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Engineering model for sand transport;
TRANSPOR 2000

Research Report
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CLIENT: RIKZ of Rijkswaterstaat

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ABSTRACT:
The report contains the following elements:

- description of recent improvements of TRANSPOR model for computation of sand transport under combined wave and current conditions, focussing on:
  - effect of ripples on (near-bed) reference concentration,
  - effect of ripples on sediment mixing and hence on time-averaged concentration profiles and suspended transport,
  - effect of wave breaking on sediment mixing and hence on time-averaged concentration profiles and suspended transport,
  - effect of wave-related suspended sand transport,
  - effect of graded bed material (multi-fractions),
  - calibration and verification of model based on large-scale wave tunnel, wave flume and field data sets.
- description of available large-scale laboratory and field data for calibration and validation of TRANSPOR model; data of the Delta-flume of Delft Hydraulics and data from the surf zone of Egmond beach, The Netherlands have been used,
- description of sensitivity computations using TRANSPOR model
- validation of the bed load transport model based on measured data from the wave tunnel of Delft Hydraulics for particle sizes in the range of 0.2 to 1 mm.

The most important conclusions are:

An engineering sand transport model has been formulated that can be used for the computation of sand transport in combined wave and current conditions, rippled and flat beds, uniform and graded bed materials with particle sizes larger than about 0.2 mm.

Data sets of large-scale flume and field measurements (with particle sizes in the range of 0.2 to 0.4 mm) have been compiled that can be used for comparison with model results.

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CONTENTS
TEXT PAGES: 40
TABLES: 6
FIGURES: 17
APPENDICES: 3

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Contents

1 Introduction .................................................................................................................. 1–1

1.1 Sand transport processes ....................................................................................... 1–1

1.2 Definitions ............................................................................................................... 1–2

1.3 Objectives and assumptions .................................................................................. 1–3

1.4 Acknowledgements ................................................................................................. 1–4

2 Modelling of bed load transport for uniform bed material (single fraction method) .............................................................................................................. 2–1

2.1 Bed load transport processes .................................................................................. 2–1

2.2 Bed load transport model ...................................................................................... 2–6

2.2.1 Formulations ..................................................................................................... 2–6

2.2.2 Comparison of computed bed load transport with experimental results .......... 2–7

2.2.3 Effect of particle size on computed bed load transport ................................. 2–7

3 Modelling of suspended sand transport for uniform bed material (single fraction method) ........................................................................................................ 3–1

3.1 Definitions ............................................................................................................... 3–1

3.2 Suspended sand transport processes .................................................................... 3–1

3.2.1 Time-averaged sand concentrations .................................................................. 3–1

3.2.2 Current-related suspended sand transport (q_{sw}) ........................................... 3–5

3.2.3 Wave-related suspended sand transport (q_{sw}) ............................................. 3–5

3.3 Suspended sand transport model .......................................................................... 3–8

3.3.1 Wave-related suspended transport (q_{sw}) .................................................... 3–8

3.3.2 Current-related suspended transport (q_{sw}) ................................................... 3–11

3.3.3 Calibration and verification of current-related suspended transport model ...... 3–13

3.3.4 Effect of particle size on current-related suspended transport ...................... 3–13

4 Modelling of bed load and suspended load transport for graded bed material (multi-fraction method) ......................................................................... 4–1

4.1 Introduction ............................................................................................................. 4–1
4.2 Sorting, hiding and exposure processes ........................................... 4–1

4.3 Multi-fraction method ........................................................................ 4–7

4.3.1 Description of method .................................................................... 4–7

4.3.2 Comparison of computed transport rates based on SF-method and MF-method excluding hiding effects ........................................... 4–8

4.3.3 Comparison of computed transport rates based on SF-method and MF-method including hiding effects ........................................... 4–9

4.4 Effect of graded bed material on bed load transport based on experimental and computational results ........................................... 4–15

4.5 Effect of graded bed material on suspended load transport based on experimental and computational results ........................................... 4–16

5 Conclusions ...................................................................................... 5–1

Appendices

A References

B TRANSPOR-model 1993

C Modified Isobe-Horikawa method for non-linear orbital velocities near the bed
Summary

Coastal management rely increasingly on predictions made by computational numerical models of hydrodynamic and sediment-dynamic processes and the resulting bed level changes, which are derived from the spatial (horizontal) gradients of the depth-integrated sand transport rates. The key element in these models is the formulation of the depth-integrated sand transport as a function of wave and current conditions for a given bed material.

In this report an engineering approach is described to compute the depth-integrated sand transport for combined wave and current conditions in the rippled and flat bed regime of uniform and graded bed materials. The method is based on instantaneous modelling of bed load transport and time-averaged modelling of suspended sand concentrations and associated suspended transport transport.

In 1984 the author proposed simple engineering formulations for the transport of sand in steady flow (Van Rijn, 1984a,b,c). In the period 1986 to 1993 this method was extended to the case of combined steady and oscillatory flow, resulting in the TRANSPOR-model 1993 (Van Rijn, 1993; reproduced in Appendix B of this report).

Since then, more field data sets have become available for calibration purposes, while also some laboratory data sets from large-scale wave tunnels and tanks have been presented in the literature. Based on these data sets, the engineering model has been improved. The focus points are the mixing of suspended sediment due to the presence of ripples and the effect of breaking waves on mixing processes. Furthermore, the model has been extended to include the influence of graded bed material. Special attention is given to the wave-related suspended sediment transport by the high-frequency oscillatory wave motion (short waves). Sediment transport by low-frequency waves is not addressed.

The main findings of the study are:

• bed-load transport model results show good agreement with laboratory and field data for sand in the range of 0.2 to 1 mm (uniform and graded bed material) in the flat bed regime without adjustment of model coefficients, which can be seen as an independent validation of the bed load transport model for the sheet flow regime;

• the measured bed load transport increases (factor 2) with particle size for sand in the range between 0.13 and 0.21 mm, but decreases again (factor 2) for sand larger than 0.32 mm up to 0.97 mm; the effect of particle size on bed load transport is correctly represented for sand of 0.13, 0.21 and 0.97 mm (compared to measured bed load transport); the computed bed load transport for sand of 0.32 mm is too large (about 50%);

• the direction of the net bed load transport is affected by the magnitude and direction of the steady current (if present) in relation to the strength of the wave asymmetry; a following current intensifies the net transport rate, but an opposing current may change the direction of the net transport into that of the current, if the strength of the opposing current is sufficiently large (>0.15 m/s); the net transport is opposed to the wave direction when the opposing current is larger than about 10% to 15% of the onshore peak orbital velocity (U_{current}/U_{oc}>0.10 to 0.15); these effects are correctly represented by the bed-load transport model;
• based on measured data of large-scale flume experiments with sand of 0.16 mm and 0.33 mm and ripples along the bed (ripple regime); the high-frequency wave-related suspended transport was found to be onshore-directed (in wave direction) in all tests with irregular waves; this transport component increases with increasing significant wave height, but decreases with decreasing particle size; this latter effect can be understood from the ripple dimensions; the ripples generated on the 0.33 mm sand bed were much more pronounced than those on the 0.16 mm sand resulting in larger vortex motions and stronger associated suspension processes; the data were used to calibrate the formulations of the wave-related suspended transport in the ripple regime; the phase-lag function involved appeared to be constant;
• the application of the multi-fraction method (for graded bed material) to compute the sand transport rate yields considerably larger values than the single fraction method, if the hiding-exposure factor (hiding of smaller particles between larger particles resting or moving on the bed) is neglected; the wider the size distribution of the bed material, the larger the effect; the modelling of the hiding factor has a large effect on the suspended transport rate for relatively small bed-shear stresses (low waves and weak currents); the proper formulation of the hiding factor is not quite clear from theoretical point of view; experimental data are necessary to evaluate the available concepts.

Limitations of the present model are:
• sand transport by oscillatory flow (with or without a weak steady flow <0.1 m/s) is assumed to be a quasi-steady process (no major phase lags), which means that the model formulations are less accurate for bed material with $d_{50}$ < 0.2 mm;
• sand transport by low-frequency wave motion ($T > 20$ s) is not modelled

The performance of the bed load transport model could not be evaluated for the ripple regime, because of lack of data. It is noted that the field data set of Egmond, which has been used in this study, may contain some data in the ripple regime.

The results of the current-related suspended transport model have not yet been compared with laboratory and field data. This will be done in future research by comparing measured and computed sand concentration profiles and suspended transport rates. Extended verification of the wave-related suspended transport model in the ripple regime is also necessary. Furthermore, the prediction of bed form dimensions and associated bed roughness should be studied.

The results presented in this report may be of interest to researchers of sand transport processes, users of morphological models and those interested in sand transport. Coastal managers are recommended to read the sections on the description of sand transport processes based on the analysis of laboratory and field data (1.1, 1.2, 2.1, 3.2 and 4.2).
I Introduction

1.1 Sand transport processes

Coastal management rely increasingly on predictions made by computational numerical models of hydrodynamic and sediment-dynamic processes and the resulting bed level changes, which are derived from the spatial (horizontal) gradients of the depth-integrated sand transport rates. The key element in these models is the formulation of the depth-integrated sand transport as a function of wave and current conditions for a given bed material. Understanding and modelling of sand transport processes in combined wave and current conditions are the objectives of this report.

Sand can be transported by wind-, wave-, tide- and density-driven currents (current-related transport), by the oscillatory water motion itself (wave-related transport), as caused by the deformation of short waves under the influence of decreasing water depth (wave asymmetry) or by a combination of currents and short waves. The waves generally are acting as sediment stirring agents; the sediments are transported by the mean current. Low-frequency waves (bound long waves) interacting with short waves may also contribute to the sediment transport process. Low-frequency waves such as bound long waves and surf beat may have a specific role in the shoreface and surf zone. Although the associated net transport directions are partly unknown, these types of waves may be of importance for the position of the inner breaker bars. The net direction of this type of transport depends largely on the phase differences between near-bed current velocities and sand concentrations.

In friction-dominated deeper water outside the breaker (surf) zone the transport process is generally 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) and larger bed structures (dunes, bars). Bed-load transport is dominating in areas where the mean currents are relatively weak in comparison to the wave motion (small ratio of depth-averaged velocity and peak orbital velocity). According to classical concepts, there is net onshore-directed bed-load transport under asymmetric non-breaking waves. Suspension of sediments can be caused by ripple-related vortices. Suspended load transport will become increasingly important with increasing strength of the tide- and wind-driven mean current due to the turbulence-related mixing capacity of the mean current (shearing in boundary layer). By this mechanism the sediments are mixed up from the bed-load layer to the upper layers of the flow.

In the surf zone of sandy beaches the transport generally is dominated by the waves through wave breaking and wave-induced currents in longshore and cross-shore direction. The breaking process as well as the near-bed wave-induced oscillatory water motion can bring relatively large quantities of sand into suspension (stirring) which can be transported as suspended load by net (wave-cycle averaged) currents such as tide-, wind- and density-driven currents. The longshore transport in the surf zone is also known as the longshore drift.
cross-shore direction the generation of a net near-bed return current (undertow) balancing the onshore mass flux between the wave crest and trough, may lead to a net offshore drift of sediment.

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

The nature of the sea bed (plane or rippled bed) has a fundamental role in the transport of sediments by waves and currents. The configuration of the sea bed controls the near-bed velocity profile, the shear stresses and the turbulence and thereby the mixing and transport of the sediment particles. For example, the presence of ripples reduces the near-bed velocities, but it enhances the the bed-shear stresses, turbulence and the entrainment of sediment particles resulting in larger overall suspension levels. Several types of bed forms can be identified, depending on the type of wave-current motion and the bed material composition. Focussing on fine sand in the range of 0.1 to 0.3 mm, there is sequence with the generation of rolling grain ripples to vortex ripples and finally to upper plane with sheet flow for increasing bed-shear. Rolling grain ripples are low relief ripples that are formed just beyond the stage of initiation of motion. These ripples are transformed into more pronounced vortex ripples due to the generation of sediment-laden vortices formed in the lee of the ripple crests under increasing wave motion. The vortex ripples are washed out under large storm waves (in shallow water) resulting in plane bed sheet flow characterised by a thin layer of large sediment concentrations.

1.2 Definitions

Depth-integrated sand transport is herein defined to consist of:
- bed load transport, which is the transport of sand particles in the wave boundary layer (thickness of about 0.01 m) in close contact with the bed surface, as observed in wave tunnel experiments (Ribberink, 1998);
- suspended load transport, which is the transport of sand particles above the bed load layer (of about 0.01 m).

The suspended load transport can be determined by depth-integration of the product of sand concentration and fluid velocity from the top of the bed load layer (at about 0.01 m above the bed) to the water surface.

Herein, the net (averaged over the wave period) total sediment transport is obtained as the sum of net the bed load \( q_b \) and net suspended load \( q_s \) transport rates, as follows:

\[ q_{\text{tot}} = q_b + q_s \]  \hspace{1cm} (1.1)

For practical reasons the suspended transport will be subdivided in current-related and wave-related transport components, which are more precisely defined in Section 3.1. This
division is necessary to study the effect of phase differences between velocity and suspended sediment. For bed load transport of particles larger than about 0.2 mm such as approach is not required, because there is an almost instantaneous response of bed load concentrations to near-bed velocity (Ribberink, 1998). All these aspects are considered hereafter.

1.3 Objectives and assumptions

Understanding and modelling of sand transport processes in a unified way is extremely complicated. Many attempts have been done to model both instantaneous and time-averaged velocities and sediment concentrations using detailed mathematical formulations as well as simple engineering methods. Given the complexity of the problem, all models strongly rely on calibration of coefficients using laboratory and field data sets.

In this report an engineering approach is described, based on instantaneous modelling of bed load transport and time-averaged modelling of suspended sand concentrations and associated suspended transport transport for both rippled bed and plane bed conditions. The development of an engineering model implies that the complicated physics involved is modelled by formulations that are relatively simple but yet sufficiently accurate to represent the basic features of the near-bed particle motion and entrainment of particles into suspension.

In 1984 the author proposed simple engineering formulations for the transport of sand in steady flow (Van Rijn, 1984a,b,c). In the period 1986 to 1993 this method was extended to the case of combined steady and oscillatory flow, resulting in the TRANSPOR-model 1993 (VanRijn, 1993; reproduced in Appendix B of this report). TRANSPOR 1993 computes the bed-load transport and the current-related suspended load transport for uniform bed material in case of combined current and wave conditions over both rippled and flat beds.

