and 103

Scale 1:500,000

WL | DELFT HYDRAULICS

Z2378 FIG. 2.2.1
MORPHOLOGICAL BEHAVIOUR OF SAND DUMPING NEAR HOEK VAN HOLLAND, THE NETHERLANDS

DELT HYDRAULICS

Z 2378 FIG. 2.3.1
MORPHOLOGICAL BEHAVIOUR OF SAND DUMPING NEAR WIJK AAN ZEE, THE NETHERLANDS

DELFt HYDRAULICS

Z 2378 FIG. 2.3.2
Bed level after 15 hours

- **Computed using $E_a$-method**
- **××××××××× measured**

Migration and Sedimentation of a Channel in a Flume

Delft Hydraulics

Z 2378  Fig. 3.2.1
A. TIDAL CONDITIONS

B. BED LEVELS
A. TIDAL CONDITIONS

B. BED LEVELS

SEDIMENTATION IN A TRIAL DREDGE CHANNEL IN THE EASTERN SCHELDT, THE NETHERLANDS

DELFt HYDRAULICS
SEDIMENTATION IN A TRIAL DREDGE CHANNEL IN ASAN BAY, KOREA
DELFT HYDRAULICS

A. TRIAL DREDGE CHANNEL

B. FLOW VELOCITY AT t=0

C. WIDTH-INTEGRATED SUSPENDED SAND TRANSPORT AT t=0

D. BED LEVEL PROFILES ALONG (REFRACTED) STREAMLINE
A. TRIAL DREDGE CHANNEL

B. FLOW VELOCITY AT t=0

C. WIDTH-INTEGRATED SUSPENDED SILT TRANSPORT AT t=0

D. BED LEVEL PROFILES ALONG (REFRACTED) STREAMLINE

SEDIMENTATION IN A TRIAL DREDGE CHANNEL
NEAR BAHIA BLANCA, ARGENTINA

DELFt HYDRAULICS
A. EXPERIMENTAL SET-UP

B. VARIATION OF SUSPENDED LOAD TRANSPORT

C. VARIATION OF BED LEVELS

EXPERIMENTAL SET-UP
VARIATION OF SUSPENDED LOAD TRANSPORT
AND BED LEVELS IN A CHANNEL (TEST 1)

DELTFT HYDRAULICS

Z 2378 FIG. 3.2.7
MORPHOLOGICAL BEHAVIOUR OF TRENCH (SECTION 2)
IN WAVE–CURRENT BASIN

DELFt HYDRAULICS

Z 2378 FIG. 3.2.8
KEY PLANS

BEACH PROFILE, FEB. 1982

Scales:
Horizontal: 1:10,000
Vertical: 1:200

NOTES:
All depths in metres relative to O.D.N.N.
Chainages relative to definition on Key Plan III.
Centre line is the theoretical C for the gas pipe, see Key Plan III.

Sand transport in deeper water
Dredged trench at the Danish coast
Site description

SUTRENCH
A4

WL | DELFT HYDRAULICS
Z 2378 Fig 3.2.9
Sand transport in deeper water
Dredged trench at the Danish coast
Cross-sections at CH 1700

LEGEND:
- PRE-DREDGE SURVEY, FEB 1983
- POST-DREDGE SURVEY, 3/5 - 3/6, 1983
- MONITORING OF BACKFILLING, 20/5 1983
- MONITORING OF BACKFILLING, 20/10 - 1/11, 1983
- MONITORING OF BACKFILLING, 2/4/13 1983
- MONITORING OF BACKFILLING, 6/7 1983
Sand transport in Deeper water
Backfilling of trench (Havinga, 1992)
Model Setup

SUTRENCH
T10.20.90

WL | DELFT HYDRAULICS
Z2378 FIG. 3.2.11
Sand transport in Deeper water
Backfilling of trench (Havinga, 1992)
Comparison with Calculated results: Base run - $S(x=0)=0.022$ kg/m/s
Stochastic transport in Deeper water
Backfilling of trench (Havinga, 1992)
Bed & Current related roughness-height=0.005m - S(x=0)=0.021kg/m/s

