General view on sand transport by currents and waves:
data analysis and engineering modelling for uniform and graded sand
(TRANSPOR 2000 and CROSMOR 2000 models)

LC. van Rijn

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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 the available sand transport data have been analysed and an engineering approach has been 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.

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 (current and waves), resulting in the TRANSPOR-model 1993 (VanRijn, 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 summarized below.

- An engineering sand transport model (TRANSPOR2000) has been formulated that can be used for the computation of sand transport in combined steady and oscillatory flow (waves and current), rippled and flat beds, uniform and graded bed materials with particle sizes between 0.1 and 2 mm.
- The calibration and the verification of the TRANSPOR2000 model is based on a large data base (see Appendix E) including about 1700 data sets for steady flow (rivers and estuaries) and about 120 data sets for combined steady and oscillatory flow (coastal seas) in depths up to 15 m and particle sizes between 0.1 and 2 mm.
- The net bed-load transport rate in conditions with combined steady and oscillatory flow over a sand bed can be reasonably well described (within factor 2 to 3) by time-averaging (over the wave period) of the instantaneous transport rates using a quasi-steady bed-load transport formula approach.
- The 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.
- The measured bed-load transport increases (factor 2) with particle size for sand in the range between 0.1 and 0.2 mm, but decreases again (factor 2) for sand larger than about 0.3 mm up to 1 mm. The effect of particle size on bed-load transport is reasonably well represented by the model.
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.

The bed-load transport is mainly affected by the grain roughness. The overall bed-form roughness also has some (weak) influence on the bed-load transport in case of combined steady and oscillatory flow because of its effect on the near-bed velocity profile.

The wave-related suspended transport component is modelled by an expression based on an instantaneous response of the suspended sediment concentrations and transport to the near-bed orbital velocity. Large-scale wave tank data have been used to calibrate the empirical coefficient involved; this coefficient is found to be constant for all test results considered (two grain sizes 0.16 and 0.33 mm; irregular waves).

The high-frequency wave-related suspended transport is found to be onshore-directed (in wave direction) in conditions with irregular waves. This transport component increases with increasing significant wave height, but decreases with decreasing particle size. This latter effect is related to the ripple dimensions; the ripples generated in conditions with a relatively coarse sand bed (0.3-0.5 mm sand) are more pronounced resulting in stronger vortex motions and associated suspension processes.

The current-related suspended transport is based on the modelling of the time-averaged velocity profile and the time-averaged sand concentration profile. Various empirical coefficients have been recalibrated using data from large-scale wave tank experiments. Important parameters for the suspended transport are the current-related and the wave-related bed-form roughness ($k_{c, b}$ and $k_{w, b}$); both parameters are input parameters.

For conditions with combined steady and oscillatory flow (current and waves) the computed sand transport rates are strongly dependent on the wave-related bed-roughness value in the low velocity range of 0.2 to 0.6 m/s, for which the effect of wave-induced mixing of sediment dominates over turbulence-induced mixing. The effect of bed roughness is less important in conditions with dominating steady flow. The computed transport rates are in reasonably good agreement with measured values, provided that the proper bed roughness value is taken (in the range of 0.01 to 0.05 m). Generally, a bed roughness value of 0.02 m yields the best results.

The sand transport for graded bed material can be computed by using a multi-fraction method (MF-method); the sand transport rate of each size fraction of the bed material is computed 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 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 bed-load transport and the suspended transport strongly depend on the dimensionless bed-shear stress parameter. Sand concentration profiles measured in a small flume for combined oscillatory and steady flow over a fine graded sediment bed have been used for determination of the proper expression of the dimensionless bed-shear stress parameter.
• The multi-fraction (MF) method yields somewhat smaller (maximum 50%) bed-load transport rates than the single fraction (SF) method. The MF-method yields substantially larger suspended transport than the SF-method, varying between a factor 3 for the lower transport regime without waves to a factor of 1.5 for the upper transport regime with waves. The increase of the suspended transport rate according to the MF-method is caused by the relatively large contribution of the finer fractions to the total suspended transport rate.

• In case of well-mixed bed material the application of a constant grain roughness related to the \(d_{50}\) of the mixture seems to be the most logic approach. In case of bed material consisting of segregated fractions it may be better to relate the grain roughness to the grain size of the individual fractions of the bed material mixture.

• The suspended sand transport based on the SF-method can be adjusted to that of the MF-method by using a smaller suspended sediment size \(d_s\) compared to the \(d_{50}\) of the bed material. The ratio of \(d_s\) and \(d_{50,\text{bed}}\) varies between 0.4 for a weak current over a coarse graded sediment bed and 0.85 for a strong current over a fine graded sediment bed. The ratio of \(d_s\) and \(d_{50,\text{bed}}\) varies between 0.7 and 1 for conditions with combined current and waves.

• The application of the MF-method is found to be most appropriate for graded bed material in conditions with weak currents and relatively low waves (\(H/h<0.2\)), because of the relatively large contribution of the finer fractions in the transport process (winnowing effects) resulting in relatively large suspended transport rates (larger than those of the SF-method).

The limitations of the TRANSPOR2000 model are:

• the 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 (fine sand bed);

• the current-related and the wave-related suspended transport rates for combined steady and oscillatory flow are strongly dependent on the wave-related bed-form roughness; the latter parameter, which is an input value of the model, can not be estimated with sufficient accuracy; bed-form and bed-roughness predictors are still missing;

• the high-frequency wave-related suspended transport is uncertain due to lack of sufficient field data for verification;

• the sand transport by low-frequency wave motion (\(T>20\) s) is not modelled.

The following recommendations are given:

• more field measurements of sand concentrations in the surf zone (bar crest zone) are required to study the wave-related mixing of sediment due to oscillatory flow for bed materials in the range of 0.2 to 1 mm;

• more field measurements of the high-frequency oscillatory suspended transport rates are required to better evaluate this transport component; it is of crucial importance for long-term profile development;

• bed roughness has a strong effect on the suspended transport in conditions with combined steady and oscillatory flow; field measurements of bed-form characteristics and associated effective bed-form roughness (based on analysis of the time-averaged velocity profiles) in the surf zone are required to include these effects in the transport models; measurements of sand transport in field conditions should always include ripple
size measurements; the bed-form data should be used to develop bed-form and bed-
roughness predictors;

- the bed-form dimensions and associated bed roughness are variable along the cross-shore profile (relatively large values in trough zone); these effects should be better modelled, because they are of crucial importance for the modelling of bar development;

- field measurements of bed-load transport in the ripple regime during fairweather conditions are required, because this transport process may dominate during these conditions;

- the modified Isobe-Horikawa method (Appendix C) for the prediction of the asymmetry of the near-bed orbital velocity is relatively simple and works reasonably well for the LIP tests (Delta flume experiments); the method should be more extensively tested for field conditions, focussing on the effect of the wave period and the bed slope.

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 morphology (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. In 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 sediments to move offshore while fair-weather waves and swell return the sediments 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 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 a 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 rippled 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:

\[
q_{\text{tot}} = q_b + q_s \tag{1.2.1}
\]

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 an 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 sand transport in conditions with 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 is 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. 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) is assumed to be a quasi-steady process (no major phase lags), which means that the model formulations are less accurate for relatively fine bed material (say d_{50} < 0.2 mm);
- sand transport by oscillatory flow (with or without a weak steady flow) 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 calibration and validation of the bed-load transport and suspended transport model for uniform and graded bed material in the ripple and flat bed regime. Furthermore, model applications will be given for sand transport at constant depth and along the cross-shore profile of a uniform coast.

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

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The COAST3D- project is acknowledged for providing the sand transport data measured in 1998 at the Egmond field site, The Netherlands.

E. Gallagher of the Oceanographic Department of Naval Postgraduate School, Monterey, California, USA and S. Elgar of the Woodshole Oceanographic Institution, Woodshole, Massachusetts, USA are gratefully acknowledged for providing the data of the Duck94 experiment.
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 data of 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).

*Experiments in large scale oscillating water tunnel LOWT*

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 2.1.1 (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, are presented in Table 2.1.2.

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 (2nd Stokes) with and without a steady current (following and opposing) over a sand bed (almost uniform sand) with particle sizes in the range of 0.13 to 0.97 mm. The measured net sand transport predominantly consists 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 sand concentration 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 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 (2nd Stokes waves with $U_{peak,onsore}$ of about 1.5 m/s). Phase-lag effects between near-bed velocities and sand concentrations resulted in negative sand transport against the wave propagation direction.
<table>
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<th>Mean velocity at 0.1m above bed (m/s)</th>
<th>Peak forward orbital velocity (m/s)</th>
<th>Peak backward orbital velocity (m/s)</th>
<th>Wave period (s)</th>
<th>Grain size, $d_{50}$ and $d_{40}$ (mm)</th>
<th>Angle between wave and current ($^\circ$)</th>
<th>Measured net bed load transport (kg/s/m)</th>
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<tr>
<td>I1</td>
<td>0.25</td>
<td>1.5</td>
<td>1.5</td>
<td>7.2</td>
<td>0.32; 0.46</td>
<td>0</td>
<td>0.25</td>
</tr>
<tr>
<td>P6</td>
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<td>1.09</td>
<td>0.57</td>
<td>6.5</td>
<td>0.23; 1.0</td>
<td>0</td>
<td>0.052</td>
</tr>
<tr>
<td>P7</td>
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<td>1.23</td>
<td>0.65</td>
<td>6.5</td>
<td>0.23; 1.0</td>
<td>0</td>
<td>0.081</td>
</tr>
<tr>
<td>P9</td>
<td>0</td>
<td>1.6</td>
<td>0.85</td>
<td>6.5</td>
<td>0.23; 1.0</td>
<td>0</td>
<td>0.17</td>
</tr>
<tr>
<td>PSB1</td>
<td>0.25</td>
<td>1.5</td>
<td>1.5</td>
<td>7.2</td>
<td>0.97; 1.0</td>
<td>0</td>
<td>0.15</td>
</tr>
</tbody>
</table>

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
4. P6, P7 and P9 are tests with bimodal sand mixture (70% of 0.21 mm and 30% of 0.97 mm)

Table 2.1.1  
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)

A selection of the basic data of the tests is given in Table 2.1.1 and is shown in Figure 2.1.1. Based on Figure 2.1.1 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 2.1.1 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 ($U_{current}/U_{occ}>0.10$ to $0.15$). In other cases this ratio may be different, because it depends on the asymmetry of the orbital velocity.
Figure 2.1.1
Top: Net bed load transport as function of peak orbital velocity (0.13 and 0.21 mm); measured data have been used for calibration of Equation 2.2.2
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)
Figure 2.1.1 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.

Computed bed-load transport rates are discussed in Section 2.2.3.

Egmond site, 1992, The Netherlands

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 tidal range is maximum 2 m. 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 positioned in the swash/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 alternately positioned on 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 2.1.2.

Figure 2.1.2 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 ($U_{\text{undertow}}/U_{1/3,\text{on}}<0.15$). 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 2.1.1 Middle).

Computed bed-load transport rates are discussed in Section 2.2.3.
<table>
<thead>
<tr>
<th>Test</th>
<th>h  (m)</th>
<th>Hs  (m)</th>
<th>Tp  (s)</th>
<th>uCR  (m/s)</th>
<th>U1/3,on  (m/s)</th>
<th>U1/3,off  (m/s)</th>
<th>d50  (mm)</th>
<th>d90  (mm)</th>
<th>qk,cross  (kg/s/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>04-11</td>
<td>1.4</td>
<td>0.7</td>
<td>7.7</td>
<td>-0.35</td>
<td>1.05</td>
<td>0.75</td>
<td>0.3</td>
<td>0.5</td>
<td>-0.04</td>
</tr>
<tr>
<td>04-11</td>
<td>1.05</td>
<td>0.45</td>
<td>8.3</td>
<td>-0.3</td>
<td>0.9</td>
<td>0.7</td>
<td>0.3</td>
<td>0.5</td>
<td>-0.03</td>
</tr>
<tr>
<td>06-11</td>
<td>1.0</td>
<td>0.55</td>
<td>7.0</td>
<td>-0.25</td>
<td>1.2</td>
<td>0.9</td>
<td>0.3</td>
<td>0.5</td>
<td>-0.03</td>
</tr>
<tr>
<td>05-11</td>
<td>1.0</td>
<td>0.65</td>
<td>7.0</td>
<td>-0.2</td>
<td>1.2</td>
<td>0.9</td>
<td>0.3</td>
<td>0.5</td>
<td>-0.03</td>
</tr>
<tr>
<td>08-11</td>
<td>1.4</td>
<td>0.8</td>
<td>8.1</td>
<td>-0.15</td>
<td>1.25</td>
<td>0.8</td>
<td>0.3</td>
<td>0.5</td>
<td>0.03</td>
</tr>
<tr>
<td>06-11</td>
<td>1.5</td>
<td>0.8</td>
<td>5.8</td>
<td>-0.1</td>
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<td>0.95</td>
<td>0.3</td>
<td>0.5</td>
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</tr>
<tr>
<td>08-11</td>
<td>1.25</td>
<td>0.75</td>
<td>8.2</td>
<td>-0.1</td>
<td>1.3</td>
<td>0.9</td>
<td>0.3</td>
<td>0.5</td>
<td>0.04</td>
</tr>
<tr>
<td>10-11</td>
<td>1.35</td>
<td>1.0</td>
<td>7.4</td>
<td>-0.1</td>
<td>1.7</td>
<td>1.15</td>
<td>0.3</td>
<td>0.5</td>
<td>0.1</td>
</tr>
<tr>
<td>06-11</td>
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<td>0.7</td>
<td>5.9</td>
<td>-0.05</td>
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<td>09-11</td>
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<td>0.75</td>
<td>0.3</td>
<td>0.5</td>
<td>0.01</td>
</tr>
</tbody>
</table>

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

**Table 2.1.2** Net cross-shore bed-load transport in surf zone of Egmond beach, The Netherlands

![Graph](image)

**Figure 2.1.2** Net cross-shore bed-load transport at Egmond beach, The Netherlands

**Sable Island bank, the Scotian Shelf, Canada**

Amos et al. (1999) measured bed state, ripple migration and bed form transport of fine sand with \( d_{50} \) of 0.23 mm in 22 m of water on Sable Island bank, Scotian Shelf, Canada. Near-bed wave and steady flows (at about 1 m above the bed) were also recorded during a period of 12 days (see Table 2.1.3). The instrument package was mounted in a free-standing frame. Sand transport occurred under conditions where oscillatory and steady flows were orthogonal. The near-bed current was dominated by semi-diurnal tidal flows reaching up to 0.35 m/s. Two periods (days 182 and 186) of moderate waves with \( H_s \) between 1 and 1.5 m (wave period of about 9 s) were present in the measurement records. The latter part of the experiment was
relatively wave-free (days 188 to 192). Bed form transport was derived from ripple migration rates and estimated ripple heights. The measured values are shown in Figure 2.1.3. The bedload transport rates are relatively small just beyond initiation of motion. The bed load transport rate increases strongly (factor 5) with increasing velocities in the range of 0.25 to 0.35 m/s. The bed-load transport increases strongly (factor 5) with increasing wave height from 0.5 to 1 m.
Computed bed-load transport rates are discussed in Section 2.2.3.

<table>
<thead>
<tr>
<th>Test code</th>
<th>$H_s$ (m)</th>
<th>$T_p$ (s)</th>
<th>$u_{r-1 m}$ (m/s)</th>
<th>$q_b$ (kg/s/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Day 188-192</td>
<td>0.5</td>
<td>9</td>
<td>0.25</td>
<td>0.00025</td>
</tr>
<tr>
<td>Day 188-192</td>
<td>0.5</td>
<td>9</td>
<td>0.3</td>
<td>0.0005</td>
</tr>
<tr>
<td>Day 188-192</td>
<td>0.5</td>
<td>9</td>
<td>0.35</td>
<td>0.001</td>
</tr>
<tr>
<td>Day 182,186</td>
<td>1 to 1.5</td>
<td>9</td>
<td>0.22</td>
<td>0.00075</td>
</tr>
<tr>
<td>Day 182,186</td>
<td>1 to 1.5</td>
<td>9</td>
<td>0.3</td>
<td>0.001</td>
</tr>
</tbody>
</table>

Table 2.1.3  Bed-form transport in 22 m of water on Sable Island bank, Scotian Shelf, Canada

![Graph showing bed form transport in 22 m of water on Sable Island bank, Scotian Shelf, Canada](image)

Figure 2.1.3 Measured and computed bed-load transport rates for conditions just beyond initiation of motion in the ripple regime
2.2 Bed load transport model; single fraction method

2.2.1 Formulations

The net bed-load transport rate in conditions with uniform bed material is obtained by time-averaging (over the wave period $T$) of the instantaneous transport rate using a bed-load transport formula (quasi-steady approach), as follows:

$$q_b = \frac{1}{T} \int q_{b,i} \, dt \quad (2.2.1)$$

with $q_{b,i} = F(\text{instantaneous hydrodynamic and sediment transport parameters}).$

The applied bed-load transport formula is a parameterization 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 ($\text{kg/s/m}$) is related to the instantaneous bed-shear stress, which is based on the instantaneous velocity vector (including both wave-related and current-related components) defined at a small height above the bed. The formula applied, reads as:

$$q_{b} = \gamma \rho_s d_{50} D_*^{-0.3} \left[ \left[ \frac{U_{b,cw}}{U_{b,cw}} \right]^0.5 \left[ \frac{\tau_{b,cw}}{\tau_{b,cr}} \right] \right]$$

in which: $\tau_{b,cw}=\text{instantaneous grain-related bed-shear stress due to both currents and waves} = 0.5 \rho \tau_{cw} (U_{b,cw})^2$, $U_{b,cw}=\text{instantaneous velocity due to currents and waves at edge of wave boundary layer (see Appendix B)}$, $\tau_{cw}=\text{grain friction coefficient due to currents and waves} = \alpha \beta \tau_c + (1-\alpha) \tau_w$, $\tau_c=\text{current-related grain friction coefficient}$, $\tau_w=\text{wave-related grain friction coefficient}$, $\alpha=\text{coefficient related to relative strength of wave and current motion}$, $\beta=\text{wave-current-interaction coefficient (Appendix B)}$, $\tau_{b,cr}=\text{critical bed-shear stress according to Shields}$, $\rho_s=\text{sediment density}$, $\rho=\text{fluid density}$, $d_{50}=\text{particle size}$, $D_*=\text{dimensionless particle size}$, $\gamma=\text{coefficient}=0.5$, $\eta=\text{exponent}=1$.

The grain roughness is assumed to be $k_{b,grain} = \varepsilon d_{50}$ with $\varepsilon = 3$ for $d_{50} < 0.5 \text{mm}$; $\varepsilon = 1$ for $d_{50} > 1 \text{mm}$ and $\varepsilon = 3$ to 1 for intermediate values (van Rijn, 1993). The bed-load transport is assumed to be mainly affected by the grain roughness, but the overall bed-form roughness also has some (weak) influence on the bed-load transport in case of combined steady and oscillatory flow because of its effect on the near-bed velocity profile. Analysis of sensitivity computations for combined steady and oscillatory flow shows that the bed-load transport is reduced by about 15% for an increase of the bed-form roughness by a factor of 5 ($k_{b,cr} = 0.05$ m in stead of 0.01 m).

The measured bed-load transport rates of the wave tunnel tests B7, B8, B9 and C1 (Table 2.1.1) have been used to determine the $\gamma$ and $\eta$ coefficients, yielding $\gamma = 0.5$ and $\eta = 1$. These values have been used as standard values in the present study. The results of the calibration
are shown in Figure 2.1.1Top. Equation (2.2.2), which is applied in the TRANSPOR2000 model is slightly different from that applied in the TRANSPOR1993 model. Results of both models are compared in Section 2.2.3.

Proper predictive modelling of the wave-related (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). Previously, 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 Input data

The input data of the TRANSPOR2000 model are:

- $h$ = water depth (m)
- $v_{r}$ = depth-averaged and time-averaged current velocity due to tide, wind, waves (m/s)
- $u_{0}$ = depth-averaged and time-averaged return current below wave crest (m/s)
- $u_{b}$ = time-averaged velocity at edge of wave boundary layer generated by waves (m/s)
- $H_{s}$ = significant wave height (m)
- $T_{p}$ = peak period of wave spectrum (s)
- $\Phi$ = angle (anti-clockwise) between current direction and wave propagation direction (0 - 360 degrees , 0=360=following; current and wave in same direction, 180=opposing; wave direction opposite to current direction)
- $d_{50}$ = median particle diameter of bed material (m)
- $d_{90}$ = 90% particle diameter of bed material (m)
- $d_{r}$ = representative particle diameter of suspended sediment (m)
- $P_{mud}$ = percentage of mud in bed material (in %)
- $k_{c,c}$ = current-related bed-form roughness height (m)
- $k_{w,w}$ = wave-related bed-form roughness height (m)
- $S_{f}$ = fluid salinity (fresh water = 0 promille ; sea water = 30 promille)
- $B_{lb}$ = tangens of bed slope in current direction
- $B_{ly}$ = tangens of bed slope normal to current direction

The velocity components $v_{r}$ and $u_{0}$ are composed to one vector value, yielding:

$$V = (v_{r}^2 + u_{0}^2 + 2v_{r}u_{0}\cos(\Phi))^{0.5},$$

which is used to compute the current-related parameters.

The current-related and wave-related bed-form roughness values are important input parameters, which have a significant effect on the suspended sand transport and a minor effect on the bed-load transport due to their influence on the velocity profile and hence on the sediment mixing capacity of both the steady and oscillatory flow. Basically, the bed- form roughness parameters are a function of bed material composition and hydrodynamic conditions and should be described by predictors for bed-form dimensions and bed roughness. However, at present stage of research there are no reliable bed-form and bed roughness predictors and therefore the bed roughness parameters are used as input parameters in the present model. This implies that model users have to come up with
realistic estimates of theses parameters or use them as calibration parameters if transport data or morphological data are available.

2.2.3 Comparison of computed bed load transport with experimental results

Field data for rivers (steady flow)
Field data of bed-load transport for sand in the range of 0.25 to 0.5 mm in steady flow conditions with a depth of about 5 m are available for the Nile River in Egypt and the Rhine-Waal River in the Netherlands (Van Rijn, 1991, 1992; Gaweesh and Van Rijn, 1994; Abdel Fattah, 1997). The Nile data have been measured in depths of 3.5 to 5 m, current velocities in the range of 0.35 to 0.85 m/s and $d_{50}$-values in the range of 0.25 to 0.45 mm. The Rhine-Waal data have been measured in depths of 4 to 5 m, current velocities in the range of 0.45 to 1 m/s and $d_{50}$-values of about 0.53 mm. The measured bed-load transport rates are shown in Figure 2.2.1.

![Figure 2.2.1 Bed-load transport for steady flow according to TRANSPOR2000 model and TRANSPOR1993 model; computed values based on depth of $h= 5$ m; $d_{50} = 0.25$ mm and 0.5 mm](image-url)
Individual data points have been clustered as much as possible into data groups of current velocity to reduce the scatter. The values within the groups have been averaged to obtain representative group-averaged values. The variation range of the velocity within a group is about 10% of the mean value; the variation range of the corresponding bed-load transport rates is as large as 50%.

Three different bed-load transport expressions have been used to compute the bed-load transport for steady flow:

- bed-load transport expression implemented in TRANSPOR2000 model (Eq. 2.2.2);
- bed-load transport expression implemented in TRANSPOR1993 model (Appendix B);
- bed-load transport expression for river flow (Eq. 7.2.44, Van Rijn, 1993).

The bed material is taken as: $d_{50} = 0.25$ mm ($d_{90} = 0.5$ mm) and $d_{so} = 0.5$ mm ($d_{so} = 1$ mm).

The input data are taken as: current velocities in the range of 0.4-2 m/s, depth = 5 m, $k_c = 0.03$ m, temperature = 15 degrees, salinity = 0 promille.

The computed results are shown in Figure 2.2.1. The bed-load transport rates according to TRANSPOR2000 and TRANSPOR1993 are within 25% for the velocity range 0.7 to 2 m/s. Much larger discrepancies are present for conditions just beyond initiation of motion. The bed-load transport rates according to the TRANSPOR2000 model are considerably larger (factor 2 to 3) for velocities of 0.4 and 0.5 m/s. The bed-load transport expression for river flow (Eq. 7.2.44, Van Rijn, 1993) yields transport rates, which are considerably smaller (factor 2 to 3) than those of the TRANSPOR2000 model for current velocities < 0.8 m/s and considerably larger (factor 2) than those of the TRANSPOR2000 model for current velocities > 0.8 m/s. The computed values of the TRANSPOR2000 model are in reasonably good agreement with the measured bed-load transport rates over the full range of current velocities (0.35 to 1 m/s).

Finally, the TRANSPOR2000 model has been used to compute the bed-load transport rates for a field data set compiled by Van den Berg (1986), see Table 2.2.1. The data set consists of sand with $d_{50}$-values in the range of 0.43 to 1.05 mm, water depths between 0.75 and 10 m and current velocities between 0.45 and 1.55 m/s. All computed values are within a factor of 2 from the measured values, see Table 2.2.1.

<table>
<thead>
<tr>
<th>Rivers</th>
<th>$d_{50}$</th>
<th>$d_{90}$</th>
<th>$h$</th>
<th>$V$</th>
<th>$T_e$</th>
<th>Measured bed load transport (kg/s/m)</th>
<th>Computed bed load transport (kg/s/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dommel, The Netherlands</td>
<td>0.43</td>
<td>0.75</td>
<td>0.75</td>
<td>0.45</td>
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<td>0.007</td>
<td>0.0074</td>
</tr>
<tr>
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<td>0.75</td>
<td>0.75</td>
<td>0.55</td>
<td>16</td>
<td>0.012</td>
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</tr>
<tr>
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<td>0.75</td>
<td>0.75</td>
<td>0.6</td>
<td>16</td>
<td>0.015</td>
<td>0.024</td>
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<td>Skive Karup, Denmark</td>
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<td>0.025</td>
</tr>
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<td>0.028</td>
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<tr>
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<td>0.061</td>
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<td>1.75</td>
<td>9.8</td>
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<td>0.154</td>
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<tr>
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<td>1.4</td>
<td>11</td>
<td>0.1</td>
<td>0.105</td>
<td></td>
</tr>
</tbody>
</table>

Table 2.2.1  Summary of field data for river flow (Van den berg, 1986)
**Experiments of large scale oscillating water tunnel (oscillatory flow)**

The bed-load transport model was applied to compute the net bed-load transport rates for some experiments (except the calibration data B7, B8, B9 and C1) from the large wave tunnel of Delft Hydraulics (Table 2.1.1). 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 grain roughness was taken equal to $k_{s,grain} = 3d_{50}$. The overall 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 $15^\circ$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 2.1.1) at 0.1 m above the bed. Computed and measured bed-load transport rates are shown in Figure 2.1.1 Middle. Nearly all computed values are within 50% of the measured values. The bed-load transport model also gives the correct transport direction, see Figure 2.1.1 Middle. The effect of particle diameter on bed load transport is reasonably well represented for $d_{50}=0.13$, 0.21 and 0.32 mm (see Fig. 2.1.1 Bottom), taking the error range of the measured transport rates into account. The computed bed-load transport for $d_{50}=0.21$ mm is somewhat too small (40%), whereas for $d_{50}=0.97$ mm it is somewhat too large (about 40%).

**Field data for coastal sea (steady and oscillatory flow), Egmond site, 1992, The Netherlands**

Equation (2.2.2) was used to compute the net bed-load transport rates for conditions measured at the Egmond beach (Table 2.1.2). The measured undertow velocity and the measured peak onshore and offshore orbital velocities were used as input data. Other input data were: bed roughness= 0.03 m, water temperature= 10 °C. The computed values are shown in Figure 2.1.2; about 50% of the computed results are within a factor 2 of the measured values and 100% is within a factor 3, which is a rather encouraging result. It is noted that the inaccuracy of the measured bed-load transport rates is rather large. The computed net transport changes from onshore-directed to offshore-directed when the offshore-directed undertow is larger than about 15% of the onshore peak orbital velocity ($U_{undertow}/U_{1/3,oc}>0.15$). The measured values show a similar behaviour.

**Field data for coastal sea (steady and oscillatory flow), Sable Island bank, Scotian Shelf, Canada**

Equation (2.2.2) was used to compute bed-load transport rates (Table 2.1.3) for conditions near Sable Island bank, Scotian Shelf, Canada (Amos et al., 1999) using measured input data (water depth, current velocity, wave height, wave period, sand size). The bed roughness was assumed to be equal to the ripple height (0.03 m) given by Amos et al. (1999). For moderate wave conditions the wave height was assumed to be $H_s = 1$ m; whereas $H_t = 0.5$ m was used for wave-free conditions. Other input data are: $d_{50} = 0.23$ mm, $d_{90} = 0.5$ mm, angle current-waves= 90°; temperature= 10 °C, salinity= 30 promille. The computed bed-load transport is shown in Figure 2.1.3. Comparison of measured and computed values shows quite good agreement (within the error ranges) for both cases.

### 2.2.4 Effect of particle size on computed bed load transport

To demonstrate the effect of particle size on the bed-load transport, sensitivity computations for particle sizes 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, $d_{90} = 2d_{50}$, wave-current angle= $90^\circ$, 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, $u_{c,\text{cross}} = 0$ 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, $u_{c,\text{cross}} = 0$ 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, $u_{c,\text{cross}} = -0.5$ m/s, $k_{s,c} = k_{s,w} = 0.01$ m;

The longshore and cross-shore currents are assumed to increase for increasing wave-energy. The bed-form related roughness ($k_{s,c}$ and $k_{s,w}$) are assumed to decrease for increasing wave energy (simulating the transition from the ripple to the sheet flow regime). The bed-load transport is mainly affected by grain roughness. The latter parameter has been varied to show its effect on the bed-load transport. Three approaches have been used: the standard approach with $k_{s,\text{grain}}$ between $3d_{90}$ and $1d_{90}$ (see Eq. 2.2.2), $k_{s,\text{grain}} = 3d_{90}$ and $k_{s,\text{grain}} = 1d_{90}$ for all conditions.

The bed-load transport vector as a function of particle size is shown in Figure 2.2.2. The bed-load transport vector includes both the wave-related and current-related transport components. The bed-load transport for the L-E event only includes the wave-related transport component, because the current velocity is zero ($v = 0$ m/s).

The results are:

- the bed-load transport increases with increasing energy conditions (factor 10 to 20 from low-energy to high-energy conditions);
- the bed-load transport increases with increasing grain roughness; $k_{s,\text{grain}} = 3d_{90}$ yields transport rates which are about 1.5 to 2 times larger than those for $k_{s,\text{grain}} = 1d_{90}$;
- the bed-load transport increases significantly with increasing particle size up to about 0.5 mm;
- the bed-load transport is constant ($k_{s,\text{grain}} = 3d_{90}$) or decreases weakly ($k_{s,\text{grain}} = 1d_{90}$) with increasing particle size for values larger than about 1 mm;
- the bed-load transport decreases significantly between 0.5 mm and 1 mm, if the grain roughness is reduced from $3d_{90}$ to $1d_{90}$ (standard approach).

To some extent, these results are in agreement with the particle size trend of Figure 2.1.1 Bottom. The model results show however a maximum bed-load transport rate for a particle size of about 0.5 mm, whereas the wave tunnel data suggests a value of about 0.3 mm.

The effect of particle size on suspended load transport is discussed in Section 3.3.6.
Figure 2.2.2  Effect of particle size on computed bed-load transport (TRANSPOR 2000)
2.3 Conclusions

Bed load transport processes

- The study of bed-load transport in coastal conditions is rather problematic, because reliable data of bed-load transport in field conditions are hardly available. Therefore, the bed load transport under waves and currents generally is studied through experiments in wave tunnels and tanks.

- Bed-load transport experiments under sheet flow conditions have been performed in the large-scale wave tunnel of Delft Hydraulics; the experimental conditions concern regular symmetric and asymmetric waves (2nd 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 0.97 mm.

- The net bed load transport in the sheet flow regime has the same direction as the largest peak orbital velocity near the bed and the net bed load transport rate is proportional to the power 3.5 of the peak forward (onshore) orbital velocity.

- The direction of the net bed-load transport due to oscillatory flow 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 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.

- The net bed-load 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 results in a decrease of the transport rate by a factor of 2.

- Bed-load transport measurements using a mechanical trap sampler have been performed in the surf zone near Egmond (The Netherlands). The net transport is offshore-directed when the offshore-directed undertow is larger than about 15% of the onshore peak orbital velocity and onshore-directed for other conditions, which is in reasonable agreement with the results of the wave tunnel experiments.

- Bed-load transport rates have been derived from ripple migration measurements in 22 m depth near Sable Island bank, Scotian Shelf, Canada. The near-bed current is dominated by semi-diurnal tidal flows reaching up to 0.35 m/s. The wave conditions are moderate with $H_s$ between 1 and 1.5 m. The bed-load transport rates are found to be relatively small during conditions just beyond initiation of motion. The bed-load transport rate increases strongly (factor 5) with increasing velocities in the range of 0.25 and 0.35 m/s. The bed-load transport increases strongly (factor 5) with increasing wave height from 0.5 to 1 m.
**Bed load transport model**

- The net bed-load transport rate in conditions with combined steady and oscillatory flow over a sand bed can be reasonably well described (within factor 2 to 3) by time-averaging (over the wave period) of the instantaneous transport rates using a quasi-steady bed-load transport formula approach; the bed-load transport model results show good agreement with laboratory and field data for steady and oscillatory flow with sand in the range of 0.15 to 1 mm in the ripple and flat bed regime without adjustment of model coefficients.