The TRANSPOR 1993 sand transport model has been parameterized into two simple expressions for bed load and suspended load transport under steady flow conditions, which have been implemented in the DELFT2D model system. This parameterized version for steady flow has been adjusted for combined wave-current conditions by Soulsby (1997) and has recently been implemented in the DELFT 2D system. Furthermore, the TRANSPOR 1993 sand transport model is being used in the cross-shore model UNIBEST 2.0. The weak points of TRANSPOR 1993 are:

- underestimation of time-averaged sand concentrations and suspended transport for low waves in the ripple regime, because ripple-related mixing is not accurately modelled;
- underestimation of time-averaged sand concentrations and suspended transport for breaking wave conditions, because this effect was not yet taken into account;
- exclusion of wave-related suspended sand transport;
- exclusion of graded bed material.

As high-quality field data were lacking at that time, it was not possible to calibrate the model over the full range of wave conditions from small non-breaking waves to relatively large breaking waves. Since then, more field data sets have become available for calibration purposes, while also some laboratory data sets from large-scale wave tunnels and tanks have been presented in the literature (see Ribberink, 1998). Based on these data sets, the engineering model has been improved, which will be described in this report. The focus
points are the mixing of suspended sediment due to the presence of ripples and the effect of breaking waves on mixing processes. Furthermore, the model has been extended to include the influence of graded bed material. Special attention is given to the wave-related suspended sediment transport by the high-frequency oscillatory wave motion (short waves). Sediment transport by low-frequency waves is not addressed. Summarizing, the model improvements since 1993 are:

- effect of ripples on (near-bed) reference concentration,
- effect of ripples on sediment mixing and hence on time-averaged concentration profiles and suspended transport,
- effect of wave breaking on sediment mixing and hence on time-averaged concentration profiles and suspended transport,
- effect of wave-related suspended sand transport,
- effect of graded bed material (multi-fractions),
- calibration and verification of model based on large-scale wave tunnel, wave flume and field data sets.

The basic assumptions of the present research are:

- sand transport by oscillatory flow (with or without a weak steady flow <0.1 m/s) is assumed to be a quasi-steady process (no major phase lags), which means that the model formulations are less accurate for bed material with $d_{50} < 0.2$ mm;
- sand transport by oscillatory flow (with or without a weak steady flow <0.1 m/s) in the ripple regime is largely unknown, because large-scale laboratory and field data are lacking; limited attention is given to this regime, although a first attempt is made to model wave-related suspended transport in the ripple regime;
- sand transport due to low-frequency wave motion ($T > 20$ s) is not modelled.

This research report presents the description of the improved model formulations; the description of data sets available for model calibration and the validation of the bed-load transport model for uniform and graded bed material in the ripple and flat bed regime. The calibration and validation of the suspended transport model will be described later (future work).

The results presented in this report are of interest to researchers of sand transport processes, users of morphological models and those interested in sand transport. Coastal managers are recommended to read the sections on the description of sand transport processes based on the analysis of laboratory and field data (1.1, 1.2, 2.1, 3.2 and 4.2).

1.4 Acknowledgements

This work is jointly funded by the MAST-3 SEDMOC project (Sediment Transport Modelling in Marine Coastal Environments) in the framework of the EU-sponsored Marine Science and Technology Programme (MAST-3) under contract MAS3-CT97-0115, the Dutch Ministry of Transport and Public Works (Rijkswaterstaat) under contract no. RKZ-725 (Part III) and the Scientific Research Programme of Delft Hydraulics.
2 Modelling of bed-load transport for uniform bed material (single fraction method)

2.1 Bed load transport processes

The study of bed load transport in coastal conditions (combined waves and current) is rather problematic, because reliable instruments for measuring bed load transport in field conditions are hardly available. Therefore, the bed load transport under waves and current generally is studied through experiments in wave tunnels and tanks (for overview, see Ribberink, 1998).

Herein, the experimental results of Ribberink and coworkers (based on tests in the large-scale wave tunnel of Delft Hydraulics) are used to demonstrate the most important aspects of bed load transport. A selection of the experimental results are given in Table 2.1 (Ribberink, 1998; Dohmen-Janssen, 1999; Hassan et al., 1999). Some field data obtained along the beach of Egmond (The Netherlands, Wolf, 1997) using a mechanical bag-type sampling instrument, have also been used, see Table 2.2.

Ribberink (1998) and coworkers performed experiments on bed load transport under sheet flow conditions in a large-scale wave tunnel (water temperature between 15 and 20 C). The experiments concern regular symmetric and asymmetric waves (2$^{nd}$ Stokes) with and without a steady current (following and opposing) over a sand bed (almost uniform sand) with diameters in the range of 0.13 to 0.32 mm. The measured net sand transport rates predominantly consist of bed-load transport in the sheet flow layer; a minor amount (10\% to 20\%) of suspended load transport may be included as shown by Grasmeijer et al.,(1999) based on detailed measurements with an optical instrument. Therefore, the sand transport in the sheet flow regime is herein termed bed-load transport. The thickness of the bed-load layer (sheet flow layer) is herein assumed to be of the order the thickness of the wave boundary layer (about 0.01 to 0.02 m). The net time-averaged bed-load transport in the middle section of the wave tunnel is derived from the sand volumes collected in the traps on both ends of the wave tunnel and the volume changes of the sand bed (based on bed level soundings before and after each test). Ribberink (1998) has shown that the bed-load transport in the sheet flow regime near the bed can be represented as a quasi-steady process for sand particles larger than about 0.2 mm, which means that the instantaneous sand transport is proportional to some power of the instantaneous near-bed velocity or better shear stress. Serious non-steady effects have been observed during some experiments in the wave tunnel with fine sand of 0.13 mm in the upper sheet flow regime (2$^{nd}$ Stokes waves with $U_{peak}$, max of about 1.5 m/s). Phase -lag effects between near-bed velocities and concentrations resulted in negative sand transport against the wave propagation direction. These experimental results have not been used in the present study.
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<th>Peak forward orbital velocity (m/s)</th>
<th>Peak backward orbital velocity (m/s)</th>
<th>Wave period (s)</th>
<th>Grain size, d50 (m)</th>
<th>Angle between wave and current (°)</th>
<th>Measured net bed load transport (kg/s/m)</th>
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Remarks:
1. positive = onshore direction (largest peak velocity)
   negative = offshore direction (smallest peak velocity)
2. regular asymmetric waves; peak velocities are exclusive mean velocity
3. mean current velocity is measured at 0.1 m above the bed

Table 2.1 Net bed-load transport in sheet flow regime in large wave tunnel of Delft Hydraulics (Ribberink, 1998, Dohmen-Janssen, 1999 and Hassan et al., 1999)
Figure 2.1

Top: Net bed load transport as function of peak orbital velocity (0.13 and 0.21 mm)
Middle: Net bed load transport as function of mean current velocity (0.21 mm)
Bottom: Net bed load transport as function of particle size (0.13, 0.21, 0.32, 0.97 mm)
A selection of the basic data of the tests is given in Table 2.1 and is shown in Figure 2.1. Based on Figure 2.1 Top, the net bed load transport in the sheet flow regime is found to have the same direction as the largest peak orbital velocity near the bed and its magnitude is proportional to power 3.5 of the peak forward (onshore) orbital velocity. Figure 2.1 Middle shows that the direction of the net bed load transport is affected by the magnitude and direction of the steady current (if present) in relation to the strength of the wave asymmetry. A following current intensifies the net transport rate, but an opposing current may change the direction of the net transport into that of the current, if the strength of the opposing is sufficiently large (>0.15 m/s). For these experiments the net transport is opposed to the wave direction when the opposing current is larger than about 10% to 15% of the onshore peak orbital velocity ($U_{current}/U_{on}$>0.10 to 0.15). In other cases this ratio may be different, because it depends on the asymmetry of the orbital velocity. Figure 2.1 Bottom demonstrates that the net sand transport rate is affected by the particle size. The transport rate increases by almost a factor 2 for particle size increasing from 0.13 mm to 0.21 mm. A further increase of the particle size from 0.21 mm to 0.97 mm seems to give a decrease of the transport rate by a factor of 2. All tests were in the flat bed regime with exception of the test with 0.97 mm sand. During this latter test relatively large ripples were present with height of about 0.2 m and length of about 1.1 m.

Wolf (1997) using a bag-type sampler, performed bed-load transport measurements in the surf zone near Egmond (The Netherlands). Simultaneously, water level recordings and flow velocity measurements were carried out. The local bed material consisted of sand with $d_{50}$=0.3 mm. The bed-load sampler, (width= 0.094 m, inside height=0.044 m) was attached to a steel rod to position the sampler on the bed. It was operated manually from a small platform in the surf zone at depths between 1 and 2 m. Given the dimensions of the sampler, the transport of sand particles in a layer of about 0.04 m is measured. This transport is herein termed bed-load transport, although some part of the suspended load transport will be included. Each bed-load measurement had a duration of about 3 minutes. After that the sampler was raised and the sand content was removed from the bag and another measurement was done. The opening of the sampler was alternatingly positioned in onshore and offshore direction. The transport rates were averaged over periods of 25 to 40 minutes. The standard deviation of the transport rates collected during these periods was about 20%. Based on calibration in the wave tunnel of Delft Hydraulics (Van der Lee, 1994), the bag-type bed load sampler was found to have an efficiency of about 0.5 (undersampling!). To account for this, the measured data were multiplied with a factor of 2. Another uncertainty is the inclusion of an unknown portion of the suspended load transport close to the bed. Given these uncertainties, the measured transport rates should be seen as 'order of magnitude' estimates. A selection of the dataset of Wolf (1997) is given in Table 2.2.

Figure 2.2 shows the net cross-shore bed load transport as a function of the ratio of the undertow velocity and the peak onshore orbital velocity for the Egmond data. A reasonably clear correlation can be observed; the net transport is offshore-directed when the offshore-directed undertow is larger than about 15% of the onshore peak orbital velocity ($U_{under}/U_{1/3, on}$>0.15). For other conditions the net transport is onshore-directed. This finding seems to be in agreement with the results of the wave tunnel experiments (Figure 2.1 Middle).
<table>
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<th>h (m)</th>
<th>$H_s$ (m)</th>
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<th>$u_{CR}$ (m/s)</th>
<th>$U_{1/3,om}$ (m/s)</th>
<th>$U_{1/3,off}$ (m/s)</th>
<th>$d_{50}$ (mm)</th>
<th>$d_{30}$ (mm)</th>
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</tbody>
</table>

$U_{1/3,om} =$ significant peak onshore orbital velocity  
$U_{1/3,off} =$ significant peak offshore orbital velocity  
+ = onshore-directed, - = offshore-directed

Table 2.2 Net cross-shore bed-load transport in surf zone of Egmond beach, The Netherlands

![Figure 2.2](image-url)  
Figure 2.2 Net cross-shore bed-load transport at Egmond beach, The Netherlands
2.2 Bed load transport model

2.2.1 Formulations

The net bed-load transport rate is computed by time-averaging (over the wave period) of the instantaneous transport rate using a bed-load transport formula (quasi-steady approach), as follows:

\[ q_b = \frac{1}{T} \int q_{bt} \, dt \]  \hspace{1cm} (2.1)

with \( q_{bt} \) = \( F \) (instantaneous hydrodynamic and sediment transport parameters), Van Rijn, 1993, see Appendix B.

The applied bed-load transport formula is a parametrization of a detailed grain saltation model representing the basic forces acting on a bed-load particle for steady flow (Van Rijn, 1984a, 1993). This approach is generalized to the regime of combined current and wave conditions by using the concept of the instantaneous bed shear stress. The instantaneous bed-load transport rate (kg/s/m) is related to the instantaneous bed-shear stress, which is based on the instantaneous velocity vector (including both wave and current-related components) defined at a small height above the bed. The formula applied, reads as:

\[ q_b = 0.25 \gamma \rho \cdot d_{so} D_s^{-0.3} \left[ \frac{\tau_{b,cw}'}{\rho} \right]^{0.5} \left[ \frac{\tau_{b,cw}'}{\tau_{b,cr}} \right]^{1.5} \]  \hspace{1cm} (2.2)

in which: \( \tau_{b,cw}' \) = instantaneous grain-related bed-shear stress due to currents and waves; \( \rho \cdot f_{cw} (U_{b,cw})^2 \), \( U_{b,cw} \) = instantaneous velocity due to currents and waves at edge of wave boundary layer, \( f_{cw} \) = grain friction coefficient due to currents and waves; \( \alpha \beta f_c' + (1-\alpha) f_w ' \), \( f_c = \) current-related grain friction coefficient (based on \( k_{x,grain} = 3d_{so} \)), \( f_w ' = \) wave-related grain friction coefficient (based on \( k_{x,grain} = 3d_{so} \)), \( \alpha = \) coefficient related to relative strength of wave and current motion, \( \beta = \) wave-current-interaction coefficient (Appendix B), \( \tau_{b,cr} \) = critical bed-shear stress according to Shields, \( \rho_s \) = sediment density, \( \rho \) = fluid density, \( d_{so} \) = particle size, \( D_s = \) dimensionless particle size, \( \gamma = 1 - (H_i/h)^{0.5} \) = calibration coefficient (\( \gamma_{min} = 0.3 \)) based on wave tunnel data, \( H_i = \) significant wave height, \( h = \) water depth.

It is assumed that the bed-load transport only depends on grain roughness and not on bed form roughness. Research on sand transport in the ripple regime is necessary to get a better understanding of ripple effects.

Proper predictive modelling of the oscillating transport components basically requires an accurate description of the near-bed orbital fluid velocity, especially in conditions with shoaling and breaking waves (non-linear wave motion). So far, these effects have been described by using linear wave theory with an empirical correction factor derived from measured data at Egmond beach, The Netherlands (Van Rijn, 1993, see Appendix B). Recently, this method has been replaced by the modified Isobe-Horikawa method (Grasmeijer and Van Rijn, 1998), which is described in Appendix C.
2.2.2 Comparison of computed bed load transport with experimental results

The bed-load transport model has been compared with experimental results from the large wave tunnel of Delft Hydraulics (Table 2.1). The observed peak orbital velocities were represented by assuming a water depth of 5 m and a wave height that gave the correct peak orbital velocities (as measured in the wave tunnel). The bed roughness was taken as $k_{s,c}=k_{s,w}=0.01$ m. The $d_{50}$ of the sand material (almost uniform sand) was taken as $2d_{50}$. The water temperature was assumed to be 15 °C. The longshore current was set to 0 m/s. The depth-averaged current was set to a value that gave the correct velocity (see Table 2.1) at 0.1 m above the bed. The computed and measured bed-load transport rates are shown in Figure 2.1. Nearly all computed values are within 50% of the measured values. The bed load transport model also gives the correct transport direction, see Figure 2.1 Middle. The effect of particle diameter on bed load transport is correctly represented for $d_{50}=0.13, 0.21$ mm and 0.97 mm (see Fig. 2.1 Bottom). The computed bed load transport for $d_{50}=0.32$ mm is however considerably too large (about 60%)

Equation (2.2) was also used to compute the net bed-load transport rates measured at the Egmond beach (Table 2.2). The measured undertow velocity and the measured peak onshore and offshore orbital velocities were used as input data. Other input data were: bed roughness=0.01 m, water temperature= 10 °C. The computed values are shown in Figure 2.2; most computed results are within a factor 2 of the measured values, which is a rather encouraging result.