SUTRENCHE RUN T10
T10.20.90 - Section 2

WL I DELFT HYDRAULICS

Z2378 FIG. 3.2.13
Sand transport in Deeper water
Backfilling of trench (Havinga, 1992)
Bed & Current related roughness-height=0.02m - S(x=0)=0.022 kg/m/s

SUTRENCH | RUN T11
T10.20.90 - Section 2

WL I DELFT HYDRAULICS
Z2378 | FIG. 3.2.14
Sand transport in Deeper water
Backfilling of trench (Havinga, 1992)
Sediment fall velocity \( W_d = 0.004 \text{ m/s} \), \( S(x=0) = 0.022 \text{kg/m/s} \)

**Velocities at inflow**

**Concentrations at inflow**

**Residual transports on initial profile**

**Bottom profiles**

**Measured (after 25hr30)**

**Calculated (after 25hr30)**

**Initial profile**

---

**SUTRENCHE** | **RUN T12**
--- | ---
T10.20.90 - Section 2

---

**WL I DELFT HYdraulics**

---

**Z2378** | **FIG. 3.2.15**
Sand transport in Deeper water
Backfilling of trench (Havinga, 1992)
Sediment fall velocity \( (W_d)=0.008 \text{ m/s} \) - \( S(x=0)=0.021 \text{kg/m/s} \)
Sand transport in Deeper water
Backfilling of trench (Havinga, 1992)
Corrected to minimum measured transport - $S(x=0)=0.018\text{kg/m/s}$

---

**Velocities at inflow**

**Concentrations at inflow**

**Residual transports on initial profile**

**Bottom profiles**

---

**SUTRENCH**

**RUN T14**

**T10.20.90 - Section 2**
Sand transport in Deeper water
Backfilling of trench (Havinga, 1992)
Corrected to maximum measured transport - S(x=0)=0.024kg/m/s
Sand transport in Deeper water
Backfilling of trench (Havinga, 1992)
Increased discharge with 5% - S(x=0)=0.021kg/m/s
Sand transport in Deeper water
Backfilling of trench (Havinga, 1992)
Incr. disch. (5%) & red. sed. conc. \(C_d=1.36\) - \(S(x=0)=0.018\text{kg/m}^3\)

SUTRENCH | RUN T17
--- | ---
T10.20.90 - Section 2

WL | DELFT HYDRAULICS
--- | ---
Z2378 | FIG. 3.2.20
Sand transport in Deeper water
Backfilling of trench (Havinga, 1992)
Comparison with Calculated results
Sand transport in Deeper water
Backfilling of trench (Havinga, 1992)
Comparison with Calculated results
Sand transport in Deeper water
Backfilling of trench (Havinga, 1992)
Comparison with Calculated results
Sand transport in Deeper water
Dredged trench at the Danish coast
Model Setup

SUTRENCHE
Mangor (1984); Ch1700

WL I DELFT HYDRAULICS
Z2378 FIG. 3.2.24
Sand transport in Deeper water
Dredged trench at the Danish coast
Backfilling of trench during calm conditions

SUTRENCHE  RUN D08
Mangor (1984); Ch1700

WL | DELFT HYDRAULICS
Z2378 | FIG. 3.2.26
Sand transport in Deeper water
Dredged trench at the Danish coast
Baserun

SUTRENCH    RUN D09
Mangor (1984); Ch1700

WL I DELFT HYDRAULICS
Sand transport in Deeper water
Dredged trench at the Danish coast
$D_{50}=0.14 \text{ mm } D_{90}=0.20 \text{ mm } W_s=0.014 \text{ m/s}$

SUTRENCHE RUN D11
Mangor (1984); Ch1700

WL | DELFT HYDRAULICS
Z2378 | FIG. 3.2.29
SOUTH

Morphodynamic behaviour of trench

NORTH

Yearly transports on initial profile (Residual)

North

Yearly transports on initial profile (Ebb and Flood)

South

Sedimentation-Erosion [m]

Sedimentation

Erosion

Sand transport in Deeper water
Euro-channel
Results from Hoitink (Erratum, 1998)