- The bed-load transport is mainly affected by the grain roughness. The overall bed-form roughness also has some (weak) influence on the bed-load transport in case of combined steady and oscillatory flow because of its effect on the near-bed velocity profile.

- The effect of particle size on bed-load transport can be reasonably well represented for particle sizes in the range of about 0.15 to 1 mm.
3 Modelling of suspended sand transport for uniform bed material; single fraction method

3.1 Definitions

The net depth-integrated suspended sand transport is defined as the sum of the net current-related ($q_{sc}$) and the net wave-related ($q_{sw}$) transport components, as follows:

$$q_s = q_{sc} + q_{sw} = \int vc \, dz + \int <(V-v)(C-c)> \, dz$$  \hspace{1cm} (3.1.1)

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

The current-related suspended transport ($q_{sc}$) is defined as the advective transport of sediment particles by the time-averaged (mean) current velocities (longshore currents, rip currents, undertow currents). Thus, the transport of sediment which is carried by the steady flow. In conditions with waves 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 enhanced due to the stirring action of the waves. These effects are included in the current-related transport.

The wave-related suspended sediment transport ($q_{sw}$) 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. The wave-related transport components are defined in the plane of orbital motion.

For practical reasons the current-related and the wave-related transport components are studied separately. Furthermore, 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 Identification of data sets

*Currents only (river flow and tidal flow)*
Reliable field data sets from major rivers and estuaries available in the literature are shown in Table 3.2.1. Only data sets with water depths larger than 1 m have been selected. The particle size range is 0.1 to 2.3 mm. The depth-averaged current velocity range is 0.3 to 2.2 m/s. Individual data points have been clustered as much as possible into data groups of water depth and current velocity to reduce the scatter. In some cases all available data points have been used. The values within the groups have been averaged to obtain representative group-averaged values. The variation range of the velocity and water depth within a group is about 10% of the mean value; the variation range of the corresponding suspended transport rates is as large as 50%. Plots of transport against current velocity for various particle size groups are shown and discussed in Section 3.2.3.

<table>
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<th>Source</th>
<th>N</th>
<th>d₅₀ (mm)</th>
<th>h (m)</th>
<th>V (m/s)</th>
<th>BF</th>
<th>Transport</th>
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<td>0.15-0.4</td>
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<td>1-2</td>
<td>1.2-2</td>
<td>n.m.</td>
<td>SLT</td>
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<td>4-6</td>
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<td>9-11</td>
<td>1.4-2</td>
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<td>0.2-0.6</td>
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</table>

N = number of cases (number of groups between brackets)

h = water depth

V = depth-averaged current velocity

BF = bed forms (R=ripples, D=dunes and F= flat bed, n.m. = not measured)

BLT = bed load transport, SLT= suspended transport

Table 3.2.1 Summary of field data for river and tidal flow conditions

Current and wave conditions (coastal seas)

Reliable data of suspended transport processes in coastal conditions are rather scarce. Field measurements have been carried out at the Maplin Sands and Boscombe Bay sites in the UK, at the Duck surf zone site in the USA and at the Egmond surf zone site in The Netherlands. Some data are available for a tidal flat (Galgeplaat) in the Eastern Scheldt Estuary, The Netherlands. Furthermore, data sets measured in the large-scale wave tank (Delta flume of Delph Hydraulics) are available to study the vertical distribution of time-averaged sand concentrations. The data refer to time-averaged sand concentrations measured by pump sampling, optical and acoustical instruments. Time-averaged velocity profiles have also been measured at the field sites by electro-magnetic instruments. Based on both
parameters (velocities and concentrations), the depth-integrated suspended transport can be estimated.

In all, data sets are available for the following cases (see Tables 3.2.2 to 3.2.8):

- **Large scale wave tank of Delft Hydraulics;**
  \( h = 1 \text{ to } 5 \text{ m, } H = 0.3 \text{ to } 1.5 \text{ m, } v = 0 \text{ m/s, } d_{50} = 0.15 \text{ to } 0.35 \text{ mm}; \)
  The wave tank has a length of about 200 m, a width of 5 m and a depth of 7 m. Irregular waves up to \( H \leq 1.5 \text{ m} \) can be generated in the wave tank. Time-averaged sand concentrations have been measured by use of a pump sampling system at various levels above the bed (between 0.02 m and the water surface); wave-related suspended transport rates between the lowest and highest measurement points are available for some tests.

- **Maplin Sands, 1973-1975 and Boscombe Bay, 1977-1978, UK;**
  \( h = 3 \text{ to } 6 \text{ m, } H = 0.5 \text{ to } 1 \text{ m, } v = 0.2 \text{ to } 0.8 \text{ m/s, } d_{50} = 0.14 \text{ to } 0.25 \text{ mm}; \)
  Maplin Sands is located at the northern side of the Outer Thames Estuary. The measurements were taken from a platform positioned on the sloping flank of a large scale sandy shoal. It forms the northern edge of a natural deep channel. The water depths range from 2 to 7 m. The tidal range is maximum 5 m.
  The Boscombe Pier is located in the Boscombe Bay and lies approximately mid-way between Poole Harbour and Hengistbury Head. The coastline is characterised by an easterly drift of sand and shingle. The pier has an overall length of 215 m and is oriented normal to the local coastline. The water depths vary in the range of 3 to 6 m. The tidal range is maximum 2 m. The bed is sandy; the upper beach has groynes.
  At both sites the flow field and sediment concentration were taken from a trackway mounted on the seaward end of the pier/platform. The instruments were mounted on a trolley, which was winched up and down the trackway. At the bottom of the trolley was a thin bed contact disc to detect the local bed. The measurement points were at regular distances between 0.05 m above the bed and the water surface. The flow velocity was measured by using electro-magnetic current meters. The time-averaged sand concentrations were measured by using a pump sampling system. Sample volumes of 20 litres were pumped. The sand particles were separated off by using a filter of 0.04 mm.

  \( h = 1 \text{ to } 6 \text{ m, } H = 0.2 \text{ to } 1.5 \text{ m, } v = 0.1 \text{ to } 0.7 \text{ m/s, } d_{50} = 0.25 \text{ to } 0.35 \text{ mm}; \)
  The Egmond site is located in the central part of the Dutch North Sea coast and consists of a sandy beach (about 0.3 mm sand). The local morphology is 2.5 dimensional exhibiting two longshore bars intersected by local rip channels; the bars are aligned parallel to the shore most of the time, but crescentic bar forms do also occur. The wave climate is dominated by sea waves with a mean annual significant offshore wave height of about 1.1 m. The tidal range varies between 1.4 m (neap) and 2 m (spring). The tidal peak currents in the offshore zone are about 0.5 m/s; the flood current to north is slightly larger than the ebb current to the south.
  In the period 1989-1992, wave height, flow velocity and sand concentration measurements have been carried at various locations in the swash zone using a small movable platform. Instantaneous fluid velocities have been measured by electro-magnetic current meters and acoustical sensors; time-averaged concentrations have been measured by use of a pump sampling system at various levels above the bed (between 0.05 m and the water surface).
  In 1998, wave height, flow velocity and sand concentration measurements have been carried out at various locations at the inner bar in the surf zone using the CRIS trailer connected to the WESP. Instantaneous fluid velocities have been measured by electro-
magnetic current meters and acoustic sensors; instantaneous sand concentrations by acoustic sensors; bed form profiles by an acoustic profiler. A pump system was available for collection of water-sediment samples (calibration). The WESP is an approximately 15 m high amphibious 3-wheel vehicle; the CRIS is a 3.5 m square and 2.5 m high trailer. The instruments on the CRIS are attached to a movable arm, which can be adjusted in vertical direction to position the sensors at the desired elevation above the bed. Sand transport measurements were performed at eight elevations from about 0.02 to 1.0 m above the bed.

- **Tidal flat Galgeplaat, Eastern Scheldt, The Netherlands, 1983:**
  h = 2 to 4 m, H_s = 0.7 m, v = 0.3 to 0.7 m/s, d_50 = 0.15 mm; tidal range is about 2 m. The bed surface of the tidal flat generally consists of fine sand covered with ripple type bed forms. Time-averaged concentrations have been measured by use of a pump sampling system at 5 to 10 levels between 0.02 m and 0.5 m above the bed operated from a platform on the tidal flat. Time-averaged fluid velocities have been measured at one height above the bed (0.3 m) by an electromagentic current meter.

- **Duck site, USA;**
  To be described later.

Individual data points have been clustered into groups as much as possible. The data parameters are the average values of the groups; the variation range of the water depth is about 10% of the mean value; the variation range of the current velocity is about 25%. The variation range of the suspended transport is as large as 50% of the mean value.

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<th>Source</th>
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<th>H_s (m)</th>
<th>T_p (s)</th>
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<th>d_50 (mm)</th>
<th>d_5 (mm)</th>
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<td>0.21</td>
<td>0 (pl)</td>
<td>0 (pl)</td>
<td>0.01</td>
<td>-</td>
<td>10</td>
</tr>
</tbody>
</table>

h = water depth, H_s = significant wave height, T_p = peak wave period
d_5 = representative suspended sand size (estimated)
k_w = wave-related bed form roughness (not measured, but estimated)
q_s,CR = wave-related (high freq.) cross-shore suspended sand transport (- offshore, + onshore)
Δ_h = bed form height (pl= plane bed), λ_h = bed form length, Te = temperature (Celsius)

**Table 3.2.2** Summary of wave-related suspended sand transport data for large-scale wave tank (Delta flume of Delft Hydraulics)
<table>
<thead>
<tr>
<th>Source</th>
<th>$h$ (m)</th>
<th>$H_s$ (m)</th>
<th>$T_z$ (s)</th>
<th>$v$ (m/s)</th>
<th>$\varphi$ (deg)</th>
<th>$d_{50}$ (mm)</th>
<th>$d_{90}$ (mm)</th>
<th>$d_s$</th>
<th>Bed forms</th>
<th>$q_{c,e}$ (kg/s/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Maplin Sands</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1973-1975</td>
<td>0.5</td>
<td>0.5</td>
<td>0.45</td>
<td>0-30; 150-180</td>
<td>0.14</td>
<td>0.3</td>
<td>0.09</td>
<td>Ripples</td>
<td>0.13</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.5</td>
<td>0.55</td>
<td>0-30; 150-180</td>
<td>0.14</td>
<td>0.3</td>
<td>0.09</td>
<td>Ripples</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.5</td>
<td>0.65</td>
<td>0-30; 150-180</td>
<td>0.14</td>
<td>0.3</td>
<td>0.09</td>
<td>Ripples</td>
<td>0.46</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>3.5</td>
<td>0.75</td>
<td>0-30; 150-180</td>
<td>0.14</td>
<td>0.3</td>
<td>0.09</td>
<td>Ripples</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.9</td>
<td>3.1</td>
<td>0.34</td>
<td>0-30; 150-180</td>
<td>0.14</td>
<td>0.3</td>
<td>0.09</td>
<td>Ripples</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.9</td>
<td>3.1</td>
<td>0.45</td>
<td>0-30; 150-180</td>
<td>0.14</td>
<td>0.3</td>
<td>0.09</td>
<td>Ripples</td>
<td>0.6</td>
</tr>
<tr>
<td><strong>Boscombe Bay</strong></td>
<td>3.7</td>
<td>0.8</td>
<td>6.3</td>
<td>0.3</td>
<td>150-180</td>
<td>0.25</td>
<td>0.45</td>
<td>0.18</td>
<td>Ripples</td>
<td>0.055</td>
</tr>
<tr>
<td>(pier)</td>
<td>1977-1978</td>
<td></td>
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<tr>
<td></td>
<td>4.8</td>
<td>0.45</td>
<td>7.2</td>
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<td>0.45</td>
<td>0.17</td>
<td>Ripples</td>
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<td>0.23</td>
<td>Ripples</td>
<td>0.0022</td>
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<td>4.9</td>
<td>0.95</td>
<td>6.8</td>
<td>0.24</td>
<td>150-180</td>
<td>0.25</td>
<td>0.45</td>
<td>0.2</td>
<td>Ripples</td>
<td>0.055</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>1</td>
<td>6.6</td>
<td>0.3</td>
<td>150-180</td>
<td>0.25</td>
<td>0.45</td>
<td>0.2</td>
<td>Ripples</td>
<td>0.08</td>
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<td>5.3</td>
<td>1.05</td>
<td>6.5</td>
<td>0.4</td>
<td>150-180</td>
<td>0.25</td>
<td>0.45</td>
<td>0.2</td>
<td>Ripples</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Temperature=5-10 °C, Salinity= 30 promille

$h$ = water depth, $H_s$ = significant wave height, $T_z$ = wave period
$\varphi$ = angle between waves and current
$v$ = depth-mean velocity vector
$d_s$ = representative suspended sand size (measured)
$q_{c,e}$ = current-related suspended sand transport vector

**Table 3.2.3** Summary of current-related suspended sand transport data at British sites (Whitehouse et al., 1996, 1997)

<table>
<thead>
<tr>
<th>Source</th>
<th>$h$ (m)</th>
<th>$H_s$ (m)</th>
<th>$T_P$ (s)</th>
<th>$u_L$ (m/s)</th>
<th>$u_{CR}$ (m/s)</th>
<th>$d_{50}$ (mm)</th>
<th>$d_{90}$ (mm)</th>
<th>$q_{c,e}$ (kg/s/m)</th>
<th>$q_{c,e,CR}$ (kg/s/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Egmond 1989-1990; 3A</td>
<td>1.25</td>
<td>0.5</td>
<td>4</td>
<td>0.1</td>
<td>-0.08</td>
<td>0.3</td>
<td>0.6</td>
<td>0.005</td>
<td>-0.0043</td>
</tr>
<tr>
<td>Egmond 1989-1990; 3B</td>
<td>1.25</td>
<td>0.5</td>
<td>4</td>
<td>0.22</td>
<td>-0.18</td>
<td>0.3</td>
<td>0.6</td>
<td>0.015</td>
<td>-0.012</td>
</tr>
<tr>
<td>Egmond 1989-1990; 3C</td>
<td>1.1</td>
<td>0.33</td>
<td>3.3</td>
<td>0.24</td>
<td>-1</td>
<td>0.3</td>
<td>0.6</td>
<td>0.012</td>
<td>-0.005</td>
</tr>
<tr>
<td>Egmond 1989-1990; 3D</td>
<td>1.05</td>
<td>0.23</td>
<td>3.3</td>
<td>0.06</td>
<td>-0.15</td>
<td>0.3</td>
<td>0.6</td>
<td>0.0025</td>
<td>-0.0035</td>
</tr>
<tr>
<td>Egmond 1989-1990; 3E</td>
<td>1.12</td>
<td>0.62</td>
<td>4</td>
<td>0.55</td>
<td>-0.35</td>
<td>0.3</td>
<td>0.6</td>
<td>0.35</td>
<td>-0.2</td>
</tr>
<tr>
<td>Egmond 1992; 4A</td>
<td>1.3</td>
<td>0.70</td>
<td>7</td>
<td>0.12</td>
<td>-0.1</td>
<td>0.35</td>
<td>0.8</td>
<td>0.07</td>
<td>-0.07</td>
</tr>
<tr>
<td>Egmond 1992; 4B</td>
<td>1.3</td>
<td>0.65</td>
<td>7</td>
<td>0.35</td>
<td>-0.35</td>
<td>0.35</td>
<td>0.8</td>
<td>0.35</td>
<td>-0.30</td>
</tr>
<tr>
<td>Egmond 1992; 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>0.05</td>
<td>-0.015</td>
</tr>
<tr>
<td>Egmond 1992; 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>0.22</td>
<td>-0.1</td>
</tr>
<tr>
<td>Egmond 1992; 4G</td>
<td>1.6</td>
<td>0.9</td>
<td>7</td>
<td>0.45</td>
<td>-0.3</td>
<td>0.35</td>
<td>0.8</td>
<td>0.5</td>
<td>-0.4</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 current velocity in longshore direction
$u_{CR}$ = depth-mean undertow velocity (- offshore, + onshore) in cross-mouth direction
$d_{s}$ = representative suspended sand size (not measured, but estimated $d_{s}=0.8d_{so}$)
$k_{sw}$ = wave-related bed form roughness (not measured, but estimated to be 0.03 m)
$k_{sc}$ = current-related bed form roughness (not measured, but estimated to be 0.03 m)
$q_{c,e}$ = current-related longshore suspended sand transport
$q_{c,e,CR}$ = current-related cross-shore suspended sand transport (- offshore, + onshore)

**Table 3.2.4** Summary of current-related suspended sand transport data at Egmond swash and surf zone 1989-1992, The Netherlands (Kroon, 1994 and Wolf, 1997).
<table>
<thead>
<tr>
<th>Source</th>
<th>h (m)</th>
<th>H₀ (m)</th>
<th>Tₑ (s)</th>
<th>v (m/s)</th>
<th>φ (deg.)</th>
<th>dₜₘ (mm)</th>
<th>dₘₙ (mm)</th>
<th>dₜₘ (mm)</th>
<th>Bed forms</th>
<th>qₑₑ (kg/s/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Egmond 1998</td>
<td>5.1</td>
<td>1</td>
<td>7.3</td>
<td>0.1</td>
<td>90</td>
<td>0.25</td>
<td>0.5</td>
<td>0.2</td>
<td>Ripples</td>
<td>0.009</td>
</tr>
<tr>
<td></td>
<td>5.3</td>
<td>1.2</td>
<td>8.1</td>
<td>0.2</td>
<td>90</td>
<td>0.25</td>
<td>0.5</td>
<td>0.2</td>
<td>Ripples</td>
<td>0.014</td>
</tr>
<tr>
<td></td>
<td>4.4</td>
<td>1.4</td>
<td>6.1</td>
<td>0.45</td>
<td>90</td>
<td>0.25</td>
<td>0.5</td>
<td>0.2</td>
<td>Ripples</td>
<td>0.049</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1</td>
<td>6.5</td>
<td>0.2</td>
<td>90</td>
<td>0.25</td>
<td>0.5</td>
<td>0.2</td>
<td>Ripples/Flat</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1</td>
<td>6.5</td>
<td>0.45</td>
<td>90</td>
<td>0.25</td>
<td>0.5</td>
<td>0.2</td>
<td>Ripples/Flat</td>
<td>0.12</td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td>0.65</td>
<td>4.6</td>
<td>0.3</td>
<td>90</td>
<td>0.25</td>
<td>0.5</td>
<td>0.2</td>
<td>Ripples/Flat</td>
<td>0.031</td>
</tr>
<tr>
<td></td>
<td>1.4</td>
<td>0.65</td>
<td>7</td>
<td>0.1</td>
<td>90</td>
<td>0.25</td>
<td>0.5</td>
<td>0.2</td>
<td>Ripples/Flat</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>1.9</td>
<td>0.7</td>
<td>7.7</td>
<td>0.3</td>
<td>90</td>
<td>0.25</td>
<td>0.5</td>
<td>0.2</td>
<td>Ripples/Flat</td>
<td>0.11</td>
</tr>
</tbody>
</table>

Temperature=5-10 °C, Salinity= 30 promille

h = water depth, H₀ = significant wave height, Tₑ = wave period
φ = angle between waves and current
v = depth-mean longshore velocity vector
dₜₘ = representative suspended sand size (measured)
qₑₑ = current-related suspended longshore sand transport vector

Table 3.2.5  Summary of current-related longshore suspended sand transport data at Egmond surf zone for water depth classes 4-5 m, 3 m and 1-2 m (Grasmeijer, 2001)
<table>
<thead>
<tr>
<th>Source</th>
<th>h</th>
<th>Hs</th>
<th>Tz</th>
<th>v</th>
<th>φ</th>
<th>d50</th>
<th>d90</th>
<th>d95</th>
<th>Bed forms</th>
<th>qcc</th>
</tr>
</thead>
<tbody>
<tr>
<td>Galgeplaat, Eastern Scheldt Estuary, 1983</td>
<td>3.45</td>
<td>0.69</td>
<td>3.5</td>
<td>0.6</td>
<td>n.m.</td>
<td>0.15</td>
<td>0.21</td>
<td>0.12</td>
<td>Ripples</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>4.01</td>
<td>0.69</td>
<td>3.5</td>
<td>0.65</td>
<td>n.m.</td>
<td>0.15</td>
<td>0.21</td>
<td>0.12</td>
<td>Ripples</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td>3.68</td>
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<td>0.35</td>
<td>n.m.</td>
<td>0.15</td>
<td>0.21</td>
<td>0.12</td>
<td>Ripples</td>
<td>0.08</td>
</tr>
</tbody>
</table>

Temperature=5-10 °C, Salinity= 30 promille

h = water depth, Hs = significant wave height, Tz = wave period
φ = angle between waves and current
v = depth-mean longshore velocity vector
d50 = representative suspended sand size (estimated)
qcc = current-related suspended sand transport vector

**Table 3.2.6** Summary of current-related suspended sand transport data at tidal flat Galgeplaat, Eastern Scheldt Estuary, (Van Rijn, 2000)

<table>
<thead>
<tr>
<th>Class</th>
<th>U1/3,an</th>
<th>U1/3,off</th>
<th>Tp</th>
<th>uL</th>
<th>ucr</th>
<th>d50</th>
<th>d90</th>
<th>d95</th>
<th>Bed forms</th>
<th>qcc.I</th>
<th>qcc.CR</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m/s</td>
<td>m/s</td>
<td>s</td>
<td>m/s</td>
<td>m/s</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>kg/s/m</td>
<td>kg/s/m</td>
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</table>

**Table 3.2.7** Summary of current-related suspended sand transport data at Egmond surf zone for six peak orbital velocity classes, The Netherlands (Grasmeijer 2001)

<table>
<thead>
<tr>
<th>Class</th>
<th>U1/3,an</th>
<th>U1/3,off</th>
<th>Tp</th>
<th>uL</th>
<th>ucr</th>
<th>d50</th>
<th>d90</th>
<th>d95</th>
<th>Bed forms</th>
<th>qcc.I</th>
<th>qcc.CR</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>m/s</td>
<td>m/s</td>
<td>s</td>
<td>m/s</td>
<td>m/s</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>kg/s/m</td>
<td>kg/s/m</td>
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<td></td>
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</tr>
</tbody>
</table>

**Table 3.2.8** Summary of current-related suspended sand transport data at Duck site, USA
3.2.2 Time-averaged sand concentrations in combined current and wave conditions

Large scale wave tank (Delta flume) 1991-1997 and Egmond site, 1989-1992

Examples of measured sand concentration profiles from experiments in the Delta flume of DH and from the field data at the Egmond site are shown in Figures 3.2.1 to 3.2.4. Based on analysis of the measured sand concentration profiles (time-averaged values of about 15 min.), the following features are given:

- the near-bed concentrations increase with increasing relative wave height (H_r/h) and the concentration profile becomes more uniform for increasing relative wave height,
- the near-bed concentrations (at 0.05 m above the bed) are about:
  - 0.2 to 0.5 kg/m³ for H_r/h=0.2 to 0.3,
  - 0.5 to 1.0 kg/m³ for H_r/h=0.3 to 0.4,
  - 1.0 to 3.0 kg/m³ for H_r/h=0.4 to 0.5,
  - 2.0 to 5.0 kg/m³ for H_r/h=0.5 to 0.9,
- the concentrations are confined to the near-bed region (z/h<0.1) for non-breaking waves (H_r/h< 0.3); the concentration gradient is approximately constant implying a constant near-bed sediment mixing coefficient,
- the concentration profile consists of a two layer system (z/h<0.1 and >0.1) for relative wave heights (H_r/h) between 0.3 and 0.5; the concentration gradient of each layer is approximately constant,
- the concentration profile is almost uniform for relatively large breaking waves (H_r/h between 0.5 and 1),
- the presence of relatively large current velocities (>0.5 m/s) has a strong effect on the concentration profile; both the near-bed concentrations and the concentrations in the outer layer are enhanced (compare Case 3E and 2F) due to increased bed-shear stresses and increased mixing capacity,
- the presence of bed forms has a strong effect on the concentration profile; comparison of concentration profiles of Case 2F (H_r/h=0.55 over a flat bed, Figure 3.2.2) and Case 2C (H_r/h=0.19 over a rippled bed) shows the presence of similar concentrations in the near-bed region;
- the variability of time-averaged concentrations is relatively large; factor 5 near the bed and factor 2 in the outer layer (see Case 2G of Figure 3.2.2 and Cases 4B to 4E of Figure 3.2.4).
Figure 3.2.1  Measured sand concentration profiles for Tests 1A, 1B, 1C and 1E of Delta flume experiments; data used for calibration of suspended transport model
Figure 3.2.2  Measured sand concentration profiles for Tests 2A, 2C, 2F, 2G, 2H and 2I of Delta flume experiments; data used for calibration of suspended transport model
Figure 3.2.3  Measured sand concentration profiles for data sets 3A, 3B, 3C and 3D of Egmond surf zone 1989-1992, The Netherlands; data used for validation of suspended transport model
Figure 3.2.4 Measured sand concentration profiles for data sets 4B, 4C, 4E and 4G of Egmond surf zone, The Netherlands; data used for validation of suspended transport model
**Boscombe Bay, 1977-1978, UK**

Sand concentration profiles measured at the Boscombe Bay site in UK (Whitehouse et al., 1997) are shown in Figure 3.2.5.

The following features are given:

- the near-bed concentrations increase with increasing relative wave height (Hₜ/h) and with increasing current velocity;
- the near-bed concentrations (at 0.05 m above the bed) are about:
  - 0.03 to 0.08 kg/m³ for Hₜ/h=0.1;
  - 0.5 to 1.0 kg/m³ for Hₜ/h=0.2;
- the concentration profile is somewhat more uniform for increasing current velocity (from 0.2 to 0.4 m/s) due to current-induced mixing.

![Graph showing sand concentration profiles](image)

**Figure 3.2.5**  *Measured sand concentration profiles for Boscombe Bay, UK*

**Egmond site, 1998, The Netherlands**

To be described later

**Duck site, USA**

To be described later

### 3.2.3 Current-related suspended sand transport (qₑₑ)

**Current only (river flow and tidal flow)**

The most basic case is the suspended transport by a current without waves (river and tidal flow). Field data sets from major rivers and estuaries available in the literature (see Table 3.2.1, Section 3.2.1) have been analysed to study the relationship between sand transport, current velocity, water depth and sand diameter. Individual data points have been clustered as much as possible into data groups of water depth and current velocity to reduce the scatter. The data points within the groups have been averaged to obtain representative values. The variation range of the water depth within a group is about 10% of the mean...
value; the variation range of the current velocity is 10% to 25%; the variation range of the corresponding suspended transport rates is as large as 50%.

Figures 3.2.6 to 3.2.9 show the suspended sand transport rates (including variation ranges) as a function of depth-averaged velocity for four sand diameter classes (0.14-0.18 mm; 0.18-0.4 mm; 0.4-0.6 mm and 2.5-3 mm). Most of the data refer to suspended sand transport. Both bed-load and suspended transport rates have been measured in the Nile River and in the Rhine-Waal River. Analysis of the results shows that the effect of water depth (between 1 and 15 m) on sand transport is of minor importance. The sand transport is dominantly dependent on the current velocity; the sand transport increases from about 0.001 to 10 kg/s/m for velocities increasing from 0.4 to 2.2 m/s. Bed-load transport dominates at low velocities in the Nile River. Trend lines of sand transport against velocity are also shown in Figures 3.2.6 to 3.2.8 and are summarized in Figure 3.2.9. This latter graph shows that the effect of sand diameter (between 0.14 and 0.6 mm) on sand transport is rather strong for low velocities, but gradually reduces for larger velocities (larger than about 1.4 m/s). The suspended transport of fine sand in the range of 0.14-0.18 mm is relatively large at low velocities (0.4-0.5 m/s) compared that of sand in the range of 0.18-0.4 mm. This may be related to the effect of ripple type bed forms, which may be rather pronounced in conditions with fine sand bed of 0.14-0.18 mm. Another cause may be the presence of selective transport processes in the sense that the finer fractions of the sand bed are winnowed and more easily transported in suspension. Furthermore, most of the data in the low velocity range have been measured in non-equilibrium tidal flow conditions with settling sediments (Maplin Sands and Foulness, UK; see Figure 3.2.6). The suspended size of the samples taken at Maplin Sands is about 0.08 mm, which is rather small compared to the bed material size ($d_{95}$ $d_{50,bed}$). This may be an indication that the fines originate from non local sources and hence that the measured transport rates are somewhat too large.

![Graph showing sand transport as function of current velocity for $d_{50}=0.14-0.18$ mm](image)

**Figure 3.2.6 Sand transport as function of current velocity for $d_{50}=0.14-0.18$ mm**

Results of laboratory experiments show somewhat smaller suspended transport rates of sand in the range of 0.14-0.18 mm at low velocities (Van Rijn, 1986). For example, the suspended transport was about 0.03 kg/s/m at a depth-averaged velocity of 0.5 m/s in a flume with depth of 0.4 m and a ripple-covered sand bed ($k_s = 0.025$ m). This value is a factor 2 smaller.
than that of the trendline of Figure 3.2.6 suggesting that the field data may have been collected in non-equilibrium (overload) conditions. More data sets in the fine sand range at low velocities are necessary to better understand the suspended transport of fine sand at low velocities.

Figure 3.2.7 Sand transport as function of current velocity for $d_{50} = 0.18-0.4$ mm; most data refer to suspended transport; bed load transport is included for Nile River

Figure 3.2.8 Sand transport as function of current velocity for $d_{50} = 0.4-0.6$ mm and 2.8 mm; bed load transport is included for Nile River and Waal River
Coastal conditions: Maplin Sands and Boscombe Bay, UK and Egmond, The Netherlands

Field data sets for combined current and wave conditions in the coastal zone are given in Tables 3.2.3 to 3.2.8 (Section 3.2.1).

Figures 3.2.10 and 3.2.11 show the suspended sand transport rates (including variation ranges) as a function of depth-averaged velocity and wave height for two sand diameter classes (0.14-0.18 mm; 0.18-0.4 mm). The trend lines are also shown. The trend lines for conditions with current only (Hc=0 m) are shown for comparison. The sand transport in the coastal zone is found to be strongly dependent on the relative wave height (Hc/h), particularly for current velocities in the range 0.2 to 0.5 m/s. The transport of sand in the size range 0.18-0.4 mm increases by a factor of 10 to 20 when waves with a relative wave height of about 0.2 are superimposed on a current of about 0.4 m/s (Figure 3.2.10Top). This factor may increase to about 50 for a relative wave height of about 0.33 (Figure 3.2.10 Bottom). Thus, the waves are very effective in the stirring of sand from the bed into the water column. The trend lines seem to indicate that the increase of the transport due to the wave effect decreases with increasing current velocity, particularly for finer sand (0.14-0.18 mm).

Figure 3.2.10Top shows that the increase of the transport of 0.14-0.18 mm sand is not more than a factor of 2 to 3 when waves with a relative wave height of 0.2 are superimposed on a current of 0.8 m/s. Information of the suspended sand size is given in Table 3.2.3. The suspended sand size is about 0.65d50 at the site Maplin Sands with a rather fine sand bed of d50=0.14 mm. The suspended sediments at this site are rather fine and may also originate from other (non local) sources. The suspended sand size varies between 0.8 and 0.9d50 for the other sites with sand beds in the range of d50= 0.25 to 0.35 mm.
Figure 3.2.10 Effect of waves on suspended transport for $d_{50}=0.14-0.18$ mm;
(Maplin Sands, UK and Galgeplaat, Eastern Scheldt, The Netherlands)
Top: Sand transport by waves and currents in water depth of 5 m,
Bottom: Sand transport by waves and currents in water depth of 3 m.
Figure 3.2.11  Effect of waves on suspended sand transport for $d_{50}=0.18-0.4$ mm (Egmond, The Netherlands and Boscombe Bay, UK),

Top: Sand transport by waves and currents in water depth of 5 m, 
Middle: Sand transport by waves and currents in water depth of 3 m, 
Bottom: Sand transport by waves and currents in water depth of 1-2 m.
More detailed information of the effect of relative wave height ($H_s/h=0.2$, 0.3, 0.4, 0.5 and 0.65) on the longshore suspended transport is given in Figure 3.2.12 for the Egmond dataset of Kroon (1994) and Wolf (1997). The transport rates show a considerable increase for breaking wave conditions with $H_s/h$ larger than about 0.4. The transport rate increases with depth-mean velocity to the power 2.5.

![Graph showing depth-integrated suspended transport rates](image)

**Figure 3.2.12** Depth-integrated suspended transport rates (data and trend lines) as a function of depth-averaged longshore velocity; Egmond surf zone 1989-1992, The Netherlands

**Coastal conditions: Egmond 1998, The Netherlands**
To be described later

**Coastal conditions: Duck site, USA**
To be described later

### 3.2.4 Wave-related suspended sand transport ($q_{s,w}$)

**Large scale wave tank of Delft Hydraulics**
Values of the depth-integrated wave-related suspended transport are hardly available. Therefore, experiments in the large-scale Delta flume (length= 200 m, depth= 7 m, width= 5 m) of Delft Hydraulics have been carried out to study the wave-related suspended transport under controlled conditions (Chung and Grasmeijer, 1999). The experimental conditions (ripple regime) are given in Table 3.2.1.
A horizontal sand bed layer was placed in the wave tank 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 acoustic 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 bed-form 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 3.2.1).