2.2.3 Effect of particle size on computed bed load transport

To demonstrate the effect of particle size on the bed load transport and suspended load transport, sensitivity computations for particle diameters in the range between 0.1 and 3 mm have been made for three hydrodynamic regimes: low-energy, medium-energy and high-energy conditions using a water depth $h= 5$ m, $d_{50}= 2d_{50}$, temperature= 15 °C and salinity = 30 promille. The wave height, current velocity and bed roughness are, as follows:

**L-E:** $H_{sig}= 1$ m, $T_p= 7$ s, $v_{long}= 0$ m/s, $u_{cross}= 0$ m/s, $k_{s,c}=k_{s,w}=0.05$ m;
**M-E:** $H_{sig}= 1.5$ m, $T_p= 7$ s, $v_{long}= 0.1$ m/s, $u_{cross}= 0$ m/s, $k_{s,c}=k_{s,w}=0.03$ m;
**H-E:** $H_{sig}= 3$ m, $T_p= 7$ s, $v_{long}= 1$ m/s, $u_{cross}=-0.5$ m/s, $k_{s,c}=k_{s,w}=0.01$ m;

The longshore and cross-shore currents were assumed to increase for increasing wave energy. The bed form related roughness ($k_{s,c}$ and $k_{s,w}$) was assumed to decrease for increasing wave energy (simulating the transition from the ripple to the sheet flow regime). The bed form related roughness only affects the suspended load transport.

The bed load transport as a function of particle diameter is shown in Figure 2.3. The effect of particle diameter on the suspended load transport is discussed in Section 3.3.4. The results are:
• **low-energy event:** bed-load transport is dominant; bed-load transport increases with increasing particle diameter up to about 0.6 mm and decreases with particle diameter for values larger than about 0.8 mm;

• **high-energy event:** bed-load transport increases with particle diameter up to 0.6 mm and decreases for values larger than 0.8 mm; bed load transport is of minor importance;

• **medium-energy event:** bed-load transport is dominant except for \( d_{50} = 0.1 \) mm and increases with particle size up to 0.6 mm and decreases with particle diameter for values larger than 0.8 mm.

To some extent, these results are in agreement with the particle size trend of Figure 2.1 Bottom. The model results show however a maximum bed load transport rate for a particle size of about 0.6 mm, whereas the tunnel data suggest a value of about 0.3 mm. The effect of particle diameter on suspended transport is discussed in Section 3.3.4.
Figure 2.3  Effect of particle size on computed transport rate
3 Modelling of suspended sand transport for uniform bed material (single fraction method)

3.1 Definitions

The net suspended sand transport is defined as the sum of the net current-related \( q_{c,c} \) and the net wave-related \( q_{w,w} \) transport components, as follows:

\[
q_b = q_{c,c} + q_{w,w} = \int vc \, dz + \int \langle (V-v)(C-c) \rangle \, dz
\]  

(3.1)

in which: \( q_{c,c} \) = 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 velocity, \( V \) = instantaneous velocity vector, \( C \) = instantaneous concentration and \( c \) = time-averaged concentration and \( \langle \cdot \rangle \) averaging over time, \( \int \) the integral from the top of bed-load layer to the water surface.

The current-related suspended transport \( q_{c,c} \) is defined as the advective transport of sediment particles by the time-averaged (mean) current velocities (longshore currents, rip currents, undertow currents). Thus, the transport of sediment which is carried by the steady flow. In case of waves that are superimposed on the current both the current velocities and the sediment concentrations will be affected by the wave motion. It is known that the wave motion reduces the current velocities near the bed, but the near-bed concentrations are strongly increased due to the stirring action of the waves. These effects are included in the current-related transport.

The wave-related suspended sediment transport \( q_{w,w} \) is defined as the transport of sediment particles by the high-frequency oscillating fluid components (cross-shore orbital motion). Low-frequency transport contributions are herein neglected.

For practical reasons the current-related and the wave-related transport components are studied separately. This allows the evaluation of the relative magnitude of both components, which is of significant importance for modelling purposes.

3.2 Suspended sand transport processes

3.2.1 Time-averaged sand concentrations

Data measured in a large-scale wave flume (Delta flume of Delft Hydraulics) and in the surf zone of Egmond beach, The Netherlands have been used to study the vertical distribution of time-averaged sand concentrations. The basic data are given in Tables 3.1 and 3.2. Some examples of measured sand concentration profiles from experiments in the Delta flume of DH are shown in Figures 3.1 to 3.4. The basic features will be discussed later.
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<th>$d_{s}$</th>
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<th>$q_{w,s,CR}$</th>
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Table 3.1 Summary of wave-related suspended sand transport data for large-scale wave flume (Delta flume of Delft Hydraulics); $d_s$ and $k_{s,w}$ are estimated; current-related transport rates are not reported.

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<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td>0.35</td>
<td>-0.25</td>
</tr>
<tr>
<td>4C</td>
<td>1.55</td>
<td>0.6</td>
<td>5.5</td>
<td>0.35</td>
<td>-0.1</td>
<td>0.35</td>
<td>0.8</td>
<td>0.28</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td>0.05</td>
<td>-0.015</td>
</tr>
<tr>
<td>4D</td>
<td>1.9</td>
<td>0.3</td>
<td>5.5</td>
<td>0.1</td>
<td>-0.03</td>
<td>0.35</td>
<td>0.8</td>
<td>0.28</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td>0.006</td>
<td>-0.003</td>
</tr>
<tr>
<td>4E</td>
<td>1.3</td>
<td>0.85</td>
<td>7.3</td>
<td>0.2</td>
<td>-0.1</td>
<td>0.35</td>
<td>0.8</td>
<td>0.28</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td>0.3</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

Angle between waves and current $\varphi=90^\circ$; Temperature=10 °C; Salinity= 30 promille

$h$ = water depth, $H_s$ = significant wave height, $T_p$ = peak wave period

$u_L$ = depth-mean longshore current velocity

$u_{CR}$ = depth-mean undertow velocity (- offshore, + onshore)

$d_s$ = representative suspended sand size (estimated)

$k_{s,w}$ = wave-related bed form roughness (estimated)

$k_{s,t}$ = current-related bed form roughness (estimated)

$q_{l,s}$ = current-related longshore suspended sand transport

$q_{w,s,CR}$ = current-related cross-shore suspended sand transport (- offshore, + onshore)

$q_{w,s,CR}$ = wave-related (high freq.) cross-shore suspended sand transport (- offshore, + onshore)

$\Delta_p$ = bed form height (pl = plane bed), $\lambda_p$ = bed form length

Table 3.2 Summary of current-related suspended sand transport data at Egmond surf zone, The Netherlands (Kroon, 1994; Wolf 1997); wave-related transport rates are not available.
Figure 3.1  Measured sand concentration profiles for Tests 1A and 1B

Figure 3.2  Measured sand concentration profiles for Tests 1C, 1D and 1E
Figure 3.3  Measured sand concentration profiles for Tests 2A to 2F

Figure 3.4  Measured sand concentration profiles for Tests 2H and 2I
3.2.2 Current-related suspended sand transport ($q_{sc}$)

Information of this transport component can be obtained from the field data of Egmond measured by Kroon (1994) and Wolf (1997). Each data point represents a certain class of conditions indicated by the variation ranges. Figure 3.5 shows depth-integrated suspended transport rates as a function of depth-averaged longshore velocity. The relative wave height has been used to group the data; three ranges are given: $H_s/h=0.15-0.23$, 0.3-0.4 and 0.5-0.65. The transport rates show a considerable increase for breaking wave conditions with $H_s/h$ larger than 0.4. The transport rate increases with depth-mean velocity to the power 2.5.

![Figure 3.5: Depth-integrated suspended transport rates as a function of depth-averaged longshore velocity.](image)

3.2.3 Wave-related suspended sand transport ($q_{sw}$)

Information of this transport component can be obtained from the field data collected at the Dutch coast. Kroon (1994) and Wolf (1997) measured instantaneous velocities and sand concentrations in the inner surf/swash zone (water depths between 0.5 and 1.5 m) at the beach site of Egmond. Houwman and Ruessink (1996) performed similar measurements in the shoreface zone and in the surf zone (water depths between 4 and 9 m) at the site of Terschelling. At both sites the sediments are in the range between 0.15 and 0.3 mm. Analysis of the available data reveals that the $q_{sw}$-component generally is onshore-directed near the bed and significant compared to the (offshore-directed)$q_{sc}$-component.

Typical values are:
- In the inner surf/swash zone: $|q_{sw}|=0.2$ to 0.3 $|q_{sc}|$
- In the shoreface and surf zone: $q_{sw}=0.5$ to 1 $q_{sc}$
Experiments in the large-scale Delta flume (length = 233 m, depth = 7 m, width = 5 m) of Delft Hydraulics have been done to study the wave-related suspended transport under controlled conditions (Chung et al., 1999; Chung and Grasmeijer, 1999). The experimental conditions (ripple regime) are given in Table 3.1.

A horizontal sand bed layer was placed in the wave flume from position x = 100 meters to x = 140 meters. The water depth was about 4.5 m in all experiments. Two types of sand have been used: fine sand with median diameter of 0.16 mm and coarse sand with median diameter of 0.33 mm. The experimental set-up is two dimensional, but local processes are three dimensional due to the generation of ripples on the bed. Irregular waves with a single-topped spectrum were generated in the flume. The significant wave heights varied in the range of H_s = 1 to 1.5 m. The peak wave period was 5 s.

An acoustical instrument (ASTM) was used to measure the instantaneous fluid velocities and sand concentrations at five points above the bed simultaneously. The measurement levels above bed were (most tests): z = 0.075; 0.125; 0.225; 0.475 and 1.075 m above bed. The precise position of the instrument sensors with respect to the ripple crests could not be measured. Two pump sampling systems (2x5 intake nozzles) located along the flume wall (intake nozzles at about 0.3 m from the wall) and close to the ASTM-instrument were used to measure time-averaged sand concentrations. A sand ripple profiler of Proudman Oceanographic Laboratory (UK) was used to measure bedform dimensions. Bed forms were also inspected by visual observation after drainage of the flume. The ASTM-instrument and pump nozzles were mounted in a tripod, which was placed on the horizontal sand bed at location x = 125 m. During each test the instruments were operated for about 15 minutes to sample over a representative wave record. Each test was repeated many times to include the effect of the (migrating) ripples on the instantaneous sand concentrations and fluid velocities. The sampling records had a maximum duration of about 2 hours. In all, 35 tests have been done, which have been grouped to 5 data sets (Table 3.1).

The measured instantaneous velocity and sediment concentration at each level above the bed have been separated into time-averaged, high frequency and low frequency components. Using these parameters, the various suspended transport components have been computed: current-related, high-frequency and low-frequency wave-related and net transport (Chung et al., 1999; Chung and Grasmeijer, 1999). The current-related suspended transport in these experiments are caused by the presence of a very weak net offshore-directed current in the near-bed layer, which is generated due to interaction of the wave-boundary layer hydrodynamics with the rippled bed. Analysis of the results showed that, in general, the high-frequency wave-related transport rates are slightly dominant and tend to be directed onshore. The current-related transport rates are slightly smaller than the high-frequency wave-related transport rates and are directed offshore. The low-frequency wave-related transport rates are of minor importance and have a tendency for offshore direction similar to the current-related transport components. The suspended sediment transport mainly occurs in the near-bed layer with thickness of about 0.3 to 0.5 m, which is roughly equivalent to 10 to 20 times the ripple height.

In order to provide an evaluation of depth-integrated transport rates, the transport terms have been integrated between the lowest and highest measurement points. The results (mean values and standard errors) for all available tests (35) are shown in Figure 3.6.
Figure 3.6  Depth-integrated wave-related and current-related suspended transport as a function of significant wave height and sand size; extrapolated transport rates between \( z = 0.075 \) and \( 0.01 \text{ m} \).

In Figure 3.6 the current-related and the high-frequency wave-related suspended transport rates are shown as a function of wave height and sand size. In all conditions with irregular waves the wave-related suspended transport is directed onshore (in wave direction), whereas the current-related suspended transport is directed offshore. From Figure 3.6 it is easy to see that the wave-related suspended transport increases with wave height and decreases with decreasing particle size. This latter effect can be understood from the ripple dimensions; the ripples generated on the 0.33 mm sand bed were much more pronounced than those on the 0.16 mm sand bed (see Table 3.1) resulting in larger vortex motions and stronger associated suspension processes. The standard error of the wave-related transport is relatively large for the case with sand bed of 0.16 mm and significant wave height of 1.5 m, expressing relatively large variability because only 3 data records of 15 minutes were available (data set V). It stresses the importance of relatively long data sets in case of a rippled bed.

To evaluate the magnitude of the high-frequency suspended transport in the unmeasured layer between \( z = 0.075\text{m} \) and \( 0.01\text{m} \) (the latter value being roughly the edge of bed load layer), the suspended sediment transport terms have been extrapolated from the measured data in the two lowest points using an exponential extrapolation method. It is realized that this type of extrapolation is rather tricky, but the main aim is to get a rough estimate of the
suspended transport in the unmeasured zone. Figure 3.6 shows the measured values between \( z = 0.075 \text{ m} \) and \( 1.075 \text{ m} \) as well as the extrapolated values below \( z = 0.075 \text{ m} \) (down to \( 0.01 \text{ m} \)). Comparison of the magnitudes of measured and extrapolated transport rates shows the following:

- coarse sand with \( d_{so} = 0.33 \text{ mm} \): the extrapolated high-frequency suspended sand transport below \( z = 0.075 \text{ m} \) is about 0.2 times the measured high-frequency transport rate above \( z = 0.075 \text{ m} \) for wave height \( H_i = 1 \text{ m} \) and about 0.7 times the measured transport rate for \( H_s = 1.25 \text{ m} \);

- fine sand with \( d_{so} = 0.16 \text{ mm} \): the extrapolated high-frequency suspended sediment transport below \( z = 0.075 \text{ m} \) is about 1.5 times the measured transport rate above \( z = 0.075 \text{ m} \) for wave height \( H_i = 1 \text{ m} \) and about 0.7 times the measured transport rate for \( H_s = 1.25 \text{ m} \).