SUTRENCH
Eurochannel

WL I DELFT HYDRAULICS

Z2378 FIG. 3.3.1
Morphodynamic behaviour of trench

Yearly transports on initial profile (Residual)

Yearly transports on initial profile (Ebb and Flood)

Sedimentation-Erosion [m]

Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 3 years simulations
Base run; Depth=7 m - Width=600 m - Slopes=4%

SUTRENCH
Eurochannel

WL I DELFT HYDRAULICS

Z2378
FIG. 3.3.3
Sand transport in Deeper water - Sensitivity analysis
Definition of inflection points
Sand transport in Deeper water - Sensitivity analysis
Definitions of Trench characteristics
SOUTH

Morphodynamic behaviour of trench

-8.0
-6.0
-4.0
-2.0
0.0
-2.0
-4.0
-6.0
-8.0

0 500 1000 1500 2000 2500 3000 3500 4000
X [m]

Definition of slope
Centre of gravity of trench
Initial profile
Profile after 3 years

NORTH

Yearly transports on initial profile

140.0
120.0
100.0
80.0
60.0
40.0
20.0
0.0
-20.0
-40.0

0 500 1000 1500 2000 2500 3000 3500 4000
X [m]

Suspended transport
Bottom transport
Total transport

Relative transports

8.0
6.0
4.0
2.0
0.0

0 500 1000 1500 2000 2500 3000 3500 4000
X [m]

Suspended transport
Bottom transport
Total transport

Sedimentation-Erosion [m]

2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5

0 500 1000 1500 2000 2500 3000 3500 4000
X [m]

Base run
De-LOW

Sand transport in Deeper water
Sensitivity analysis: Effect of process and model parameters
Dₚₐ=0.15 m (Decreased by 50 %)
Morphodynamic behaviour of trench

Yearly transports on initial profile

Relative transports

Sedimentation-Erosion [m]

Sand transport in Deeper water
Sensitivity analysis: Effect of process and model parameters
Dₚₑ=0.45 m (increased by 150 %)

SUTRENCH

Eurochannel

WL I DELFT HYDRAULICS

Z2378 FIG. 3.3.7
Sensitivity analysis: Effect of process and model parameters

\[ E_{w,\text{bed}} = 0.002 \text{ m}^2/\text{s} \] and \[ E_{w,\text{max}} = 0.0175 \text{ m}^2/\text{s} \] (Decreased by 50 %)
Sand transport in Deeper water
Sensitivity analysis: Effect of process and model parameters
$E_{w,\text{bed}}=0.006 \text{ m}^2/\text{s}$ and $E_{w,\text{max}}=0.0525 \text{ m}^2/\text{s}$ (increased by 150%)
Sand transport in Deeper water
Sensitivity analysis: Effect of process and model parameters
$H_s=1.125 \text{ m} \text{ (Decreased by 50\%)}$
SOUTH

Morphodynamic behaviour of trench

NORTH

Definition of slope
Centre of gravity of trench
Initial profile
Profile after 3 years

Yearly transports on initial profile

North

Suspended transport
Bottom transport
Total transport

Relative transports

Suspended transport
Bottom transport
Total transport

Sedimentation-Erosion [m]

Base run
Hs-HIGH

Sand transport in Deeper water
Sensitivity analysis: Effect of process and model parameters
H_s=3.375 m (increased by 150 %)

SUTRENCH

Eurochannel

WL I DELFT HYDRAULICS

Z2378

FIG. 3.3.12
Sand transport in Deeper water
Sensitivity analysis: Effect of process and model parameters
\( T_p = 4.4 \) s (Decreased by 67 %)
SOUTH

Morphodynamic behaviour of trench

NORTH

Yearly transports on initial profile

Relative transports

Sedimentation-Erosion [m]

Sand transport in Deeper water
Sensitivity analysis: Effect of process and model parameters
T_P=9.9 s (Increased by 150 %)

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Z2378 | FIG. 3.3.14
Morphodynamic behaviour of trench

Yearly transports on initial profile

Relative transports

Sedimentation-Erosion

Sand transport in Deeper water
Sensitivity analysis: Effect of process and model parameters

\[ D_0 = 100 \times 10^{-6} \text{ m}; D_0 = 200 \times 10^{-6} \text{ m}; W_s = 0.01 \text{ m/s (Decreased by 50%)} \]