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 wave-related and low-frequency wave-related and net transport (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 the offshore direction similar to the current-related transport component. 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 obtain the depth-integrated transport rates, the transport terms have been integrated between the lowest and highest measurement points. The results (mean values and errors) for all available tests are shown in Figure 3.2.13. The measured values represent the depth-integrated values between the lowest measurement point $z = 0.075$ m and the highest point $z = 1.075$ m. The wave-related suspended transport in the unmeasured zone between $z = 0.01$ m and $z = 0.075$ m has been estimated by extrapolation (Chung and Grasmeijer, 1999). The total wave-related suspended transport (in both the measured and unmeasured zones) is about 1.5 times the measured values shown in Figure 3.2.13. 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. Based on these results, it is
evident that the high-frequency suspended sand transport rate below 0.075 m down to 0.01 m is an essential part of the total depth-integrated transport rate and can not be neglected. For field measurements this has the consequence that measured values down to 0.01 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.

![Graph showing wave-related suspended transport rate vs. significant wave height](image)

**Figure 3.2.13** Depth-integrated wave-related suspended transport (measured and computed) as a function of significant wave height and sand size

In Figure 3.2.13 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). From Figure 3.2.13 it can be observed that the wave-related suspended transport increases with increasing 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 are much more pronounced than those on the 0.16 mm sand bed (see Table 3.2.1) resulting in larger vortex motions and stronger associated suspension processes. The standard error of the wave-related transport is relatively large (about 50%) 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.

**Egmond site 1989-1992, The Netherlands**

Some information of the wave-related 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 at one or two points above the bed 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 local (in a specific point above bed) \(q_{ww}\)-component generally is onshore-directed near the bed and significant compared to the (offshore-directed) \(q_{wc}\)-component. Typical values are:
In the inner surf/swash zone: \[ |q_{hs}| = 0.2 \text{ to } 0.3 \ |q_{hs}|
\]
In the shoreface and surf zone: \[ |q_{hs}| = 0.5 \text{ to } 1 \ |q_{hs}|
\]
The vertical resolution of the data is not sufficient to obtain depth-integrated values of the wave-related suspended transport.

_Egmond site 1998, The Netherlands_
To be described later.

### 3.3 Suspended sand transport model

#### 3.3.1 Wave-related suspended transport \((q_{hs})\)

Modelling of the wave-related suspended transport \((q_{hs})\) for a sand bed covered with ripple type bed forms, basically requires the simultaneous (numerical) solution of both the time-dependent momentum equation for the oscillatory fluid flow and the time-dependent advection-diffusion equation for suspended sediment particles.

For the two-dimensional vertical plane this latter equation reads as:

\[
\frac{\partial C}{\partial t} + \frac{\partial [(UC - (v_{wz} \frac{\partial C}{\partial x})] / \partial x + \partial [(W - w_s)C - (v_{wz} \frac{\partial C}{\partial z}) / \partial z = 0}
\]

(3.3.1a)

with: \(C\) = instantaneous sand concentration (volume); \(U\), \(W\) = horizontal and vertical instantaneous fluid velocities; \(w_s\) = fall velocity of suspended sand; \(v_{wz}\), \(v_{wz}\) = sediment mixing coefficient in horizontal \(x\) and vertical \(z\) directions; \(t\) = time; \(x\) = horizontal coordinate and \(z\) = vertical coordinate.

The oscillatory flow along a rippled bed is rather complicated due to the generation, advection and diffusion of the near-bed vortices including the sediment particles carried by the vortices. Numerical simulation of the detailed vortex motions requires the application of sophisticated turbulence-models on a fine grid structure. Furthermore, the shape and dimensions of the ripples should be known a priori (boundary conditions). Using this approach, the instantaneous fluid flow and suspended transport due to combined steady and oscillatory flow over a rippled bed can be solved in an integrated way, which is a great advantage of this method. A major 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). This detailed approach is at an early stage of research and beyond the scope of the present study.

For a plane bed without bed forms the advection-diffusion equation for the suspended concentrations can be simplified to:

\[
\frac{\partial C}{\partial t} + \frac{\partial [(-w_s)C - (v_{wz} \frac{\partial C}{\partial z}) / \partial z = 0}
\]

(3.3.1b)

This unsteady model approach is applied by many researchers to simulate the suspended concentrations inside and outside the wave boundary layer over a plane bed (see overview of Dohmen-Janssen, 1999). This approach may also be used to simulate the time-averaged sand concentrations over a rippled, provided that the overall effect of the bed forms on the
sediment mixing coefficients is taken into account (Chung and Grasmeijer, 1999). Their results also show that the wave-related suspended transport can not be simulated accurately by Equation (3.3.1b). Herein, an engineering approach is proposed to estimate the wave-related suspended transport. This method (implemented in the TRANSPOR2000 model) has been introduced by Houwman and Ruessink (1996). Experimental data are required to determine the empirical coefficient involved.

The wave-related suspended transport component is modelled as:

\[ q_{k,w} = \gamma \left[ \left( U_{on} \right)^4 - \left( U_{off} \right)^4 \right] \left[ \left( U_{on} \right)^3 + \left( U_{off} \right)^3 \right] \int \text{c} \ dz \]  

(3.3.2)

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

Equation (3.3.2) is based on an instantaneous response of the suspended sand concentrations (C) and transport \( q_{k,w} \) to the near-bed orbital velocity (C proportional to \( U^3 \) and \( q_k \) to \( U^4 \)). 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 suspended 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).

Simulation of the wave-related suspended transport according to Eq. (3.3.2) requires computation of the time-averaged sand concentration profile (see Section 3.3.2) and integration of the time-averaged sand concentration profile 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 and Grasmeijer, 1999) show 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 and Grasmeijer (1999) have determined the \( \gamma \)-function by fitting of Eq (3.3.2) to the measured wave-related transport rates (Table 3.3.1 and Figure 3.2.13). 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. It is noted that the \( \gamma \)-value of 0.2 is based on data with rather pronounced ripples observed in a large scale 2D wave tank. The \( \gamma \)-value
may be considerably smaller (say between 0.1 and 0.2) for field conditions with less pronounced 3D-ripples.

Equation (3.3.2) implemented in the TRANSPOR2000 model has also been used to compute the wave-related suspended transport for the five Delta flume cases 1A to 1E. The near-bed orbital velocities during the onshore and offshore phase of the wave cycle have been represented by sine-functions based on the measured near-bed peak orbital velocities. Input values are shown in Table 3.3.1. The computed wave-related suspended transport rates are shown in Figure 3.2.13. The computed values are roughly 1.5 to 2 times the measured values for the 0.16 mm sand, which is a rather good result realizing that the measured transport rates do not include the values in the unmeasured zone below z= 0.01 m (see also discussion in Section 3.2.4). The computed values for the 0.33 mm sand are somewhat too small (about 50%).

<table>
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<th>$U_{k,am}$ (m/s)</th>
<th>$U_{k,off}$ (m/s)</th>
<th>$d_{50}$ (mm)</th>
<th>$d_{90}$ (mm)</th>
<th>$d_s$ (mm)</th>
<th>$k_{s,w}$ (m)</th>
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</tr>
</tbody>
</table>

Table 3.3.1 Measured and computed wave-related suspended transport rates for Delta flume experiments ($h= 4.5$ m, $T_p= 5$ s, $d_s$ and $k_{s,w}$ are estimated)

Proper predictive modelling of the oscillating suspended transport component ($Q_{s,w}$) requires an accurate description of the near-bed orbital fluid velocity, especially in conditions with shoaling and breaking waves (non-linear wave motion). Previously, 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 $Q_{bcw}$, $Q_c$ and $Q_{s,w}$ ($Q_{rel}=Q_{bcw}+Q_c+Q_{s,w}$) in cross-shore direction; some sensitivity computations have been made for uniform bed material of 0.2 mm and a water depth of 5 m (using TRANSPOR 2000). The transport components are defined as: $Q_{bcw}$ =mean and oscillating bed-load transport , $Q_{s,w}$ = oscillating suspended transport (only high-frequency) and $Q_c$ = mean current-related suspended transport. The $\gamma$-factor of the $Q_{s,w}$ transport component has been taken as 0.2 to obtain values of the right order of magnitude compared to measured data.

Figure 3.3.1 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 (tide- or wind-driven) depth-averaged current velocities are taken as $v= 0.5$, 0.6, 0.7, 0.8 and 1 m/s for $H_s/h=0.2$, 0.3, 0.4, 0.5 and 0.6. The cross-shore undertow is computed according to the expression given in Appendix B resulting in values between $u_r=-0.04$ to -0.43 m/s. The bed roughness is $k_{s,c}=k_{s,w}=0.02$ m for all conditions. The water temperature is 15 °C, the salinity is 30 promille and $\gamma= 0.2$. 

---

Images and tables are not transcribed.
The following results can be observed (see Fig. 3.3.1):

- the bed-load transport is onshore-directed for \( H/h \leq 0.4 \) and offshore-directed for \( H/h = 0.5 \) and 0.6;
- the wave-related suspended transport is onshore-directed; the oscillatory suspended transport is much larger than (factor 5 to 10) than the bed-load transport for 0.2 mm sediment;
- the onshore-directed wave-related suspended transport is slightly larger than the offshore-directed current-related suspended transport for \( H/h = 0.2 \) and smaller than the offshore-directed current-related suspended transport for \( H/h \geq 0.3 \);
- the net total transport is onshore-directed for \( H/h = 0.2 \); net total transport is offshore-directed for \( H/h \geq 0.3 \).

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 net cross-shore transport.

![Graph of cross-shore transport rate vs. relative wave height](image)

**Figure 3.3.1** Computed 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 (\( q_{c,i} \))

*Description of model*

Modelling of the current-related suspended transport requires modelling of both the time-averaged velocity profile and the time-averaged sand concentration profile. The suspended transport in the main current direction is computed as: \( q_{c,i} = \int v_{R,x} c_z\,dz \) with \( c_z \) = time-averaged sand concentration at height \( z \) and \( v_{R,x} \) = current velocity at height \( z \) in main current direction.

The current velocity profile in the main current direction (see Fig. A1 of Appendix B) 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). In both layers the velocity profile is assumed to be logarithmic.
The suspended transport due to the undertow current is computed as: 

\[ q_{c,2} = \int u_{z} \cdot c_{z} \, dz \]

with \( c_{z} \) = time-averaged sand concentration at height \( z \) and \( u_{z} \) = undertow velocity at height \( z \) above the bed. The current velocity profile of the undertow current (due to breaking waves), which is opposite to the wave direction (see Fig. A1 of Appendix B), is represented as a three-layer system. In the two layers near the bed the velocity profile is assumed to be logarithmic. In the upper layer \((z/h>0.5)\) the velocity profile is assumed to decrease to zero at the water surface according to a third power distribution. The velocity profile of the undertow current is described in Appendix D. It is noted that the TRANSPOR 1993 model is based on a logarithmic profile for the undertow current.

The new three-layer velocity profile for the undertow results in somewhat larger suspended transport rates (about 30\%) due to the increase of the near-bed velocities.

Both current-related suspended transport components \( (q_{c,1} \) and \( q_{c,2} \)) are summed by vectorial addition.

The total suspended transport is obtained by vectorial summation of \( q_{c} \) and \( q_{cw} \).

The time-averaged (over the wave period) advection-diffusion equation is applied to compute the equilibrium time-averaged sand concentration profile due to the combined effect of steady and oscillatory flow. This equation reads as:

\[ w_{s,m} + \epsilon_{s,cw} \frac{dc}{dz} = 0 \quad (3.3.3) \]

in which: \( w_{s,m} \) = fall velocity of suspended sediment in a fluid-sediment mixture (m/s), 
\( \epsilon_{s,cw} \) = sediment mixing coefficient for combined steady and oscillatory flow (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 steady and oscillatory flow the sediment mixing coefficient is modeled as:

\[ \epsilon_{s,cw} = (\epsilon_{s,m})^2 + (\epsilon_{s,c})^2 \quad (3.3.4) \]

in which: \( \epsilon_{s,m} \) = wave-related mixing coefficient (m²/s), \( \epsilon_{s,c} \) = current-related mixing coefficient due to main current and undertow current (m²/s); the effect of the sediment particles on the mixing of fluid momentum is taken into account by the \( \beta \)-factor, which depends on the particle fall velocity and the bed-shear velocity \( \epsilon_{s,c} = \beta \epsilon_{f,c} \).

Some parameters of the wave-related sediment coefficient (as described in Appendix B) have been modified based on analysis of the experimental data (Table 3.2.2). The vertical distribution has not been changed.

\[ \delta_s = \max(5\gamma_{br} \delta_w, 10\gamma_{br} k_{s,w}) \quad \text{with limits } 0.1 \leq \delta_s \leq 0.5 \, \text{m} \quad (3.3.5) \]

with: \( \delta_s \) = thickness of effective near-bed sediment mixing layer, \( \delta_w \) = thickness of wave boundary layer, \( k_{s,w} \) = wave-related bed roughness and \( \gamma_{br} = 1 + (H_s/h-0.4)^{0.5} \) empirical coefficient related to wave breaking \( (\gamma_{br} = 1 \text{ for } H_s/h \leq 0.4) \)
The wave-related sediment mixing coefficient in the upper half of the water column has been modified into (multiplied by the breaking coefficient $\gamma_{br}$):

$$
e_{a,w,max} = 0.035 \ \gamma_{br} \ H/T_p \ \text{with} \ e_{a,w,max} \leq 0.05 \ \text{m}^2/\text{s}$$

(3.3.6a)

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

$$
e_{a,w,bed} = 0.004 \ \delta_a \ D_s \ U_0$$

(3.3.6b)

in which: $D_s$ = particle parameter, $U_0$ = near-bed peak orbital velocity. The near-bed mixing parameter $e_{a,w,bed}$ was found to be dependent on the particle parameter $D_s$, based on analysis of sand concentration profiles of experiments with bed material in the range of 0.1 to 0.5 mm. The near-bed mixing appears to increase with increasing particle size, which may be an indication of the dominant influence of centrifugal forces acting on the particles due to strong turbulence-induced vortex motions resulting in an increase of the effective mixing of sediment particles. As no information is available for bed materials larger than 0.5 mm, the application of Eq. (3.3.6b) for these conditions is highly uncertain. More research is necessary for accurate prediction of the wave-induced suspended transport for relatively coarse materials (>0.5 mm; coarse sand and gravel beds).

Numerical solution of the advection-diffusion equation (Eq. 3.3.3) requires the 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^{3.5}}{D^{0.3}} \ \text{with} \ c_a \leq 0.05 \ \text{(approx. 130 kg/m}^3\))$$

(3.3.7)

in which: $D_s$ = 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 0.02 m.

The $T$-parameter is:

$$
T = (\tau_{b,cr} - \tau_{b,cr})/ \tau_{b,cr}
$$

(3.3.8)

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):

$$
\tau'_{b,cr} = \tau'_{b,c} + \tau'_{b,w}
$$

(3.3.9)

in which: $\tau'_{b,c}$ = $\alpha_c \ \tau_{b,c}$ = effective current-related bed-shear stress (N/m²), and $\tau'_{b,w}$ = $\mu_w \ \tau_{b,w}$ = effective wave-related bed-shear stress (N/m²), $\mu_w$ = efficiency factor and $\alpha_c$ = wave-current interaction factor; grain-related friction factor depends on $d_{50}$. 
The wave-related efficiency factor \( \mu_{w,a} \) is an important parameter, because it strongly affects the reference concentration near the bed. This parameter will probably depend on the bed form and bed roughness characteristics, but the functional relationship involved is not yet known. Therefore, the \( \mu_{w,a} \) factor has been used as a calibration parameter to get a better estimate of the near-bed concentration. As the bed forms are related to the relative wave height (ripples for small values of \( H_s/h \) and plane bed for large values of \( H_s/h \)), the \( \mu_{w,a} \) factor is supposed to be related to the relative wave height. Based on analysis of experimental data (Table 3.2.2), the \( \mu_{w,a} \) factor 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 (\( k_{s,c} \) and \( k_{s,w} \)). 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 (\( k_{s,c} \) and \( k_{s,w} \)) 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 must be taken into account. This is done, as follows (see Appendix B):

- multiplying the critical bed-shear stress with the Schoklitsch-factor
  \[ k_1 = \sin (\phi + \beta) / \sin(\phi) \]
  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 \( k_1 > 1 \) 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
  \[ k_2 = 1/(1 + \tan\beta / \tan\phi) \]
  \( \tan\beta \) is positive for uphill transport yielding \( k_2 < 1 \) 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.

Schematization of wave spectrum

To analyze the effect of the representation of the wave spectrum on the current-related (longshore) sand transport rates, sensitivity computations (using TRANSPOR 1993 model, see Van Rijn, 1997) have been made for a specific case with a constant water depth \( h = 3 \) m using a single representative wave height approach and a multi wave height approach based on a Rayleigh distribution. The bed material is \( d_{50} = 0.0002 \) m and \( d_{60} = 0.0003 \) m and the bed roughness is taken to be \( k_{s,c} = k_{s,w} = 0.01 \) m. The irregular waves (normal to the coast) are assumed to have a Rayleigh-distribution with \( H_{rms} = 0.71 \) m, \( H_\beta = 1 \) m and \( T_p = 7 \) s. The longshore current velocities have been varied in the range between 0.2 and 2 m/s. The water temperature is taken as 15 °C and the salinity as 30 promille.
The current-related sand transport rates have been computed in three ways:
1. based on a single representative wave height equal to $H_{rms}$,
2. based on a single representative wave height equal to $H_{1/3}$,
3. based on multi-wave height approach, schematizing the Rayleigh-distribution to 8 wave height classes between 0.2 and 1.6 m and wave periods between 6.2 and 7.6 s.

The results are presented in Figure 3.3.2, showing the ratio of the transport rates based on the single wave approach and the multi-wave approach. Application of the significant wave height $H_{1/3}$ as the representative wave height yields transport rates which are considerably larger than those of the multi-wave approach (factor 2 for low velocities). Application of the $H_{rms}$ leads to transport rates, which are much smaller (factor 2 for low velocities). The discrepancies are relatively large for low current velocities when the waves are the dominant factor in the transport process. For high current velocities the proper representation of the wave field becomes less important with respect to the transport process.

It is noted that the single wave height $H_{1/3}$ has been used before to calibrate the applied sand transport calculation method (Van Rijn, 1993) and the $H_{1/3}$-value is therefore supposed to give the best results. Application of the multi wave height approach will lead to an error (underestimation) of the order of a factor 2 for low velocities. Application of $H_{rms}$ as the representative wave height will lead to an error (underestimation) of the order of a factor 4 for low velocities compared to the transport rates based on $H_{1/3}$.

![Figure 3.3.2](image_url)

**Figure 3.3.2**  *Influence of single and multi-wave approach on current-related sand transport*

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

As discussed in Section 3.3.2, the modified parameters $\delta_n$, $\gamma_{br}$, $\mu_{wa}$ of Eqs. (3.3.5, 3.3.6 and 3.3.9) have been obtained by fitting of computed and measured time-averaged sand
concentration profiles. The sand concentration profiles measured in the large-scale Delta flume (10 cases; Table 3.2.2 and Figures 3.2.1 and 3.2.2) have been used for calibration of the parameters. The wave-related efficiency factor $\mu_{w,a}$ is an important parameter, because it strongly affects the reference concentration near the bed. The ‘old’ $\mu_{w,c}$-coefficient (TRANSPOR 1993) yielded $c_r$-values which were too small for rippled bed conditions ($H/h<0.5$). As the bed forms are related to the the relative wave height (ripples for small values of $H/h$ and plane bed for large values of $H/h$), the $\mu_{w,a}$ factor has been related to the relative wave height $H/h$. The effect of breaking wave conditions on the concentration profile was taken into account by modifying the thickness of the near-bed mixing layer $\delta_k$ and by multiplying the sediment mixing coefficient with a breaking parameter $\gamma_{br}$.

### 3.3.4 Verification of current-related suspended transport model for conditions with currents only

**Sand in the range of 0.14-0.18 mm**

Data of measured sand transport rates in field conditions with currents only have been presented in Section 3.2.3. The trendline representing the measured transport rates and the variation ranges of a factor 2 are shown in Figure 3.3.3. The measured data refer to suspended sand transport.

The TRANSPOR 2000 model has been applied to compute the suspended and total load transport for a water depth of 8 m and a sand bed with $d_{50}=0.16$ mm and $d_{50}=0.3$ mm. The bed roughness has been varied as $k_s=0.03$ and $0.05$ m. The suspended sand size has been taken as: $d_s=0.6-1d_{50}$. It is assumed that the very fine fractions of the bed material are winnowed from the bed at very low velocities, yielding a relatively small $d_s$-value and that the coarser fractions are picked up from the bed at larger velocities, yielding a relatively large $d_s$-value. Other data are: temperature= 15 degrees, salinity= 0 promille. The computed suspended load transport rates are shown in Figure 3.3.3. The computed total load transport shows reasonably good agreement (within factor 2) with the trendline of the measured values for velocities in the range of 0.7 to 1.8 m/s. The computed values appear to be significantly too small for velocities in the range of 0.4 to 0.6 m/s. The bed roughness in the range of $k_s=0.03$ to $0.05$ m has not much effect on the computed values. The computed suspended transport is relatively small at low velocities (0.4-0.5 m/s) compared the measured transport rates. This may be related to the effect of ripple type bed forms, which may be rather pronounced in conditions with fine sand bed of 0.14-0.18 mm. These ripple effects may not be properly taken into account by the model.

Another cause for discrepancies between computed and measured values may be the presence of selective transport processes in the sense that the finer fractions of the sand bed are winnowed and more easily transported in suspension. Furthermore, most of the data in the low velocity range have been measured in non-equilibrium tidal flow conditions (overloading) with settling sediments (Maplin Sands and Foulness, UK; see Figure 3.2.6).
Figure 3.3.3  Depth-integrated suspended load transport as function of current velocity for sand in the range 0.14-0.18 mm; computed values based on depth of $h=8$ m and $d_{50}=0.16$ mm

Sand in range of 0.18-0.4 mm

Data of measured sand transport rates in field conditions with currents only have been presented in Section 3.2.3. The trendline representing the measured transport rates and the variation ranges of a factor 2 are shown in Figure 3.3.4. The measured data generally refer to suspended sand transport, but bed load transport is included for the transport rates in the velocity range 0.4 to 0.6 m/s.

The TRANSPOR 2000 model has been applied to compute the suspended and total load transport for a water depth of 5 m and a sand bed with $d_{50}=0.25$ mm and $d_{60}=0.5$ mm. The bed roughness has been varied as $k_r=0.03$, 0.05 and 0.1 m. The suspended sand size has been taken as: $d_s=d_{50,\text{bed}}$. Other data are: temperature= 15 degrees, salinity= 0 promille. The computed suspended and total load transport rates are shown in Figure 3.3.4.

Effect of bed roughness

The computed total load transport shows reasonably good agreement (within factor 2, see Figure 3.3.4) with the trendline of the measured values. The bed roughness in the range of $k_r=0.03$ to 0.1 m has not much effect on the computed values. The computed suspended transport based on $d_s=d_{50}$ is significantly smaller than the computed total load transport at low velocities close to initiation of motion (maximum factor 10 at velocity of 0.4 m/s). For these conditions the bed load transport is dominant in the velocity range of 0.4 to 0.7 m/s; the bed load transport depends on grain roughness.

Effect of water depth

The effect of water depth on the measured transport rates can hardly be detected within the scatter range of the data (see Figure 3.2.7). The effect of water depth on the computed transport rates has been studied by sensitivity computations based on depth values in the range of 1 and 10 m. The results ($k_r=0.05$ m = constant) are presented in Figure 3.3.5. The ratio of the transport rate at $h=1$ m and at $h=10$ m as a function of velocity is shown in Figure 3.3.6.

The effect of the water depth on the bed-load transport is rather significant, particularly at low velocities of 0.4 to 0.6 m/s. The bed-load transport at a depth of 1 m is a factor 10 to 3
larger than that at a depth of 10 m for velocities in the range of 0.4 to 0.6 m/s. This effect reduces to a factor 2 for a velocity of 0.8 m/s. The effect is caused by the effect of depth on the grain-related bed-shear stress, which can be expressed as $\tau'_b = \rho g (u/C)^2$ with $u$= depth-mean velocity, $\rho$= fluid density, $g$= acceleration of gravity, and $C' = 18 \log(12 \ h/3d_0)=$ grain Chezy-roughness depending on depth $h$. Taking $h$ in the range from 1 to 10 m and $d_0=0.0005$ m, the grain Chezy-roughness varies between 70 and 90 m$^{0.5}$/s. Hence, the grain-related bed-shear stress roughly decreases by a factor 2 for the depth increasing from 1 to 10 m ($C'$ increasing from 70 to 90 m$^{0.5}$/s).

The effect of the water depth on the suspended load transport is even more significant at low velocities of 0.4 to 0.6 m/s. The suspended transport at a depth of 1 m is a factor 30 to 5 larger than that at a depth of 10 m for velocities in the range of 0.4 to 0.6 m/s. This effect reduces to a factor 2 for a velocity of 1.1 m/s and to a factor 1.1 for a velocity of 2 m/s. The relatively strong effect of the water depth on the suspended transport at low velocities is not that serious, because bed-load transport is dominant for velocities up to 0.6 m/s.

The effect of water depth on the suspended transport rates at low velocities seems to be more significant for the computed values than for the measured transport rates. This may be the result of some compensating processes, which are not represented sufficiently well by the model. For example, the bed-form dimensions and hence the bed-form roughness may increase for increasing water depths yielding larger transport rates. The model does not show this behaviour, as the model results are not very much affected by bed roughness (see Figure 3.3.4).

Overall, the effect of water depth on the transport rate can be neglected (variation within a factor 2) for current velocities larger than about 0.9 m/s.

**Effect of suspended size $d_s$**

The effect of suspended sand size has been studied by sensitivity computations based on $d_c=0.6-1d_{50}$. In this case it is assumed that the very fine fractions of the bed material are winnowed from the bed at very low velocities and that the coarser fractions are picked up from the bed at larger velocities (see also Tables 3.2.3 and 3.2.5). The results are shown in Figure 3.3.7. The computed suspended transport at relatively low velocities increases considerably by taking the winnowing effects into account ($d_c=0.6-1d_{50}$) in stead of $d_c=d_{50,bed}$.

The TRANSPOR 2000 multi-fraction model (MF model, see Chapter 4) has also been used to compute the suspended transport. The hydrodynamic input data are the same; the bed material has been represented by 6 fractions, as follows: 0.075 mm (10%), 0.15 mm (20%), 0.25 mm (20%), 0.35 (20%), 0.45 mm (20%), and 0.55 mm (10%), yielding a $d_{50}$ of 0.25 mm. The computed total load transport rates show remarkably good agreement with the trend line of the measured transport rates (Figure 3.3.7); the computed values are somewhat too large for velocities larger than 0.6 m/s.
Figure 3.3.4  Depth-integrated suspended load and total load transport as function of current velocity for sand in the range 0.18-0.4 mm; computed values based on depth of \( h = 5 \) m, \( d_{50} = 0.25 \) mm; effect of bed roughness \( k_s = 0.03 \) to 0.1 m

Figure 3.3.5  Depth-integrated suspended load and total load transport as function of current velocity for sand in the range 0.18-0.4 mm; computed values based on \( k_s = 0.05 \) m, \( d_{50} = 0.25 \) mm; effect of water depth \( h = 1 \) to 10 m
Figure 3.3.6  Ratio of computed transport rates at $h = 1$ m and at $h = 10$ m, $k_s = 0.05$ m, $d_{50} = 0.25$ mm

Figure 3.3.7  Depth-integrated suspended load and total load transport as function of current velocity for sand in the range 0.18-0.4 mm; computed values based on $h = 5$ m, $k_s = 0.03$ m, $d_{50} = 0.25$ mm; effect of $d_s = d_{50}$ and $d_s = 0.6-1.0 d_{50}$
Sand in range of 0.4-0.6 mm

Figure 3.3.8 shows measured and computed total load transport rates for field conditions with currents only (see Table 3.2.1). The measured data are represented by the trendline including the variation ranges of a factor 2, see Figure 3.3.8. The measured data generally refer to suspended sand transport, but bed-load transport is included for the transport rates in the velocity range 0.4 to 0.8 m/s.

The TRANSPOR 2000 model has been applied to compute the suspended and total load transport for a water depth of 8 m and a sand bed with $d_{50}=0.5$ mm and $d_{60}=1$ mm. The bed roughness has been taken as $k_s=0.03$ m. The suspended sand size has been taken as: $d_i=0.6-0.8d_{50}$. Other data are: temperature= 15 degrees, salinity= 0 promille. The computed total load transport rates are shown in Figure 3.3.8. The computed total load transport shows reasonably good agreement (roughly within factor 2) with the trendline of the measured values. The bed-load transport is dominant in the velocity range of 0.4 to 1.0 m/s; the bed-load transport depends on grain roughness.

The TRANSPOR 2000 multi-fraction model (MF model, see Chapter 4) has also been used to compute the suspended transport. The hydrodynamic input data are the same; the bed material has been represented by 6 fractions, as follows: 0.15 mm (10%), 0.25 mm (20%), 0.5 mm (20%), 0.7 mm (20%), 0.9 mm (20%) and 1.1 mm (10%), yielding a $d_{50}$ of 0.5 mm. The computed total load transport rates show remarkably good agreement with the trend line of the measured transport rates (Figure 3.3.8); the computed values are somewhat too large at low velocities in the range of 0.5 to 0.7 m/s.

![Graph showing depth-integrated total load transport as function of current velocity for sand in the range of 0.4-0.6 mm; computed values based on depth of $h=8$ m, $d_{50}=0.5$ mm, $k_s=0.03$ m; effect of SF and MF methods](image-url)
3.3.5 Verification of current-related suspended transport model for combined wave and current conditions

*Sand in range of 0.18-0.4 mm: data from Boscombe Bay, UK, 1977-1978 and Egmond, The Netherlands, 1989-1992, 1998*

The data of measured suspended sand transport rates at the sites of Boscombe Bay and Egmond have been presented in Section 3.2.3. Figure 3.3.9 shows the measured suspended sand transport in a water depth of about 5 m with significant wave height of about $H_\text{c} = 1$ m. For reference the trendline of measured transport rates in a current without waves is shown (including variation ranges of a factor 2).

The TRANSPOR 2000 model has been applied to compute the suspended transport for a water depth of 5 m and a sand bed with $d_{50}=0.25$ mm and $d_{90}=0.5$ mm. The bed roughness has been varied as $k_c = 0.01, 0.02$ and $0.03$ m. The suspended sand size has been taken as: $d_s = 0.8d_{50}$ according to the measured values (see Tables 3.2.3 and 3.2.5). Other data are: $T_p = 7$ s, $\varphi =$ angle between wave and current direction= 90 to 165 degrees, temperature= 10 degrees, salinity= 30 promille. The computed suspended sand transport rates for $H_c = 1$ m are shown in Figure 3.3.9. The computed suspended transport rates show reasonable agreement (within factor 2 to 3) with the Egmond data for $k_c = 0.01-0.02$ m and with the Boscombe data for $k_c = 0.03-0.05$ m. The computed values are strongly dependent on the wave-related bed roughness in the low velocity range of 0.2 to 0.6 m/s, for which the effect of wave-induced mixing of sediment dominates over turbulence-induced mixing. The effect of bed roughness is less important in conditions with a dominating current (see Figure 3.3.9). The wave-related bed roughness represents the effect of the bed ripples on the transport process in the coastal zone. Given the strong effect of bed roughness and hence ripple dimensions on the transport process, it is essential to have information of the ripple characteristics in field conditions. Thus, measurements of sand transport in field conditions should always include ripple size measurements.

The suspend transport rates computed for conditions without waves show good agreement with the trendline of measured values.
Figure 3.3.9  Depth-integrated suspended sand transport as function of current velocity for sand in the range of 0.18-0.4 mm; measured data from Boscombe Bay 1988 and Egmond 1998; computed values based on depth of h = 5 m and d₅₀ = 0.25 mm; effect of bed roughness kₛ.

Figure 3.3.10 shows the measured suspended sand transport at the Egmond site in a water depth of about 3 m with significant wave height of about Hₛ = 1 m. For reference the trendline of measured transport rates in a current without waves is shown. The TRANSPOR 2000 model has been applied to compute the suspended transport for a water depth of 3 m and a sand bed with d₅₀ = 0.25 mm and d₅₀ = 0.5 mm. The bed roughness has been varied as kₛ = 0.01, 0.02 and 0.03 m. The suspended sand size has been taken as: dₛ = 0.8d₅₀ according to the measured values (see Table 3.2.5). Other data are: Tₛ = 7 s, φ = 90 degrees, temperature = 10 degrees, salinity = 30 promille. The computed suspended sand transport rates for Hₛ = 1 m are shown in Figure 3.3.10. The computed suspended transport rates show reasonable agreement (within factor 2) with the Egmond data for kₛ = 0.02 m.
**Figure 3.3.10** Depth-integrated suspended sand transport as function of current velocity for sand in the range of 0.18-0.4 mm; measured data from Egmond 1998; computed values based on depth of $h=3$ m and $d_{50}=0.25$ mm; effect of bed roughness $k_s$.

**Figure 3.3.11** Depth-integrated suspended sand transport as function of current velocity for sand in the range of 0.18-0.4 mm; measured data from Egmond 1989-1992, 1998; computed values based on depth of $h=1.5$ m and $d_{50}=0.25$ mm.

Figure 3.3.11 shows the measured suspended sand transport at the Egmond site in a water depth of about 1 to 2 m with relative wave heights of about $H_s/h=0.4-0.5$. For reference the trendline of measured transport rates in a current without waves is shown.