Based on these results, it is evident that the high-frequency suspended sand transport rates below \( 0.075 \text{ m} \) down to \( 0.01 \text{ m} \) is an essential part of the total depth-integrated transport rate and can not be neglected. Similar results were obtained for the time-averaged suspended transport rates (not shown). For field measurements this has the consequence that measured values down to \( 0.01 \text{ m} \) above the bed are required to determine accurate values of the total depth-integrated suspended transport rate, which is a challenging task for field workers.

### 3.3 Suspended sand transport model

#### 3.3.1 Wave-related suspended transport \((q_{s,w})\)

Modelling of \( q_{s,w} \) basically requires numerical solution of the time-dependent advection-diffusion equation for suspended sand (unsteady model):

\[
\frac{\partial C}{\partial t} - \frac{\partial [w_i C - (\varepsilon_s \frac{\partial C}{\partial z})]}{\partial z} = 0
\]

(3.2)

with: \( C \) = instantaneous sand concentration (volume), \( w_i \) = fall velocity of suspended sand, \( \varepsilon_s \) =vertical sediment mixing coefficient, \( t \) = time, \( z \) = vertical coordinate.

This unsteady model approach is applied by many researchers (see overview of Dohmen-Janssen, 1999). Using this approach, the instantaneous suspended transport due to combined steady and oscillatory flow can be solved in an integrated way, which is a great advantage of this method. A major drawback of this approach however is that it does not give very accurate results of the wave-related sand transport in the ripple regime (Chung et al., 1999). Another drawback is the relatively large computational time involved, when it is applied in a numerical morphological model system with feed back to changing bed levels and hence hydrodynamics (loop system). To overcome this problem, herein an engineering approach is proposed, which has been found to give sufficiently accurate and efficient results for the available data range.

The engineering method has been introduced by Houwman and Ruessink (1996). The wave-related suspended transport component can be modelled as:
\[ q_{b,w} = \gamma \left[ (U_{on})^2 - (U_{off})^2 \right] \left[ \frac{1}{(U_{on})^2 + (U_{off})^2} \right] \int c \, dz \] (3.3)

with: \( U_{on} = U_{b,f} \) = near-bed peak orbital velocity in onshore direction (in wave direction) and \( U_{off} = U_{b,b} \) = near-bed peak orbital velocity in offshore direction (against wave direction), \( c \) = time-averaged concentration and \( \gamma \) = phase lag function.

Equation (3.3) is based on an instantaneous response of the suspended sediment concentrations (C) and transport \( (q_{b,w}) \) to the near-bed orbital velocity \( (C \propto U^3 \text{ and } q \propto U^4) \). This may be valid for the near-bed layer (say 1 to 5 times the wave boundary layer thickness), but at higher levels a delayed response of the sand concentrations (phase lag effects) will be more realistic, particularly for fine sediments. For very fine sediment the wave-related transport may even be opposite to the wave propagation direction.

Phase lag effects are supposed to be accounted for by the \( \gamma \)-function. As phase lag effects are related to the wave conditions, sand size and bed geometry, the \( \gamma \)-function is supposed to be a complicated function of the former parameters (yielding negative values for very fine sand). A detailed discussion of phase lag effects and functions is given by Dohmen-Janssen (1999).

Computation of the wave-related suspended transport according to Eq. (3.3) requires integration of the time-averaged sand concentrations in vertical direction. Herein, the integration is taken over a near-bed layer with a thickness equal to about 0.5 m, assuming that the suspended sand above this layer is not much effected by the high-frequency wave motion with periods in the range of \( T = 5 \) to 10 s. This assumption is satisfied if the fall time of a suspended sand particle over a distance of 0.5 m is much larger than the wave period \( (T_{fall}=0.5/w_s \text{ yielding about } 25 \text{ s for } d=0.2 \text{ mm with } w_s=0.02 \text{ m/s}) \). Furthermore, the data of the Delta flume (Chung et al., 1999) shows that most of the wave-related suspended transport occurs in the near-bed layer with a thickness of about 0.5 m (10 to 20 times the ripple height).

Chung et al. (1999) and Grasmeijer et al. (1999) have determined the \( \gamma \)-function by fitting of Eq (3.3) to the wave-related transport rates measured in the Delta flume (Table 3.1 and Figure 3.6). The peak onshore and offshore orbital velocities as well as the time-averaged sand concentrations were taken from the measured data. Amazingly, the \( \gamma \)-function was found to be a constant value of about 0.2 for all test results (relative standard error of about 30 %). Any influence of the wave conditions and/or the sand size on the \( \gamma \)-function could not be detected, implying relatively small phase lag effects for the five data sets used.

Proper predictive modelling of the oscillating suspended transport component \( (q_{b,w}) \) requires an accurate description of the near-bed orbital fluid velocity, especially in conditions with shoaling and breaking waves (non-linear wave motion). So far, these effects have been described by using linear wave theory with an empirical correction factor derived from measured data at Egmond beach, The Netherlands (Van Rijn, 1993, see Appendix B). Recently, this method has been replaced by the modified Isobe-Horikawa method (Grasmeijer and Van Rijn, 1998), which is described in Appendix C.
**Relative importance of wave-related suspended sand transport**

To get a better understanding of the relative importance of the three transport components \( q_{b, cw} \) = mean and oscillating bed-load transport, \( q_{h, w} \) = oscillating suspended transport (only high-frequency) and \( q_{s, c} \) = mean current-related suspended transport; some sensitivity computations have been made for uniform bed material of 0.2 mm and a water depth of 5 m (using TRANSPOR 1993). The \( \gamma \)-factor of the \( q_{h, w} \) transport component is taken as 0.3 to obtain values of the right order of magnitude compared to measured data.

Figure 3.7 shows computed values of the three transport components in the cross-shore direction and the net transport in the cross-shore direction as a function of relative wave height \( H_s / h \). The wave period is 7 s. The waves are normal to the coast. The longshore (tidal- or wind-driven) depth-averaged current velocities are taken as 0.7 m/s. The cross-shore undertow is computed according to the expression given in Appendix B. The bed roughness is \( k_{s, c} = k_{s, w} = 0.01 \) m for all conditions. The water temperature is 15 °C, the salinity is 30 promille and \( \gamma = 0.3 \).

The following results can be observed (see Fig. 3.7):

- bed-load transport and wave-related suspended transport are both onshore-directed; osc. suspended transport is a factor 5 to 10 larger than the bed-load transport for 0.2 mm sediment;
- onshore-directed wave-related suspended transport is slightly larger than the offshore-directed current-related suspended transport for \( H_s / h < 0.3 \) resulting in net onshore-directed suspended transport for \( H_s / h < 0.3 \);
- net total transport is onshore-directed for \( H_s / h < 0.3 \); net total transport is offshore-directed for \( H_s / h > 0.3 \).

Since the net total transport in cross-shore direction is a delicate balance of all cross-shore transport components involved, the \( \gamma \)-factor of the wave-related suspended transport has a great influence on the net transport. Furthermore, also the low-frequency transport component (neglected in this study) may have some effect on the cross-shore transport balance.

![Figure 3.7](image)

**Figure 3.7** Cross-shore transport components as a function of relative wave height for 0.2 mm-sediment (water depth = 5 m)
3.3.2 Current-related suspended transport \((q_{sc})\)

Modelling of the current-related suspended transport requires modelling of both the time-averaged velocity profile and the time-averaged sand concentration profile.

The current velocity profile is represented as a two-layer system to account for the wave effects in the near-bed layer (Van Rijn and Kroon, 1992; Van Rijn, 1993; see Appendix B for details).

The time-averaged (over the wave period) advection-diffusion equation is applied to compute the equilibrium time-averaged sand concentration profile in conditions with current-related and wave-related mixing. This equation reads as:

\[
w_{s,m} + \varepsilon_{s,cw} \frac{dc}{dz} = 0
\]  
(3.4)

in which: \(w_{s,m}\) = fall velocity of suspended sediment in a fluid-sediment mixture (m/s), \(\varepsilon_{s,cw}\) = sediment mixing coefficient for combined current and waves (m²/s), \(c\) = time-averaged concentration at height \(z\) above the bed (kg/m³); hindered settling and turbulence damping effects are taken into account.

For combined current and wave conditions the sediment mixing coefficient is modeled as:

\[
\varepsilon_{s,cw} = ((\varepsilon_{s,c})^2 + (\varepsilon_{s,w})^2)^{0.5}
\]  
(3.5)

in which: \(\varepsilon_{s,w}\) = wave-related mixing coefficient (m²/s), \(\varepsilon_{s,c}\) = current-related mixing coefficient (m²/s)

Some parameters of the wave-related sediment coefficient (as described in Appendix B) have been modified. The vertical distribution has not been changed.

\[
\delta_{b} = \max(5\gamma_{br} \delta_{w}, 10\gamma_{br} k_{sw}) \text{ with limits } 0.1 \leq \delta_{b} \leq 0.5 \text{ m}
\]  
(3.6)

with: \(\delta_{b}\) = thickness of effective near-bed sediment mixing layer, \(\delta_{w}\) = thickness of wave boundary layer, \(k_{sw}\) = wave-related bed roughness and \(\gamma_{br} = 1 + (H_{b}/h-0.4)^{0.5}\) = empirical coefficient related to wave breaking \((\gamma_{br} = 1 \text{ for } H_{b}/h \leq 0.4)\)

The wave-related sediment mixing coefficient in the upper half of the water column has been modified into (multiplied by the breaking coefficient \(\gamma_{br}\)):

\[
\varepsilon_{s,w,max} = 0.035 \gamma_{br} h H_{b}/T_{p} \text{ with } \varepsilon_{s,w,\text{max}} \leq 0.05 \text{ m}^2/\text{s}
\]  
(3.7a)

The wave-related sediment mixing coefficient near the bed is described by:

\[
\varepsilon_{s,w,\text{bed}} = 0.004 \delta_{b} D_{s} U_{b}
\]  
(3.7b)

in which: \(D_{s}\) = particle parameter, \(U_{b}\) = near-bed peak orbital velocity.
Numerical solution of the advection-diffusion equation (Eq. 3.4) requires specification of the concentration at a certain elevation above the bed (reference concentration). The reference concentration (volume) is given by:

\[ c_a = 0.015 \frac{d_{50}}{a} \frac{T^{1.5}}{D^{0.3}} \text{ with } c_a \leq 0.05 \text{ (approx. } 130 \text{ kg/m}^3) \]  \hspace{1cm} (3.8)

in which: \( D \) = dimensionless particle parameter (-), \( T \) = dimensionless bed-shear stress parameter (-), \( a \) = reference level (m), \( a \) is taken equal to the bed roughness height \( k_s \) with a minimum value of 0.01 m.

The T-parameter is, as follows:

\[ T = \left( \frac{\tau'_{b,cr}}{\tau_{b,cr}} \right) \]  \hspace{1cm} (3.9)

in which: \( \tau'_{b,cr} \) = time-averaged effective bed-shear stress (N/m²), \( \tau_{b,cr} \) = time-averaged critical bed-shear stress according to Shields (N/m²)

The magnitude of the time-averaged bed-shear stress, which is independent of the angle between the wave- and current direction, is given by (Appendix B):

\[ \tau'_{b,cr} = \mu_c \alpha_{cw} \tau_{bc} + \tau'_{bw} \]  \hspace{1cm} (3.10)

in which: \( \tau'_{bc} = \mu_c \alpha_{cw} \tau_{bc} \) = effective current-related bed-shear stress (N/m²), and \( \tau'_{bw} = \mu_w \tau_{bw} \) = effective wave-related bed-shear stress (N/m²), \( \mu \) = efficiency factor and \( \alpha \) = wave-current interaction factor; grain-related friction factor depends on \( d_{50} \).

The wave-related efficiency factor \( \mu_{w,a} \) (see Appendix B) has been modified into \( \mu_{w,a} = 0.125(1.5 - H_s/h)^2 \) with minimum value of 0.063. This yields a better description of the reference concentration for relatively small wave heights in the ripple regime.

Important parameters for the suspended load transport are the current-related and the wave-related bed form roughness (\( k_{sc} \) and \( k_{sw} \)). These parameters are directly related to the size and geometry of the bed forms (ripples). The wave-related bed form roughness is also related to the near-bed orbital excursion. If the ripple length is much larger than the orbital excursion, the wave-related bed form roughness is relatively small because the ripple-related vortices will be relatively weak. At present stage of research both parameters (\( k_{sc} \) and \( k_{sw} \)) are used as input parameters. Research is necessary to get better estimates of these parameters. Basically, bed form and bed roughness predictors are required.

For morphological computations, the effect of the local bed slope on the transport rate should be taken into account. This is done, as follows (see Appendix B):

- multiplying the critical bed-shear stress with the Schoklitsch-factor:
  \( k_f = \sin (\phi + \beta)/\sin(\phi) \), in which \( \phi \) = dynamic friction angle (\( \tan(\phi) \) is about 0.6) and \( \beta \) = local
slope angle; the angle $\beta$ is positive for uphill transport yielding $k_s > 1$ and hence an increase of the critical bed-shear stress and a decrease of the transport rate; $\beta$ is negative for downhill transport;

- multiplying the net bed-load and suspended load transport with the Bagnold factor $k_s = \frac{1}{1 + \tan \beta / \tan \phi}$; $\tan \beta$ is positive for uphill transport yielding $k_s < 1$ and hence a decrease of the transport rate; $\tan \beta$ is negative for downhill transport.

Basically, the Bagnold-factor should be applied to the instantaneous transport rates within the wave cycle and not to the net time-averaged transport rate. The former approach may lead to a rather strong effect of the bed slope on the net bed-load transport. This is caused by the fact that the net bed-load transport is the difference of two large transport quantities related to the forward and backward phases of the wave cycle.