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FIG. 3.3.15
Sand transport in Deeper water
Sensitivity analysis: Effect of process and model parameters
\(D_{50}=300 \times 10^{-6} \text{ m}; \ D_{90}=450 \times 10^{-6} \text{ m}; \ W_a=0.04 \text{ m/s (Increased by 150 %)}\)

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Z2378 FIG. 3.3.16
SOUTH

Morphodynamic behaviour of trench

-8.0
-6.0
-4.0
-2.0
0.0

Depth [m]

0 500 1000 1500 2000 2500 3000 3500 4000

X [m]

Definition of slope
O Centre of gravity of trench
Initial profile
Profile after 3 years

NORTH

Yearly transports on initial profile

North

60.0
40.0
20.0
0.0

Transports [m³/year]

3500
4000

3500
4000

Suspended transport
Bottom transport
Total transport

South

-20.0
-40.0

Relative transports

2.0
1.5
1.0
0.5
0.0

Relative transports [-]

3500
4000

3500
4000

Suspended transport
Bottom transport
Total transport

Sedimentation-Erosion [m]

1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5

3500
4000

3500
4000

Base run
Rw-LOW

Sand transport in Deeper water
Sensitivity analysis: Effect of process and model parameters
Rw=0.005 m (Decreased by 50 %)

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Z2378 FIG. 3.3.17
Sand transport in Deeper water
Sensitivity analysis: Effect of process and model parameters
$R_{c}=0.025$ m (Decreased by 50 %)
Sand transport in Deeper water
Sensitivity analysis: Effect of process-model parameters
R_c=0.075 m (increased by 150 %)
Sand transport in Deeper water
Sensitivity analysis: Effect of process and model parameters
$Z_a=0.025 \text{ m} \text{ (Decreased by 50 \%)}$
Sand transport in Deeper water
Sensitivity analysis - Influence of tide schematisation
Comparison of schematisations
Morphodynamic behaviour of trench after 3 years

Yearly transports on initial profile (7 steps)

Yearly transports on initial profile (2 steps)

Sedimentation erosion after 3 years

Sand transport in Deeper water
Sensitivity analysis: Influence of tide schematisation
Comparison of transports and bottom profiles
Morphodynamic behaviour of trench

Yearly transports on initial profile (Residual)

Yearly transports on initial profile (Ebb and Flood)

Sedimentation-Erosion [m]

Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 3 years simulations
Trench Width=1200 m (width is twice width of base run)
Morphodynamic behaviour of trench

Yearly transports on initial profile (Residual)

Yearly transports on initial profile (Ebb and Flood)

Sedimentation-Erosion [m]

Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 3 years simulations
Trench Width=300 m (width is half width of base run)
Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 3 years simulations
Trench depth=3.5 m (depth is half depth of base run)
Morphodynamic behaviour of trench

Yearly transports on initial profile (Residual)

Yearly transports on initial profile (Ebb and Flood)

Sedimentation-Erosion [m]

Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 3 years simulations
Trench depth=14 m (depth is twice depth of base run)
Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 3 years simulations
Trench slopes=2 % (slope is half slope of base run)

SOUTH

Morphodynamic behaviour of trench

Depth [m]

0.0
-2.0
-4.0
-6.0
-8.0

0 500 1000 1500 2000 2500 3000 3500 4000

X [m]

Definition of slope
Centre of gravity of trench
Initial profile
Profile after 3 years

NORTH

Yearly transports on initial profile (Residual)

Transport [m³/m³/year]

100.0
80.0
60.0
40.0
20.0
0.0
-20.0
-40.0

North

South

Suspended transport
Bottom transport
Total transport

Yearly transports on initial profile (Ebb and Flood)

Transport [m³/m³/year]

120.0
90.0
60.0
30.0
0.0
-30.0
-60.0

Ebb
Flood
Suspended transport
Bottom transport
Total transport

Sedimentation-Erosion [m]

Sedimentation

2.0
1.5
1.0
0.5
0.0

Erosion

-0.5
-1.0
-1.5
-2.0

X [m]