The TRANSPOR 2000 model has been applied to compute the suspended transport for a water depth of 1.5 m and a sand bed with $d_{50}=0.25$ mm and $d_{50}=0.5$ mm. The bed roughness has been taken as $k_s=0.02$ m. The suspended sand size has been taken as: $d_s=0.8d_{50}$ according to the measured values (see Table 3.2.5). Other data are: $T_p=7$ s, $\varphi=90$ degrees, temperature= 10 degrees, salinity= 30 promille. The computed suspended sand
transport rates are also shown in Figure 3.3.11. The computed suspended transport rates show reasonable agreement (within factor 2 to 3) with the Egmond data for \( k_s = 0.02 \) m.

**Sand in range of 0.18-0.4 mm: data from Egmond swash zone, The Netherlands, 1989-1992**

The field data sets of Egmond surf zone (10 cases; Table 3.2.4 and Figures 3.2.3 and 3.2.4) have been used to verify the overall suspended transport model. The bed roughness values have been estimated to be \( k_{sc} = k_{sw} = 0.03 \) m. The suspended sediment size has been taken as \( d_s = 0.8d_{50} \). The computed sand concentration profiles are much too large for two Cases (3A and 3B), but show reasonable agreement with measured concentration profiles for the other eight Cases.

The computed transport rates are given in Table 3.3.2. The computed bed-load transport includes both the wave-related and the current-related transport components; the computed suspended transport only includes the current-related component (thus \( \gamma = 0 \), see Eq. 3.3.2).

In longshore direction the computed bed-load transport is relatively small and is not more than about 10% to 25% of the computed suspended transport \( (q_b/q_s = 0.1 \) to 0.2) for nearly all cases. An exception is the computed longshore bed-load transport of case 3D, which is relatively large compared to the longshore suspended transport \( (q_b/q_s = 0.45) \), because of the low wave-energy conditions. In cross-shore direction the computed bed-load transport generally is onshore-directed, if the offshore-directed undertow current is relatively weak \( (<0.1 \text{m/s}) \). The computed cross-shore bed-load transport is relatively small (negligible), if the undertow current is relatively strong \( (>0.3 \text{m/s}) \); Cases 3E, 4B, 4G).

Figure 3.3.12 shows a comparison of computed and measured suspended transport rates in longshore direction. In most cases (80%) the computed suspended transport is within a factor 2 of the measured values, which is a rather good result. Similar results have been obtained for the computed suspended transport in cross-shore direction (not shown).

The results of the TRANSPOR 1993 model are also shown in Figure 3.3.12. The computed results of the improved model (TRANSPOR 2000) are much better than those of the ‘old’ model (TRANSPOR 1993). Analysis of the results of the ‘old’ model shows that all computed values are much smaller than the measured values; only in three cases the computed suspended transport rates are within a factor 2 of the measured transport rates (Cases 3A, 3B and 3E). The computed transport rates of the ‘old’ model are much too small (factor 5) for strongly breaking waves; the near-bed concentrations are reasonably well predicted but the concentrations higher up in the water column are greatly underpredicted, because the effect of breaking waves on the effective mixing is not included in the ‘old’ model. Including this latter effect, the results of the improved model are much better for the Cases (4A to 4G) with strongly breaking waves.

The results of the BAGNOLD-BAILARD formula \( (q_c = 0.5e_{pp} f_c u [(\rho_r - \rho)w_s g]^1/3 |U|^3 |U| \), in kg/m/s, with \( U = \) instantaneous near-bed velocity, \( f_c = \) grain-friction factor, \( w_s = \) fall velocity of suspended sand) for the longshore suspended transport are also shown in Figure 3.3.12. The grain-friction factor is taken as \( k_{s\text{gran}} = 5d_{50} \) and the empirical coefficient is taken as \( c_c = 0.025 \) in line with Aagaard et al. (1998) and Thornton et al. (1996). The bed is assumed to be flat \( (k_{sc} = k_{sw} = 5d_{50}) \). Generally, the computed longshore suspended transport rates are much too small, particularly for breaking wave conditions (factor 5 to 10). This was also noticed by Van Rijn (1997). The results of the B-B formula are considerably better, if the
grain-related friction factor is replaced by the bedform-related friction factor with $k_c = 0.03$ m. This latter approach yields a substantial increase of the computed longshore suspended transport rates (about factor 2.5), but the computed transport rates are still considerably too small for breaking wave conditions (factor 3). The computed transport rates (according to the B-B formula) in cross-shore direction can not be compared to the measured current-related transport rates, because the computed results do include the wave-related transport components.

Figure 3.3.13 shows the ratio of the measured and computed (based on TRANSPOR 2000) longshore suspended transport as function of depth-averaged longshore current and relative wave height to identify possible systematic errors. As can be observed, there are no serious systematic errors in the computed suspended transport due to wave height and/or current velocity.

<table>
<thead>
<tr>
<th>Case</th>
<th>Computed longshore bed-load transport $q_{L,b,ew}$ (kg/s/m)</th>
<th>Computed cross-shore bed-load transport</th>
<th>Computed current-related longshore suspended transport $q_{L,cr,ew}$ (kg/s/m)</th>
<th>Computed current-related cross-shore suspended transport $q_{CR,A,c}$ (kg/s/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3A</td>
<td>0.0035</td>
<td>0.015</td>
<td>0.016</td>
<td>-0.016</td>
</tr>
<tr>
<td>3B</td>
<td>0.0082</td>
<td>-0.0004</td>
<td>0.043</td>
<td>-0.043</td>
</tr>
<tr>
<td>3C</td>
<td>0.0043</td>
<td>0.00098</td>
<td>0.019</td>
<td>-0.0095</td>
</tr>
<tr>
<td>3D</td>
<td>0.00063</td>
<td>-0.0040</td>
<td>0.0014</td>
<td>-0.0044</td>
</tr>
<tr>
<td>3E</td>
<td>0.047</td>
<td>-0.041</td>
<td>0.5</td>
<td>-0.37</td>
</tr>
<tr>
<td>4A</td>
<td>0.0092</td>
<td>0.046</td>
<td>0.042</td>
<td>-0.042</td>
</tr>
<tr>
<td>4B</td>
<td>0.031</td>
<td>-0.038</td>
<td>0.18</td>
<td>-0.21</td>
</tr>
<tr>
<td>4C</td>
<td>0.018</td>
<td>-0.02</td>
<td>0.078</td>
<td>-0.027</td>
</tr>
<tr>
<td>4E</td>
<td>0.021</td>
<td>0.071</td>
<td>0.11</td>
<td>-0.063</td>
</tr>
<tr>
<td>4G</td>
<td>0.052</td>
<td>-0.016</td>
<td>0.42</td>
<td>-0.33</td>
</tr>
</tbody>
</table>

*Table 3.3.2* Computed bed-load and suspended transport rates (TRANSPOR 2000) for Egmond surf zone 1989-1992
Measured longshore suspended transport (kg/s/m)

Computed longshore suspended transport (kg/s/m)

Van Rijn (TRANSPOR 2000)
Van Rijn (TRANSPOR 1993)
Bagnold-Bailard (es=0.025)

Figure 3.3.12  Measured and computed longshore suspended transport, Egmond surf zone

Ratio of measured and computed longshore suspended transport as function of depth-averaged longshore current and relative wave height; Egmond surf zone

Sand in range of 0.18-0.4 mm: data from Egmond surf zone, The Netherlands, 1998
To be described later.............
Sand in range of 0.14-0.18 mm: data from Maplin Sands, UK, 1973-1975

The data of measured suspended sand transport rates at the site of Maplin Sands (UK) have been presented in Section 3.2.3. Figure 3.3.14 shows the measured suspended sand transport in a water depth of about 5 m with significant wave height of about 0.5 and 1 m. For reference the trendline of measured transport rates in a current without waves is shown. The TRANSPOR 2000 model has been applied to compute the suspended transport for a water depth of 5 m with significant wave height of 1 m and a sand bed with d_{50}=0.14 mm and d_{50}=0.3 mm. The bed roughness has been varied as k_{s}=0.01 m and 0.02 m. The suspended sand size has been taken as: d_{s}=0.65 d_{50}=0.09 mm according to the measured values (see Table 3.2.3). Other data are: T_{p}=4 s, \varphi= angle between current and wave direction= 15 degrees, temperature= 10 degrees, salinity= 30 promille. The computed suspended sand transport rates are also shown in Figure 3.3.14. The computed suspended transport rates are systematically too large (factor 2) for k_{s}=0.01 m. The computed values are much too large for k_{s}=0.02 m.

The suspended transport rate computed for conditions without waves are also shown. Compared to the trendline values of measured suspended transport rates, the computed values are significantly too small for low velocities in the range of 0.4 to 0.6 m/s. Possible causes for discrepancies are discussed in Section 3.2.3.

![Graph showing depth-integrated suspended sand transport as function of current velocity for sand in the range of 0.14-0.18 mm; measured data from Maplin Sands 1973-1975; computed values based on depth of h=5 m and d_{50}=0.14 mm; effect of k_{s}](image)

Figure 3.3.14 Depth-integrated suspended sand transport as function of current velocity for sand in the range of 0.14-0.18 mm; measured data from Maplin Sands 1973-1975; computed values based on depth of h=5 m and d_{50}=0.14 mm; effect of k_{s}.

Figure 3.3.15 shows the measured suspended sand transport rates at the site of Maplin Sands, UK (depth=3 m, d_{50}=0.14 mm, d_{s}=0.09 mm, H_{s}=0.9 m, H_{s}/h=0.33) and at the Site of Galgeplaat, Eastern Scheldt, The Netherlands (depth=3.5 m, d_{50}=0.15 mm, d_{s}=0.12 mm, H_{s}=0.7 m, H_{s}/h=0.2). For reference the trendline of measured transport rates in a current without waves is also shown. The TRANSPOR 2000 model has been applied to compute the suspended transport for both sites. The bed roughness has been varied as k_{s}=0.01 m and 0.02 m. The suspended sand size is taken equal to the measured values. Other data are: T_{p}=4 s, \varphi= angle between current and
wave direction= 15 degrees for Maplin Sands and 90 degrees for the Galgeplaat, temperature= 10 degrees, salinity= 30 promille. The computed suspended sand transport rates are also shown in Figure 3.3.15. The computed suspended transport rates show good agreement (within factor 2) for bed roughness in the range of 0.01 to 0.02 m. The suspend transport rates computed for conditions without waves are also shown.

Figure 3.3.15 Depth-integrated suspended sand transport as function of current velocity for sand in the range of 0.14-0.18 mm; measured data from Maplin Sands 1973-1975 and Galgeplaat 1983; computed values based on measured field data; effect of $k_s$

Sand in range of 0.14-0.18 mm: data from Duck surf zone, USA, 1991
To be described later.............

Sand in range of 0.10-0.14 mm: data from Duck shelf zone, USA, 1991
Another dataset used for verification of the TRANSPOR2000 model is the field data set of Madsen et al., 1993. An instrumented bottom boundary layer tripod was deployed on the inner shelf at a depth of about 13 m off the U.S. Army Corps of Engineers Field Research Facility at Duck (USA) over a 2-week period that included the severe Halloween Storm of late October 1991. The bottom sediments consisted of fine sands with $d_{50}$ of about 0.1 mm. OBS sensors were used to measure the suspended sediment concentrations at several elevations above the bed. The OBS signals were corrected for electronic drift and for background concentrations due to the presence of very fine silt and clay in the water. It is noted that the OBS signals may be rather inaccurate, as the sensors are very sensitive to fine sediments in the water. Herein, the data of bursts 50 to 80 have been averaged to obtain a group-averaged value including variation ranges, yielding the following values: $h=$ water depth= 13 m (±0.5), $H_s=$ sign. wave height= 3.75 m (±0.25), $T=$ wave period= 11 s (±1), $v=$ depth-averaged current= 0.6 m/s (±0.1), angle w-c= 80 degr., $d_{50}$= 0.1 mm. The measured concentrations are shown in Figure 3.3.16; the variation range of the average values is about ± 50%. The measured wind-induced current velocity at $z= 0.29$ m above the bed is about
0.35 (±0.1) m/s. The bed was assumed to be flat during the storm with an effective bed roughness value of \( k_s = 0.01 \) m. Other values used are: \( d_{50} = 0.2 \) mm, temperature= 15 °C, salinity= 30 promille. Maximum computed concentrations are obtained for the smallest water depth, the largest wave height and current velocity. Similarly, minimum concentrations are obtained for the largest water depth, the smallest wave height and current velocity. Given the variation ranges of computed and measured values, the agreement is satisfactory.

![Diagram showing concentration variation](image)

**Figure 3.3.16** Measured and computed sand concentrations at a depth of 13 m on the inner shelf off DUCK, USA (Madsen et al., 1991)

### 3.3.6 Effect of particle size on suspended transport

To show the effect of particle size on the suspended load transport and bed-load transport, computations (using TRANSPOR 2000) have been made for a specific case (see Section 2.2.4). The bed-form roughness is specified in Section 2.2.4 (standard values are \( k_s = 0.05 \) m for low-energy case; \( k_s = 0.03 \) m for medium-energy case and \( k_s = 0.01 \) m for high-energy case). The suspended sediment size is taken equal to the \( d_{50} \) of the bed material (\( d_{50} = d_{50} \)). To study the effect of the bed-form roughness on the suspended transport, the bed-form roughness was also taken twice as large. An increase of the bed-form roughness results in a decrease of the near-bed current velocities, an increase of the near-bed mixing coefficients and an increase of the effective bed-shear stress and hence the near-bed reference concentration. Furthermore, the reference concentration is inversely proportional to the bed-roughness value \( k_s \), because the reference level is taken equal to the bed-roughness value (\( a = k_s \), see Eq. 3.3.7). Thus, the reference concentration decreases for increasing \( k_s \)-value. All these parameters affect the suspended transport.

Figure 3.3.17 shows the suspended transport vector (\( q_s \)) including both the current-related and the wave-related transport components as a function of particle diameter. The bed-load transport vector is also shown (for discussion of bed load transport, see Section 2.2.4)
The results for the suspended transport are:

- **low-energy event:** the current-related suspended transport is zero because there is no
  longshore current; the wave-related suspended transport (oscillatory component) is
  dominant for particle sizes smaller than about 0.3 mm; this transport component is
  approximately constant for d<0.3 mm and decreases significantly for larger particle sizes;
  the wave-related suspended transport increases with about 30% (particles between 0.5
  and 1 mm) for increased bed-form roughness (from k_s= 0.05 to 0.1 m) mainly due to the
  increase of the bed-shear stress and hence the reference concentration (the near-bed
  mixing is about constant); it is noted that the suspended transport is rather small
  compared to the bed-load transport for particles larger than about 0.5 mm; the modelling
  of these small suspended transport values is highly uncertain;

- **high-energy event:** the suspended transport (due to steady and oscillatory flow
  components) is dominant compared to the bed-load transport; the suspended transport
  rate decreases significantly for increasing particle diameter; the suspended transport
  decreases with about 30% to 40% for increased bed-form roughness (from k_s= 0.01 to
  0.02 m) mainly due to the relatively large decrease of the reference concentration
  (although the near-bed mixing increases);

- **medium-energy event:** the suspended transport (due to steady and oscillatory flow
  components) is dominant for particle sizes smaller than about 0.4 mm; the suspended
  transport decreases significantly for increasing particle size; the suspended transport
  increases with about 30% (particles <1 mm) for increased bed-form roughness (from k_s= 0.03 to
  0.06 m) mainly due to the increase of the near-bed mixing coefficient (the reference concentration decreases slightly); it is noted that the suspended transport is rather small compared to the bed-load transport for particles larger than about 0.5 mm.

The modelling of the suspended transport for conditions with bed materials larger than about
0.5 mm is highly uncertain, because the effects of these relatively large particles on the
mixing process is not very well known. Analysis results of sand concentration profiles
measured in currents show that the current-related sediment mixing coefficient is larger than
the fluid mixing coefficient (β-factor>1, Van Rijn, 1993). Furthermore, the effect increases
with increasing particle size. These effects may be caused by the dominant influence of the
vortex-induced centrifugal forces acting on the particles and forcing them to the outside of the
vortices with a consequent increase of the effective mixing length. The wave-related sediment
mixing in the near-bed zone (\(\varepsilon_{w,\text{bed}}\); Eq. 3.3.6b) shows similar effects. This parameter is
modelled by an empirical expression (based on data from experiments with bed material in
the range of 0.1 to 0.5 mm): \(\varepsilon_{w,\text{bed}} = 0.004 \, \delta_e \, D_\text{w} \, U_\text{b} \) with D_\text{w} = particle parameter, U_\text{b} = near-
bed peak orbital velocity and \(\delta_e\) = thickness of mixing layer (Eq. 3.3.6b). Based on this
empirical expression, the wave-related sediment mixing in the near-bed zone increases with
increasing particle size.

The effect of particle size on suspended transport has been studied for particles in the range
of 0.1 to 3 mm (see Figure 3.3.17). Thus, the wave-related mixing coefficient has been
applied well outside its validity range of 0.1 to 0.5 mm. To evaluate the inaccuracies
involved, the suspended transport has also been computed for a modified wave-related
mixing coefficient (using Eq. 3.3.6b with D_\text{w} =15, if D_\text{w} >15). This reduces the wave-related
mixing and hence the wave-related suspended transport for particles larger than about 0.5
mm, as can be observed in Figure 3.3.18. The suspended transport is substantially reduced
(factor 2 to 3) for particle sizes larger than about 0.5 mm.
More research is necessary for accurate prediction of the wave-induced suspended transport for relatively coarse materials (>0.5 mm; coarse sand and gravel beds).

![Graph showing transport rate vs. particle size for different energy events](image)

**Graph Details:**
- **Low-energy event**
- **Medium-energy event**
- **High-energy event**

- **Transport rate (kg/s/m)**
- **Particle size (mm)**
Figure 3.3.17  Effect of particle size on computed suspended transport and bed-load transport rates (TRANSPOR 2000); Low-Energy event: $k_s = 0.05$ m; Medium-Energy event: $k_s = 0.03$ m; High-Energy event: $k_s = 0.01$ m (standard values)

Figure 3.3.18  Effect of wave-related mixing coefficient on computed suspended transport; bed roughness $k_s = 0.05$, $0.03$ and $0.01$ m for L-E, M-E and H-E events (TRANSPOR 2000)

3.4 Conclusions

Suspended transport processes (currents only)
- Reliable field data sets from major rivers and estuaries (currents only) are available in the literature for water depths larger than 1 m, particle sizes in the range of 0.1 to 2 mm, depth-averaged current velocities in the range of 0.3 to 2.2 m/s; most data refer to suspended transport processes.

- The sand transport data have been clustered into four particle size classes (0.14-0.18 mm; 0.18-0.4 mm; 0.4-0.6 mm and 2.5-3 mm). The effect of water depth (between 1 and 15 m) on sand transport appears to be of minor importance. The sand transport is strongly dependent on the current velocity; the sand transport increases from about 0.001 to 10 kg/s/m for velocities increasing from 0.4 to 2.2 m/s. Bed-load transport dominates at low velocities.

- The effect of particle size (between 0.14 and 0.6 mm) on sand transport is rather strong for low velocities, but reduces gradually for larger velocities (larger than about 1.4 m/s). The suspended transport of fine sand in the range of 0.14-0.18 mm is relatively large at low velocities (0.4-0.5 m/s) compared that of sand in the range of 0.18-0.4 mm. This may be related to the effect of ripple type bed forms, which may be rather pronounced in conditions with a fine sand bed of 0.14-0.18 mm. Another cause may be the presence of selective transport processes in the sense that the finer fractions of the sand bed are winnowed and more easily transported in suspension. Furthermore, most of the data in the low velocity range have been measured in non-equilibrium tidal flow conditions with settling sediments (overloading).

**Suspended transport processes (combined current and waves)**

- Reliable data of suspended transport processes in coastal conditions are rather scarce. Field measurements have been carried out at sites in Britain (Maplin Sands, Boscombe Bay), USA (Duck beach) and The Netherlands (Galgpolit, Eastern Scheldt and Egmond beach). Furthermore, data sets measured in the large-scale wave tank (Delta flume of Delft Hydraulics) are available to study the vertical distribution of time-averaged sand concentrations.

- Analysis of the measured sand concentration profiles shows that:
  - the near-bed concentrations increase with increasing relative wave height ($H_w/h$) and the concentration profile becomes more uniform for increasing relative wave height;
  - the near-bed concentrations (at 0.05 m above the bed) are between 0.2 and 5 kg/m$^3$ for $H_w/h$=0.2 to 0.9;
  - the concentrations are confined to the near-bed region ($z/h$<0.1) for non-breaking waves ($H_w/h$<0.3);
  - the concentration profile consists of a two-layer system ($z/h$<0.1 and >0.1) for relative wave heights ($H_w/h$) between 0.3 and 0.5;
  - the concentration profile is almost uniform for relatively large breaking waves ($H_w/h$ between 0.5 and 1);
  - the presence of relatively large current velocities (>0.5 m/s) has a strong effect on the concentration profile; both the near-bed concentrations and the concentrations in the outer layer are increased due to increased bed-shear stresses and increased mixing.
capacity;
- the presence of bed forms has a strong effect on the concentration profile.

- The current-related suspended sand transport in the coastal zone is found to be strongly dependent on the relative wave height \(H_o/h\), particularly for current velocities in the range 0.2 to 0.5 m/s. The transport of suspended sand in the size range 0.18-0.4 mm increases by a factor of 10 to 20 when waves with a relative wave height of about 0.2 are superimposed on a current of about 0.4 m/s. This factor may increase to about 50 for a relative wave height of about 0.33. The increase of the suspended transport due to the wave effect decreases with increasing current velocity, particularly for finer sand (0.14-0.18 mm). The increase of the suspended transport of 0.14-0.18 mm sand is not more than a factor of 2 to 3 when waves with a relative wave height of 0.2 are superimposed on a current of 0.8 m/s. The suspended transport rates show a considerable increase for breaking wave conditions with \(H_o/h\) larger than about 0.4.

- The high-frequency wave-related suspended transport is found to be onshore-directed (in the wave direction) in conditions with irregular waves. This transport component increases with increasing significant wave height, but decreases with decreasing particle size. This latter effect is related to the ripple dimensions; the ripples are more pronounced in conditions with a relatively coarse sand bed (0.3 to 0.5 mm) resulting in stronger vortex motions and associated suspension processes and hence increased wave-related transport rates.

**Suspended transport model**

- The wave-related suspended transport component is modelled by an expression based on an instantaneous response of the suspended sediment concentrations and transport to the near-bed orbital velocity. Large scale wave tank data have been used to calibrate the empirical coefficient involved. This coefficient is found to be constant for all test results considered (two grain sizes 0.16 and 0.33 mm).

- The current-related suspended transport is based on the modelling of the time-averaged velocity profile and the time-averaged sand concentration profile. The time-averaged (over the wave period) advection-diffusion equation is applied to compute the time-averaged sand concentration profile for combined current and wave conditions. Various empirical coefficients have been recalibrated using data from large scale wave tank experiments. Important parameters for the suspended transport are the current-related and the wave-related bed form roughness \((k_{s_c} \text{ and } k_{s_w})\). These parameters are directly related to the size and geometry of the bed forms (ripples). At present stage of research both parameters \((k_{s_c} \text{ and } k_{s_w})\) are used as input parameters.

- For conditions with currents only (no waves) the computed transport rates of sediments in the range of 0.14 to 0.6 mm show reasonably good agreement (within factor 2) with measured values for velocities in the range of 0.7 to 1.8 m/s. The computed values in the particle size range of 0.14 to 0.18 mm may be somewhat too small for velocities in the range of 0.4 to 0.6 m/s. This may be related to the effect of ripple type bed forms, which
may be rather pronounced in conditions with fine sand bed of 0.14-0.18 mm. These ripple effects may not be properly taken into account by the model.

- Bed roughness values in the range of $k_s = 0.03$ to 0.1 m have not much effect on the computed transport rates for conditions with currents only. The effect of the water depth on the suspended transport is rather significant at low velocities of 0.4 to 0.6 m/s. The suspended transport at a depth of 1 m is a factor 30 to 5 larger than the suspended transport at a depth of 10 m for velocities in the range of 0.4 to 0.6 m/s. This effect reduces to a factor 2 for a velocity of 1.1 m/s and to a factor 1.1 for a velocity of 2 m/s. The relatively strong effect of the water depth on the suspended transport at low velocities is not that serious, because bed-load transport is dominant for velocities up to 0.6 m/s. The suspended size has a rather strong effect on the suspended transport, particularly at low velocities.

- For conditions with combined current and waves the computed sand transport rates are strongly dependent on the wave-related bed roughness in the low velocity range of 0.2 to 0.6 m/s, for which the effect of wave-induced mixing of sediment dominates over turbulence-induced mixing. The effect of bed roughness is less important in conditions with a dominating current. The computed transport rates are in reasonably good agreement with measured values, provided that the proper bed roughness value is taken (in the range of 0.01 to 0.05 m). Generally, a bed roughness value of 0.02 m yields the best results.

- Sensitivity computations for a depth of 5 m and a sand bed of 0.2 mm show that bed-load transport is onshore-directed for $H_s/h \leq 0.4$ and offshore-directed for $H_s/h > 0.4$; the wave-related suspended transport is onshore-directed and much larger than (factor 5 to 10) than the bed-load transport; the onshore-directed wave-related suspended transport is slightly larger than the offshore-directed current-related suspended transport for $H_s/h = 0.2$ and smaller than the offshore-directed current-related suspended transport for $H_s/h \geq 0.3$; the net total transport is onshore-directed for $H_s/h = 0.2$; the net total transport is offshore-directed for $H_s/h \geq 0.3$. 

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_{50} and fraction size=100%) to determine the sand transport rate. This method is known as the Single-Fraction method (SF-method). Herein, a Multi-Fraction method (MF-method) is presented that can be used to determine the sand transport rate for graded bed material by schematizing the bed material in multiple fractions.

A first understanding of the difference in transport rate according to the single fraction method (SF) and multi-fraction method (MF) can be obtained by using a simple transport formula of the type \( q_s \propto v^\alpha d^\beta \) (\( v=\)velocity, \( d=\)diameter). The exponent \( \alpha \) may vary between \( \alpha=2 \) or \( \alpha=2.5 \). The effect of hiding is neglected.

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

<table>
<thead>
<tr>
<th>( p_i )</th>
<th>( d_i )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>0.5d</td>
</tr>
<tr>
<td>0.15</td>
<td>0.666d</td>
</tr>
<tr>
<td>0.2</td>
<td>0.8d</td>
</tr>
<tr>
<td>0.2</td>
<td>1d</td>
</tr>
<tr>
<td>1.25d</td>
<td>1.5d</td>
</tr>
<tr>
<td>2d</td>
<td>2d</td>
</tr>
</tbody>
</table>

the MF transport can be expressed in terms of the SF transport yielding: \( q_{s,N=7}=1.26q_{s,N=1} \) for all current velocities.

Taking a wider (symmetric) size distribution with \( d_i=0.25d \), 0.5d, 0.75d, 1d, 1.33d, 1.5d, 2d and \( d_i=4d \) with similar percentages, it follows that \( q_{s,N=7}=2.1q_{s,N=1} \).

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 \( d^3 \) (\( \alpha=2 \)) in stead of \( d^2 \), the MF method also yields \( q_{s,N=7}=1.26q_{s,N=1} \) (or \( q_{s,N=7}=2.1q_{s,N=1} \) for a wider size distribution). Thus, the transport rates are the same, but in the latter case (\( q_s \) proportional to \( d^3 \)) the transport of finer particles is dominant (suspended load) in the transport process, whereas in the former case (\( q_s \) proportional to \( d^2 \)) the transport of coarser particles is dominant.
In natural conditions the bed material generally has a slightly asymmetric size distribution. Furthermore, hiding and sorting processes do occur, which should be taken into account. The effect of these phenomena on the transport process of graded sediment is not quite clear. The transport rate of graded sediment may increase or decrease compared to the transport rate of ‘uniform’ sediment of the same median diameter ($d_{50}$). The modelling of transport processes for graded sediments using a multi fraction method will be studied in this Chapter 4.

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; $q_s \approx d^m$ 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. 2.2.2.

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; $p=\text{protrusion of a particle above others and d= diameter.}$ 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:
p/d= 0.8 \quad \frac{\tau_{cr}}{\tau_{cr,p=0}} = 0.09-0.12 \\
p/d= 0.6 \quad \frac{\tau_{cr}}{\tau_{cr,p=0}} = 0.12-0.15 \\
p/d= 0.4 \quad \frac{\tau_{cr}}{\tau_{cr,p=0}} = 0.20-0.25 \\
p/d= 0.2 \quad \frac{\tau_{cr}}{\tau_{cr,p=0}} = 0.45-0.55 \\
p/d=-0.2 \quad \frac{\tau_{cr}}{\tau_{cr,p=0}} = 2.5-2 \\

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 \( d_s/h; \) \( d_s = \) 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 in the literature.

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 McAradell (1993) performed laboratory experiments with bimodal sand-gravel mixtures (\( d_{10}=0.3 \) mm, \( d_{30}=8 \) mm, \( d_{60}=30 \) 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 at 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 at 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 \( p_i/F_i > 1 \), with \( p_i = \) proportion of each fraction in the transported material and \( F_i = \) 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 (\( q_{in} \)) are normalized to their availability (\( F_i \)) in the bed material and plotted as a function of the bed-shear stress (\( q_{in}/F_i \) against \( \tau_s \)). 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 (\( \tau_{s,\text{crit}} \)) 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 4.2.1 shows the critical bed-shear stress as a function of particle diameter based on the results of Misri et al. (184), Wilcock (1993), Wilcock et al. (1988), Petit (1994) and Kuhnle (1993) for unimodal sediment (sand or gravel) mixtures and Wilcock et al. (1993) and Kuhnle (1993) for bimodal (sand and gravel) 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. 4.2.1 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 unimodal mixture and a relatively strong increase of the critical bed-shear stress for the bimodal mixtures. The finest fractions (d<1 mm) of the Wilcock 1993 mixtures and the Kuhnle 1993 mixtures also seem to have a somewhat higher critical shear stress. The datasets of Wilcock et al. 1988 and Kuhnle 1993 for unimodal mixtures show an almost constant critical bed-shear stresses (horizontal line) for sand in the range of 0.5 to 2 mm. A horizontal line in Fig. 4.2.1 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 bed-load particles is the same as that of the original bed material under all conditions. Based on the data of Fig. 4.2.1, the concept of equal mobility may be reasonably correct for unimodal (sand or gravel) sediments between 0.2 and 10 mm. The curves cross the Shields curve (uniform sediment) at approximately the median diameter (d\textsubscript{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 material of the same size, while the smaller size fractions require higher bed-shear stresses than for uniform material of the same size. The reason for this is that the larger 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 the particles of uniform bed-material. Kuhnle (1993) performed experiments with bimodal sand-gravel mixtures; the percentages of gravel were varied (10%, 25% and 40%). The mobility of the sand particles was not found to change much for increasing percentages of gravel, but the mobility of the gravel particles decreased rapidly for increasing percentage of gravel. Thus, bimodal sediment shows combined behaviour of uniform and mixed sediment. If the bed sediment is more bimodal, then the sand and gravel fractions show different behaviour. The gravel fractions show slightly size-selective transport, while the sand part shows equal mobility. These trends found for bimodal sediment cannot be explained with a simple hiding-exposure concept assuming that the bed surface is thoroughly mixed. A more plausible explanation is local size segregation in the form of longitudinal sand ribbons on gravel beds, which have been observed in all considered experiments. If fine sediment is concentrated in more or less homogeneous patches, then the critical shear stress will approach that of uniform sediment. The hiding-exposure phenomenon then acts to such extent that the sediment within a sand patch exhibits equal mobility. The coarse fractions of the bed surface will be more exposed than in the case of uniform coarse material, resulting in relatively large critical bed-shear stress but still considerably smaller than for the uniform coarse material (considerably below the Shields curve). Part of the coarse material is buried beneath the fine patches, which decreases the transport rate of the coarse material.

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 is caused by relatively large bed-shear stresses (curve sloping upward to the left, see Fig. 4.2.1). Thus, in the sand-size range the larger grains may be selectively removed, leaving behind the finer grains. The dataset of Rakoczi (1975) confirms that finer and coarser particles in the gravel range \((d_{50} = 1.4 \text{ and } 5 \text{ mm})\) 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 \((0.5 \text{ mm})\) the finer grains were eroded before the medium and coarser grains were set into motion (Rakoczi, 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. 4.2.1 can be used to derive the exposure or hiding factor \((\xi)\) for particles in a mixture. Two approaches are possible:

- \(d_{50}\) method introduced by Egiazaroff (1965); the hiding factor is defined as the ratio of the dimensionless critical bed-shear stress of fraction diameter \(d\) and the dimensionless critical bed-shear stress according to Shields based on the median diameter \(d_{50}\) (see Eq 4.2.1a)

- \(d_i\)-method; the hiding factor is defined as the ratio of the dimensionless critical bed-shear stress of fraction diameter \(d\) and the dimensionless critical bed-shear stress according to Shields based on the fraction diameter \(d_i\) (see Eq 4.2.1b).

Definitions of the hiding-exposure factor:

\[
\begin{align*}
\xi_{50,0} &= \frac{\theta_{cr,0}}{\theta_{cr,d_{50},shields}} = \left(\frac{\tau_{b,cr,0}}{\tau_{b,cr,d_{50},shields}}\right) \left(\frac{d_{50}}{d}\right) = F(d/d_{50}) \quad \text{(4.2.1a)} \\
\xi_{cr} &= \frac{\theta_{cr}}{\theta_{cr,d_{i},shields}} = \frac{\tau_{b,cr}}{\tau_{b,cr,d_{i},shields}} = F(d_i/d_{50}) \quad \text{(4.2.1b)}
\end{align*}
\]


in which: \( \tau_{b,cr,i} \) = critical bed-shear stress of fraction \( d_i \) within a mixture, \( \tau_{b,cr,i,shields} \) = critical bed-shear stress of fraction \( d_i \) based on Shields curve, \( \tau_{b,cr,d_{50},shields} \) = critical bed-shear stress of \( d_{50} \) based on Shields curve, \( \theta = \) dimensionless critical bed-shear stress = \( \tau_{b,cr}/(\rho_s \cdot g) \).