### 3.3.3 Calibration and verification of current-related suspended transport model

The modified parameters $\delta_0$, $\gamma_{ker}$, $\mu_{w,a}$ of Eqs (3.6, 3.7 and 3.10) have been obtained by fitting of computed and measured time-averaged sand concentration profiles. The large-scale flume data (Table 3.1) has been used for calibration of the parameters. The field data of Egmond (Table 3.2) has been used to verify the overall suspended load transport model. The results of the calibration and verification procedure will be described at a later stage.

### 3.3.4 Effect of particle size on current-related suspended transport

To show the effect of particle size on the current-related suspended load transport, computations (using TRANSPOR 1993) have been made for a specific case (see Section 2.2.3). The suspended transport ($q_{s,e}$) as a function of particle diameter is shown in Figure 2.3.

The results are:

- **low-energy event**: suspended load transport is zero;
- **high-energy event**: suspended load transport is dominant and the transport rate decreases with increasing particle diameter;
- **medium-energy event**: suspended load transport decreases with increasing particle size.
4 Modelling of bed load and suspended load transport for graded bed material (multi-fraction method)

4.1 Introduction

The sediment bed of the coastal zone usually exhibits a large horizontal variation of sediment sizes. Local variations related to the presence of bed forms (differences in size at the top and in the trough) may occur, but cross-shore sorting between the beach, the surf zone and deeper water due to selective transport processes is a more important process in nature (fining in seaward direction). Vertical sorting is the process governing the vertical exchange of sediment particles between the various bed layers. These sorting effects can only be represented by taking the full size composition of the bed material, which may vary horizontally and vertically, into account.

Usually, mathematical modelling of sand transport and morphology is based on the application of a single sediment fraction \(d=d_0\) and fraction size \(=100\%\) to determine the sand transport rate. Herein, a multi-fraction method is presented that can be used to determine the sand transport rate for graded bed material.

4.2 Sorting, hiding and exposure processes

The process by which grains of different diameter move to a certain position in the coastal zone for given hydrodynamic conditions is termed grain selection or sorting. Grain sorting is related to the selective movement of sediment particles in a mixture near incipient motion at low bed-shear stresses and during generalized transport at higher shear stresses. A basic question is whether the initial movement of a particular size fraction within the total distribution of sizes is affected by the presence of the other size fractions or in other words: is the initial movement of a particular fraction equal to that of uniform material of the same size? Another question is the behaviour of the fractions when all particles of the bed surface are fully mobilized.

Two effects are important:
- the degree of exposure of sediment particles of unequal size within a mixture (hiding of smaller particles resting or moving between the larger particles);
- the non-linear dependence of transport on particle diameter, for example:
  suspended load transport is inversely proportional to grain size; \(q_i \approx d^m\) with m-coefficient between -0.5 and -2; bed-load transport may increase with grain size in the fine particle range (between 0.1 and 0.5 mm) and may decrease with grain size for coarse particles (larger than 0.5 mm), see Fig. 2.3.

The degree of exposure of a grain with respect to surrounding grains obviously is the most important parameter determining the bed-shear stress for initiation of motion, as shown by Fenton and Abbott (1977). They studied the effect of relative protrusion \(p/d\) on the initial movement of grains in the transitional and fully turbulent regime; \(p=\) protrusion of a particle
above others and \( d \) = diameter. Test grains were placed on top of a rod between similar grains glued to the flume bottom. The rod was then screwed upwards, pushing the grain into the flow until it was swept away. This was repeated twenty times for each test. Two types of grains were used: 2.5 mm diameter angular polystyrene grains and 5 to 10 mm well-rounded pea gravel. Relative protrusion was varied in the range between -0.2 and 0.8. A negative relative protrusion is a configuration with the top of the grain below that of the adjacent grains. The maximum relative protrusion of 0.8 is that of a grain sitting above one of the interstices formed by the other grains. The critical bed-shear stress for incipient motion (compared to that for zero protrusion \( p=0 \)) was found to decrease for increasing positive relative protrusion and to increase for negative relative protrusion values. Their results can be summarized, as:

\[
\begin{align*}
\frac{\tau_d}{\tau_{cr, P=0}} & = 0.09-0.12 \\
\frac{\tau_d}{\tau_{cr, P=0}} & = 0.12-0.15 \\
\frac{\tau_d}{\tau_{cr, P=0}} & = 0.20-0.25 \\
\frac{\tau_d}{\tau_{cr, P=0}} & = 0.45-0.55 \\
\frac{\tau_d}{\tau_{cr, P=0}} & = 2.5-2
\end{align*}
\]

From a comparison of the data of Fenton and Abbott and the data of Shields, it follows that the Shields curve represents conditions with relative protrusions in the range of 0.1 for larger particles to 0.3 for smaller particles.

Carling (1983) found that relative protrusion is an important parameter for large grain sizes in natural shallow streams with poorly-sorted bed material (between 0.2 and 200 mm). The dimensionless threshold values of the larger grains were found to be in the range of \( \theta = 0.01 \) to 0.1. The threshold values decreased for increasing relative protrusion (expressed as \( d_h/h \); \( d_h \) = mean size of 5 largest grains of sample, \( h \) = flow depth) in the range of 0.1 to 0.5.

For steady flow in gravel-bed rivers some researchers (Parker et al., 1982) have found that all sizes in a mixture begin moving at nearly the same bed-shear stress (equal mobility concept). Others (Komar, 1996) have shown that the bed-load material is becoming coarser for increasing bed-shear stresses to approach the composition of the bed material in the upper transport regime.

The critical bed-shear stress of individual fractions in a mixture is difficult to both define and measure. Different approaches to the problem have been presented.

Komar (1996) and others associate the critical condition for entrainment of gravel-type sediment with the maximum particle size \( (d_{max}) \) in the bed load sample. Analysis of data shows a relationship of the type: \( \tau_{cr,d_{max}} = (d_{max})^{0.6} \), which is known as a flow-competence relationship. This information is of potential use in evaluating the flood discharge from the maximum particle diameter found in deposits, although these large sizes may also be introduced by bank erosion or as debris from side slopes. A critical review has been given by Wilcock (1992). He states that any information derived from the largest sampled grains is very sensitive to sample size and the origin of the largest grains.

Wilcock and McArdell (1993) performed laboratory experiments with bimodal sand-gravel mixtures \( (d_{90}=0.3 \text{ mm}, d_{50}=8 \text{ mm}, d_{10}=30 \text{ mm}) \) and derived the critical bed-shear stress for incipient motion from fractional transport rates. Their findings are:

- at the lowest flows (0.4 m/s in depth of about 0.1 m) the transport is composed almost entirely of sand sizes (<2 mm), organized into irregular flow-parallel stripes;
- as flow increases over this low-transport range, the bed coarsens because the surface pores are progressively filled with sand;
• as flow strength increases to the upper regime, an increasing proportion of the gravel fractions are entrained into transport and long, narrow dune-type bed forms develop; the body of the bed forms (length= 1 m, height= 0.02 m) consists of sand grains (<1 mm); most grains are deposited in the region of the advancing front; gravel grains move as single grains and rapidly traverse the bed form without stopping; the largest gravel grains often overpass several bed forms; the finest sand fractions move over the bed form in suspension;

• at relatively high flow strengths (1 m/s in depth of 0.1 m) the bed surface becomes consistently finer, because more fine sand is made available for transport (not sheltered anymore by the gravel grains).

The fractional transport rates (scaled by their proportion on the bed surface) was observed to fall into two groups: one consisting of fully mobilized finer grains moving at a more or less constant fractional transport rate and another consisting of immobile or partially mobile coarser grains moving at much smaller transport rates. The grain size separating (partially) mobile and immobile grains increases with increasing flow strength from about 0.5 to 10 mm. Complete mobilization of a certain size fraction occurs at roughly twice the bed-shear stress necessary for incipient motion. Fully mobile is defined as $p_{bl}/F_{<1}$, with $p_{bl}$ proportion of each fraction in the transported material and $F_{<1}$ proportion of each fraction on the bed surface. The bed-shear stress for incipient motion of the sand particles (0.3 to 1 mm) was approximately constant at about 0.5 N/m²; the values of the gravel grains (1 to 70 mm) were found to increase from 0.5 to 7 N/m².

Wilcock (1993) and others analyzed measured transport rates of individual fractions for conditions just beyond initiation of motion. The fractional bed-load transport rates ($q_{bl}$) are normalized to their availability ($F_{<1}$) in the bed material and plotted as a function of the bed-shear stress ($q_{bl}/F_{<1}$ against $F_{<1}$). A small threshold transport (say 0.0001 kg/s/m) is introduced as a threshold criterion to find the threshold bed-shear stress for initiation of motion ($F_{<1,th}$) of each particular fraction. This is done to avoid the uncertainty involved in extrapolating a curve fitted to the fractional transport rates. Wilcock studied the critical bed-shear stress of individual size fractions in unimodal and bimodal sediments. Flume and field data from the literature and new flume data were analyzed. The fluid bed-shear stress was defined as the skin-friction stress when bed forms were present.

Petit (1994) studied the motion of marked individual gravel particles ($d_{50}$ between 12 and 39 mm) in a flume. The bed-shear stresses were evaluated from measured near-bed velocity gradients, when initial movement of marked particles was observed to occur. Two types of movements were defined: initiation of movement (about 20% of the bed surface in motion) and generalized movement. Flume runs were conducted with different slopes and discharges. Before each run, the marked particles of different sizes were arranged on the bed at different locations along the flume. The particles were placed on a bed consisting of similar particles. Velocity measurements were done at these locations at moments of movement. The bed-shear stresses derived from the measured velocity gradients were plotted as a function of particle diameter. The data showed the presence of two or more parallel curves representing an upper and a lower limit. The upper limit represents the limit of shear stress above which movement of material is certain (all marked particles in motion); the lower limit represents the limit of shear stress below which movement is rare (80% of particles in rest).

Figure 4.1 shows the critical bed-shear stress as a function of particle diameter based on the results of Wilcock (1993), Wilcock et al (1988) and Petit (1994) for unimodal sediment mixtures. The Shields curve for uniform sediment is also shown. The data of Petit represents the average value of the two limiting shear stresses.
Most of the data in Fig. 4.1 refer to relatively coarse sediment material (d>1 mm). The data sets show a slight increase of the critical bed-shear stress for the coarser fraction sizes within the mixture. The finest fractions (d<1 mm) of the Wilcock 1993 dataset seem to have a somewhat higher critical shear stress. The dataset of Wilcock et al. 1988 shows constant critical bed-shear stresses (horizontal line) for sand in the range of 0.5 to 1 mm. A horizontal line in Fig. 4.1 implies equal mobility of all size fractions; all grain sizes of the mixture are set into motion at the same bed-shear stress. In that case the composition of the transported bedload particles is the same as that of the original bed material under all conditions. Based on the data of Fig. 4.1, the concept of equal mobility may be reasonably correct for gravel-type sediments between 2 and 10 mm. The curves cross the Shields curve (uniform sediment) at approximately the median diameter (d_{50}), except for the dataset of Wilcock et al. 1988. As regards the data crossing the Shields curve, the larger sizes are set into motion at bed-shear stresses that are smaller than required for uniform sizes, while the smaller size fractions require higher bed-shear stresses than for uniform material. The reason for this is that the largest sizes within a mixture are more exposed to the flow, while the smaller sizes tend to be sheltered from the flow by the larger particles. Thus, the larger particles in a mixture are substantially more mobile than in the uniform-bed case. Komar (1996) states that the selective mobility pattern of fine sand material is opposite to that found in coarser gravel material. In sandy bed material the entrainment of the finest fractions may be caused by relatively large bed-shear stresses (curve sloping downward to the right, see
Fig. 4.1). Thus, in the sand-size range the larger grains may be selectively removed, leaving behind the finer grains. The dataset (MIT-funi, d<sub>so</sub>=0.67 mm) of Wilcock et al 1988 (see Fig. 4.1) does not confirm the findings of Komar. The dataset of Rakoczii (1975) confirms that finer and coarser particles in the gravel range (d<sub>so</sub>=1.4 and 5 mm) start moving at practically the same value of the bed-shear stress. Thus, equal mobility of grains may be present for gravel-type mixtures. For sandy material (0.5 mm) the finer grains were eroded before the medium and coarser grains were set into motion (Rakoczii, 1975). This latter behaviour is opposite to the findings of Komar. More experimental data in the fine sand range is needed to determine the critical bed-shear stress of mixtures in the sand range.

The available data of Fig. 4.1 can be used to derive the exposure or hiding factor for particles in a mixture, as follows:

\[
\xi = \frac{\tau_{b,c,r}}{\tau_{b,c,r,shields}} = F(d/d_{so})
\]  (4.1)

in which: \(\tau_{b,c,r}\) = critical bed-shear stress of fraction \(d\), within a mixture and \(\tau_{b,c,r,shields}\) = critical bed-shear stress of fraction \(d\), based on Shields curve.

The computed values of the exposure or hiding factor are shown in Figure 4.2. The hiding factor or exposure factor of Egiazaroff (1965), defined as a multiplication factor to the critical shear stress, is given by:

\[
\xi = \left[ \frac{\log(19)}{\log(19 \ d/d_{so})} \right]^2
\]  (4.2)

in which: \(d\) = mean particle diameter of size class \(i\), \(d_{so}\) = median diameter of bed material mixture.

A graph of Eq.(4.2) is given in Figure 4.2. Originally, Egiazaroff used the average diameter \(d_{ma}\) in Eq.(4.2) in stead of \(d_{so}\). In the present approach the \(d_{so}\) is used to be consistent with the single fraction transport formulas in which generally the \(d_{so}\)-value is being used as the representative parameter.

According to Komar (1996), the initiation of motion of a grain by the flowing fluid is related to the pivoting angle of the grain about one of its contact points with underlying grains, see Fig. 4.2. Experiments have been performed to determine how the pivoting angle depends on grain shape, grain size, grain orientation and imbrication. Grain imbrication refers to the orientation of relatively flat ellipsoidal grains (pebbles), when the long axial diameter is oriented transverse to the main flow direction and the intermediate axial diameter is dipping upstream, creating a totally different geometry for grain pivoting. The pivoting angles were measured on an apparatus consisting of a board which can be progressively inclined, the angle of the board being equal to the pivoting angle at the instant of grain movement. The pivoting angle can be expressed, as: \(\phi = \phi_{o} (d/d_{so})^{f}\) with \(\phi_{o}\) = pivoting angle for uniform grains.