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FIG. 3.3.31
Morphodynamic behaviour of trench

Yearly transports on initial profile (Residual)

Yearly transports on initial profile (Ebb and Flood)

Sedimentation-Erosion [m]

Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 3 years simulations
Trench slopes=8 % (slope is twice slope of base run)
Morphodynamic behaviour of trench

Yearly transports on initial profile (Residual)

Yearly transports on initial profile (Ebb and Flood)

Sedimentation-Erosion [m]
SOUTH

Morphodynamic behaviour of trench

NORTH

Definition of slope
○ Centre of gravity of trench
- Initial profile
- Profile after 50 years

Yearly transports on initial profile (Residual)

North

Transport [m³/year]

0 500 1000 1500 2000 2500 3000 3500 4000

-40
-20
0
20
40
60
80
100

- - - - Suspended transport
- - - - Bottom transport
- - - - Total transport

Yearly transports on initial profile (Ebb and Flood)

Transport [m³/year]

0 500 1000 1500 2000 2500 3000 3500 4000

-60
-30
0
30
60
90
120

- - - - Ebb
- - - - Flood
- - - - Suspended transport
- - - - Bottom transport
- - - - Total transport

Sedimentation-Erosion [m]

Sedimentation

0 500 1000 1500 2000 2500 3000 3500 4000

-8
-6
-4
-2
0
2
4
6
8
10

Erosion

Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 50 years simulations
Trench Width=1200 m (width is twice width of base run)

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Z2378 | FIG. 4.2.3
Morphodynamic behaviour of trench

Yearly transports on initial profile (Residual)

Yearly transports on initial profile (Ebb and Flood)

Sedimentation-Erosion [m]

Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 50 years simulations
Trench Width=300 m (width is half width of base run)
Total trapping efficiency

Dredging trapping efficiency

Relative trench width

Relative trench depth

Migration of centre of gravity

Relative Sedimentation-Erosion

Sand transport in Deeper water - Sensitivity analysis
Development of output parameters in time
Base Run Trench width = 300 m

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Z2378 FIG. 4.2.7
SOUTH

Morphodynamic behaviour of trench

- Depth [m]
  - Definition of slope
  - Centre of gravity of trench
  - Initial profile
  - Profile after 50 years

NORTH

Yearly transports on initial profile (Residual)

- Transports [m³/yr]
  - Suspended transport
  - Bottom transport
  - Total transport

Yearly transports on initial profile (Ebb and Flood)

- Transports [m³/yr]
  - Ebb
  - Flood
  - Suspended transport
  - Bottom transport
  - Total transport

Sedimentation-Erosion [m]

- Sedimentation [m]
  - Erosion [m]

Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 50 years simulations
Trench depth=14 m (depth is twice depth of base run)

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Z2378 | FIG. 4.2.8
Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 50 years simulations
Trench depth=10 m (depth is 10/7 times depth of base run)

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Z2378 FIG. 4.2.9
Morphodynamic behaviour of trench

Yearly transports on initial profile (Residual)

Yearly transports on initial profile (Ebb and Flood)

Sedimentation-Erosion [m]

Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 50 years simulations
Trench depth=3.5 m (depth is half depth of base run)
Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 50 years simulations
Trench depth=2 m (depth is 2/7 times depth of base run)
Sand transport in Deeper water - Sensitivity analysis
Development of output parameters in time

- Base Run
- Trench depth = 10 m
Sand transport in Deeper water - Sensitivity analysis
Development of output parameters in time

- Base Run
- Trench depth = 3.5 m

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FIG. 4.2.14
Sand transport in Deeper water - Sensitivity analysis
Development of output parameters in time
☐ Base Run ☆ Trench depth = 2 m

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Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 50 years simulations
Trench slopes=8 % (slope is twice slope of base run)
Morphodynamic behaviour of trench

Yearly transports on initial profile (Residual)

Yearly transports on initial profile (Ebb and Flood)

Sedimentation-Erosion [m]

Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 50 years simulations
Trench slopes=2 % (slope is half slope of base run)
Sand transport in Deeper water - Sensitivity analysis