The computed values of the exposure or hiding factor are shown in Figure 4.2.2. The hiding factor or exposure factor of Egiazarov (1965), defined as a multiplication factor to the dimensionless critical shear stress, is given by:

\[
\xi_{d,0} = \frac{\theta_{cr,i}}{\theta_{cr,d_{50}}} = \left( \frac{\log(19)}{\log(19 d/d_{50})} \right)^2
\]  

(4.2.2)

in which: \( d_i \) = mean particle diameter of size class \( i \), \( d_{50} \) = median diameter of bed material mixture.

Equation (4.2.2) is shown in Figure 4.2.2Top.

Originally, Egiazarov used the average diameter \( d_m \) in Eq.(4.2.2) in stead of \( d_{50} \). In the present approach the \( d_{50} \) is used to be consistent with the single fraction transport formulas in which generally the \( d_{50} \)-value is being used as the representative parameter. The hiding factor of Egiazarov yields values, which are somewhat too large for the finest fractions (\( d_i/d_{50} < 0.2 \)) and also for the coarsest fractions (\( d_i/d_{50} > 3 \)).

The data in Fig. 4.2.2Top can be represented reasonably well by a linear expression: \( \xi_{d,0} = (d_i/d_{50})^f \).

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. 4.2.2. 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 in an apparatus consisting of a board which was progressively inclined. The angle of the board at the instant of grain movement was defined to be the pivoting angle. The pivoting angle can be expressed, as: \( \phi = \phi_o \cdot (d_i/d_{50})^f \) with \( \phi_o \) = pivoting angle for uniform grains.

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

- spheres (2.3 to 38 mm) \( \phi_o = 20^\circ \) to \( 35^\circ \), \( f = -0.55 \) to \( -0.75 \)
- ellipsoidal natural pebbles (4 to 50 mm) \( \phi_o = 30^\circ \) to \( 40^\circ \), \( f = -0.35 \) 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 \( \phi_o \)-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 \( \phi_o = 50^\circ \)
- diameter= 1 mm \( \phi_o = 47^\circ \)
- diameter = 10 mm \( \phi_0 = 38^\circ \)

An exposure factor can also be derived from the work of Komar (1996), yielding:

\[
\xi_{i,q} = \left[ \frac{\tan(\phi_0(d_i/d_{50})^{0.3})}{\tan(\phi_0)} \right] \quad \text{for } f = 0.3
\]  

(4.2.3)

in which: \( d_i \) = mean particle diameter of size class \( i \), \( d_{50} \) = median diameter of bed material mixture, \( \phi_0 = 61.5^\circ \) = angle of repose (or pivoting angle = angle between vertical line and a line through one of the contact points of the grains, see Fig. 4.2.2) for uniform grains; the \( \xi_{i,q} \) factor goes to infinity for \( d_i/d_{50} \) approaching about 0.283.

Equation (4.2.3) is shown in Fig. 4.2.2Top. For \( d_i/d_{50} < 1 \) the angle of repose increases resulting in an increase of the hiding factor.

The hiding factor of Egiazarov 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 approaches of Egiazarov and Komar do not account for vortex-induced pick-up of smaller grains hiding between larger grains (negative relative protrusion).

The hiding factor data based on the \( d_i \)-method (Fig. 4.2.2Middle) can be very well represented by a linear expression: \( \xi_{i,q} = (d_i/d_{50})^{-1} \).

Buffington and Montgomery (1997) have presented an extensive overview of most existing empirical hiding factors. They conclude that no universal relation can be found.
Figure 4.2.2  Hiding factor according to Egiazaroff (1965), Komar (1996) and others

Top:    Hiding-exposure factor based on $d_{50}$-method
Middle: Hiding-exposure factor based on $d_i$-method
Bottom: Pivoting angles
Using Eq. (4.2.1a), the critical bed-shear stress for fraction i can be written as:

\[ \tau_{b,cr,i} = \xi_{i,0} (d_i / d_{50}) \tau_{b,cr,d_{50},shields} \]  \hspace{1cm} (4.2.4)

In this method the critical bed-shear stress for fraction i is based on the critical bed-shear stress of the \( d_{50} \) of the mixture (herein termed as \( d_{50} \)-method). The hiding factor of Egiazarov (see Fig. 4.2.2Top) is used as the standard method to represent the hiding-exposure effects. The application of the \( d_{50} \)-method for the 400 \( \mu \)m-mixture is shown in Table 4.2.1. As can be seen, the \( \tau_{b,cr,i} \) parameter is increasing for increasing fraction diameters using the hiding factor of Egiazarov. The \( \tau_{b,cr,i} \) values are constant using a linear hiding factor. The results are also shown in Figure 4.2.3. The behaviour of the \( d_{50} \)-method according to Eq (4.2.4) is opposite to the \( d_i \)-method of Eq (4.2.5).

Using Eq. (4.2.1b), the critical bed-shear stress for fraction i can be written as:

\[ \tau_{b,cr,i} = \xi_{i,0} \tau_{b,cr,d_i,shields} \]  \hspace{1cm} (4.2.5)

In this method the critical bed-shear stress for fraction i is based on the critical bed-shear stress according to Shields using the \( d_i \)-values (herein termed as \( d_i \)-method). Data of Wilcock (1993) and Petit (1994) have been used to determine the \( \xi_{i,0} \) parameter (see Figs. 4.2.1 and 4.2.2Middle) resulting in \( \xi_{i,0} = (d_i / d_{50})^{-1} \). Basically this method yields a nearly constant critical bed-shear stress \( \tau_{b,cr,i} \) for all fractions of a mixture with diameters larger than about 0.5 mm.

The application of the \( d_i \)-method for mixtures with particle diameters smaller than 0.5 mm results in an increase of \( \tau_{b,cr,i} \) for decreasing fraction diameter as a consequence of the form of the Shields’ curve for smaller diameters. To explain this, an example is given for a bed material mixture of \( d_{50} = 400 \mu \)m, see Table 4.2.2. As can be seen, the \( \tau_{b,cr,i} \) parameter is about constant for the fractions with diameters larger than 0.5 mm, but increases strongly for the fractions with diameters smaller than 0.5 mm. The \( \tau_{b,cr,i} \) values are also shown in Figure 4.2.3.

<table>
<thead>
<tr>
<th>400-( \mu )m sediment</th>
<th>( d_{50} = 0.4 ) mm</th>
<th>( d_{50} = 0.86 ) mm</th>
<th>( \tau_{b,cr,d_{50},shields} )</th>
<th>( d_{50} )-method</th>
</tr>
</thead>
<tbody>
<tr>
<td>fraction (( \mu )m)</td>
<td>( d_i ) (( \mu )m)</td>
<td>( p_i ) (%)</td>
<td>( d_i / d_{50} )</td>
<td>( \xi_{i,0} ) Linear</td>
</tr>
<tr>
<td>75-175</td>
<td>125</td>
<td>5</td>
<td>0.31</td>
<td>3.22</td>
</tr>
<tr>
<td>175-275</td>
<td>225</td>
<td>15</td>
<td>0.56</td>
<td>1.79</td>
</tr>
<tr>
<td>275-375</td>
<td>325</td>
<td>20</td>
<td>0.81</td>
<td>1.23</td>
</tr>
<tr>
<td>375-575</td>
<td>475</td>
<td>20</td>
<td>1.19</td>
<td>0.84</td>
</tr>
<tr>
<td>575-775</td>
<td>675</td>
<td>20</td>
<td>1.69</td>
<td>0.59</td>
</tr>
<tr>
<td>775-1125</td>
<td>950</td>
<td>15</td>
<td>2.38</td>
<td>0.42</td>
</tr>
<tr>
<td>1125-1175</td>
<td>1150</td>
<td>5</td>
<td>2.88</td>
<td>0.35</td>
</tr>
</tbody>
</table>

*Table 4.2.1 Hiding factor and critical bed-shear stress; \( d_{50} \)-method*
Finally, it is remarked that the results presented in Fig. 4.2.3 for the 400 µm-mixture are based on various assumptions. Comparison with measured data is required to select the most realistic method. In the absence of data the d50-method is supposed to give the most realistic results and will be used as the standard approach in the multi-fraction method (see Section 4.3).

<table>
<thead>
<tr>
<th>400-µm sediment d50=0.4 mm</th>
<th>d1-method</th>
</tr>
</thead>
<tbody>
<tr>
<td>fraction (µm)</td>
<td>d1 (µm)</td>
</tr>
<tr>
<td>75-175</td>
<td>125</td>
</tr>
<tr>
<td>175-275</td>
<td>225</td>
</tr>
<tr>
<td>275-375</td>
<td>325</td>
</tr>
<tr>
<td>375-575</td>
<td>475</td>
</tr>
<tr>
<td>575-775</td>
<td>675</td>
</tr>
<tr>
<td>775-1125</td>
<td>950</td>
</tr>
<tr>
<td>1125-1715</td>
<td>1150</td>
</tr>
</tbody>
</table>

Table 4.2.2  Hiding factor and critical bed-shear stress; d1-method

<table>
<thead>
<tr>
<th>Grain diameter di (mm)</th>
<th>Critical bed-shear stress (N/m²)</th>
</tr>
</thead>
</table>

Figure 4.2.3  Computed critical bed-shear stress for fractions of 400 µm-mixture using four different methods
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 may 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 \((\xi_i)\) to account for the non-uniformity effects. This correction is necessary because the coarser particles are more exposed to the near-bed steady and oscillatory flow (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 theoretical correction factor of Egiazaroff (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:

\[
\begin{align*}
q_b &= \Sigma p_i \cdot q_{b,i} \quad \text{and} \\
q_s &= \Sigma p_i \cdot q_{s,i}
\end{align*}
\]  
(4.3.1)

(4.3.2)

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

Both the bed-load transport and the suspended transport strongly depend on the dimensionless bed-shear stress parameter \(T\), defined as: \(T=(\theta-\theta_0)/\theta_{cr}=(\tau_b-\tau_{b,cr})/\tau_{b,cr}\) for uniform sediment (see Equations 2.2.2 and 3.3.8). In conditions with graded sediment it is not clear in what way the \(T\)-parameter should be modelled. Various approaches can be used. This will be studied in Section 4.3.2.

The multi-fraction method as proposed in the present study will result 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). Most single-fraction methods have been calibrated against field data and are therefore believed to produce realistic results. If this is accepted, then the multi-fraction method should yield about the same transport rate as that computed by the single fraction method for reasonably uniform sediment. 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 approach may however not be correct for specific conditions such as close to initiation of motion and/or for very wide size distributions of the bed material and will therefore not be used in this study.

4.3.2 Analysis of T-parameter for multi fraction method; sensitivity and calibration computations

Effect of critical bed-shear stress on transport rates
Based on the work of Kleinhans and Van Rijn (2000) for bed-load transport of coarse graded sediment, the T₁ parameter is best modelled, as follows (no hiding factor in the denominator):

Method 1: \[ T₁ = \frac{[\theta_1 \cdot \xi_1 \cdot \theta_{cr,d50}]}{\theta_{cr,d50}} = \left[ \frac{\tau_{b-} \cdot \xi_1 \cdot (d_i/d_{50}) \tau_{b,cr,d50}}{[(d_i/d_{50}) \tau_{b,cr,d50}]} \right] \] \hspace{1cm} (4.3.3a)

with \[ \theta_1 = \tau_{b-}/((\rho_c-\rho)gd) \] = effective mobility parameter, \[ \theta_{cr,d50} = \tau_{b,cr,d50}/((\rho_c-\rho)gd_{50}) \] = critical mobility parameter based on \( d_{50} \), \[ \tau_{b-} \] = effective bed-shear stress based on \( k_{a,grain} = 3d_{50} \), \( \tau_{b,cr,d50} \) = critical shear stress based on \( d_{50} \) (d_{50}-method), \[ \xi_i \] = exposure factor according to Egiazaroff function or other function, see Fig. 4.2.2.

The critical bed-shear stress can also be related to the fractional particle size:

Method 2: \[ T₁ = \frac{[\theta_1 \cdot \xi_i \cdot \theta_{cr,di}]}{\theta_{cr,di}} = \frac{[\tau_{b-} \cdot \xi_i \cdot \tau_{b,cr,di}]}{\tau_{b,cr,di}} \] \hspace{1cm} (4.3.3b)

with \[ \theta_1 = \tau_{b-}/((\rho_c-\rho)gd) \] = effective mobility parameter, \[ \theta_{cr,di} = \tau_{b,cr,di}/((\rho_c-\rho)gd_{i}) \] = critical mobility parameter based on \( d_i \), \( \tau_{b,cr,di} \) = critical shear stress based on \( d_i \) (d_{i}-method), \[ \xi_i \] = exposure factor according to Egiazaroff function or other function, see Fig. 4.2.2.

In the present study focussing on suspended transport, five other approaches have been used to model the T₁-parameter, as follows:

Method 3: \[ T₁ = \frac{[\tau_{b-} \cdot \xi_i \cdot \tau_{b,cr,d50}]}{\tau_{b,cr,d50}} \] ; method 1 without \( d/d_{50} \) \hspace{1cm} (4.3.3c)

Method 4: \[ T₁ = \frac{[\tau_{b-} \cdot \tau_{b,cr,d50}]}{\tau_{b,cr,d50}} \] ; method 3 without hiding factor (h.f.) \hspace{1cm} (4.3.3d)

Method 5: \[ T₁ = \frac{[\tau_{b-} \cdot (d_i/d_{50}) \tau_{b,cr,d50}]}{[(d_i/d_{50}) \tau_{b,cr,d50}]} \] ; method 1 without h. f. \hspace{1cm} (4.3.3e)

Method 6: \[ T₁ = \frac{[\tau_{b-} \cdot \xi_i \cdot (d_i/d_{50}) \tau_{b,cr,d50}]}{[\tau_{b,cr,d50}]} \] ; method 1 without \( d/d_{50} \) in denominator \hspace{1cm} (4.3.3f)

Method 7: \[ T₁ = \frac{[\tau_{b-} \cdot \xi_i \cdot (d_i/d_{50}) \tau_{b,cr,d50}]}{[(d_i/d_{50})^{\eta} \tau_{b,cr,d50}]} \] ; method 1 with modified \( d_i/d_{50} \) in denominator \( (\eta=0.5 \) based on calibration using measured concentrations; see below)
Figure 4.3.1  
Dimensionless $T_r$-parameter as function of particle diameter

Method 1 has the best theoretical foundation; the other methods 3 to 7 are modifications of method 1.

The $T_r$-values according to methods 1, 2, 3 and 7 are shown in Figure 4.3.1 for a practical case of relatively fine graded bed material: $\tau'_i=6.7$ N/m$^2$; $d_{50}=200$ μm (using 41 fractions between 50 and 450 μm); $\tau_{cr,d_{50}}=0.161$ N/m$^2$. Methods 1 and 7 yield relatively large $T_r$-values for the smaller fraction diameters (smaller than the $d_{50}$) and relatively small $T_r$-values for the larger fraction diameters (larger than $d_{50}$); method 7 is based on $\eta=0.5$ as found from calibration (see below). Method 2 yields slightly decreasing $T_r$-values (from 40 to 30) for increasing particle diameter. Method 3 yields slightly increasing $T_r$-values (from 38 to 41) for increasing particle diameter, which is not realistic.

Sensitivity computations

The effect of the critical bed-shear stress on the bed-load and suspended transport according to the MF-method has been studied by performing sensitivity computations using methods 1, 2 and 7, as described by Eqs. (4.3.3a, 4.3.3b and 4.3.3g). For reference the transport according to the SF-method is also given.

These computations have been made for two mixtures: 200 μm and 400 μm (see Table 4.3.1). Two flow conditions are considered:
- water depth=3 m and current velocity=0.5 m/s with bed roughness $k_e=0.05$ m,
- water depth=3 m and current velocity of 1.5 m/s with $k_e=0.01$ m.

The water temperature was taken to be 15 °C and the salinity 30 promille.
Table 4.3.1  Bed material composition for 200-μm and 400-μm mixtures

The results (see Tables 4.3.2 and 4.3.3) show:

**200 μm bed material**
- the MF-methods yield bed-load transport rates which are approximately the same as those of the SF-method;
- the MF-method 1 yields suspended transport rates, which are a factor 2 to 3 larger than the MF-method 2 and the SF-method;
- the MF-method 1 yields slightly larger (30%) suspended transport than the MF-method 7;
- the MF-methods yield reference concentrations that are approximately the same as those of the SF-method;

**400 μm bed material**
- the MF-methods 1 and 7 yield slightly smaller (10%) bed-load transport rates than the SF-method for \( v = 0.5 \); the bed-load transport of MF-method 2 is about 10% to 20% smaller than that of the SF-method;
- the MF-method 1 yields suspended transport rates, which are a factor 2 to 5 larger than the MF-method 2;
- the MF-method 1 yields suspended transport rates, which are a factor 1.5 to 2 larger than the MF-method 7;
- the MF-methods 1 and 7 yields reference concentrations that are approximately the same as those of the SF-method; thus the relatively large suspended transport rates of MF-methods 1 and 7 are mainly caused by the mixing process acting on the finer fractions.

These analysis results show that the effect of the critical bed-shear stress (using methods, 1, 2 and 7) on the total bed-load transport according to the MF-approach is relatively small (maximum 20%). Figure 4.3.2 shows the effect of the critical bed-shear stress on the fractional transport rates (bed load and suspended transport). The critical bed-shear stress based on the \( d_{50} \)-method (method 1) yields relatively small critical bed-shear stresses (see Figure 4.2.3) for the smaller fraction diameters and hence relatively large transport rates for the smaller fraction diameters, both for the bed-load transport and the suspended transport. The bed-load transport of the fraction 0.225 mm based on the \( d_l \)-method (method 2) is a factor 2 smaller than that based on the \( d_{50} \)-method (method 1). The bed-load transport of the fraction 0.125 mm based on the \( d_l \)-method (method 2) is zero, whereas that based on the \( d_{50} \)-
method (method 1) is non zero. The suspended transport shows similar effects. Method 7 yields similar effects as method 1.

Sensitivity computations using the mean particle diameter $d_m$ (as proposed by Egiazaroff, 1965) in stead of the $d_{50}$ in Eq.(4.3.3a) have also been made for two flow conditions: $v = 0.5$ and 1.5 m/s. This resulted in 10% to 25%-larger bed load and suspended transport rates for the 200 μm-sediment ($d_m = 225 μm$) and 400 μm-sediment ($d_m = 535 μm$). For the 800 μm-sediment ($d_m = 1135 μm$, see Table 4.3.7) the transport rates based on the $d_m$-method are 30% smaller than that based on the $d_{50}$-method in case of $v = 1.5$ m/s; the transport rates based on the $d_m$-method are a factor 3 to 4 smaller for $v = 0.5$ m/s (just beyond initiation of motion). Thus, there is a large effect for conditions just beyond initiation. Herein, it is assumed that the transport rates based on the $d_{50}$-method are more realistic than those based on the $d_m$-method.
| Mixture 200 µm (d_{so}=0.20 mm  d_{so}=0.31 mm) |
|---|---|---|---|
| v=0.5 m/s | h=3 m | k_{s}=0.05 m |

<table>
<thead>
<tr>
<th>MF method</th>
<th>(7 frac.)</th>
<th>SF method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meth. 1</td>
<td>Meth. 7</td>
<td>Meth. 2</td>
</tr>
<tr>
<td>τ_{b,cr,i} modified using d_{50}</td>
<td>τ_{b,cr,i} using d_{i} Egia h.f.</td>
<td>d_{so}=0.20 mm</td>
</tr>
<tr>
<td>q_{b} (kg/s/m)</td>
<td>q_{s} (kg/s/m)</td>
<td>q_{b} (kg/s/m)</td>
</tr>
<tr>
<td>0.0040</td>
<td>0.013</td>
<td>0.0040</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Ref. concentration</th>
</tr>
</thead>
<tbody>
<tr>
<td>c_{s}=0.127 kg/m^{3}</td>
</tr>
<tr>
<td>c_{s}=0.118 kg/m^{3}</td>
</tr>
<tr>
<td>c_{s}=0.118 kg/m^{3}</td>
</tr>
<tr>
<td>c_{s}=0.125 kg/m^{3}</td>
</tr>
</tbody>
</table>

| Mixture 200 µm (d_{so}=0.20 mm  d_{so}=0.31 mm) |
|---|---|---|---|
| v=1.5 m/s | h=3 m | k_{s}=0.01 m |

<table>
<thead>
<tr>
<th>MF method</th>
<th>(7 frac.)</th>
<th>SF method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meth. 1</td>
<td>Meth. 7</td>
<td>Meth. 2</td>
</tr>
<tr>
<td>τ_{b,cr,i} using d_{50} Egia h.f.</td>
<td>τ_{b,cr,i} using d_{i} Egia h.f.</td>
<td>d_{so}=0.20 mm</td>
</tr>
<tr>
<td>q_{b} (kg/s/m)</td>
<td>q_{s} (kg/s/m)</td>
<td>q_{b} (kg/s/m)</td>
</tr>
<tr>
<td>0.178</td>
<td>8.13</td>
<td>0.182</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Ref. concentration</th>
</tr>
</thead>
<tbody>
<tr>
<td>c_{s}=20.2 kg/m^{3}</td>
</tr>
<tr>
<td>c_{s}=19.7 kg/m^{3}</td>
</tr>
<tr>
<td>c_{s}=19.1 kg/m^{3}</td>
</tr>
<tr>
<td>c_{s}=19.9 kg/m^{3}</td>
</tr>
</tbody>
</table>

Table 4.3.2  Effect of critical bed shear stress and hiding factor on bed-load and suspended transport according to MF-method and comparison of transport rates according to MF- and SF-methods; 200 µm mixture
Mixture 400 μm (d_{50}=0.40 mm d_{95}=0.86 mm)

| v=0.5 m/s | h=3 m | k_s=0.05 m |

<table>
<thead>
<tr>
<th>MF method</th>
<th>(7 frac.)</th>
<th>SF method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meth. 1 (\tau_{b,cr,i}) using (d_{50}) Egia h.f.</td>
<td>Meth. 7 (\tau_{b,cr,i}) modified using (d_{50}) Egia h.f.</td>
<td>Meth. 2 (\tau_{b,cr,i}) using (d_{i}) Egia h.f.</td>
</tr>
<tr>
<td>(q_b) (kg/s/m)</td>
<td>(q_s) (kg/s/m)</td>
<td>(d_{50}) (mm)</td>
</tr>
<tr>
<td>0.0060</td>
<td>0.0052</td>
<td>0.0061</td>
</tr>
</tbody>
</table>

Refer. concentration
- \(c_a=0.175\) kg/m\(^3\)
- \(c_s=0.154\) kg/m\(^3\)
- \(c_s=0.116\) kg/m\(^3\)
- \(c_s=0.178\) kg/m\(^3\)

Mixture 400 μm (d_{50}=0.40 mm d_{95}=0.86 mm)

| v=1.5 m/s | h=3 m | k_s=0.01 m |

<table>
<thead>
<tr>
<th>MF method</th>
<th>(7 frac.)</th>
<th>SF method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meth. 1 (\tau_{b,cr,i}) using (d_{50}) Egia h.f.</td>
<td>Meth. 7 (\tau_{b,cr,i}) modified using (d_{50}) Egia h.f.</td>
<td>Meth. 2 (\tau_{b,cr,i}) using (d_{i}) Egia h.f.</td>
</tr>
<tr>
<td>(q_b) (kg/s/m)</td>
<td>(q_s) (kg/s/m)</td>
<td>(q_b) (kg/s/m)</td>
</tr>
<tr>
<td>0.30</td>
<td>3.20</td>
<td>0.32</td>
</tr>
</tbody>
</table>

Refer. concentration
- \(c_a=30.0\) kg/m\(^3\)
- \(c_s=29.5\) kg/m\(^3\)
- \(c_s=24.2\) kg/m\(^3\)
- \(c_s=30.0\) kg/m\(^3\)

Table 4.3.3 Effect of critical bed shear stress and hiding factor on bed-load and suspended transport according to MF-method and comparison of transport rates according to MF- and SF-methods; 400 μm mixture
<table>
<thead>
<tr>
<th>Fractions</th>
<th>MF method</th>
<th>( q_b ) (kg/s/m)</th>
<th>( q_s ) (kg/s/m)</th>
<th>( q_b ) (kg/s/m)</th>
<th>( q_s ) (kg/s/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (125( \mu m ))</td>
<td>Method 1 ( \tau_{bcr,i} ) ( d_{50} ) and Egia h.f.</td>
<td>0.00053</td>
<td>0.0033</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2 (225( \mu m ))</td>
<td>Method 2 ( \tau_{bcr,i} ) ( d_{50} ) and Egia h.f.</td>
<td>0.0013</td>
<td>0.0012</td>
<td>0.00063</td>
<td>0.00038</td>
</tr>
<tr>
<td>3 (325( \mu m ))</td>
<td></td>
<td>0.0015</td>
<td>0.00043</td>
<td>0.00116</td>
<td>0.00029</td>
</tr>
<tr>
<td>4 (475( \mu m ))</td>
<td></td>
<td>0.0012</td>
<td>0.00015</td>
<td>0.00138</td>
<td>0.00018</td>
</tr>
<tr>
<td>5 (675( \mu m ))</td>
<td></td>
<td>0.00092</td>
<td>0.00006</td>
<td>0.00117</td>
<td>0.000087</td>
</tr>
<tr>
<td>6 (950( \mu m ))</td>
<td></td>
<td>0.00048</td>
<td>0.000018</td>
<td>0.00058</td>
<td>0.000024</td>
</tr>
<tr>
<td>7 (1150( \mu m ))</td>
<td></td>
<td>0.00012</td>
<td>0.0000027</td>
<td>0.000128</td>
<td>0.0000028</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>0.0060</td>
<td>0.0052</td>
<td>0.00505</td>
<td>0.00097</td>
</tr>
</tbody>
</table>

**Table 4.3.4** Effect of critical bed-shear stress on fractional bed load and suspended transport rates according to MF-method, 400\( \mu m \)-mixture, \( v=0.5 \text{ m/s} \)
Effect of critical bed-shear stress on fractional bed load and suspended transport rates according to MF-method, 400 µm-mixture, v = 0.5 m/s

Effect of grain shear stress on transport rate

The effect of the grain roughness on the transport rates according to the MF-method (based on Eq 4.3.3a; method 1) has been studied by using $k_{\text{grain},i} = 3d_i$ in stead of $k_{\text{grain},i} = 3d_{90}$. The latter approach implies a constant grain roughness and hence grain shear stress ($\tau_4$) for all fractions. The results for the 200 µm bed material ($v = 1.5$ m/s) are shown in Table 4.3.5. The grain roughness of $k_{\text{grain}} = 3d_{90}$ yields transport rates that are about 30% larger than that based on $k_{\text{grain},i} = 3d_i$ for the fractions with relatively small diameters (fractions 1, 2 and 3). The total suspended transport rate based on $k_{\text{grain}} = 3d_{90}$ is about 20% larger. The total bed-load transport rate is about 10% larger.

It is not quite clear how the grain roughness should be modelled in case of mixtures consisting of graded sediments. Two situations may occur: well-mixed bed material or non-mixed bed material with segregation of the individual fractions. In case of well-mixed bed material the application of a constant grain roughness $k_{\text{grain}} = \alpha d_{90}$ ($\alpha$ in range of 1 to 3)
seems to be the most logic approach. In case of bed material consisting of segregated fractions it may be better to relate the grain roughness to the size of the individual fractions of the bed material mixture ($k_{s,\text{grain}} = 3d_i$).

<table>
<thead>
<tr>
<th>Fractions</th>
<th>MF ( \tau_{b,\text{crit}} ) using ( d_50 )</th>
<th>method with Egiazaroff</th>
<th>hiding factor ( k_{s,\text{grain}} = 3d_i )</th>
<th>( q_b ) (kg/s/m)</th>
<th>( q_e ) (kg/s/m)</th>
<th>( q_h ) (kg/s/m)</th>
<th>( q_s ) (kg/s/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.012</td>
<td>3.56</td>
<td>0.0083</td>
<td>2.58</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.031</td>
<td>2.65</td>
<td>0.024</td>
<td>2.13</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0.037</td>
<td>1.03</td>
<td>0.032</td>
<td>0.90</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>0.035</td>
<td>0.46</td>
<td>0.031</td>
<td>0.43</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>0.033</td>
<td>0.29</td>
<td>0.031</td>
<td>0.28</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>0.023</td>
<td>0.12</td>
<td>0.023</td>
<td>0.12</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>0.0071</td>
<td>0.019</td>
<td>0.0078</td>
<td>0.02</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>0.178</td>
<td>8.13</td>
<td>0.159</td>
<td>6.46</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reference concentration ( c_a )</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( c_a = 20.2 \text{ kg/m}^3 )</td>
<td>( c_a = 17.5 \text{ kg/m}^3 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.3.5 Effect of grain-roughness on bed load and suspended transport according to MF-method, 200\( \mu \text{m} \)-mixture, \( v = 1.5 \text{ m/s} \)

**Calibration of T-parameter based on experimental results from laboratory flume**

Sand concentration profiles measured in a small flume for combined current and wave conditions over a fine graded sediment bed (Jacobs and Dekker, 2000 and Sistermans, 2000) have been used for evaluation of the proper expression of the \( T_i \)-parameter (methods 1 to 7). The experiments have been carried out in a wave-current flume of the Delft University of Technology. Two types of sand have been used: uniform sand with \( d_{50} \) of about 0.16 mm and graded sand with \( d_{30} \) of about 0.2 mm. The water depth was about 0.5 m in all tests. The significant wave heights were in the range of 0.12 to 0.2 m. The depth-averaged current velocities were in the range of 0.1 to 0.3 m/s (following current). The basic data are given in Table 4.3.6. Instantaneous velocities and sand concentrations at various elevations above the bed have been measured by use of an acoustic instrument. Instantaneous fluid velocities have also been measured by use of an electro-magnetic velocity meter. Time-averaged sand concentration profiles have been obtained by using a pump sampling instrument consisting of 10 intake tubes (internal opening of 3 mm; sampling time of about 20 min). The suspended sand sizes based on analysis of samples in a settling tube are given in Table 4.3.6. Ripple dimensions have been determined by use of a bed profile follower. Figure 4.3.3 shows measured sand concentration profiles (based on the pumped concentrations) for uniform and graded bed material with \( H_i = 0.15 \text{ m} \) and \( v = 0.22 \text{ m/s (TOP)} \) and \( H_i = 0.19 \text{ m} \) and \( v = 0.2 \text{ m/s (Bottom)} \). As can be observed, the sand concentrations are largest for the graded sediment bed. The near-bed concentrations are
about 50% larger for the graded sediment bed; the sand concentrations higher up in the water column are much larger (factor 2 to 4) for the graded sediment bed, which is caused by the winnowing of the fine sediments from the bed. The SF-method has been applied to compute the sand concentration profile for the uniform sand bed (suspended sand size is assumed to be equal to the \(d_{50}\) of the bed material) and the MF-method (10 fractions) has been used for the graded sand bed. The reference level has been set to \(\sigma = 0.5\Delta_s\) (with \(\Delta_s\) = ripple height). The bed-form roughness has been set to \(k_s = \Delta_s\). The results are shown in Figure 4.3.3. The computed sand concentrations based on the SF-method show reasonably good agreement with the measured concentrations for the uniform sand.