Some values given by Li and Komar (1986) are:

- spheres (2.3 to 38 mm) \(\phi_{o} = 20^\circ \text{ to } 35^\circ, f = -0.55 \text{ to } -0.75\)
- ellipsoidal natural pebbles (4 to 50 mm) \(\phi_{o} = 30^\circ \text{ to } 40^\circ, f = -0.35 \text{ to } -0.55\)
- ellipsoidal pebbles (4 to 50 mm, imbrication) \(\phi_{o} = 63^\circ, f = -0.3\)
- angular gravel (5.8 to 47 mm) \(\phi_{o} = 50^\circ, f = -0.35\)
- sand-sized material \(\phi_{o} = 50^\circ, f = -0.35\)
The $\phi_r$-value was found to decrease with increasing diameter and increasing sphericity (spherical grains have lower pivoting angles). The $f$-value increases with increasing sphericity. The diameter effect can be illustrated by the following values for spheres:

- Diameter = 0.25 mm  $\phi_r = 50^\circ$
- Diameter = 1 mm  $\phi_r = 47^\circ$
- Diameter = 10 mm  $\phi_r = 38^\circ$

An exposure factor can also be derived from the work of Komar (1996), yielding for $f = -0.3$:

$$
\xi_i = \left[ \tan(\phi_i(d_i/d_{50})^{0.3}) \right] / \left[ \tan(\phi_o) \right] \tag{4.3}
$$

in which: $d_i =$ mean particle diameter of size class i, $d_{50} =$ median diameter of bed material mixture, $\phi_o = 61.5^\circ;$ angle of repose (or pivoting angle = angle between vertical line and a line through one of the contact points of the grains, see Fig. 4.2) for uniform grains; the $\xi_r$-factor goes to infinity for $d_i/d_{50}$ approaching about 0.283.

Equation (4.3) is shown in Fig. 4.2. For $d_i/d_{50}<1$ the angle of repose increases resulting in an increase of the hiding factor.

The hiding factor of Egiazaroff yields considerably smaller values than that of Komar for $d_i/d_{50}<1$, but both factors yield almost the same results for $d_i/d_{50}>1$.

The data in Fig. 4.2 can be represented reasonably well by a linear expression: $\xi_i = (d_i/d_{50})^{-1}$.

The approaches of Egiazaroff and Komar do not account for vortex-induced pick-up of smaller grains hiding between larger grains (negative relative protrusion).
4.3 Multi-fraction method

4.3.1 Description of method

Bed material in natural conditions consists of non-uniform sediment particles. The effect of the non-uniformity of the bed material will result in selective transport processes (sorting). Generally, the approach is to divide the bed material in a number of size fractions and to compute the sand transport rate of each size fraction using an existing single fraction method (replacing the median diameter of the bed material by the mean diameter of each fraction) with a correction factor ($\xi$) to account for the non-uniformity effects. This correction is necessary because the coarser particles are more exposed to the near-bed current and wave motion than the finer particles which are somewhat sheltered by the coarser particles (hiding effect). The interaction of the size fractions can be represented by increasing the critical shear stress of the finer particles and decreasing the critical shear stress of the coarser particles. Herein, the correction factor of Egiazarovff (1965) is used.

Armouring will occur if the coarser particles are immobile, whereas the smaller grains are eroded (despite their increased critical bed-shear stress) until the developing armour layer prevents further pick-up of finer underlying sediments. During higher flow velocities the armour layer may also be mobilized (mobile armour layer). Bed forms of finer sediments may migrate over the coarser armour layer.

The total sand transport rate for all size fractions can be obtained by summation of the transport rates per fraction taking the probability of occurrence of each size fraction into account, as follows:

$$q_b = \Sigma p_i q_{b,i} \quad \text{and}$$

$$q_k = \Sigma p_i q_{k,i}$$

(4.4)

(4.5)
in which: $p_i =$ probability of occurrence of size fraction $i$, $\Sigma =$ summation over $N$ fractions, $N =$ number of size fractions

The bed-shear stress parameter is computed, as follows:

$$ T = \left( \tau_i - \xi_i \tau_{ci,i} \right) / \xi_i \tau_{ci,i} \quad (4.6a) $$

Another approach may be:

$$ T = \left( \tau_i - \xi_i \tau_{ci,d50} \right) / \xi_i \tau_{ci,d50} \quad (4.6b) $$

with $\tau_{ci,d50} =$ critical shear stress of mixture based on $d_{50}$ (constant critical shear stress), $\tau_{ci,i} =$ critical shear stress of fraction $i$ based on $d_i$ and $\xi_i =$ exposure factor according to Egiazaroff, see Fig. 4.2.

From a physical point of view the application of Eq.(4.6a) seems to be the most appropriate choice. This approach is in line with the definitions given in Section 4.2. The results of sensitivity computations using Eqs. (4.6a) and (4.6b) are presented in Section 4.3.2 and 4.3.3. The multi-fraction method as used in the present study results in a transport rate larger or smaller than that based on the single-fraction method, depending on the processes considered (initiation of motion, hiding effects, bed load or suspended load).

Another approach may be to assume that the multi-fraction transport should be the same as that computed by the single fraction method. Most single-fraction methods have been calibrated against field data and are therefore believed to produce realistic results. This may, however, not be true for specific conditions such as close to initiation of motion and/or for very wide size distributions. The transport rate according to the MF method can be made equal to that of the SF method by reducing the transport rate per fraction (based on the relative contribution of the transport rate per fraction) until the sum is equal to that of the single fraction method. This latter approach has not been used herein.

4.3.2 Comparison of computed transport rates based on SF-method and MF-method excluding hiding effects

The difference in the transport rates according to the single fraction method (SF) and multi-fraction method (MF) can be evaluated by using a simple transport formula of the type $q_i = \nu^2 d^\alpha$ ($\nu =$ velocity, $d =$ diameter). The exponent $\alpha$ is taken as $\alpha = -2$ and $\alpha = 2$ to study the effect of particle size. The effect of hiding is neglected.

Taking $\alpha = -2$ and a symmetrical size distribution ($N = 7$ fractions), as follows:

$$
\begin{align*}
p_i &= 0.05 \quad p_5 = 0.15 \quad p_3 = 0.2 \quad p_1 = 0.2 \quad p_6 = 0.2 \quad p_7 = 0.15 \quad p_8 = 0.05 \\
d_1 &= 0.5 d \quad d_3 = 0.666d \quad d_5 = 0.8d \quad d_7 = 1d \quad d_5 = 1.25d \quad d_3 = 1.5d \quad d_7 = 2d
\end{align*}
$$

the MF transport can be expressed in terms of the SF transport yielding: $q_{sN=7} = 1.26 q_{sN=1}$.

Taking a wider symmetrical size distribution with $d_i = 0.25d$, $0.5d$, $0.75d$, $1d$, $1.33d$, $1.5d$, $2d$ and $d_7 = 4d$ with similar percentages, it follows that $q_{sN=7} = 2.1 q_{sN=1}$.

Thus, if the transport rate is proportional to $d^2$, the application of the MF method (without the hiding effect) yields a larger transport rate than that based on the SF method with a representative diameter $d$. Furthermore, the wider the size distribution, the larger is the effect.
When the transport rate is proportional to \( d^2 \) (\( \alpha = 2 \)) in stead of \( d^2 \), the MF method also yields \( q_{N=7} = 1.26 \) \( q_{N=1} \) (or \( q_{N=7} = 2.1 \) \( q_{N=1} \) for a wider size distribution). Thus, the transport rates are the same, but in the latter case (\( q_i \) proportional to \( d^2 \)) the finer particles are dominant (suspended load) in the transport process, whereas in the former case (\( q_i \) proportional to \( d^3 \)) the coarser particles are dominant.

For asymmetrical size distributions the transport rate based on the MF-fraction method (without hiding effect) may become smaller or larger than that based on the SF-method depending on the type of asymmetry (more fine or more coarse material) and on the transport-diameter relationship.

The influence of the hiding (or exposure) factor is a reduction of the transport rate in the case of dominant suspended load transport because the critical bed-shear stress of the finer particles (hiding between the coarser particles) is enlarged. Thus, for suspended load transport the hiding effect is opposite to the diameter effect.

### 4.3.3 Comparison of computed transport rates based on SF-method and MF-method including hiding effects

To compare the results of both the SF and MF-methods (as described in Sections 2.2, 3.3 and 4.3), computations have been made for three types of bed material using seven fractions (\( N = 7 \)), as given in Table 4.1. It may be noted that the applied bed material distributions (Table 4.1) represent conditions with somewhat more fine material compared to sediment with a log-normal distribution.

The water depth was taken as \( h = 3 \) m. The current velocity was varied in the range of 0.3 to 2 m/s. The wave heights were taken as \( H_s = 0 \) m and \( H_c = 2 \) m with a period of \( T_c = 7 \) s. For the case with 800 \( \mu m \)-sediment a wave height of \( H_s = 1 \) m (instead of 2 m) was taken to show the effect of a smaller wave height. The angle between the wave and current directions was 90°. The current-related and wave-related bed roughness heights were assumed to be equal and varied in the range of 0.05 to 0.01 m, depending on the hydraulic conditions (smaller values at higher bed-shear stresses). The water temperature was taken to be 15 °C and the salinity 30 promille.

Sensitivity computations using Eq.(4.6a) and (4.6b) have been made to analyze the effect of both a constant critical shear stress and a fraction-dependent critical shear stress on the transport rates.

Application of Eq.(4.6b) to compute transport rates for 200 \( \mu m \)-sediment, yielded relatively large bed-load and suspended load transport rates at low velocities (factor 10 to 20 larger than that based on Eq.(4.6a) ). In the higher transport regime the computed transport rates based on Eq.(4.6b) were about 20% to 50% larger.

For the 400 \( \mu m \)-sediment the computed bed-load transport values were a factor 20 larger in the lower transport regime and a factor 2 in the higher regime. The suspended load transport rates computed for the 400 \( \mu m \)-sediment showed a similar behaviour as those for the 200 \( \mu m \)-sediment. Herein, the transport rates based on Eq.(4.6a) are believed to be more realistic than those based on Eq.(4.6b). Therefore, this latter method has not been used for further analysis.

Sensitivity computations using the mean particle diameter \( d_m \) (as proposed by Egiazaroff, 1965) in stead of the \( d_0 \) in Eq.(4.6a) have also been made. This resulted in 20%-smaller transport rates for the 200 \( \mu m \)-sediment and 50%-smaller transport rates for the 800 \( \mu m \)-sediment. As the \( d_m \)-value is larger than the \( d_0 \)-value, the hiding factor is also larger yielding a larger critical bed-shear stress and hence a smaller transport rate.
Computation of the suspended load transport using the single fraction method requires information of the fall velocity of the suspended sediment. Two options are possible: the suspended sediment size \( d_i \) can be taken equal or smaller than the bed material size \( d_{50,\text{bed}} \). Usually, the suspended sediment size varies between 0.7 and 1 \( d_0 \) of the bed material depending on the hydraulic conditions (higher values for increasing bed-shear stress, see Van Rijn, 1993). As regards the SF-method, two series of computations have been made; \( d_i = d_{50,\text{bed}} \) for all conditions and \( d_i = 0.7 \ d_{50,\text{bed}} \) at small bed-shear stresses increasing to \( d_i = d_{50,\text{bed}} \) at high values of the bed-shear stress. In the latter case the SF-method may be seen as a quasi two-fraction method because the sediment size to compute the bed-load transport is different from the sediment size to compute the suspended load transport.

<table>
<thead>
<tr>
<th>200-( \mu )m sediment</th>
<th>400-( \mu )m sediment</th>
<th>800-( \mu )m sediment</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d_{50} = 200 ) ( \mu )m</td>
<td>( d_{50} = 400 ) ( \mu )m</td>
<td>( d_{50} = 800 ) ( \mu )m</td>
</tr>
<tr>
<td>( d_{50} = 310 ) ( \mu )m</td>
<td>( d_{50} = 860 ) ( \mu )m</td>
<td>( d_{50} = 1835 ) ( \mu )m</td>
</tr>
<tr>
<td>Fraction (( \mu )m)</td>
<td>( d_i ) (( \mu )m)</td>
<td>( p_i ) (%)</td>
</tr>
<tr>
<td>50-150</td>
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<td>5</td>
</tr>
<tr>
<td>100-150</td>
<td>125</td>
<td>15</td>
</tr>
<tr>
<td>150-200</td>
<td>175</td>
<td>20</td>
</tr>
<tr>
<td>200-250</td>
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<td>20</td>
</tr>
<tr>
<td>250-300</td>
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<td>20</td>
</tr>
<tr>
<td>300-350</td>
<td>325</td>
<td>15</td>
</tr>
<tr>
<td>350-450</td>
<td>400</td>
<td>5</td>
</tr>
</tbody>
</table>

**Table 4.1**  
**Bed material composition**

The computation results are presented in Figures 4.3, 4.4, 4.5 and 4.6.

Figure 4.3 shows the current-related suspended load transport rates as a function of the current velocity and the wave height for the 200-\( \mu \)m-sediment material, using \( d_i = d_0 \) as input value for the suspended sediment size of the single fraction (SF) method. Comparing the results of the SF and MF methods, slightly higher transport rates are obtained by using the SF-method except for a current velocity larger than 1.25 m/s (\( H_i = 0 \) m).
Figure 4.3  Current-related suspended load transport using single and multi fraction methods for 200 μm-sediment

Figures 4.4, 4.5 and 4.6 show the ratio of the transport rates according to both methods. The bed-load transport includes the wave and current-related transport components; the suspended transport only includes the current-related transport component (wave-related has been neglected, γ=0).

Bed-load transport ratios \( q_{b,N=U}/q_{b,N=7} \) as well as suspended load transport ratios \( q_{s,N=U}/q_{s,N=7} \) are presented. N=1 refers to the single fraction method; N=7 refers to the multi-fraction method.

The deviations between the MF and SF-method are largest in the lower transport regime (bed-shear stress slightly greater than critical shear stress) and smallest in the higher transport regime when the bed-shear stress is much larger than the critical bed-shear stress.