Development of output parameters in time

- Base Run
- Trench slopes = 8%

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Sand transport in Deeper water - Sensitivity analysis
Development of output parameters in time

- Base Run
- Trench slopes = 2%

**Total trapping efficiency**

- Y-axis: \( TE_{tot} \)
- X-axis: Time [yr]

**Dredging trapping efficiency**

- Y-axis: \( TE_{dredging} \)
- X-axis: Time [yr]

**Relative trench width**

- Y-axis: Width [m]
- X-axis: Time [yr]

**Relative trench depth**

- Y-axis: Depth [m]
- X-axis: Time [yr]

**Migration of centre of gravity**

- Y-axis: \( D_{cog} \) [m]
- X-axis: Time [yr]

**Relative Sedimentation-Erosion**

- Y-axis: Sed-Ero [m]
- X-axis: Time [yr]
Sand transport in Deeper water - Sensitivity analysis
Influence of dimensions of trench - 50 years simulations
text=Water depth=25 m (MSL) (1.25 times water depth of Base run)
Sand transport in Deeper water - Sensitivity analysis
Development of output parameters in time

- Base Run: Water depth (MSL) = 25 m

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The large oscillating water tunnel

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Z2378 | Fig. 5.1.1
- flow discharge $Q$ and piston motion
- flow velocities $U(z,t)$
- sediment concentrations $C(z,t)$
- net sediment transport rate $q_s(x)$ (mass conservation technique)
Schematic diagram of transverse suction system (pump sampler)
Mechanical dimensions and beam pattern of optical backscatterance sensor
Calibration curves of OBS sensors for two different sand sizes

D_{50} = 0.12 mm

D_{50} = 0.19 mm
Influence of grain size (D50) on slope of calibration curve of OBS's
Calibration curves of EMF sensor D234
Still water offset determined during the experiments: 0.013 m/s
Comparison of signal from EMF and laser doppler velocity meter (upper)
Part of EMF signal used for further computations (lower)
Velocity and concentration signal
Test B2-3
Velocity and concentration signal
Test B7-1
Velocity and concentration signal
Test B8-1
Velocity and concentration signal
Test E2-1
Velocity and concentration signal
Test E4-1

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Velocity and concentration signal
Test J1-1
Velocity and concentration signal
Test H2-1

Velocity (m/s)
-1.5
-1.0
-0.5
0.0
0.5
1.0
1.5

EMF; z = 0.045 m

Time (hh:mm:ss)
15:15:00
15:15:10
15:15:20
15:15:30
15:15:40
15:15:50
15:16:00

Concentration (g/l)
0.0
2.0
4.0
6.0
8.0
10.0
12.0

OBS 350; z = 0.025 m
OBS 355; z = 0.045 m

Time (hh:mm:ss)
15:15:00
15:15:10
15:15:20
15:15:30
15:15:40
15:15:50
15:16:00

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Z2378  Fig. 5.4.8
Velocity and concentration signal
Test H5-1

Velocity (m/s)

EMF; z = 0.046 m

Concentration (g/l)

OBS 350; z = 0.026 m
OBS 355; z = 0.046 m

Time (hh:mm:ss)
Velocity and concentration signal
Test H8-1
Concentrations

test B00-1, B00-2 and B00-3

Concentrations


Time-averaged suspended sediment concentration profile
Tests B00 and B2

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B00 and B2

Z2378  Fig. 5.4.11
Concentrations
Test B7-1, B7-2 and B7-3

Concentrations
Test B8-1 and B8-2

Time-averaged suspended sediment concentration profile
Tests B7 and B8
Time-averaged suspended sediment concentration profile
Tests E2 and E4
Concentrations

test J1-1 and J1-2

Concentrations

test H2-1, H2-2 and H2-3

Time-averaged suspended sediment concentration profile
Tests J1 and H2

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Time-averaged suspended sediment concentration profile
Tests H5 and H8

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Z2378 Fig. 5.4.15
Calculated and measured transport rates for the unmeasured zone near the bed.

**m = 5.00, n = 1.65**

**Δ m = 5.35, n = 1.65**

**x m = 5.00, n = 1.78**

**sand transport in unmeasured zone near bed (kg/m/s)**