The effect of the \(T\)-parameter (methods 1 to 7) on the computed sand concentration profiles is shown in Figure 4.3.3. The results of method 1, 6 and 7 are shown in Figure 4.3.3. The reference concentration is only weakly affected by the various methods. The concentrations based on method 1 show reasonably good agreement with the measured near-bed concentrations for the graded sand, but the concentrations higher up in the water column are somewhat overpredicted. The reference concentration of method 1 is slightly larger than that of the SF-method. Methods 2, 3, 4 and 6 yield significantly smaller concentrations, mainly because the \(T\)-parameter of the finer fractions is reduced and hence the reference concentration and the concentration profiles of the finer fractions are reduced in the total concentration profile. The hiding factor has not so much effect far beyond the critical conditions for initiation of motion (compare results of methods 1 and 5 and methods 3 and 4). Method 7 yields a concentration profile similar to the measured profile, which has been obtained by using \((d/d_{50})^{0.5}\) in the denominator (calibration). Method 7 is the calibrated version of method 1 to improve the results. Method 7 is preferred and will be used as the standard method.
### Table 4.3.6  Basic data of experiments with uniform sand bed and graded sand bed in small-scale wave-current flume (Jacobs and Dekker, 2000; Sistermans, 2000)

<table>
<thead>
<tr>
<th></th>
<th>Exp 1-l graded</th>
<th>M218u1</th>
<th>Exp 2-l graded</th>
<th>M220u1</th>
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</thead>
<tbody>
<tr>
<td>h (m)</td>
<td>0.515</td>
<td>0.506</td>
<td>0.518</td>
<td>0.516</td>
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<td>H1 (mm, %)</td>
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<td>0.152</td>
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<tr>
<td>Tp (s)</td>
<td>2.5</td>
<td></td>
<td>2.6</td>
<td></td>
</tr>
<tr>
<td>v (m/s)</td>
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<td></td>
<td>0.21</td>
<td></td>
</tr>
<tr>
<td>d0 (mm)</td>
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<td>0.16</td>
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</tr>
<tr>
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<tr>
<td>d0 (mm)</td>
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</tr>
<tr>
<td>d (mm)</td>
<td>0.275</td>
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<td>0.130</td>
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</tr>
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<td>Δλ (m)</td>
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<td>0.022</td>
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<tr>
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<td>0.15</td>
<td></td>
</tr>
<tr>
<td>Tc (°C)</td>
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<td>26</td>
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</tr>
<tr>
<td>z (m)</td>
<td>0.024</td>
<td>0.038</td>
<td>0.03</td>
<td>0.033</td>
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<tr>
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<td>0.874</td>
<td>1.95</td>
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<td>0.048</td>
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<td>1.58</td>
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<td>0.073</td>
<td>0.075</td>
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<td>0.3</td>
<td>0.98</td>
<td>0.5</td>
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<td>0.098</td>
<td>0.13</td>
<td>0.093</td>
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<td>c (kg/m³)</td>
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<td>0.17</td>
<td>0.69</td>
<td>0.32</td>
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<td>0.163</td>
<td>0.16</td>
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<td>0.158</td>
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<td>c (kg/m³)</td>
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<td>0.03</td>
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<td>z (m)</td>
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<td>0.238</td>
<td>0.24</td>
<td>0.233</td>
</tr>
<tr>
<td>c (kg/m³)</td>
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<td>0.015</td>
<td>0.16</td>
<td>0.065</td>
</tr>
<tr>
<td>z (m)</td>
<td>0.274</td>
<td>0.288</td>
<td>0.29</td>
<td>0.283</td>
</tr>
<tr>
<td>c (kg/m³)</td>
<td>0.065</td>
<td>0.011</td>
<td>0.11</td>
<td>0.057</td>
</tr>
</tbody>
</table>
Figure 4.3.3  
Sand concentration profiles for uniform and graded bed material; calibration of T-parameter based on two experiments in a small-scale flume.
4.3.3 Comparison of computed transport rates based on MF-method and SF-method

To compare the results of the MF-method and SF-method (as described in Sections 2.2, 3.3 and 4.3), computations have been made for three types of bed material consisting of seven sand fractions (N=7), as given in Table 4.3.7. It is noted that the applied bed material distributions (Table 4.3.7) represent conditions with somewhat more fine sediment compared to bed material with a log-normal distribution. The multi-fraction method is based on: method 7 (Eq. 4.3.3g); \( k_{s,\text{grain}} = 3d_{90} \) and the hiding factor of Egiazarov.

The water depth is \( h = 3 \) m. The current velocity is in the range of 0.3 to 2 m/s. The wave heights are \( H_s = 0 \) m and \( H_s = 1 \) m with a period of \( T_s = 7 \) s. The angle between the wave and current direction is 90°. The current-related and wave-related bed roughness heights are: \( k_s = 0.05 \) m for \( v = 0.3, 0.4, 0.5 \) m/s, \( k_s = 0.03 \) m for \( v = 0.75, 1.0, 1.25 \) m/s, \( k_s = 0.02 \) m for \( v = 1.5 \) m/s and \( k_s = 0.01 \) m for \( v = 2 \) m/s. The water temperature is 15 °C and the salinity is 30 promille.

Computation of the suspended transport using the single fraction method requires information of the suspended sediment, which is taken equal to the bed material size (\( d_s = d_{50,\text{bed}} \)).

<table>
<thead>
<tr>
<th>200-µm bed material</th>
<th>400-µm bed material</th>
<th>800-µm bed material</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>(\text{fraction})</td>
<td>(d_i)</td>
<td>(p_i)</td>
</tr>
<tr>
<td>(µm)</td>
<td>(µm)</td>
<td>(%)</td>
</tr>
<tr>
<td>50-150</td>
<td>75</td>
<td>5</td>
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<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>
<td>225</td>
<td>20</td>
</tr>
<tr>
<td>250-300</td>
<td>275</td>
<td>20</td>
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<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 4.3.7 Bed material composition

**Bed load and suspended load transport**

Figure 4.3.4 shows the computed current-related suspended transport rates as a function of the current velocity and the wave height for the 200 µm-bed material based on the single fraction (SF) method and the multi-fraction (MF) method. The MF-method yields substantially larger transport rates varying between a factor 3 for the lower transport regime (\( H_s = 0 \) m and \( v = 0.4 \) m/s) and a factor of 1.5 for the upper transport regime (\( H_s = 1 \) m). The increase of the suspended transport rate according to the MF-method is caused by the relatively large contribution of the finer fractions to the total suspended transport rate.
Figure 4.3.4  Current-related suspended transport using single fraction method and multi fraction method (method 7; Eq. 4.3.3g) for 200 µm-bed material

Figure 4.3.5 shows the ratio of the transport rates according to the SF and MF methods. Bed-load transport ratios \( \left( \frac{q_{b,N-1}}{q_{b,N-1}} \right) \) as well as suspended transport ratios \( \left( \frac{q_{s,N-1}}{q_{s,N-1}} \right) \) are presented; N=1 refers to the single fraction method and N=7 refers to the multi-fraction method. 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, \( \gamma = 0 \)).

The results for the 200, 400 and 800 µm-bed material (Figure 4.3.5) show the following trends:

**Bed load transport**
- the bed-load transport rates according to the MF-method are somewhat smaller (maximum 50% for 800 µm-bed material) at small current velocities without waves (\( H_c = 0 \) m);
- the bed-load transport rates according to the MF-method are almost the same as those of the SF-method for conditions with waves (\( H_c = 1 \) m);

**Suspended transport**
- for conditions without waves (\( H_c = 0 \) m) the suspended transport rates according to the MF-method are significantly larger (factor 2 to 3) than those of the SF-method, particularly at small velocities (see also Figure 4.3.5Top);
- for conditions without waves (\( H_c = 0 \) m) the suspended transport rates according to the MF-method are the same or slightly larger (factor 1 to 2);
for conditions without waves (H_w= 0 m) the transport ratio (q_{N=1}/q_{N=2}) shows an increasing trend for increasing current velocity and a decreasing trend for conditions with waves (H_w= 1 m); for conditions with very large current velocities (decreasing effect of wave-induced mixing compared to current-induced mixing) the transport ratio seems to approach a constant value of about 0.5 for 800 µm-bed material and about 0.6 for 200 µm-bed material;

• for conditions with waves (H_w= 1 m) and relatively small current velocities the suspended transport rates according to the MF-method are only slightly larger (0 to 20%) than those of the SF-method; all fractions participate in the transport process because of the dominant wave-induced pick-up and mixing of particles.

Examples of concentration profiles for 200, 400 and 800 µm-bed material are shown in Figure 4.3.6.

For conditions with waves and a weak current (dominant wave effect) the SF-method and the MF-method yield approximately the same near-bed concentrations (see Figure 4.3.6 Middle and Bottom) and the same suspended transport rates. This can be understood by analysis of the near-bed wave-related sediment mixing coefficient. This parameter is modelled by an empirical expression (based on experiments with bed material in the range of 0.1 to 0.5 mm; Van Rijn 1993): \( c_{w,bed} = 0.004 \, \delta \, D_s \, U_b \), with \( D_s \) = particle parameter, \( U_b \) = near-bed peak orbital velocity and \( \delta \) = thickness of mixing layer (Eq. 3.3.6b). Based on this empirical expression, the wave-related sediment mixing in the near-bed zone shows a linear increase with increasing particle size. This phenomenon may be caused by the dominant influence of the vortex-induced centrifugal forces acting on the particles and forcing them to the outside of the vortices with a consequent increase of the effective mixing length. A similar effect has been found for current-related sediment mixing (\( \beta \)-factor>1, Van Rijn, 1993).

The gradient of the near-bed concentration profile is given by: \( dc/dz = (w_s/c_{w,bed}) \) c. Thus, the gradient depends on the ratio \( w_s/c_{w,bed} \).

The near-bed sediment mixing coefficient depends on hydrodynamic parameters (\( U_b, \delta \)) and shows a linear increase with increasing particle size.

The fall velocity (\( w_f \)) shows an almost linear increase with particle size for particles in the range between 0.2 and 2 mm.

Hence, the ratio \( w_s/c_{w,bed} \) is almost independent of particle size. This means that the gradient of the near-bed concentration profile is almost independent of the particle size and that both the SF and MF method will produce approximately the same results (see Figure 4.3.6 Middle and Bottom). Thus, the gradient of the near-bed concentration profile mainly depends on the hydrodynamic parameters.

The MF-method and SF-method yield the same gradient in the near-bed zone, but the MF-method yields larger concentrations higher up in the water column due to the contribution of the finer fractions. This latter effect is not so important for the transport rate, because most sediment is transported in the near-bed zone. Furthermore, the reference concentration \( (c_r) \) is almost the same for the SF-method and the MF-method (Figure 4.3.6). Therefore, the MF-method and the SF-method yield approximately the same suspended transport rates for conditions with dominating waves (relatively large waves and weak current).

It is noted that the modelling of the wave-induced suspended transport for conditions with bed materials larger than about 0.5 mm is highly uncertain, because the effects of these relatively large particles on the mixing process is not very well known.
Furthermore, it is noted that the present results based on the MF-method are significantly different from those presented by Van Rijn (1997a). At that earlier stage the MF-method was based on method 2 (Eq. 4.3.3b; d₄-method) to compute the critical bed-shear stress, which resulted in significantly smaller suspended transport rates based on MF-method compared to those based on the SF-method in the lower transport regime: qₑ, N=1 > qₑ, N=7. The present results yield: qₑ,N=1 < qₑ,N=7.

**Suspended sediment size dₑ**

The suspended sand transport based on the SF-method can be adjusted to that of the MF-method by using a smaller suspended sediment size dₑ compared to the d₅₀ of the bed material. The results for all cases are given in Table 4.3.8. The ratio of dₑ and d₅₀,bed varies between 0.4 for a weak current over a coarse graded sediment bed and 0.85 for a strong current over a fine graded sediment bed. The ratio of dₑ and d₅₀,bed varies between 0.7 and 1 for conditions with combined current and waves. Examples of concentration profiles are given in Figure 4.3.6. Information of measured values for the Egmond coastal field site (surf zone) is given by Grasmeijer (2001). The ratio of dₑ and d₅₀,bed was about 0.8 (dₑ= 0.2 mm and d₅₀,bed= 0.24 mm) in April-May and October-November 1998.

Using this approach, 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. A method to estimate the representative suspended sediment size has been proposed by Van Rijn (1984, 1993). The present results yield somewhat smaller values for dₑ.

The application of the MF-method is most appropriate for graded bed material in conditions with weak currents and relatively low waves (Hₗ/h<0.2), because of the relatively large contribution of the finer fractions in the transport process resulting in relatively large suspended transport rates (larger than those of the SF-method).

| Bed material | waves & Hₗ= 0 m | | waves & Hₗ= 1 m |
|--------------|-----------------|-----------------|
|               | current         | current         |
|               | v=0.5 m/s      | v=1 m/s         | v=2 m/s         |
|               | v=0.5 m/s      | v=1 m/s         | v=2 m/s         |
| 200-μm        | dₑ/d₅₀=0.8      | 0.8             | 0.85            | 0.95           | 0.9            | 0.85            |
| 400-μm        | 0.65            | 0.7             | 0.75            | 0.9            | 0.85           | 0.8             |
| 800-μm        | 0.4             | 0.45            | 0.5             | 0.9            | 0.8            | 0.7             |

**Table 4.3.8** Ratio of suspended sediment size dₑ and d₅₀ of bed material resulting in approximately the same suspended transport based on the SF and MF-methods
**Figure 4.3.5**  Ratio of transport rates according to the SF and MF methods for 200, 400 and 800 μm-bed material; depth of $h = 3$ m; MF-method based on method 7 (Eq. 4.3.3g)
Figure 4.3.6 Sand concentration profiles based on SF-method and MF-method 7

Top: 200μm-bed; \( H_r = 0 \) m, \( v = 0.5 \) m/s;
Middle 1: 200μm-bed; \( H_r = 1 \) m, \( v = 0.5 \) m/s;
Middle 2: 400μm-bed; \( H_r = 1 \) m, \( v = 0.5 \) m/s;
Bottom: 800μm-bed; \( H_r = 1 \) m, \( v = 0.5 \) m/s.
4.4 Effect of graded bed material on sand transport based on experimental and computational results

**Bed-load transport in river flow conditions (laboratory and field data)**

The bed-load transport of coarse graded bed material (range of 0.1 to 100 mm) in river flow conditions based on various MF-methods has been studied by Kleinhaus and Van Rijn (2000). Data from flume experiments and from small-scale creeks in the USA have been used to improve the bed-load predictor of Meyer-Peter and Mueller and that of Van Rijn. The hiding-exposure function of Egiazarov was found to work well and a new hindrance factor was introduced to model conditions with a partly immobile bed. In many rivers with a sand-gravel bed the sand fraction is in motion, whereas the gravel fraction is on the threshold of motion during most conditions. Stochastic methods taking the turbulence characteristics into account have been applied to model bed-load transport near the threshold of motion.

**Bed load and suspended load transport in river flow conditions (field data)**

The TRANSPOR 2000 multi-fraction model (MF model, see Chapter 4) has been used to compute the suspended transport for sand in the range of 0.18-0.4 mm and current velocities in the range of 0.4-1.25 m/s. The hydrodynamic input data are taken as: depth = 5 m, $k_c = 0.03$ m, temperature = 15 degrees, salinity = 0 promille. The bed material has been represented by 6 fractions, as follows: 0.075 mm (10%), 0.15 mm (20%), 0.25 mm (20%), 0.35 (20%), 0.45 mm (20%), and 0.55 mm (10%), yielding a $d_{50}$ of 0.25 mm and $d_{90}$ = 0.55 mm. The results are shown in Figure 3.3.7 of Section 3.3.4. The computed total load transport rates show remarkably good agreement with the trend line of the measured transport rates; the computed values are somewhat too large for velocities larger than 0.6 m/s.

Similar computations have been made for sand in the range of 0.4-0.6 mm and current velocities in the range of 0.4-1.8 m/s. The hydrodynamic input data are taken as: depth = 8 m, $k_c = 0.03$ m, temperature = 15 degrees, salinity = 0 promille. The bed material has been represented by 6 fractions, as follows: 0.15 mm (10%), 0.25 mm (20%), 0.5 (20%), 0.7 mm (20%), 0.9 mm (20%) and 1.1 mm (10%), yielding a $d_{50}$ of 0.5 mm and $d_{90}$ = 0.9 mm. The results are shown in Figure 3.3.8 of Section 3.3.4. The computed total load transport rates show remarkably good agreement with the trend line of the measured transport rates; the computed values are somewhat too large at low velocities in the range of 0.5 to 0.7 m/s.

**Suspended load transport in river flow conditions (Enoree river, USA)**

Anderson (1942) collected suspended sediment samples in the Enoree river in the USA. At the point of collection the river was straight and regular with almost vertical banks (width of 15 m). The sand bed consisted of graded bed material with sizes in the range of 0.06 to 5 mm. The $d_{50}$ was in the range of 0.5 to 0.9 mm; the $d_{90}$ was in the range of 1 to 4 mm. Representative values are given in Table 4.4.1. The water depth was in the range of 0.9 to 1.5 m; the flow velocity was in the range of 0.55 to 0.85 m/s. The water surface slope was determined from two recording-gages located about 300 m apart at the upper and lower ends of the reach. The Chezy-equation has been used to determine the effective bed-roughness of Nikuradse (see Table 4.4.1). The effective bed-roughness values are rather large suggesting the presence of rather large bed forms (dunes). Bed forms were not discussed by Anderson. The suspended sediment sampler consisted of a number of pint milk bottles attached at regular intervals to a cable.
Sand concentration profiles have been computed for three cases (see Table 4.4.1) using \( N=8 \) fractions and \( N=1 \) fraction; in the latter case the suspended sand size \( d_s \) has been taken equal to the \( d_{50} \) of the bed material (\( d_s=d_{50}=0.655 \) mm). The reference concentration \( c_r \) has been applied at \( a=0.05h \). The current-related bed roughness (\( k_{er} \)) is given in Table 4.4.1.

The measured and computed concentrations are shown in Figure 4.4.1. For all three cases the near-bed concentrations are substantially overpredicted (factor 2 to 3), but the sand concentrations in the upper portion (90%) of the flow depth are rather well represented by the MF-method for depth-averaged velocities of 0.75 and 0.85 m/s. The concentrations higher up in the water column for a velocity of 0.55 m/s are considerably underpredicted. It is noted that the computed near-bed concentrations represent a spatially-averaged value along the bed forms, whereas the measured concentrations are local values at some location along the bed forms. The SF-method yields significantly underestimated sand concentrations in the upper portion of the flow depth.
<table>
<thead>
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<th>Bed material composition</th>
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<td>$d_{50} =$ 0.655 mm</td>
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<tr>
<td>$d_{90} =$ 1.3 mm</td>
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</tbody>
</table>

<table>
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<tr>
<th>Fraction sizes (mm)</th>
<th>Percentages</th>
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</thead>
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<td>0.1; 0.15; 0.21; 0.3; 0.42; 0.6; 0.85; 1.5</td>
<td>0.005; 0.01; 0.015; 0.045; 0.15; 0.225; 0.225; 0.325</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Date</th>
<th>Flow depth h (m)</th>
<th>Flow velocity v (m/s)</th>
<th>Slope S (-)</th>
<th>Bed roughness k_s (m)</th>
<th>Temp. (°C)</th>
</tr>
</thead>
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<tr>
<td>19 Febr. 1940; SN 2.40</td>
<td>1.52</td>
<td>0.85</td>
<td>0.00074</td>
<td>0.7</td>
<td>8</td>
</tr>
<tr>
<td>19 Febr. 1940; SN 17.00</td>
<td>1.28</td>
<td>0.75</td>
<td>0.00074</td>
<td>0.8</td>
<td>8</td>
</tr>
<tr>
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<td>0.55</td>
<td>0.00057</td>
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<td>24</td>
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</table>

<table>
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<tr>
<th>Measured sand concentrations</th>
<th>Enoree River</th>
</tr>
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<tbody>
<tr>
<td>19 Febr. 1940; SN 2.40</td>
<td>Height above bed (m)</td>
</tr>
<tr>
<td></td>
<td>0.035; 0.09; 0.15; 0.27; 0.37; 0.5; 0.62</td>
</tr>
<tr>
<td></td>
<td>Sand concentrations (kg/m³)</td>
</tr>
<tr>
<td></td>
<td>0.35; 0.25; 0.18; 0.12; 0.082; 0.065; 0.045</td>
</tr>
<tr>
<td>19 Febr. 1940; SN 17.00</td>
<td>Height above bed (m)</td>
</tr>
<tr>
<td></td>
<td>0.035; 0.11; 0.18; 0.31; 0.43; 0.61; 0.75; 0.88</td>
</tr>
<tr>
<td></td>
<td>Sand concentrations (kg/m³)</td>
</tr>
<tr>
<td></td>
<td>0.42; 0.17; 0.1; 0.07; 0.043; 0.038; 0.02; 0.016</td>
</tr>
<tr>
<td>19 Febr. 1940; SN 16.07</td>
<td>Height above bed (m)</td>
</tr>
<tr>
<td></td>
<td>0.035; 0.14; 0.22; 0.41; 0.58; 0.78</td>
</tr>
<tr>
<td></td>
<td>Sand concentrations (kg/m³)</td>
</tr>
<tr>
<td></td>
<td>0.11; 0.051; 0.048; 0.019; 0.016; 0.01</td>
</tr>
</tbody>
</table>

Table 4.4.1  Data of Enoree River, USA
Figure 4.4.1  Measured and computed sand concentration profiles for graded bed material in Enoree river; USA
Bed load transport in coastal conditions (laboratory data)

Tests have been carried out 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 2.1.1. The $d_{50}$ was about 0.24 mm and the $d_{90}$ was about 1 mm. All experimental results are in the sheet flow regime. The measured net transport rates under conditions of asymmetric wave motion are given in Table 2.1.1 and Figure 4.4.2. For comparison the results of Tests C1, B8 and B9 with uniform sand of 0.21 mm are also shown in Figure 4.4.2. As can be observed, the measured net transport rates are almost the same (within transport error range). Thus, the presence of a coarse fraction (30% with diameter of 1 mm) has almost no influence on the net transport rate.

Equation (2.2.2) has been used to compute the net transport rates using the single-fraction and the multi-fraction approach. The multi-fraction method is based on subdivision of the bed material in a number of size fractions; the transport rate of each size fraction is computed 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 (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 (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 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 single-fraction approach based on $d_{50}=0.24$ mm and $d_{90}=1$ mm yields an overestimation of about 30% to 60%, as can be seen in Figure 4.4.2. The 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 ($k_{s}=3d_{90}$ with $d_{90}=1$ mm), yielding relatively large bed-shear stresses for the tests P6, P7 and P9.

To apply the multi-fraction method, the mixture has been schematized in three fractions: two fine fractions (30% of $d_f=0.18$ mm, 40% of $d_f=0.3$ mm) and one coarse fraction (30% of $d_c=1$ mm). Method 1 (Eq. 4.3.3a) and method 7 (Eq. 4.3.3g) have been applied to compute the critical bed-shear stress with the hiding factor according to Egiazarov. The multi-fraction method yields somewhat smaller transport rates than the single-fraction method. The transport rates based on the MF-method are in better agreement with the measured values, see Figure 4.4.3. The bed-load transport of the fine fraction is obtained as $q_{b,\text{fraction1}}+q_{b,\text{fraction2}}$; the bed-load transport of the coarse fraction is represented by the transport of fraction 3 ($q_{b,\text{fraction3}}$). Neglecting the hiding factor, the computed transport rates (per fraction) are almost the same as those taking the hiding factor into account. Thus, the hiding factor has no effect on the transport rates for these conditions, because the effective bed-shear stresses in the sheet flow regime are much larger than the critical bed-shear stresses for all fractions.

The application of methods 1 and 7 with $k_{s,\text{grain}}=3d_{90}$ for the critical bed-shear stress results in a substantial overestimation of the transport rate of the fine fraction, see Figure 4.4.3. The application of both methods with $k_{s,\text{grain}}=3d_i$ for the critical bed-shear stress results in a substantial reduction of the fine fraction transport, see Figures 4.4.2 and 4.4.3. The transport rates of both the fine and coarse fraction are about the same (both about 50% of the total).

The best agreement (error of about 20%) is obtained by using method 7 (Eq. 4.3.3g; $d_{90}$-method) with $k_{s,\text{grain}}=3d_i$, see Figs. 4.4.2 and 4.4.3. The fact that this latter approach yields better results, is probably related to the segregation processes, which occurred during most
tests. Initially the bed material in the tunnel was well-mixed, but after the tests the coarser fraction was larger at the downwave end (in the direction of the largest peak orbital velocity, onshore direction) of the tunnel and smaller at the upwave end of the tunnel (Kroekenstoel, 1999). This is an indication of segregation of the fine and coarse fractions along the bed of the tunnel. In this case it may be better to relate the grain roughness to the size of the individual fractions of the bed material mixture \( k_{s,grain,i} = 3d_i \) rather than to the overall \( d_{90} \) of the initial bed material.

Overall, it is concluded that the multi-fraction method produces better results than the single fraction method for the bimodal sand mixture tested in the wave tunnel. The critical bed-shear stress based on method 7 yields the best results. The grain roughness can be best represented by \( k_{s,grain} = 3d_i \). The hiding factor is not important for the sheet flow regime (large bed-shear stresses).

![Graph showing net bed load transport rates](image)

**Figure 4.4.2** Effect of graded bed material on net bed-load transport rates (wave tunnel data of Delft Hydraulics)
Figure 4.4.3  Comparison of computed and measured transport rates per fraction (wave tunnel data P6, P7, P9 of Delft Hydraulics)
**Suspended load transport in coastal conditions (laboratory data)**

Experiments over a horizontal sand bed have been carried out in a small-scale wave-curent flume of the Fluids Mechanics Laboratory of the Delft University of Technology. (Jacobs and Dekker, 2000 and Sistmans, 2000) Two types of sand have been used in the experimental program: uniform sand with \(d_{50}\) of about 0.16 mm and graded sand with \(d_{50}\) of about 0.2 mm. The water depth was about 0.5 m in all tests. The hydrodynamic conditions are: irregular waves superimposed on a following current. The significant wave heights are in the range of 0.12 to 0.2 m. The depth-averaged current velocities are in the range of 0.1 to 0.3 m/s (following current). Time-averaged suspended sand concentrations and suspended transport rates have been measured. Instantaneous velocities and sand concentrations at various elevations above the bed have been measured by use of an acoustic instrument. Instantaneous fluid velocities have also been measured by use of an electro-magnetic velocity meter. Time-averaged sand concentration profiles have been obtained by using a pump sampling instrument consisting of 10 intake tubes (internal opening of 3 mm; sampling time of about 20 min). The basic data of characteristic tests are given in Tables 4.4.2 and 4.4.3. The suspended sand sizes based on analysis in a settling tube, are also given in Tables 4.4.2 and 4.4.3. The measured suspended sand size is about \(d_0 = 0.7\) to 0.9 \(d_{50,\text{bed}}\) for the uniform bed material and about \(d_0 = 0.35\) to 0.45 \(d_{50,\text{bed}}\) for the graded bed material. Ripple dimensions have been determined by use of a bed profile follower.

Figure 4.4.4 shows measured sand concentration profiles (based on the pumped concentrations) for waves with \(H_c = 0.15 m\) and 0.18 m over uniform and graded bed material. The experimental conditions are given in each plot. As can be observed by comparing the results of Figure 4.4.4Top and Middle \((H_c=0.15 m\) for both cases), the near-bed concentrations are significantly larger (factor 2) for the graded sediment bed (Middle); the sand concentrations higher up in the water column are somewhat larger for the graded sediment bed, which is caused by the winnowing of the fine sediments from the bed.

Figure 4.4.5 shows measured concentration profiles for combined wave and current conditions (3 tests). As can be observed, the concentrations are more uniformly distributed over the depth due to the mixing capacity of the current.

The SF-method has been applied to compute the sand concentration profile for the uniform sand bed (assuming that the suspended sand size \(d_0\) is equal to the \(d_{50}\) of the bed material) and for the graded sediment bed (assuming that the suspended sand size \(d_0\) is smaller than the \(d_{50}\) of the bed material).

The MF-method (10 fractions) has been used for the graded sand bed. The T-parameter is based on method 7 (standard method). The reference level has been set to \(a = 0.5\Delta\) (with \(\Delta\) = ripple height). The bed-form roughness has been set to \(k_s = \Delta\). The results are shown in Figures 4.4.4 and 4.4.5 for 6 cases.

The results are:

Waves alone (Figure 4.4.4)
- the computed sand concentrations based on the SF-method show reasonably good agreement with the measured concentrations for the uniform sand (Figure 4.4.4Top);
- the computed sand concentrations based on the MF-method show reasonably good agreement with the measured concentrations in the near-bed region for the graded sand bed (Figure 4.4.4Middle and Bottom), but the computed concentrations higher up in the water column are much too large compared to the measured values; the winnowing effect of the fine fractions is overestimated by the model;

Combined current and waves (Figure 4.4.5)
• the computed sand concentrations based on the MF-method show reasonably good agreement with the measured concentrations for the graded sand;
• the computed sand concentrations based on the MF-method show an increase of a factor 2 in the near-bed region to a factor 3 higher up the water column by increasing the bed roughness from $k_e=1 \Delta_e$ to $1.5 \Delta_e$;
• the computed sand concentrations based on the SF-method show reasonably good agreement with the measured values for the graded sediment, if the suspended sediment size is taken as $d_s = 0.65$ $d_{50, \text{bed}}$, the measured suspended sediment sizes vary between $d_s = 0.35$ $d_{50, \text{bed}}$ and $0.45$ $d_{50, \text{bed}}$.

<table>
<thead>
<tr>
<th>M218g graded</th>
<th>M220g graded</th>
<th>M418g graded</th>
</tr>
</thead>
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<tr>
<td>$h=0.5$ m</td>
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<td>$h=0.52$ m</td>
</tr>
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</tr>
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<td>$T_p=2.7$ s</td>
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</tr>
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<td>$d_{50}=0.26$ mm</td>
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<td>$d_{50}=0.42$ mm</td>
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<td>$\Delta_e=0.022$ m</td>
<td>$\Delta_e=0.022$ m</td>
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<tr>
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<td>$\lambda_e=0.18$ m</td>
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<td>$T_e=24^\circ C$</td>
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Table 4.4.2  Basic data of experiments with graded sand bed in small-scale wave-current flume (Tests M218g, M220g, M418g; Jacobs and Dekker, 2000)
### M015u uniform

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<th>$T_e$ (°C)</th>
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<th>$z$ (m)</th>
<th>$c$ (kg/m³)</th>
<th>$z$ (m)</th>
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<td>0.16</td>
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<th>$z$ (m)</th>
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### M018g graded

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**Table 4.4.3** Basic data of experiments with uniform sand bed and graded sand bed in small-scale wave-current flume (Tests M015u, M015g, M018g; Jacobs and Dekker, 2000)
Figure 4.4.4  Measured and computed sand concentration profiles for waves (no current) over uniform sand bed (Top) and graded sand bed (Middle and Bottom): 3 tests M015uniform, M015graded and M018graded
Figure 4.4.5  Measured and computed sand concentration profiles for combined current and waves over graded sand bed; 3 tests M218graded, M220graded and M418graded
4.5 Conclusions

Sorting, hiding and exposure processes

- 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.

- 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. The critical bed-shear stress for initiation of motion strongly depends on the degree of exposure of a grain with respect to surrounding grains. For steady flow in gravel-bed rivers it has been found that all sizes in a mixture begin to move at nearly the same bed-shear stress (equal mobility concept). For steady flow in graded sand-bed rivers the critical bed-shear stress of the finer fractions may be larger than those of the coarser fractions.

- The critical bed-shear stress of the fractions of graded bed material can be best represented by the critical Shield’s value based on the median particle diameter $d_{50}$ and the hiding-exposure factor of Egiazaroff (1965).

Sand transport model; multi-fraction method

- The sand transport for graded bed material can be computed by using a multi-fraction method (MF-method); the sand transport rate of each size fraction of the bed material is computed 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 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 between 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 bed-load transport and the suspended transport strongly depend on the dimensionless bed-shear stress parameter. Sand concentration profiles measured in a small flume for combined current and wave conditions over a fine graded sediment bed have been used for determination of the proper expression of the dimensionless bed-shear stress parameter.

- The multi-fraction (MF) method yields somewhat smaller (maximum 50%) bed-load transport rates than the single fraction (SF) method. The MF-method yields substantially larger suspended transport rates than the SF-method, varying between a factor 3 for the lower transport regime without waves to a factor of 1.5 for the upper transport regime with waves. The increase of the suspended transport rate according to the MF-method is caused by the relatively large contribution of the finer fractions to the total suspended transport rate.
• In case of well-mixed bed material the application of a constant grain roughness related to the \(d_{90}\) of the mixture seems to be the most logic approach. In case of bed material consisting of segregated fractions it may be better to relate the grain roughness to the grain size of the individual fractions of the bed material mixture.

• The suspended sand transport based on the SF-method can be adjusted to that of the MF-method by using a smaller suspended sediment size \(d_s\) compared to the \(d_{50}\) of the bed material. The ratio of \(d_s\) and \(d_{50,bed}\) varies between 0.4 for a weak current over a coarse graded sediment bed and 0.85 for a strong current over a fine graded sediment bed. The ratio of \(d_s\) and \(d_{50,bed}\) varies between 0.7 and 1 for conditions with combined current and waves.

• The application of the MF-method is found to be most appropriate for graded bed material in conditions with weak currents and relatively low waves (\(H_s/h<0.2\)), because of the relatively large contribution of the finer fractions in the transport process resulting in relatively large suspended transport rates (larger than those of the SF-method).

• The computed transport rates (based on the MF-method) for coarse sand in the range of 0.4 to 0.6 mm are in remarkably good agreement with measured transport rates for conditions with currents only.

• The computed transport rates (based on the MF-method) for bimodal sand mixture (\(d_{50}=0.24\) mm, \(d_{90}=1\) mm) are in remarkably good agreement with measured transport rates for oscillatory flow in the wave tunnel. The grain roughness can be best represented by the grain size of the individual fractions. The hiding factor is not important for the sheet flow regime (large bed-shear stresses).

• The computed sand concentrations and transport rates based on the MF-method are in reasonably good agreement with the measured values for experiments with graded sand in conditions with combined current and waves (in a small scale flume). The computed concentrations higher up in the water column are too large for conditions with oscillatory flow (no current).
5 Application of TRANSPOR 2000 model

5.1 Sand transport in constant water depth

5.1.1 Definition of cases

The TRANSPOR2000 model has been used to compute the total load transport in conditions with a constant water depth, a longshore current and waves propagating normal to the shore. The total load transport vector has been decomposed in the current-related transport component in longshore direction and the wave-related transport component in cross-shore direction. The effects of the undertow due to breaking waves and the wave-induced streaming near the bed have been neglected. The following data set has been used.