The results for the 200 μm-sediment (Figure 4.4) show the following trends:
- the bed-load transport rates according to the MF-method are slightly larger (about 10%) than those of the SF-method;
- the suspended load transport rates according to the MF-method are significantly smaller than those of the SF-method at small current velocities, especially when the \( d_s \)-value of the SF-method is varied from 0.7 to 1 \( d_s \)-value of the bed material; in the absence of waves the ratio \( q_{s,N=U}/q_{s,N=7} \) is about 1.5 to 4 at small current velocities and reduces to about 0.95 at large current velocities; in the presence of waves the ratio \( q_{s,N=U}/q_{s,N=7} \) varies in the range 1.1 to 2.2, see Figure 4.4.
Figure 4.4  Ratio of transport rates according to the SF and MF methods for 200 µm-sediment

Figure 4.5 shows the results for the 400 µm-sediment. The results are:
- the bed load transport rates according to the MF-method in the absence of waves are significantly smaller at low velocities (factor 2) and slightly smaller at higher velocities (10%); in the presence of large waves (high transport regime) both methods yield the same results;
- the suspended load transport rates according to the MF-method also are substantially smaller than those of the SF-method (factor 2.5 to 4 at low velocities without waves; factor 1.2 to 2 at low velocities with relatively large waves);
computational results for very high velocities larger than 3 m/s \( (H_i=0) \) yielded \( q_{b,N=7}=1.7q_{b,N=1} \) (not shown), because the diameter effect is much more important than the hiding effect in the upper transport regime.

\[ d_{50} = 400 \, \mu m \]
\[ H_i = 0 \, m \]
\[ h = 3 \, m \]

\[ d_{50} = 400 \, \mu m \]
\[ H_i = 2 \, m \]
\[ h = 3 \, m \]

**Figure 4.5**  *Ratio of transport rates according to the SF and MF methods for 400 \( \mu m \)-sediment*

Figure 4.6 shows the results for the 800 \( \mu m \)-sediment. The results are:
- the bed-load transport rates according to the MF-method are somewhat smaller (40%).
the suspended load transport rates according to the MF-method are substantially smaller than those of the SF-method (factor 2 to 5 at low velocities without waves; factor 2 to 3 at low velocities with waves).

Figure 4.6  *Ratio of transport rates according to the SF and MF methods for 800 μm-sediment*

For all cases the application of the multi-fraction method results in substantially smaller suspended transport rates, especially at lower velocities. This is caused by the effect of the hiding factor on the reference concentration, which is demonstrated by results of sensitivity
computations with and without hiding factor, see Table 4.2. Neglecting the hiding factor, the critical bed-shear stress of the smaller particles will be smaller and hence the reference concentration and the suspended transport will be larger, as shown for a symmetrical size distribution in Section 4.3.2. For 200μm-sediment the suspended transport (according to the multi-fraction method without hiding factor) was found to be about 50% larger (Table 4.2); for 800 μm-sediment the suspended transport was found to increase by a factor 8 to 10! (see Table 4.2). The effect of the hiding factor on the bed-load transport is rather small (15% to 30%).

Based on this, it is concluded that for coarse sediment the suspended transport rates will be rather large, if the hiding factor is neglected.

Experimental results are necessary to better evaluate the importance of the hiding effect for sand in the range of 200 to 800 μm (0.2 to 0.8 mm).

<table>
<thead>
<tr>
<th>Particle size (μm)</th>
<th>Current velocity (m/s)</th>
<th>Bed l. tr. 1 fraction (kg/s/m)</th>
<th>Bed l. tr. 7 fractions with h.f. (kg/s/m)</th>
<th>Bed l. tr. 7 fractions no h.f. (kg/s/m)</th>
<th>Susp. tr. 1 fraction (kg/s/m)</th>
<th>Susp. tr. 7 fractions with h.f. (kg/s/m)</th>
<th>Susp. tr. 7 fractions no h.f. (kg/s/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>0.5</td>
<td>0.002</td>
<td>0.0023</td>
<td>0.0017</td>
<td>0.005</td>
<td>0.0034</td>
<td>0.0055</td>
</tr>
<tr>
<td>200</td>
<td>2</td>
<td>1.2</td>
<td>1.4</td>
<td>1.2</td>
<td>16.4</td>
<td>17.2</td>
<td>24</td>
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<td>1.1</td>
<td>1.26</td>
<td>1.1</td>
<td>9</td>
</tr>
</tbody>
</table>

Table 4.2  Effect of hiding factor on transport rate (h.f. = hiding factor), no waves

4.4  Effect of graded bed material on bed load transport based on experimental and computational results

Some test have been done in the wave tunnel of Delft Hydraulics (Hassan et al., 1999) for a bimodal sand consisting of 0.21 mm sand (70%) and 0.97 mm sand (30%), see Tests P6, P7 and P9 of Table 2.1. The d50 was about 0.23 mm and the d90 was about 1 mm. The measured net transport rates under conditions of asymmetric wave motion are given in Table 2.1 and Figure 4.7. For comparison the results of Tests C1, B8 and B9 with uniform sand of 0.21 mm are also shown in Figure 4.7. As can be observed, the measured net transport rates are almost the same. Thus, the presence of a coarse fraction (30% with diameter of 1 mm) has almost no influence on the net transport rate.

Equation (2.2) has been used to compute the net transport rates using a single-fraction and a multi-fraction approach. The multi-fraction method is based on subdivision of the bed material in a number of size fractions; the sand transport rate of each size fraction is computed using an existing single fraction method (replacing the mean diameter of the bed material by the mean diameter of each fraction) with a correction factor (hiding factor) to account for the non-uniformity effects). This correction is necessary because the coarser particles are more exposed to the near-bed current and wave motion than the finer particles which are somewhat sheltered by the coarser particles (hiding effect). The interaction of the size fractions can be represented by increasing the critical shear stress of the finer particles and decreasing the critical shear stress of the coarser particles. The total sand transport rate for all size fractions can be obtained by summation of the transport rates per fraction taking
the probability of occurrence of each size fraction into account. The computed results are shown in Figure 4.7. The single-fraction approach based on $d_{50}=0.23$ mm and $d_{50}=1$ mm yields an overestimation of about 50% large for peak velocities of 1.1 and 1.2 m/s and an overestimation of a factor 2 for a peak velocity of 1.6 m/s. The relatively large overestimation of the measured transport rates in case of the single fraction method is mainly caused by the use of a relatively large grain roughness ($k_s = 3d_{50}$ with $d_{50}=1$ mm), yielding relatively large bed-shear stresses for the tests C1, B8 anf B9. The multi-fraction method yields smaller transport rates, which are in better agreement with the measured values (about 30% too large).

![Figure 4.7](image_url)  
**Figure 4.7**  
Effect of graded bed material on net bed load transport rates

### 4.5 Effect of graded bed material on suspended load transport based on experimental and computational results

Experiments have been done in a small-scale wave-current flume of the Fluids Mechanics Laboratory of the Technical University of Delft. A horizontal sand bed was present in the flume; two types of sand were used: uniform sand with $d_{50}$ of about 0.17 mm and graded sand with almost the same $d_{50}$. The water depth was about 0.5 m in all tests. Irregular waves were generated superimposed on a following current. Time-averaged suspended sand concentrations and suspended transport rates were measured. This data set will be used to study the effect of graded bed material on suspended transport (future research).
5 Conclusions

The main findings of the present report are:

- an engineering sand transport model has been formulated that can be used for the computation of sand transport in combined wave and current conditions, rippled and flat beds, uniform and graded bed materials with particle sizes larger than about 0.2 mm;
- data sets of large-scale flume and field measurements (with particle sizes in the range of 0.2 to 0.4 mm) have been compiled that can be used for comparison with model results;
- bed-load transport model results show good agreement with laboratory and field data for sand in the range of 0.2 to 0.4 mm (uniform and graded bed material) in the flat bed regime without adjustment of model coefficients, which can be seen as an independent validation of the bed load transport model for the sheet flow regime;
- the measured bed load transport increases (factor 2) with particle size for sand in the range between 0.13 and 0.21 mm, but decreases again (factor 2) for sand larger than 0.32 mm up to 0.97 mm; the effect of particle size on bed load transport is correctly represented for sand of 0.13, 0.21 and 0.97 mm (compared to measured bed load transport); the computed bed load transport for sand of 0.32 mm is too large (about 50%);
- the direction of the net bed load transport is affected by the magnitude and direction of the steady current (if present) in relation to the strength of the wave asymmetry; a following current intensifies the net transport rate, but an opposing current may change the direction of the net transport into that of the current, if the strength of the opposing is sufficiently large (>0.15 m/s); the net transport is opposed to the wave direction when the opposing current is larger than about 10% to 15% of the onshore peak orbital velocity ($U_{current}/U_{on}>0.10$ to 0.15); these effects are correctly represented by the bed-load transport model;
- based on measured data of large-scale flume experiments with sand of 0.16 mm and 0.33 mm and bed forms in the ripple regime; the high-frequency wave-related suspended transport was found to be onshore-directed (in the wave direction) in all tests with irregular waves; this transport component increases with increasing significant wave height, but decreases with decreasing particle size; this latter effect can be understood from the ripple dimensions; the ripples generated on the 0.33 mm sand bed were much more pronounced than those on the 0.16 mm sand resulting in larger vortex motions and stronger associated suspension processes; the data were used to calibrate the formulations of the wave-related suspended transport in the ripple regime; the phase-lag function involved appeared to be constant;
- the application of the multi-fraction method to compute the sand transport rate yields considerably larger values than the single fraction method, if the hiding-exposure factor (hiding of smaller particles between larger particles resting or moving on the bed) is neglected; the wider the size distribution of the bed material, the larger the effect; the modelling of the hiding factor has a large effect on the suspended transport rate for relatively small bed-shear stresses (low waves and weak currents); the proper
formulation of the hiding factor is not quite clear from theoretical point of view; experimental data are necessary to evaluate the available concepts.

Limitations of the present model are:
- sand transport by oscillatory flow (with or without a weak steady flow <0.1 m/s) is assumed to be a quasi-steady process (no major phase lags), which means that the model formulations are less accurate for bed material with $d_{50}<0.2$ mm;
- sand transport by low-frequency wave motion ($T>20$ s) is not modelled.

The performance of the bed load transport model could not be evaluated for the ripple regime, because of lack of data. It is noted that the field data set of Egmond, which has been used in this study, may contain some data in the ripple regime.

The results of the current-related suspended transport model have not yet been compared with laboratory and field data. This will be done in future research by comparing measured and computed concentration profiles and suspended transport rates. Extended verification of the wave-related suspended transport model in the ripple regime is also necessary.

It is recommended to study bed load transport in the ripple regime under irregular waves in the wave tunnel (sand larger than 0.2 mm). Irregular waves should be used to generate realistic ripple forms. Furthermore, the prediction of bed forms and associated bed roughness should be studied.
APPENDIX A
A References


Van Rijn, L.C. and Kroon, A., 1992. Sediment transport by currents and waves. 23rd ICCE, Venice, Italy


B TRANSPOR-model 1993
1. Input

- $h$ = water depth (m)
- $\bar{v}_R$ = depth-averaged velocity vector in main current direction, see Fig. A.1 (m/s)
- $\bar{u}_r$ = time-averaged and depth-averaged return velocity below wave trough compensating the mass transport between wave crest and trough (— in backward or offshore direction), see Fig. A.1 (m/s)
- $u_b$ = time-averaged near-bed velocity due to waves, wind or density-gradient (+ in forward direction, — in backward direction), see Fig. A.1 (m/s)
- $H_b$ = significant wave height (m)
- $T_p$ = (absolute) wave period of peak of spectrum (s)
- $\phi$ = angle between wave and main current direction (0-360°) (-)
- $d_{50}$ = median diameter of bed material (m)
- $d_{90}$ = 90% diameter of bed material (m)
- $d_r$ = representative diameter of suspended material (m)
- $k_{sc}$ = current-related bed roughness height (minimum $k_{sc} = 0.01$ m) (m)
- $k_{sw}$ = wave-related bed roughness height (minimum $k_{sw} = 0.01$ m) (m)
- $T_e$ = fluid temperature (°C)
- $SA$ = fluid salinity (‰)

**Figure A.1 Schematic presentation of current and wave direction**

**Remarks:**

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

- $d_r = (0.6$ to $1) \, d_{50,\text{bed}}$, see Section 8.4.3.

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

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

Chloridinity : \( \text{CL} = (\text{SA} - 0.03)/1.805 \)
Fluid density : \( \rho = 1000 + 1.455 \text{ CL} - 0.0065 (\text{Te} - 4 + 0.4 \text{ CL})^2 \)
Kinematic viscosity : \( \nu = (4/(20 + \text{Te})) \times 10^{-4} \)
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 : \( D_\ast = d_{50} [(s-1)g/\nu^2]^{1/3} \)
Shields parameter :

\[
\begin{align*}
1 < D_\ast & \leq 4 : \theta_\ast = 0.24 D_\ast^{0.64} \\
4 < D_\ast & \leq 10 : \theta_\ast = 0.14 D_\ast^{0.1} \\
10 < D_\ast & \leq 20 : \theta_\ast = 0.04 D_\ast^{0.29} \\
20 < D_\ast & \leq 150 : \theta_\ast = 0.013 D_\ast^{0.5} \\
D_\ast & > 150 : \theta_\ast = 0.055
\end{align*}
\]
Critical bed-shear stress : \( \tau_{cr} = (\rho_s - \rho) g d_{50} \theta_\ast \)
Critical depth-averaged velocity : \( \bar{u}_{cr} = 5.75[(s - 1)g d_{50}]^{0.5} (\theta_{cr})^{0.5} \log(4h/d_{50}) \)
Critical peak orbital velocity (Komar):

\[
\begin{align*}
d_{50} < 0.0005 \text{ m} : \hat{U}_{cr} & = [0.12(s-1)g (d_{50})^{0.5} (T_p)^{0.57}]^{1/3} \\
d_{50} \geq 0.0005 \text{ m} : \hat{U}_{cr} & = [1.09(s-1)g (d_{50})^{0.75} (T_p)^{0.25}]^{0.571}
\end{align*}
\]

4. Compute wave length

Wave length modified by currents : \[
\left[\frac{L'}{T_p} - \bar{v}_R \cos \phi \right]^2 = \left[\frac{gL'}{2\pi}\right] \tanh \left[\frac{2\pi h}{L'}\right]
\]

5. Compute relative wave period

The relative wave period is : \( T_p' = \frac{T_p}{1 - (\bar{v}_R T_p \cos \phi)/L'} \)

6. Compute wave parameters

Near-bed peak orbital excursion : \( \hat{A}_\delta = \frac{H_s}{2 \sinh(2\pi h/L')} \)
Near-bed peak orbital velocity : \( \hat{U}_\delta = \frac{\pi H_s}{T_p' \sinh(2\pi h/L')} \)
Wave-boundary layer thickness : \( \delta_w = 0.072 \hat{A}_\delta (\hat{A}_\delta/k_{b,w})^{0.25} \)
Near-bed peak orbital velocity in forward direction