- Water depth; \( h = 5 \) m
- Wave height (see Table 5.1.1); \( H = 0 \) to 3 m
- Wave period (see Table 5.1.1); \( T_p = 5 \) to 8 s
- Depth-averaged current velocity (see Table 5.1.1); \( u = 0.1 \) to 2 m/s
- Undertow current; \( u_r = 0 \) m/s
- Wave-induced streaming near bed; \( u_b = 0 \) m/s
- Angle between wave and current direction; \( \phi = 90 \) degrees;
- Bed material; \( d_{50} = 0.25 \) mm
  - \( d_{10} = 0.5 \) mm
- Suspended sediment size (see Table 5.1.1); \( d_s = 0.17 \) to 0.25 mm
- Bed roughness (see Table 5.1.1); \( k_w = 0.02 \) to 0.1 m
  - \( k_{w,1} = 0.02 \) to 0.1 m
- Bed slope; horizontal
- temperature; \( T_e = 15 \) °C
- salinity; \( S_A = 0 \) promille

The current-related and wave-related bed roughness are assumed to be equal; a flat bed in the upper regime is assumed to have an effective roughness of 0.02 m (approx. the thickness of the wave boundary layer). In all, 55 computations have been made. This data set has also been used by Van Rijn 1993 (Appendix A; TRANSPOR 1993).
### Table 5.1.1 Input data

<table>
<thead>
<tr>
<th>Hs=0 m</th>
<th>Tp=5 s</th>
<th>Hs=0.5 m, Tp=6 s</th>
<th>Hs=2.0 m, Tp=7 s</th>
<th>Hs=3.0 m, Tp=8 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>depth-mean current vel. v (m/s)</td>
<td>bed roughness k_s=k_w (m)</td>
<td>suspended sand size, d_i (mm)</td>
<td>depth-mean current vel. v (m/s)</td>
<td>bed roughness k_s=k_w (m)</td>
</tr>
<tr>
<td>0.1</td>
<td>0.1</td>
<td>0.17</td>
<td>0.1</td>
<td>0.02 (flat)</td>
</tr>
<tr>
<td>0.3</td>
<td>0.1</td>
<td>0.17</td>
<td>0.3</td>
<td>0.02 (flat)</td>
</tr>
<tr>
<td>0.5</td>
<td>0.1</td>
<td>0.17</td>
<td>0.5</td>
<td>0.02 (flat)</td>
</tr>
<tr>
<td>0.6</td>
<td>0.1</td>
<td>0.18</td>
<td>0.6</td>
<td>0.02 (flat)</td>
</tr>
<tr>
<td>0.7</td>
<td>0.1</td>
<td>0.19</td>
<td>0.7</td>
<td>0.02 (flat)</td>
</tr>
<tr>
<td>0.8</td>
<td>0.1</td>
<td>0.2</td>
<td>0.8</td>
<td>0.02 (flat)</td>
</tr>
<tr>
<td>1.0</td>
<td>0.1</td>
<td>0.21</td>
<td>1.0</td>
<td>0.02 (flat)</td>
</tr>
<tr>
<td>1.2</td>
<td>0.08</td>
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<tr>
<td>1.5</td>
<td>0.06</td>
<td>0.23</td>
<td>1.5</td>
<td>0.02 (flat)</td>
</tr>
<tr>
<td>1.8</td>
<td>0.03</td>
<td>0.24</td>
<td>1.8</td>
<td>0.02 (flat)</td>
</tr>
<tr>
<td>2.0</td>
<td>0.02 (flat)</td>
<td>0.25</td>
<td>2.0</td>
<td>0.02 (flat)</td>
</tr>
</tbody>
</table>

### 5.1.2 Current-related total load transport in longshore direction

The computed current-related total load transport in the longshore direction is shown in Figure 5.1.1. The depth-averaged current velocity is plotted on the horizontal axis. As can be observed, the transport rate increases for increasing current velocity and increasing wave height. It is noted that the transport rates are approximately equal for Hs= 1 and 2 m in the velocity range 0.3 to 0.8 m/s, which is caused by the relatively large change in bed roughness from k_s= 0.1 m (for Hs= 1 m) to k_s=0.02 m (for Hs=2 m), see Table 5.1.1. The present model is much more sensitive to the bed roughness value than the TRANSPOR1993 model.

Analysis of the results show that the total load transport strongly depends on the current velocity. Roughly, transport≈v^5 for Hs= 0 m; transport≈v^2.5 for Hs= 1 m and transport≈v^2 for Hs= 3 m. Thus, the power of the current velocity decreases for increasing wave height. The total load transport in longshore direction mainly consists of suspended load transport; the bed-load transport in the longshore direction is negligible small for all conditions. Figure 5.1.2 shows the total load transport rates of the TRANSPOR2000 model in comparison to the results of the TRANSPOR1993 model, the BAGNOLD-BAILARD model and the BIJKER model (b=5).

### Field data for currents (no waves)

Field data of suspended transport rates measured in the Eastern and Western Scheldt estuary (h=depth between 4 and 6 m; d_{50} between 0.18 and 0.22 mm; Voogt et al., 1991) have been analysed and are shown in Figure 5.1.2 for the case without waves (H_s= 0 m). The original datasets consist of about 200 individual data points, which have been clustered into data groups. The data within each group have been averaged to obtain one representative group-average value. The group-average values are shown in Figure 5.1.2. The variation range of the velocity within a group is about 10%; the variation ranges of the corresponding suspended transport rates are as large as 50%. Field data of the Nile River (h= 4 to 4.5 m; d_{50}= 0.25 to 0.31 mm; Abdel-Fattah, S., 1997) and the Mississippi River (h= 4.7 to 5.5 m; d_{50}= 0.28 to
0.36 mm; Peterson and Howells, 1973) are also shown in Figure 5.1.2. The latter data represent group-average values based on about 5 to 10 individual data sets.

**Field data for combined current and waves**

Field data of suspended transport rates measured in 1977 and 1978 at the Boscombe Bay site, England (Whitehouse et al., 1997) and at the Egmond site, The Netherlands (Grasmeijer, 2001) have been analysed. Data with water depths between 4.5 and 5.5 m have been selected. The bed material at both sites is sand with \( d_{50} \) of about 0.25 mm. The significant wave heights are in the range of 0.5 to 1 m. The current velocities are in the range of 0.1 to 0.5 m/s. The computed suspended transport rates of the TRANSPOR2000 model show good agreement with measured values, provided that the proper bed-form roughness in the range of 0.01 to 0.03 m is taken. The bed-form roughness values used in Table 5.1.1 appear to be much too large for conditions with combined current and waves. Plots of measured transport rates as a function of current velocity and wave height are shown in Section 3.2.3.

![Figure 5.1.1](image-url)  
*Figure 5.1.1  Total load transport as function of depth-averaged current velocity and significant wave height; TRANSPOR2000 model*
Figure 5.1.2  Total load transport as function of depth-averaged current velocity and significant wave height; TRANSPOR2000, TRANSPOR1993, BAGNOLD-BAILARD model, BIJKER model and measured data
Discussion of results
The TRANSPOR1993 and TRANSPOR2000 models yield about the same transport rates for the case without waves (Hₕ = 0 m). The models are almost the same with exception of the bed-load transport model, which has been modified slightly. TRANSPOR2000 yields bed-load transport rates, which are slightly larger for relatively small current velocities (0.4 to 0.6 m/s) and slightly smaller values for relatively large velocities (1.5 to 2 m/s). The TRANSPOR2000 model yields considerably larger (up to factor 5) total load transport rates for combined current and wave conditions compared to the results of the TRANSPOR1993 model, particularly for non-breaking wave conditions (Hₕ/h<0.4). This is mainly caused by a much stronger effect of the ripples on the effective bed-shear stress and hence on the reference concentration in the TRANSPOR2000 model.

For breaking wave conditions with Hₕ/h≥0.4 the TRANSPOR2000 model produces transport rates, which are about 30% to 50% larger than those of the TRANSPOR1993. This is mainly caused by a much stronger effect of wave breaking (through a wave breaking coefficient) on the sediment mixing coefficient and hence on the suspended concentrations in the TRANSPOR2000 model.

The BAGNOLD-BAILARD model (1981) has also been used to compute the total load transport. The B-B formula reads as:

\[ q_i = 0.5e_r p_r f_{sw} [(\rho_s - \rho)g \tan \varphi]^1 \left| U^2 \right| U \quad \text{and} \quad q_i = 0.5e_r p_r f_{sw} [(\rho_s - \rho)w_c g]^1 \left| U^3 \right| U \]

with \( q_i \) = transport rate (in kg/m/s), \( U \) = instantaneous near-bed velocity, \( f_{sw} \) = grain-friction factor, \( \tan \varphi \) = 0.6, \( w_c \) = fall velocity of suspended sand (based on suspended sand size \( d_s \) from Table 5.1.1).

The grain-friction factor is usually taken as \( k_{s,grain} = 5d_{50} \) and the empirical coefficients are taken as \( e_r = 0.21 \) and \( e_c = 0.025 \) in line with Aagaard et al. (1998) and Thornton et al. (1996). The bed-form roughness is taken as given in Table 5.1.1.

In conditions of a current without waves (Hₕ = 0 m) the B-B formula yields relative large transport rates for current velocities smaller than about 0.5 m/s due to the absence of a threshold value for initiation of sediment motion. For current velocities larger than about 0.5 m/s the B-B formula yields relatively small transport rates (up to factor 10!).

For combined current and wave conditions the transport rates according to the B-B formula are significantly smaller (factor 10 to 15) than those of the TRANSPOR2000 model. The discrepancy increases with increasing wave height, because the wave breaking effect on the suspended transport is not explicitly taken into account by the B-B model. The transport rates of the B-B formula are found to be somewhat larger (30%), if the bed is assumed to be fully flat (\( k_{s,c} = k_{s,w} = 5d_{50} \)) for all conditions in stead of the bed roughness values from Table 5.1.1.

Finally, the results of the BIKKER model (1971) are summarized (b-coefficient= 5):

In conditions of a current without waves (Hₕ=0 m);
- current velocities smaller than about 1.2 m/s: the transport rates according to BIKKER model are almost the same as those of the TRANSPOR2000 model;
- current velocities between 1.2 and 2 m/s: the transport rates of the BIKKER model are smaller than those of the TRANSPOR2000 model (maximum factor 3);
In combined current and wave conditions ($H_z$ between 0.5 and 2 m):

- current velocities between 0.9 and 1.1 m/s: the transport rates according to the BIJKER model are about the same as those of the TRANSPOR2000 model;
- current velocities smaller than 0.9 m/s: the transport rates according to BIJKER are significantly larger (maximum factor 8) than those of the TRANSPOR2000 model;
- current velocities larger than 1.1 m/s: the transport rates according to BIJKER are significantly smaller (maximum factor 3.5) than those of the TRANSPOR2000 model.

In combined current and wave conditions ($H_z$=3 m):

- current velocities up to 0.8 m/s: the transport rates of BIJKER are about the same as those of the TRANSPOR2000 model;
- current velocities between 0.8 and 2 m/s: the transport rates of BIJKER are significantly smaller (maximum factor 5) than those of the TRANSPOR2000 model.

The results of the BIJKER model and the TRANSPOR2000 model show good agreement with measured data for steady currents without waves. It is noted that the model results include the suspended and bed-load transport, while the measured transport rates only include the suspended transport rates (no bed-load transport). The computed suspended transport rates of the TRANSPOR2000 model for conditions with combined current and waves show good agreement with field data (British and Dutch sites), provided that the proper bed-form roughness in the range of 0.01 to 0.03 m is taken. The bed-form roughness values used in Table 5.1.1 appear to be much too large for conditions with combined current and waves.

5.1.3 Wave-related total load transport in cross-shore direction

The computed wave-related total load transport in the cross-shore direction is shown in Figures 5.1.3, 5.1.4 and 5.1.5. The wave-related transport is defined as the bed-load and suspended load transport due to the asymmetry of the orbital velocities near the bed. Figure 5.1.3 shows the wave-related transport as a function of depth-averaged current for three wave heights $H_z$= 0.5, 1, 2 and 3 m. The wave-related transport according to the TRANSPOR2000 model is only weakly dependent on longshore current velocity for values smaller than 0.5 m/s. For larger current velocities there is a significant increase of the transport rate with increasing current velocity due to the increase of the suspended sand concentrations and hence the oscillating suspended transport. The percentage of bed-load transport to the total transport in cross-shore direction is given in Table 5.1.2 for some cases.

<table>
<thead>
<tr>
<th></th>
<th>$H_s$=1 m</th>
<th></th>
<th></th>
<th>$H_s$=3 m</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$u=0.1$ m/s</td>
<td>30%</td>
<td>25%</td>
<td>5%</td>
<td>30%</td>
<td>25%</td>
<td>5%</td>
</tr>
<tr>
<td>$0.5$ m/s</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$2$ m/s</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$u=0.1$ m/s</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$0.5$ m/s</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$2$ m/s</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5.1.2 Percentages of bed-load transport

The results of the B-B model are also shown. The wave-related transport according to the B-B model is almost independent of longshore current velocity. The results of the TRANSPOR2000 model and the B-B model show reasonably good agreement for current velocities smaller than about 0.5 m/s (relatively small suspended sand concentrations).
Figure 5.1.3  Wave-related total load transport as function of longshore current velocity and wave height; TRANSPOR2000 model and BAGNOLD-BAILARD model

Figure 5.1.4  Wave-related total load transport as function of longshore current velocity and wave height; TRANSPOR2000 model
Figure 5.1.5  Wave-related transport as function of wave height and longshore current velocity: TRANSPOR2000 model and B-B model
Figure 5.1.4 shows the wave-related transport as a function of the significant wave height for six values of the longshore current velocity between 0.3 and 2 m/s. It shows that the results of the TRANSPOR2000 model are almost independent of the longshore current for values <0.5 m/s.

Figure 5.1.5 shows the wave-related transport according to the TRANSPOR2000 model and the B-B model as a function of the significant wave height for four values of the longshore current velocity between 0.3 and 2 m/s. There is reasonably good agreement for relatively small velocities. For larger current velocities the B-B model yields wave-related transport rates, which are significantly smaller.

5.2 Sand transport along cross-shore bottom profile

5.2.1 Introduction

The TRANSPOR2000 model (implemented in the CROSMOR2000 model) has been used to compute the sand transport distribution and morphology along the cross-shore profile of various field sites. Two experiments performed in the large-scale wave tanks are also considered.

First, a short description of the CROSMOR2000 model is given in Section 5.2.2. Next, the simulations for the LIP-1B and LIP-1C experiments in the large-scale Delta flume of Delft Hydraulics are described in Sections 5.2.3 and 5.2.4, respectively. The simulations for the Duck field site (USA) and the Katwijk field site (The Netherlands) are described in Sections 5.2.5 and 5.2.6.

5.2.2 Description of cross-shore model CROSMOR2000


The propagation and transformation of individual waves (wave by wave approach) is described by a probabilistic model (Van Rijn and Wijnberg, 1994, 1996) solving the wave energy equation for each individual wave. The individual waves shoal until an empirical criterion for breaking is satisfied. Wave height decay after breaking is modelled by using an energy dissipation method. Wave-induced set-up and set-down and breaking-associated longshore currents are also modelled. The standard wave breaking coefficient is represented as a function of local wave steepness and bottom slope. Laboratory and field data have been used to calibrate and to verify the model. Generally, the measured $H_{1/3}$-wave heights are reasonably well represented by the model in all zones from deep water to the shallow surf zone. The fraction of breaking waves is reasonably well represented by the model in the upscaling zones of the bottom profile. Verification of the model results with respect to wave-induced longshore current velocities based on other datasets was not extensive, because of lack of data. In case of a barred profile the measured longshore velocities showed a relatively uniform distribution in the (trough) zone between the bar crest and the shoreline, which could to some extent be modelled by including the space-averaged radiation force gradient, the horizontal mixing and the longshore water surface gradients related to variations in set-up. In case of a monotonically upsloping profile the
cross-shore distribution of the longshore current velocities is reasonably well represented (Van Rijn and Wijnberg, 1996). A definition sketch of wave and current directions is given in Figure 5.2.1.

Figure 5.2.1  Definition sketch

\[\text{Teta} = \text{angle between wave direction and positive y-axis}\]
\[\text{Phi} = \text{angle between wave and current direction}\]
\[\text{Alfa} = \text{angle between longshore current direction and positive y-axis} (v_{long} < 0 \text{ m/s}; \text{alфа} = 270^\circ \text{ and } v_{long} > 0 \text{ m/s}; \text{alфа} = 90^\circ)\]

Updated hydrodynamics
The CROMSOR2000-model has been applied with updated submodels for hydrodynamics and sand transport. The modifications are summarized in Table 5.2.1. The update of the submodels is related to:

- the description of the wave orbital velocity using the modified Isobe-Horikawa method;
- the modification of the wave breaking coefficient;
- the modification of the Longuet Higgins (Eulerian) streaming in the wave boundary layer;
- the effect of bed roughness and wave breaking on sediment mixing (diffusivity);
- the effect of bed ripples in the lower regime on near-bed sand concentrations.

Wave orbital velocity
Kroon (1994) and Wolf (1997) have shown that the available theories for non-linear wave motion are not accurate in describing the asymmetry of the near-bed orbital velocity. Generally, the wave velocity asymmetry in the surf zone was considerably overestimated by these theories. So far, linear wave theory with an empirical correction factor (derived from velocity data obtained at the Egmond beach site, The Netherlands, Kroon, 1994) to deal with non-linear effects was used in the CROMSOR model. The instantaneous velocities during the forward and the backward phase of the cycle are assumed to have a sinusoidal distribution. The duration period of each phase is corrected to obtain zero net flow over the full cycle (\(T_{\text{for}} + T_{\text{back}} = T\)).
Grasmeijer and van Rijn (1998) have used the semi-empirical method of Isobe and Horikawa (1982) with modified coefficients (based on data fitting using field data series from various coastal sites). This method (see Appendix C) has been implemented as the standard model for computation of the wave velocity asymmetry.

**Wave breaking coefficient**
The standard wave breaking model is based on breaking coefficients, which are a function of relative wave steepness (see Table 5.2.1). Experience so far shows that the computed wave heights of the CROSMOR model generally are somewhat too large (about 10%) for storm waves; therefore the standard wave breaking coefficients ($\theta = H_b/h$) have been reduced by about 10%.

**Longuet Higgins streaming**
The onshore-directed streaming ($U_b$) in the wave boundary layer due to viscous and turbulent diffusion was so far modelled quite simply by assuming: $U_b = 0.05$ $U_6$ with $U_6 = \text{peak value of near-bed orbital velocity based on linear wave theory}$.
The expression given by Longuet-Higgins (1953) for Eulerian streaming in the wave boundary due to viscous diffusion reads as: $U_b = \frac{\alpha}{2} U^2/C$ with $U$= near-bed peak orbital velocity and $C=L/T=\text{phase velocity}$. This expression has been implemented; the peak velocity of the near-bed orbital velocity is taken as the average of the peak forward and peak backward velocity based on the Isobe-Horikawa method. For strongly breaking storm waves the LH-streaming is slightly reduced.

**Undertow**
The depth-averaged return current ($u_r$) under the wave trough of each individual wave (summation over wave classes) is derived from linear mass transport and the water depth ($h_r$) under the trough. The mass transport is given by $0.125 \, g \, H^2/c$ with $c= (g \, h)^{0.5}$ = phase velocity in shallow water.
This yields: $u_r = - \alpha \, g^{0.5} \, H^{2/3} \, h_r$ with $\alpha = 0.125$, $h_r = (0.95 - 0.35(H/h))$ and $h = \text{depth under trough}$, based on the analysis of field data (Kroon, 1994). This approach implies a local response of the return current to the wave energy, which may not be a good representation of the physics involved. The return current is driven by a seaward-directed pressure gradient generated by the radiation stress-induced set-up of the water surface, which may lead to a delayed response of the return current. The non-local response in cross-shore direction is modelled by averaging the wave height and water depth over a short distance (equal to the wave length) seaward of the location x.
The contribution of the rollers of broken waves to the mass transport and to the generation of longshore currents (Svendsen, 1984; Dally and Osiecki, 1994) is neglected. According to Masselink and Black (1995) and to Kroon (1994), the linear mass transport approach gives reasonably good results compared to field data.
Experience so far shows that the undertow current generally is somewhat too small for strongly breaking waves above bars and slightly too large for low steepness breaking waves. Correction factors related to relative wave height (H/h) and wave steepness (H/L) have been used to account for these effects.
The vertical distribution of the undertow model was modelled as a logarithmic profile in the CROSMOR1997 model. In the updated CROSMOR2000 model the vertical distribution has been modified; the new velocity profile is schematized into three layers with a
logarithmic distribution in the lower two layers and a third power distribution in the upper layer, yielding velocities which approach to zero at the water surface (see Appendix D).

**Cross-shore tidal velocity**

The cross-shore depth-averaged current velocity due to the rise and fall of the tide is taken into account by an expression based on continuity, assuming that the water surface is horizontal over a relatively short distance.

**Sand transport**

The CROSMOR2000 model is based on the TRANSPOR2000 model for sand transport. The effect of the local cross-shore bed slope on the transport rate is taken into account by (see Van Rijn, 1993):

- multiplying the critical bed-shear stress with the Schoklitsch-factor $k_1 = \sin (\phi + \beta)/\sin(\phi)$, 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 $k_1 > 1$ 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 $k_2 = (1/(1+\tan \beta/\tan \phi))$; $\tan \beta$ is positive for uphill transport yielding $k_2 < 1$ 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 was also used, but it did 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. Unstable morphological results were obtained, when the instantaneous approach was used. Therefore, the Bagnold-factor was pragmatically applied to the net time-averaged transport rates.

The effect of bed roughness and wave breaking on the sediment mixing coefficient was not included in the TRANSPOR1993 model. Furthermore, the effect of bed ripples on near-bed reference concentration was not included. The TRANSPOR1993 model was used in the CROSMOR1997 model.
<table>
<thead>
<tr>
<th>Process/parameter</th>
<th>CROSMOR1997</th>
<th>CROSMOR2000 (updated)</th>
</tr>
</thead>
</table>
| Wave orbital velocity     | based on linear wave theory with empirical coefficient to account for wave velocity asymmetry.  
U_{\alpha}= \alpha \ U_{\alpha,\text{linear}} and  
U_{\beta}= (2 - \alpha) \ U_{\alpha,\text{linear}}  
with \( \alpha = 1 + 0.3 \) (H/h),  
H = wave height, h = water depth. | based on method of Isobe and Horikawa (1982), as modified by Grasmeijer and van Rijn (1998).                                                                 |
| at edge of wave boundary  |                                                                                                                                                |                                                                                                                                                        |
| layer                     |                                                                                                                                                |                                                                                                                                                        |
| Wave breaking coefficient | \( \gamma = 0.45 \) for tan\( \beta/tan\alpha=0 \)  
\( \gamma = 0.55 \) for tan\( \beta/tan\alpha=0.25 \)  
\( \gamma = 0.70 \) for tan\( \beta/tan\alpha=1 \)  
\( \gamma = 0.80 \) for tan\( \beta/tan\alpha=3 \)  
\( \gamma = 1.0 \) for tan\( \beta/tan\alpha=10 \)  
with tan\( \alpha = H/L \) (wave steepness)  
ten\( \beta \) = local bottom slope  
(linear interpolation for other values) | \( \gamma = 0.4 \) for tan\( \beta/tan\alpha=0 \)  
\( \gamma = 0.5 \) for tan\( \beta/tan\alpha=0.25 \)  
\( \gamma = 0.6 \) for tan\( \beta/tan\alpha=1 \)  
\( \gamma = 0.7 \) for tan\( \beta/tan\alpha=3 \)  
\( \gamma = 0.8 \) for tan\( \beta/tan\alpha=10 \)  
with tan\( \alpha = H/L \)  
ten\( \beta \) = local bottom slope |
| Longuet Higgins streaming | \( U_h=0.05 \ U_{\alpha,\text{linear}} \)                                                                                                       | \( U_h=0.75 \ (0.5 U_{\alpha,\text{r}} + 0.5 U_{\alpha,\text{h}})^2/C \) with C = phase velocity.  
LH streaming is multiplied by reduction factor for strongly breaking waves. |
| Undertow                  | \( u=0.125 g^{0.5} h^{0.5} (h-h_0) \) with \( u= \) depth-av. undertow below wave trough, \( h= \) water depth, \( h_0= \) depth below wave trough | Undertow is multiplied by enhancement factor for strongly breaking waves (H/h>0.5).  
Undertow is multiplied by reduction factor for low steepness breaking waves.  
New vertical profile (see App. D) |
| Cross-shore tidal current | not taken into account                                                                                                                          | \( u_{\text{tide}}=(x)-(h_t-h_{\alpha})/(h \Delta t) \) with \( x= \) length to boundary, \( h_t= \) tidal level at time \( t \),  
\( h_{\alpha}= \) tidal level at previous time  
\( \Delta t= \) time step  
h= water depth |
| Sand transport            | Effect of bed roughness and breaking waves on the sediment mixing coefficient is not included.  
Effect of bed ripples on near-bed reference concentration is not included. | The sediment mixing near the bed is related to the bed roughness.  
The sediment mixing distribution is related to relative wave height of breaking waves.  
Near-bed reference concentration is related to relative wave height of low waves. |

**Table 5.2.1** Modifications of updated CROSMOR2000 model


**Bed level changes and bed material composition**

Bed level changes per fraction i are described by:

\[ \rho_i (1-e) \frac{\partial z_{b,i}}{\partial t} + \frac{\partial (p_i q_{i,j})}{\partial x} = 0 \]  

(5.2.1)

with: \( z_b \)= bed level to datum, \( q_{i,j} = q_{b,i} + q_{s,i} \)= volumetric total load (bed load plus suspended load) transport per fraction i, \( p_i \)= value of fraction i, \( \rho_s \)= sediment density, \( e \)= porosity factor.

In discrete notation:

\[ \Delta z_{b,i,x,t} = [(p_i q_{i,j})_{x,\Delta x} - (p_i q_{i,j})_{x+1,\Delta x}] \Delta t / (2 \Delta x (1-e) \rho_s) \]  

(5.2.2)

with: \( \Delta t \)= time step, \( \Delta x \)= space step, \( \Delta z_{b,i,x,t} \)= bed level change at time t (positive for decreasing transport in positive x-direction, yielding deposition).

The total bed level change is obtained by summation of fractional bed level changes over all N-fractions:

\[ \Delta z_{b,x,t} = \Sigma \Delta z_{b,i,x,t} \]  

(5.2.3)

The new bed level at time t is obtained by applying an explicit Lax-Wendroff scheme, as follows:

\[ z_{b,i,x,t} = z_{b,i,x,t-\Delta t} + \gamma_b \left[ \frac{1}{2} (z_{b,x-\Delta x,t-\Delta t} + z_{b,x+\Delta x,t-\Delta t}) - z_{b,i,x,t-\Delta t} \right] \]  

(5.2.4)

with: \( \gamma_b \)= smoothing factor (between 0.001 and 0.5).

The bed material is computed in a thin (order of 0.1 m) surface mixing layer of thickness \( \delta \) applying a one-layer approach. The thickness of the surface layer is assumed to be constant in space and time and is moving in vertical direction with the bed surface in response to bed level changes (deposition upwards and erosion downwards). Thus, the surface layer is always at the top of the bed. The mixing of sediment within the surface layer is assumed to be effectuated within each time step (instantaneous mixing) by small-scale bed-form migration processes in the lower regime or by wave-induced vortices in the sheet flow regime. Conceptually, the mixing layer (active layer) represents the layer that can be reworked (sorted through) in the time step applied.

At present stage of research the bed material composition of the subsoil below the surface layer is assumed to be uniform (no layered structure) and equal to the initially specified fraction values \( p_{b,i,j} \).

This approach may represent tendencies for redistribution of sediment fractions along the cross-shore profile in a slowly varying morphology of rather uniform bed material composition. In a very dynamic morphologic environment with alternating erosional and depositional events the present approach is not realistic and book-keeping of the sediment fractions per layer is necessary (future research).
Hirano (1971) was one of the first to apply a transport relationship per size fraction in a mathematical model for simultaneous computation of bed evolution and sediment composition in rivers based on a one-layer approach. Bennett and Nordin (1977) used a two-layer approach with an active top layer and an inactive sublayer for the computation of bed evolution in rivers. When deposition or erosion of a certain thickness of a particular fraction occurs, this material is added or removed from the active layer and exchanged with the inactive sublayer and with the subsoil beneath the inactive layer. Ribberink (1987) also distinguished a sublayer (exchange layer), which is affected occasionally by deep bed-form troughs in rivers. The sediment flux per fraction between the active top layer and the inactive sublayer is related to the local bed-load transport rate, the bed-form length and the difference of size fraction i in the top and sublayer.  

To the author’s knowledge, the one-layer approach has not yet been used in cross-shore modelling of coastal profiles.

The bed material composition in the CROSMOR-model is computed according to the following procedure:

- the sediment mass \( M \) of the surface layer at \( t=0 \) is subdivided in masses \( M_{i,j} \) based on the initial fraction values \( p_{0,i,j} \), as follows: \( M_{i,j} = p_{0,i,j} M \), with \( \Sigma M_{i,j} = \Sigma (p_{0,i,j}) = M \), as \( \Sigma (p_{0,i,j}) = 1 \) and \( M= \) constant in space and time;

- the sediment mass \( M_{i,j} \) of fraction \( i \) changes due to sediment deposition or erosion at the surface of the bed; in case of deposition the mixing layer will move upward at a rate equal to the deposition rate, while an equal amount of sediment with the composition of the mixing layer will be lost at the bottom (exchange at base) of the mixing layer; in case of erosion the opposite process will take place and the mixing layer will move downward eroding itself into the subsoil, hence sediment with the composition of the subsoil will be absorbed by the mixing layer (see Figure 5.2.2);

in formula notation:

\[
M_{i,j,t+\Delta t} = M_{i,j,t} + \Delta M_{i,j,t} = (\Delta M_{i,j,t})p_{i,j,t}
\]  
(5.2.5)

with: \( \Delta M_{i,j,t} = (1-e) p_{e} \Delta z_{h,i,j,t} \) and \( \Delta M_{i,j} = (1-e) p_{e} \Delta z_{b,i,j,t} \);

Deposition (supply at top and loss at base): \( p_{e,i,e} = \) fraction value of mixing layer at time \( t \);  
Erosion (loss at top and supply from subsoil): \( p_{e,i,b} = \) initial (\( t=0 \)) fraction value.

The last term of Eq (5.2.5) represents the exchange of sediment at the base of the layer. Summation over all fractions yields:

\[
\Sigma M_{i,j,t+\Delta t} = \Sigma M_{i,j,t} + \Sigma (\Delta M_{i,j,t}) - \Sigma (\Delta M_{i,j,t} P_{i,j,t}) \text{ or,}
\]

\[
\Sigma M_{i,j,t+\Delta t} = M + \Delta M_{i,j} - \Delta M_{i,j,1} = M;
\]

thus, the mass of the surface layer remains constant.

- the new composition of the bed material of the mixing layer is given by:

\[
p_{i,j,t+\Delta t} = M_{i,j,t+\Delta t}/M
\]  
(5.2.6)
the amount of sediment of a particular size fraction to be eroded can not be larger than the amount of material of that fraction present in the active layer (availability-limited transport); the maximum fractional transport rate per time step and per unit width is $M_i/(b \Delta t)$.

Thus, if $p_{i-1} > M_i/(b \Delta t)$; then $p_{i-1} = M_i/(b \Delta t)$;

in the present version of the model a different approach is used; if due to erosion $M_{i,i+1} < 0$, then $M_{i,i+1} = 0$ and the deficit is subtracted proportionally from the remaining positive $M_i$-values, so that $M$ remains constant and $p_{i,i+1} = 1$ is recomputed ($\Sigma M_i = M$ and $\Sigma p_i = 1$ should be satisfied); after each time step the first N-1 fractions are slightly smoothed along the profile for reasons of stability and the last fraction is recomputed (satisfying $\Sigma p_i = 1$ everywhere). Analysis of computed fractional values did not show availability-limited transport conditions in the test cases considered herein (time step $\Delta t$ sufficiently small).

Finally, it is noted that there is no conservation of sediment per size fraction, because there is no book-keeping of sediment per fraction leaving or entering at the base of the mixing layer. The overall mass balance is preserved through Eqs. (5.2.1) and (5.2.3). Armouring of the bed is not taken into account. Horizontal fining due to abrasion effects (mechanical wearing) is not taken into account.

![Figure 5.2.2](Exchange of sand at base of mixing layer)

### 5.2.3 LIP-1B experiment, Delta Flume, Delft Hydraulics

**Test conditions**

Within the framework of the European Large Installations Plan (LIP) a programme of detailed measurements of hydrodynamics, sand transport and morphology along a sloping cross-shore profile has been carried out in the large-scale Deltaflume of Delft Hydraulics (Arcilla et al, 1994; Roelvink and Reniers, 1995).
Two test series were carried out:
- irregular waves over cross-shore profile without dune at the shore (Test 1A, 1B and 1C);
- irregular waves over cross-shore profile with dune at shore (Test 2A, 2B, 2E and 2C).

Herein, only Test series 1B and 1C are considered. The water level was constant in these tests (4.1 m above the concrete flume bottom). The initial profile of Test 1B was the end profile of Test 1A and so on. Test 1B represents erosive short-period storm waves; Test 1C represents accretive long-period fairweather waves.

Measured parameters are:
- wave height from pressure sensors attached to the side wall of the flume (at 10 locations);
- velocity sensors at 5 elevations above the bed from measurement carriage on top the flume (used at various positions along the flume);
- bed level soundings at regular time intervals during the tests (wave generation was stopped);
- total net sand transport derived (by integration along profile) from bed-level soundings at different time intervals.
Wave, sediment and profile parameters

The boundary conditions and input data are given in Tables 5.2.2 and 5.2.3.