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

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

\[ \alpha = 1 + 0.3 \ (H_/h) \]

Near-bed orbital velocity in backward direction

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

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

Return velocity mass transport

\[ \tilde{u}_r = -\frac{0.125 g^{0.5} (H_\delta)^2}{h^{0.5} h} \]

\[ h_t = (0.95 - 0.35 (H_/h)) h \]

Near-bed wave-induced velocity

\[ u_b = (0.05 - (\alpha_a - 0.5)) \hat{U}_\delta \]

\[ \alpha_a = \hat{U}_{b,f}/(\hat{U}_{b,f} + \hat{U}_{b,b}) \]

7. Compute apparent bed roughness

\[ k_s = k_{s,c} \exp[\gamma \hat{U}_\delta/((\bar{v}_R)^2 + (\bar{u}_r)^2)^{0.5}] , \quad k_{s,max} = 10 k_{s,c} \]

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

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

8. Compute friction factors

For Current:

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

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

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

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

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

For Waves:

\[ f_w' = \exp[-6 + 5.2(\hat{A}_b/3d_{90})^{-0.19}] \]

\[ f_w = \exp[-6 + 5.2(\hat{A}_b/k_{s,w})^{-0.19}] \]

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

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

Efficiency factor waves : \( \mu_w = \frac{f_w^{\prime}}{f_w} \)
\( \mu_{w,\alpha} = 0.6/D_s \)

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

Bed-shear stress current : \( \tau_c = \frac{1}{8} \rho \frac{f_c}{(V_R)^2 + (u_R^{\prime})^2}^{0.5} \)

Bed-shear stress waves : \( \tau_w = \frac{1}{4} \rho \frac{f_w}{(U_0)^2} \)

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

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

10. Compute bed-shear stress parameters

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

Dimensionless bed-shear stress for reference concentration at \( z = a \) : \( T_a = \frac{(\alpha_{cw} \mu_c \tau_c + \mu_{w,\alpha} \tau_w) - \tau_{cr}}{\tau_{cr}} \)
\( (T = 0 \text{ if } T < 0) \)

11. Compute velocity distribution over the depth

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

Inside wave-boundary layer, \( z < 3\delta_w \) : \( v_{R,Z} = \frac{v_\delta \ln(30z/k_{sc})}{\ln(90\delta_w/k_{sc})} \)
\( v_\delta = \frac{\bar{v}_R \ln(30\delta_w/k_a)}{-1 + \ln(30h/k_a)} \)
12. Compute sediment mixing coefficient distribution over the depth

Current, \( z < 0.5 \ h \) : \[ \varepsilon_{s,c} = \kappa \ \beta \ \frac{u_{s,c}}{h} \ \left( 1 - \frac{z}{h} \right) \]
\( z \geq 0.5 \ h \) : \[ \varepsilon_{s,c} = 0.25 \ \kappa \ \beta \ \frac{u_{s,c}}{h} \]
\[ u_{s,c} = \left( \frac{g^{0.5}}{C} \right) \left[ (\langle v_R \rangle)^2 + (\langle u_t \rangle)^2 \right]^{0.5} \]
\[ \beta = 1 + 2 \left( \frac{w_s}{u_{s,c}} \right)^2 \]
\[ \beta_{\text{max}} = 1.5 \]

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

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

13. Compute concentration distribution over the depth by numerical integration

Reference level : \( a = \text{maximum}(k_{s,c}, k_{s,w}) \)

Concentration gradient \( (z > a) \) : \[ \frac{dc}{dz} = -\frac{(1-c)^5 c \ w_s}{\varepsilon_{s,cw} \left( 1 + (c/c_o)^{0.8} - 2(c/c_o)^{0.4} \right)} \]

Bed concentration \( (z \leq a) \) : \[ c_s = 0.015 \ \frac{d_{50} T_s^{1.5}}{a D_s^{0.3}} \]
\[ c_o = 0.65 = \text{maximum volume concentration} \]
\[ w_s = \text{fall velocity of suspended sediment} \]

14. Compute time-averaged suspended load transport rates

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

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

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

Current velocities at $z = \delta$
above bed

\[
v_{R,\delta} = \frac{\bar{v}_R \ln(30\delta/k_s)}{-1 + \ln(30h/k_s)}
\]

\[
u_{x,\delta} = (u_c/v_R)v_{R,\delta}
\]

Orbital velocities (asymm.)

\[
U_{\delta,f} \text{ and } U_{\delta,b}
\]

Instantaneous velocity $x$

\[
\Sigma U_{\delta,x} = U_\delta \cos\phi + v_{R,\delta} + (u_b + u_{r,\delta}) \cos\phi
\]

Instantaneous velocity $y$

\[
\Sigma U_{\delta,y} = U_\delta \sin\phi + (u_b + u_{r,\delta}) \sin\phi
\]

Instantaneous velocity

\[
U_{\delta,R} = \left[\left(\Sigma U_{\delta,x}\right)^2 + \left(\Sigma U_{\delta,y}\right)^2\right]^{0.5}
\]

Instantaneous friction coefficient

\[
\alpha = \frac{|v_{R,\delta}|}{|v_{R,\delta}| + |U_\delta|}
\]

\[
\beta = 0.25 \left[ \frac{-1 + \ln(30h/k_{s,c})}{\ln(30\delta/k_{s,c})} \right]^2
\]

\[
f'_{cw} = \alpha \beta f'_c + (1 - \alpha) f'_w
\]

Instantaneous bed-shear stress

\[
\tau_{b,cw} = 0.5 \rho f'_{cw} (U_{\delta,R})^2
\]

Instantaneous bed-load transport

\[
\gamma = 1 - \left(\frac{H_s}{h}\right)^{0.5}, \quad \gamma_{\min} = 0.3
\]

\[
q_b = 0.25 \gamma \rho_s d_{50} D_{*}^{-0.3} \left[ \frac{v'_{b,cw}}{\rho} \right]^{0.5} \left[ \frac{\tau_{b,cw} - \tau_{b,cr}}{\tau_{b,cr}} \right]^{1.5}
\]

\[
q_{b,x} = (\Sigma U_{\delta,x}/U_{\delta,R}) q_b
\]

\[
q_{b,y} = (\Sigma U_{\delta,y}/U_{\delta,R}) q_b
\]

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

16. Compute bed form dimensions

Bed form dimensions are computed according to formulas given in Chapter 5.
Figure A.2 Instantaneous velocity vector near bed ($z = \delta$)
17. Examples

The method of Van Rijn and that of Engelund-Hansen (Eq. 7.4.9) have been used to compute the total load transport in the current direction.

The basic data are:

- water depth, \( h = 5 \text{ m} \)
- depth-averaged return current, \( \bar{u}_t = 0 \text{ m/s} \)
- time-averaged near-bed velocity, \( u_b = 0 \text{ m/s} \)
- angle between current and waves, \( \phi = 90^\circ \)
- bed material characteristics, \( d_{50} = 0.00025 \text{ m} \) \( d_{90} = 0.0005 \text{ m} \)
- fluid temperature, \( T_e = 15^\circ\text{C} \)
- fluid salinity, \( \text{SA} = 0\% \)

The other input parameters are given in the Table A.1.

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<th>( H_s = 0 \text{ m} )</th>
<th>( H_s = 0.5 \text{ m}, T_p = 5 \text{ s} )</th>
<th>( H_s = 1.0 \text{ m}, T_p = 6 \text{ s} )</th>
<th>( H_s = 2 \text{ m}, T_p = 7 \text{ s} )</th>
<th>( H_s = 3 \text{ m}, T_p = 8 \text{ s} )</th>
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<td>( d_s )</td>
<td>( \bar{v}_R )</td>
<td>( k_{b,c} = k_{b,w} )</td>
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<td>(m)</td>
<td>( \mu \text{m} )</td>
<td>(m/s)</td>
<td>(m)</td>
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<td>0.02</td>
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</table>

*Table A.1 Input parameters TRANSPOR*

The computed total load transport rates (in kg/sm) are shown in Figure A.3. The methods of Van Rijn and Engelund-Hansen show good agreement for the current alone case (\( H_s = 0 \text{ m} \)). The method of Engelund and Hansen overpredicts the transport rates at low velocities (0.3-0.6 m/s), because initiation of motion is not taken into account.
Figure A.3 Computed total load transport in current direction

- $h = 5 \text{ m}$
- $d_{50} = 250 \mu\text{m}$
- $d_{90} = 500 \mu\text{m}$
- $T_e = 15 \degree \text{C}$
- $S_a = 0$
- $\phi = 90\degree$

Van Rijn
Engelund–Hansen ($H_s=0\text{m}$)
APPENDIX C
C Modified Isobe-Horikawa method for non-linear orbital velocities near the bed

This method is described in: “Breaker bar formation and migration” by Grasmeijer and Van Rijn, ICCE 1998, Copenhagen, Denmark

The high-frequency near-bed orbital velocities (low-frequency effects are neglected) are computed using a modification of the method of Isobe and Horikawa (1982). The method of Isobe and Horikawa method is a parameterisation of fifth-order Stokes wave theory and third-order cnoidal wave theory which can be used over a wide range of wave conditions. In the original formulation the near-bed value of \( \hat{u} \) (defined as: \( u_{in} + u_{off} \)) is derived from deep water wave conditions as follows:

\[
\hat{u} = 2.0 u_{\text{linear}} \tag{1}
\]

with:

\[
r_1 = -27.3 \log_{10} \left( \frac{H_0}{L_0} \right) - 16.3 \tag{2}
\]

\[
r_2 = 1.28 \tag{3}
\]

\[
r_3 = 1 \tag{4}
\]

\[
r = r_1 - r_2 \exp \left( -r_3 \frac{h}{L_0} \right) \tag{5}
\]

\( u_{\text{linear}} \) = peak near-bed velocity computed using linear wave theory (m/s), \( H_0 \) = deep water wave height (m), \( L_0 \) = deep water wave length (m), \( h \) = local water depth (m).

The method has been modified by improving the \( r \)-factor using the local wave conditions (instead of the deep water wave height) to determine the near-bed value of \( \hat{u} \). The \( r \)-factor was found by calibration using laboratory and field data with random waves (see Table 1). This resulted in:

\[
r = 1 - 3.2 \left( \frac{H}{L} \right)^{0.65} \left( \frac{H}{L} \right)^{3.4} \tag{6}
\]

with: \( H \) = local wave height (m), \( L \) = local wave length (m), \( u_{\text{linear}} \) = near-bed velocity computed using linear waves theory.
The basic data are given in Table 1.

<table>
<thead>
<tr>
<th>Description</th>
<th>Testnumber</th>
<th>h (m)</th>
<th>H_m0 (m)</th>
<th>T_p (s)</th>
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<td>9.0</td>
<td>1.89</td>
<td>8.0</td>
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<td></td>
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<td>9.4</td>
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<tr>
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</table>

Table 1. Basic data of measurements used in calibration of r-factor.

Measured signals of surface elevation and horizontal orbital velocity near the bed were analysed using spectral analysis. High- and low-frequency oscillations were separated (by filtering) at a period of 2 times the wave spectrum peak period, T_p. The high-frequency signals were separated into shorter time series each containing 10-15 individual waves. Each of the short time series was defined as one single wave class with one representative wave height, wave period, crest velocity near the bed, and trough velocity near the bed. The mean values were chosen to represent the wave class. A comparison between measured and computed values of ū is presented in Figure 1. The broken lines indicate a 20% error band.
Figure 1. Comparison between measured and computed values of near-bed orbital velocity $\bar{u}$ defined as $u_{on} + u_{off}$.

The following formulae, Eq.(7)-Eq.(14), were derived to account for the asymmetry of the velocity profile (Isobe and Horikawa, 1982). Eq.(7)-Eq.(12) is a parameterisation of fifth-order Stokes wave theory and third-order cnoidal wave theory. Eq.(13) and Eq.(14) were introduced to take into account the deformation of the velocity profile due to bottom slope.

\[
\left( \frac{u_{on}}{\bar{u}} \right)_a = \lambda_1 + \lambda_2 \left( \frac{\bar{u}}{\sqrt{gh}} \right) + \lambda_3 \exp \left( -\lambda_4 \left( \frac{\bar{u}}{\sqrt{gh}} \right) \right)
\]

(7)

with:

\[
\lambda_1 = 0.5 - \lambda_3
\]

(8)

\[
\lambda_2 = \lambda_3 \lambda_4 + \lambda_5
\]

(9)

\[
\lambda_3 = \frac{(0.5 - \lambda_5)}{\lambda_4 - 1 + \exp(-\lambda_4)}
\]

(10)

\[
\lambda_4 = \begin{cases} 
  -15 + 1.35 \left( T \frac{g}{\sqrt{h}} \right), & T \frac{g}{\sqrt{h}} \leq 15 \\
  -2.7 + 0.53 \left( T \frac{g}{\sqrt{h}} \right), & T \frac{g}{\sqrt{h}} > 15 
\end{cases}
\]

(11)
\[ \lambda_5 = \begin{cases} 
0.0032 \left( \frac{T}{V h} \right)^2 + 0.000080 \left( \frac{T}{V h} \right)^3, & T, \frac{g}{V h} \leq 30 \\
0.0056 \left( \frac{T}{V h} \right)^2 - 0.000040 \left( \frac{T}{V h} \right)^3, & T, \frac{g}{V h} > 30 
\end{cases} \] (12)

\[ \left( \frac{u_{on}}{\hat{u}} \right)_{\text{modified}} = 0.5 + \left( \left( \frac{u_{on}}{\hat{u}} \right)_{\text{max}} - 0.5 \right) \tanh \left( \frac{\left( \frac{u_{on}}{\hat{u}} \right)_{\text{a}} - 0.5}{\left( \frac{u_{on}}{\hat{u}} \right)_{\text{max}} - 0.5} \right) \] (13)

with:

\[ \left( \frac{u_{on}}{\hat{u}} \right)_{\text{max}} = 0.62 + \frac{0.003}{\text{bed slope}} \] (14)

A comparison between preliminary computations using the present model and laboratory tests showed that the influence of the bed slope might be less pronounced. The following relation gave more realistic results:

\[ \left( \frac{u_{on}}{\hat{u}} \right)_{\text{max}} = 0.62 + \frac{0.001}{\text{bed slope}} \] (15)

The present model includes a sinusoidal distribution of the instantaneous velocities during the forward and backward phase of the wave cycle. The duration period of each phase is corrected to obtain zero net flow over the full cycle (in contrast to the original approach of Isobe and Horikawa).
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