<table>
<thead>
<tr>
<th>BOUNDARY CONDITIONS AND INPUT DATA</th>
<th>LIP1B</th>
<th>LIP1C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height ( H_{m0} ) at ( x=0 ) (m)</td>
<td>1.4</td>
<td>0.6</td>
</tr>
<tr>
<td>Peak period ( T_p ) (s)</td>
<td>5</td>
<td>8</td>
</tr>
<tr>
<td>Water depth at ( x=0 ) (m)</td>
<td>4.1</td>
<td>4.1</td>
</tr>
<tr>
<td>Duration of test (hrs)</td>
<td>18</td>
<td>12</td>
</tr>
<tr>
<td>Number wave classes</td>
<td>9</td>
<td>6</td>
</tr>
<tr>
<td>Number of sand fractions</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Sand size, ( d_{50} ) and ( d_{90} ) (m)</td>
<td>0.0002 and 0.0004</td>
<td>0.0002 and 0.0004</td>
</tr>
<tr>
<td>Fall velocity of bed material (m/s)</td>
<td>0.025</td>
<td>0.025</td>
</tr>
<tr>
<td>Factor for high freq. susp. sand transport (-)</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>Temperature (C)</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Salinity (promille)</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Porosity (-)</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>Bed roughness ( k_{w} ) (m)</td>
<td>0.01 (x&lt;135, x&gt;160)</td>
<td>0.01 (x&lt;138, x&gt;160)</td>
</tr>
<tr>
<td>Bed roughness ( k_{e} ) (m)</td>
<td>same</td>
<td>same</td>
</tr>
<tr>
<td>Space step (m)</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Minimum depth (m)</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>Time step (s)</td>
<td>2400</td>
<td>3600</td>
</tr>
</tbody>
</table>

Table 5.2.2  Boundary conditions and input data

The wave spectrum measured at the entrance of the flume is schematized in classes according to Table 5.2.3.

<table>
<thead>
<tr>
<th>( H ) (m)</th>
<th>( T ) (s)</th>
<th>( p ) (%)</th>
<th>( H ) (m)</th>
<th>( T ) (s)</th>
<th>( p ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3</td>
<td>2</td>
<td>14</td>
<td>0.12</td>
<td>4</td>
<td>23</td>
</tr>
<tr>
<td>0.55</td>
<td>3</td>
<td>13</td>
<td>0.27</td>
<td>6</td>
<td>25</td>
</tr>
<tr>
<td>0.7</td>
<td>3.7</td>
<td>13</td>
<td>0.39</td>
<td>7</td>
<td>20</td>
</tr>
<tr>
<td>0.85</td>
<td>4</td>
<td>13</td>
<td>0.51</td>
<td>8</td>
<td>18</td>
</tr>
<tr>
<td>1.0</td>
<td>4</td>
<td>14</td>
<td>0.63</td>
<td>8</td>
<td>12</td>
</tr>
<tr>
<td>1.15</td>
<td>4</td>
<td>15</td>
<td>0.78</td>
<td>8</td>
<td>2</td>
</tr>
<tr>
<td>1.3</td>
<td>4</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>1.5</td>
<td>5</td>
<td>6</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>1.9</td>
<td>5</td>
<td>4</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

Table 5.2.3  Schematization of measured wave spectrum
Results

Computed and measured results are given in Figures 5.2.3 and 5.2.4. The results are briefly described below.

Wave height
- computed significant wave heights ($H_{1/3}$) are within 5% to 10% of measured values ($H_{m0}$) in all stations.

Undertow
- computed values (depth-integrated velocity below wave trough level) show good agreement with measured values (error bars indicate variations of velocity over depth);
- computed and measured undertow velocities are maximum above the developing breaker bar around $x=135$ m.

Peak onshore and offshore high freq. orbital velocities near bed ($U_{1/3,\text{on}}$ and $U_{1/3,\text{off}}$)
- computed peak onshore orbital velocity (modified Isobe-method, see Appendix C) is about 10% too large compared to measured values, especially landward of the bar ($x=135$ m) where the waves are broken;
- computed peak offshore orbital velocity shows good agreement with measured values.

Sand transport
- measured sand transport has been derived by integration of the bed levels at $t=0$ and $t=18$ hours (Fig. 5.2.3Bottom) and thus represents the time-averaged value over the considered time interval;
- computed sand transport rates (including oscillatory high-frequency suspended transport) at $t=0$ and $t=18$ hours based on variable bed roughness are shown (Fig. 5.2.3Bottom); the computed transport is onshore-directed just seaward of the bar and offshore-directed in the bar crest-trough zone and again onshore-directed just landward of the trough zone; the computed transport rates are of the right magnitude, but the computed transport rate distribution at the bar crest is too peaked and somewhat shifted in seaward direction at the end of the test ($t=18$ hours); the effect of the oscillatory high- frequency suspended transport is shown in Figure 5.2.4Top; this effect yields onshore transport rates of about 0.02 to 0.04 kg/s/m between $x=50$ m and 140 m and should be included to obtain realistic transport rates in agreement with measured values; the gradients of the cross-shore transport rate are not much affected by the oscillatory suspended transport;
- computed sand transport rates are significantly affected by the bed roughness; the sand rates based on variable bed roughness of $k_r=0.01$ to 0.03 m (see Table 5.2.2) and a constant value of $k_r=0.01$ m are also shown in Figure 5.2.4Top; the constant value of $k_r=0.01$ m results in a considerable reduction of the sand transport rates between $x=135$ m and 145 m; the relatively large bed roughness in the trough landward of the bar crest can be explained by the presence of pronounced bed forms at that location;
- computed transport according to the Bagnold formula at $t=0$ hrs is also shown in Fig. 5.2.4Top; the sand transport is onshore-directed at all locations and differs significantly from measured values.

Bed level evolution
• bar development is reasonably well simulated, if the effective bed roughness is varied along the profile (larger bed roughness of 0.03 m in the bar trough zone between x= 142 and 155 m, roughness of 0.01 m outside this zone); the computed trough erosion is somewhat larger and is shifted somewhat more seaward compared to measured trough erosion;
• simulated bar development is not sufficient for constant bed roughness of 0.01 m (see Fig 5.2.3Top);
• simulated bar development is slightly more seaward, if oscillatory high-frequency suspended transport is neglected (Fig. 5.2.3Bottom); the effect of the oscillatory suspended transport on the short term bed evolution is not very substantial, because the transport gradients are not very much affected by this transport component; the effect on long-term bed evolution will however be more significant because the absolute value of the incoming transport (at x=0) is modified by including the oscillatory suspended transport (additional onshore transport component);
• application of the Bagnold sand transport model and variable bed roughness does not result in sufficient bar development, because this model does not produce offshore-directed sand transport.
Figure 5.2.3  Computed and measured results of LIP1B-test
Top: Bed level evolution
Middle 1: Wave height and undertow along profile
Middle 2: Peak onshore and offshore orbital velocities (near bed)
Bottom: Sand transport along profile
**Figure 3.2.4** Computed and measured results of LIP1B-test

*Top:* Sand transport

*Middle:* Bed level evolution

*Bottom:* Bed level evolution
5.2.4 LIP-1C experiment, Delta Flume, Delft Hydraulics

Test conditions
The test conditions are described in Section 5.2.3.

Results
Computed and measured results are given in Figures 5.2.5 and 5.2.6.
The results are briefly described below.

Wave height
- computed wave heights (H_{1/3}) are somewhat too large in the shoaling zone (5%).

Undertow
- computed values (depth-integrated velocity below wave trough level) show good
  agreement with measured values just landward of the bar crest; computed values are too
  small in the trough zone; computed values are too large (factor 2) seaward of the bar.

Peak onshore and offshore high freq. orbital velocities near bed (U_{1/3,on} and U_{1/3,off})
- computed peak onshore orbital velocities (modified Isobe-method, see Appendix C) are
  about 10% to 15% too large compared to measured values, especially landward of the bar
  (x=145 m) where the waves are broken;
- computed peak offshore orbital velocity are considerably too large (30%) at the crest.

Sand transport
- measured sand transport has been derived by integration of the bed levels at t=0 and t=13
  hours and thus represents the time-averaged value over 13 hours;
- computed sand transport rates at t= 0 and t= 13 hours (based \( k_c = 0.01-0.05 \) m) are
  shown; the onshore-directed transport rates are quite well represented seaward of and at
  the bar crest; the computed sand transport just landward of the bar crest is onshore
  directed, whereas the measured transport is slightly offshore directed; the sand transport
  is dominated by bed-load transport except in the trough zone (see Fig. 5.2.6Bottom); the
  effect of the oscillatory high-frequency suspended transport is shown in Figure 5.2.6Top;
  this effect yields a maximum difference of about 0.02 to 0.03 kg/s/m (onshore directed) at
  about x= 140 m and should be included to obtain more realistic transport rates at the bar
  crest;
- the transport according to the Bagnold formula is also shown in Fig. 5.2.6Top; the
  transport rates are much too large.

Bed level evolution
- bar growth can only be computed by introducing a transition of bed roughness from \( k_{c,w} =
  0.01 \) to 0.05 m over a distance of about 1 m to simulate the generation of ripples
  landward of the bar crest;
- computed bar development shows reasonably good agreement with measured bar
  development; especially when the high-frequency suspended transport is neglected.
Figure 5.2.5  Computed and measured results of LIP1C-test
Top:    Bed level evolution
Middle 1: Wave height and undertow along profile
Middle 2: Peak onshore and offshore orbital velocities (near bed)
Bottom:  Sand transport along profile
**Figure 5.2.6**  Computed and measured results of LIP1C-test

*Top:* Sand transport along profile

*Bottom:* Bed level evolution
5.2.5 Duck beach 1994, USA

Experimental conditions
The present data were obtained during the Duck94 field experiment conducted near Duck, North Carolina (USA) on a barrier island exposed to the Atlantic Ocean (Gallagher et al., 1998). The data period covered is 21 September to 20 October 1994. Bathymetry plots at 21 Sep., 4 Oct., 10 Oct., 14 Oct. and 20 Oct. are shown in Figures 5.2.7 to 5.2.11.

![Bathymetry plot near Duck, NC, USA, 21 September 1994](image)

**Figure 5.2.7** Bathymetry plot near Duck, NC, USA, 21 September 1994

![Bathymetry plot near Duck, NC, USA, 4 October 1994](image)

**Figure 5.2.8** Bathymetry plot near Duck, NC, USA, 4 October 1994
Figure 5.2.9  Bathymetry plot near Duck, NC, USA, 10 October 1994

Figure 5.2.10  Bathymetry plot near Duck, NC, USA, 14 October 1994

Figure 5.2.11  Bathymetry plot near Duck, NC, USA, 20 October 1994
The presence of a breaker bar can be observed around the cross-shore position of 200 m from the reference line. The bathymetry characteristics can be described as:

- 21 September: minor storm on 21 Sept., $H_{\text{offshore}} = 2.5$ m (maximum); the bar has a reasonably straight alignment between the main transect 945 m and the transect 1200 m; an oblique (crenicient) pattern can be observed between transects 700 m and 945 m;
- 21 September - 4 October: significant wave height $H_{\text{offshore}}$ between 1 and 1.5 m; the bar has moved offshore (about 25 m) between transects 700 and 800 m; the bar has moved onshore slightly between transects 875 and 975 m; bar alignment is almost straight;
- 4 October - 10 October: $H_{\text{offshore}}$ is maximum 1 m; bar movement is minor;
- 10 October - 14 October: $H_{\text{offshore}}$ between 1 and 1.5 m; bar has moved offshore (about 30 to 40 m); bar alignment is almost straight;
- 14 October - 16 October: $H_{\text{offshore}}$ is between 2 and 3 m (storm period);
  - 16 October - 20 October: $H_{\text{offshore}}$ is between 1.5 and 2 m; bar has moved offshore (about 50 m) between transects 875 and 975 m; bar has moved onshore near transects 700-850 m; bar has moved onshore near transects 1050-1200 m; bar alignment is crenicient.

The periods considered for modelling are: September 21-26 and October 10-20, 1994. Both hydrodynamic and morphodynamic runs have been made using the CROSMOR2000 model.

During the period September 21-26, the bar crest around the main transect at 945 m moved onshore over a distance of about 15 m. Longshore variations of the bar are minor over a distance of 50 m on both sides of the main transect 945 m. The plan view of the crest lines is given in Figure 5.2.12. The significant wave height at a depth of about 5 m varied between 0.5 and 2.5 m; a minor storm with a duration of about 12 hours occurred on September 21; most of the time (5 days) the wave height varied between 0.5 and 1 m (see Figure 5.2.22).

During the period October 10-20, the bar crest around the main transect at 945 m moved about 75 m in offshore direction. Longshore variations of the bar are minor over a distance of about 50 m on both sides of the main transect. However, a 3-dimensional bar feature can be observed on a larger scale (about 500 m); the bar crest has moved in offshore direction between the transects 850 and 1050 m due to the presence of a relatively strong offshore-directed current (rip current, see Figures 5.2.11 and 5.2.12). Beyond these transects the available data indicate that the bar crest remained more or less at the same location during the storm event of 10 to 20 Oct. 1994. The significant wave height in water depth of about 5 m varied between 0.5 and 3 m; storm conditions with wave heights between 2 and 3 m were present during 2.5 days between 14 Oct. to 17 Oct. Offshore migration occurred primarily during high-energy wave conditions. Slight onshore bar movement occurred during conditions with low-energy waves.

A basic question with respect to the application of a profile model for short-term modelling (on the event scale of days to weeks) is whether such a model can be applied to an individual transect, because longshore variability generally is so large that bed level changes of individual transects over short periods may not be significantly different in statistical sense.
The effects of longshore variability can be eliminated by longshore averaging of individual transects resulting in a longshore-averaged profile and a variation band (based on standard error: $\sigma/\sqrt{n}$ ). Bed level changes at two different data are only significant if there is no overlap of the error bands. The longshore averaging distance should be so large that the longshore rhythmicity including rip channels is fully covered. Figure 5.3.13 shows longshore-averaged transects (between transects 800 and 1150) including the error band for three periods: 21 Sept-4 Oct, 10-14 Oct. and 14-20 Oct. As can be observed the bed level changes for the period 10-14 Oct. and 14-20 Oct. are significant in the sense that the net bed level changes are much larger than the error bands due to longshore variability. The net bed level changes for the period 21 Sept. - 4 Oct. (calm weather period) are just statistically significant. The bar crest shows net accretion and the trough shows net erosion; these changes are just larger than the thickness of the error bands.

**Figure 5.2.12** Plan view of bar migration based on Grab-soundings; bar crest and trough lines are shown for October and September events

The hydrodynamic conditions can be derived from the available field measurements carried out in the main transect at 945 m.

Co-located pressure sensors and bi-directional electromagnetic current meters were deployed on fixed frames at 13 locations on a 255 m long cross-shore transect close to longshore main transect 945 m (see Figure 5.2.12). Co-located sonar altimeters were deployed at 9 locations. Current meters had an elevation between 0.4 and 1 m above the seafloor. The wave heights and currents are given as time-averaged values over 3 hours. Tide levels were derived from the water depth variations measured at offshore locations (between 480 and 887 m from reference line, see Table 5.2.4), assuming that bed level variations were negligible. The
maximum tidal variation was about 1.1 m during the September event and about 1.5 m during the October event. The mean depth was obtained by averaging over time, yielding $h_{\text{mean}} = 5$ m for station 480 m and $h_{\text{mean}} = 8.1$ m for station 887 m. The tide level was obtained as $h_t - h_{\text{mean}}$. The total water depth was obtained by summation of the bed level and tide level. Bed profiles were obtained from the CRAB soundings. The bed material varied between $d_{50} = 0.3$ mm near the waterline and $d_{50} = 0.15$ mm at about 350 m offshore.

The Duck datafile also contains the total water depth and the bed level distance to MSL at each measurement location. In the offshore zone where the bed level changes are minimum, the total water depth should be smaller than the bed level distance to MSL during ebb tide and larger during flood tide. The total water depth measured in the offshore zone however does not show this behaviour; most of the times the total water depth is larger than the bed level distance to MSL. For this reason these total water depth values have not been used, but they have been determined from the bed level soundings and the tide levels.
Figure 5.2.13 Measured cross-shore profiles between 21 Sept. and 20 Oct. 1994; longshore-averaged profiles between 800 and 1150 m including standard error bands
**Boundary data**

The boundary conditions and input data of the hydrodynamic runs given in Tables 5.2.4.

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<th></th>
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</tr>
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<tr>
<td>Wave height $H_{a=x=0}$</td>
<td>1.5</td>
<td>1.8</td>
<td>2.3</td>
<td>2.5</td>
<td>2.5</td>
<td>1.9</td>
<td>1.05</td>
</tr>
<tr>
<td>Peak period $T_p$ (s)</td>
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<td>7</td>
<td>7</td>
<td>8</td>
<td>8</td>
<td>7</td>
<td>10</td>
</tr>
<tr>
<td>Wave angle $\theta_{x=0}$ (degrees to normal, $\pm$ from north)</td>
<td>15</td>
<td>20</td>
<td>-20</td>
<td>-10</td>
<td>0</td>
<td>-15</td>
<td>-7</td>
</tr>
<tr>
<td>Water level to MSL (m)</td>
<td>0.2</td>
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<td>0.6</td>
<td>0.4</td>
<td>-0.3</td>
<td>0.1</td>
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<td>5.56</td>
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Table 5.2.4  *Boundary conditions and input data of hydrodynamic runs*
The data of the morphologic runs are given in Table 5.2.5.

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<td>0.00015 and 0.0003</td>
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<td>Factor for high freq. susp. sand transport (-)</td>
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<td>Transport factor</td>
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<td>Temperature (C)</td>
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<td>Salinity (promille)</td>
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<td>Porosity (-)</td>
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<td>Bed roughness (k_{s_{a}}) (m)</td>
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<td>Mixing coefficient (m²/s)</td>
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<td>Time step (s)</td>
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Table 5.2.5  Boundary conditions and input data of morphologic runs
Results of hydrodynamic runs; 11 October, t=7 and 13 hrs; 14 October, t=13 hrs; 15 October, t=1 and 19 hrs

Based on Figures 5.2.14 to 5.2.18, the following results are presented:

- **significant wave height**: the computed values show reasonably good agreement with measured values in the zone seaward of the bar; the computed wave heights show inaccuracies (too large or too small) up to 30% in the bar zone; the computed wave heights generally are too large (up to 20%) in the trough zone landward of the bar;
- **longshore current**: the computed values generally are fairly well represented seaward of the bar; the computed longshore current velocities show a distinct peak just landward of the bar crest, which is in reasonably good agreement with measured values; generally the computed values are too small (up to 50%) in the bar crest zone and in the trough zone landward of the bar;
- **undertow current**: the measured undertow current velocities generally show a strong peak up to -0.7 m/s (local rip current?) in the bar crest zone, which is absent in the computed values; the maximum computed undertow velocity is -0.25 m/s just seaward of the bar crest (wave breaking zone);
- **sand transport**: the computed cross-shore sand transport is offshore-directed in the bar crest zone and onshore-directed in the zones seaward and landward of the bar crest (not shown).

Results of hydrodynamic run; 22 September, t=1 hrs, 26 September, t=13 hrs

Based on Figures 5.2.19 to 5.2.20, the following results are presented:

- **significant wave height**: the computed values based on the standard breaking coefficient shows fairly good results; the computed values are about 20% too large in the bar trough zone (260<x<320 m);
- **longshore current**: the measured values on 22 Sep. are fairly well represented with exception of the offshore zone (x<100 m); the maximum longshore current velocity is about 0.8 m/s; the measured values on 26 Sep. are fairly well represented seaward of the bar; it is noted that the measured longshore current is defined at about 0.5 m above the bed, whereas the computed value represents the depth-averaged longshore current;
- **undertow current**: the measured undertow current on 22 Sep. varies roughly between 0 and -0.2 m/s; the computed undertow current velocities show similar values; the measured undertow current on 26 Sep. varies between 0.1 m/s (onshore-directed) and -0.15 m/s (offshore-directed) seaward of the bar; the computed offshore-directed values are in reasonable agreement with measured offshore-directed values; the measured undertow shows a strong peak up to -0.5 m/s (local rip current?) at the landward flank of the bar, which is absent in the computed values; it is noted that the measured undertow current is defined at about 0.5 m above the bed, whereas the computed value represents the depth-averaged undertow current;
- **sand transport**: the computed cross-shore sand transport on 22 Sep. shows a large offshore-directed peak at the bar crest (200<x<250 m); onshore-directed sand transport occurs on both flanks of the bar (not shown); the computed cross-shore sand transport on 26 Sep. is offshore-directed and rather small (not shown).

A remarkable feature is the relatively strong peak of the measured undertow current in the bar zone for all cases considered. This peak is substantially underestimated by the model. Figure 5.2.21 shows the generation of this peak current for increasing offshore wave heights.
from $H_s = 1.8$ to $2.4$ m on 22 Sept. 1994. At $t=1$ hrs the maximum undertow current is about -0.2 m/s, which has increased to about -0.45 m/s at $t=4$ and 7 hrs. The strong peak flow in the wave breaking zone suggests the generation of a local rip current on the time scale of a few hours. The offshore-directed velocities in the zone seaward of the bar crest remain of the order of -0.1 to -0.2 m/s.

**Figure 5.2.14** Measured and computed wave height, longshore velocity and cross-shore velocity; $t=7$ hrs, 11 Oct. 1994
Figure 5.2.15  Measured and computed wave height, longshore velocity and cross-shore velocity; t = 13 hrs, 11 Oct. 1994
Figure 5.2.16 Measured and computed wave height, longshore velocity and cross-shore velocity; t=13 hrs, 14 Oct. 1994
Figure 5.2.17  Measured and computed wave height, longshore velocity and cross-shore velocity; t= 1 hrs, 15 Oct. 1994
Figure 5.2.18  Measured and computed wave height, longshore velocity and cross-shore velocity; t = 19 hrs, 15 Oct. 1994
Measured data 22 sept. t=1 hrs, 1994
Wave angle=-15 degrees (from south)
Water level to MSL=-0.3 m
Hs,0= 1.9 m; Tp= 7 s
Longsh. current =-0.1 m/s (to north)
Longsh. wind= -5 m/s (to north)

Figure 5.2.19  Measured and computed wave height, longshore velocity and cross-shore velocity; t= 1 hrs, 22 Sept. 1994
Figure 5.2.20  Measured and computed wave height, longshore velocity and cross-shore velocity; $t=13$ hrs, 26 Sept. 1994
Figure 5.2.21  *Generation of cross-shore undertow current, 22 september 1994*
Results of morphologic run; 21-26 September 1994 event
Analysis of the measured bathymetry between transects 750 and 1200 m shows that:

- the bar crest between transects 800 and 1100 m is reasonably straight, but has an oblique orientation with respect to the coastline at 21 Sept. (Figures 5.2.7 and 5.2.12);
- the bar crest between transect 875 and 975 m has moved in onshore direction at 26 Sept.

The profile model CROS MOR2000 has been applied to simulate the bed level changes in the main transect at 945m. The bed roughness is given in Table 5.2.5. Relatively large wave-induced ripples are assumed to be present in the trough zone, having a wave-related bed roughness value of 0.05 m. The bed material is represented by $d_{50}=0.15$ mm and $d_{90}=0.3$ mm.

Preliminary runs have been made to calibrate the transport factor (multiplication factor to standard sand transport) yielding 0.5. The wave breaking coefficient is taken as $\gamma=0.45$. The sand transport model of Van Rijn and that of Bagnold-Bailard have been used to compute the morphologic development of the cross-shore profile. The fall velocity in the B-B model has been set to $w_f=0.017$ m/s. The calibration factor of 0.5 has been applied to both models.

Based on Figures 5.2.22 and 5.2.23, the computed results show the following features:

**Sand transport model of Van Rijn (Figure 5.2.22 and Table 5.2.6)**
- slight onshore migration of the bar, which is considerably smaller than the observed values; the sand transport due to the wave asymmetry and the undertow creates a convergence point just landward of the bar crest resulting in onshore bar migration; the relatively large bed roughness in the trough zone is necessary to generate sufficiently large offshore-directed sand transport by the undertow in the trough zone;
- generation of beach bar (about 4 m$^3$/m) near the mean waterline, which is not observed in the field data.

**Sand transport of Bagnold-Bailard (Figure 5.2.23 and Table 5.2.6)**
- significant erosion of seaward flank of bar, which is considerably larger than the observed erosion; onshore bar migration in agreement with observed values;
- generation of a large swash bar (about 16 m$^3$/m) between -0.5 m and 0.5 m depth contours near the water line, which is not observed in the field data; see also Table 5.2.6;
- about 3 m$^3$/m enters the profile across the -6 m depth contour; see Table 5.2.6.

<table>
<thead>
<tr>
<th>Case</th>
<th>Duration</th>
<th>Sand transport model</th>
<th>Volume ($m^3$; incl. pores) of sand passing -6 m depth contour (seaward boundary, x=0) + = onshore</th>
<th>Volume ($m^3$; incl. pores) of sand passing -0.5 m depth contour + = onshore</th>
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<td>Bagnold</td>
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</table>

**Table 5.2.6** Volumes of sand passing -6 m depth contour and -0.5 m depth contour for September event
Figure 5.2.22 Measured and computed bed levels for 21-26 September 1994 event and schematized offshore significant wave height; based on sand transport model of Van Rijn
Figure 5.2.23  Measured and computed bed levels for 21-26 September 1994 event and schematized offshore significant wave height; based on sand transport model of Bagnold-Bailleard
Results of morphologic run; 10-20 October 1994 event
Analysis of the measured bathymetry between transects 750 and 1200 m (Figures 5.2.9 to 5.2.11) shows a distinct 3D behaviour of the bar system, which can be summarized as:

- bar crest is almost straight at 10 Oct.;
- bar crest has moved in offshore direction between transects 800 and 1050 m (on both sides of the main transect 945 m) at 20 Oct.; the bar crest location has not changed much outside the section 800-1050 m;
- current measurements in the main transect show the presence of a relatively strong offshore-directed current of about 0.6 to 0.8 m/s (rip current) at the location of the bar crest.

The profile model CROSMOR2000 has been applied to the main transect at 945 m. The bed roughness is set to 0.02 m (see Table 5.2.5), assuming sheet flow conditions during the storm event. The bed material is represented by \(d_{so}=0.15\) mm and \(d_{sw}=0.3\) mm.

Preliminary runs have been made to calibrate the transport factor (multiplication factor to standard sand transport) yielding 0.5. The sand transport model of Van Rijn and that of Bagnold-Baillard have been used to compute the morphologic development of the cross-shore profile. The calibration factor of 0.5 has been applied to both models.

Based on Figures 5.2.24 and 5.2.25, the computed results show the following features:

**Sand transport model of Van Rijn (Figure 5.2.24 and Table 5.2.7)**

- **standard model run;** flattening of bar in combination with slight offshore and onshore deposition; the bed profile landward of \(x=300\) m is quite well simulated; about 35 m\(^3\)/m of sand has passed the -6 m depth contour in seaward direction during this storm event (due to undertow; maximum of -0.1 m/s); about 2 m\(^3\)/m of sand has passed the -0.5 m depth contour in seaward direction due to beach erosion, which is realistic for a storm event;

- **model run with modified undertow;** the undertow velocities in the model have been increased (maximum factor 2.5) in the trough zone landward of the bar (between \(x=200\) m and \(x=270\) m) to better match the observed undertow velocities resulting in more bar erosion and the development of a pronounced bar; the computed bar location is somewhat too far seawards (between \(x=100\) and 150 m) compared to the observed bar crest location; the bed profile landward of \(x=300\) m is quite well simulated; about 35 m\(^3\)/m of sand passes the -6 m depth contour in seaward direction during this storm event (due to undertow; maximum of -0.1 m/s); about 3 m\(^3\)/m of sand has passed the -0.5 m depth contour in seaward direction due to beach erosion, which is realistic for a storm event;

**Sand transport model of Bagnold-Baillard (Figure 5.2.25 and Table 5.2.7)**

- **standard model run;** slight flattening of the bar in combination with substantial onshore deposition; about 28 m\(^3\)/m has passed the -0.5 m depth contour in onshore direction to form a large shoal of sand near the water line, which is not realistic for a storm event; about 36 m\(^3\)/m of sand has passed the -6 m depth contour in onshore direction during this storm event (due to the velocity asymmetry effect, which is dominant in the B-B model);

- **model run with modified undertow;** the undertow velocities in the model have been increased (maximum factor 2.5) in the trough zone landward of the bar (between \(x=200\) m and \(x=270\) m), resulting in a large bar just seaward of the initial bar, but the computed bar has not migrated sufficiently far in seaward direction compared with the observed bar; about 26 m\(^3\)/m of sand has passed the -0.5 m depth contour in onshore direction to
form a large shoal of sand near the water line, which is not realistic for a storm event; about 35 m$^3$/m of sand has passed the -6 m depth contour in landward direction during this storm event (due to the velocity asymmetry effect, which is dominant in the B-B model).

Comparing the results of both sand transport models, it can be concluded that the Bagnold-Bailard model yields dominating onshore sand transport due to the velocity asymmetry effect. The effect of offshore sand transport due to wave stirring and advection of sand by the undertow is not properly taken into account.

The modelling results based on the sand transport model of Van Rijn confirm that a rip current is necessary to move the bar crest in offshore direction, because the undertow current at the bar crest based on the standard model has to be increased considerably (factor 2.5) to move the bar in offshore direction.

The present Duck case is a clear example that short-term modelling on the event scale of days to weeks can not be applied on individual transect, because longshore variability generally is so large that bed level changes of individual transects over short periods are not significant in statistical sense. Longshore averaging of individual transects should be applied to eliminate the effects of longshore variability. These effects can be represented by using a longshore-averaged profile and an error band. Bed level changes at two different dates are only significant if there is no overlap of the error bands. The longshore averaging distance should be so large that the longshore rhythmicity including rip channels is fully covered. For example, the width of rip channel at the Duck site is about 200 to 300 m during the 10-20 Oct. 94 event. In practice, this may result in the averaging of 10 to 20 transect with a spacing of 100 m between each transect (1 to 2 km).

<table>
<thead>
<tr>
<th>Case</th>
<th>Duration</th>
<th>Sand transport model</th>
<th>Volume (m$^3$, incl. pores) of sand passing -6 m depth contour (=seaward boundary, x=0) + = onshore</th>
<th>Volume (m$^3$, incl. pores) of sand passing -0.5 m depth contour + = onshore</th>
</tr>
</thead>
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<tr>
<td></td>
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<td>Van Rijn modified undertow</td>
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<td>Bagnold standard</td>
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<tr>
<td></td>
<td></td>
<td>Bagnold modified undertow</td>
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<td>26</td>
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</table>

Table 5.2.7  Volumes of sand passing -6 m depth contour and -0.5 m depth contour for September event
Figure 5.2.24 Measured and computed bed levels for 10-20 October 1994 event and schematized offshore significant wave height; based on sand transport model of Van Rijn
5.2.6 Sediment sorting at Katwijk beach, The Netherlands

Field data
Detailed information of cross-shore grain size variations at the Katwijk-site (about 20 km north of The Haque) along the North Sea coast of The Netherlands has been presented by Terwindt (1962). Terwindt studied grain size variations and the effect of a storm event on the cross-shore particle size distribution.

The results of Terwindt (1962) for the location Katwijk are based on the analysis of samples collected in a summer period under different hydraulic conditions (fairweather and minor storm event; 1 to 2 weeks). The maximum significant wave height outside the surf zone was estimated to be about 3 m during summer storm conditions. The summer storm period has been modelled by assuming offshore significant wave heights between 1 and 3 m during 1 week (see input data below).

The observed cross-shore grain size variations, presented in Figure 5.2.26, show the following features:
Before summer storm period
- relatively coarse material ($d_{50}$ of about 300 micron) in the shallow swash zone near the water line;
- systematic fining of sediment material in seaward direction over the width of the surf zone; seaward of the outer breaker bar the $d_{50}$ has reduced to a value of about 140 micron during periods with calm weather;
- fining of sediment from the swash zone (300 micron) to the dune top (220 micron);
**After summer storm period**

- the sediments in the outer surf zone are found to be somewhat coarser (10% to 20%; \( d_{50} \) is about 180 micron) after a summer storm period; the fraction 105-150 micron shows the greatest variations; during calm periods the fraction 105-150 micron is dominant (50% to 70%) in the bed material; after the storm period the contribution of the 105-150 fraction is reduced to about 20%; thus the finer material is washed out during conditions with higher waves and is most probably transported in suspension to deeper water where it is deposited.

Similar effects have been observed by Stuble and Cialone (1996).

**Model input and results**

The input data are, as follows:

**Wave model**

Significant wave height at depth=10 m

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<tr>
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<td>1 m</td>
</tr>
<tr>
<td>36 hrs (1.5 days)</td>
<td>3 m</td>
</tr>
<tr>
<td>60 hrs (2.5 days)</td>
<td>3 m</td>
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<td>72 hrs (3 days)</td>
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</tr>
<tr>
<td>168 hrs (7 days)</td>
<td>1 m</td>
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</tbody>
</table>

Water depth (x=0 m) \( h_0 = 10 \) m

Wave incidence angle (x=0 m) \( \theta = 10 \) degrees

Longshore tidal velocity (x=0 m) \( v_0 = 0.6; -0.5 \) m/s

Tide levels (x=0 m) \( \Delta h = 0.8; -0.8 \) m

Peak period \( T_p = 8 \) s

Number of wave classes \( NW = 5 \)

Fluid density \( \rho_w = 1030 \) kg/m³

Bottom friction \( k_{w,w} = 0.02 \) m

Water temperature \( T_e = 15 \) degrees C

Salinity \( S_a = 30 \) promille

**Sediment transport model**

Current-related bed roughness \( k_{w,c} = 0.02 \) m

Wave-related bed roughness \( k_{w,w} = 0.02 \) m

Multi fraction method (N=5) \( d_{50} = 0.14 \) to 0.28 mm

\( d_i = 0.1; 0.15; 0.2; 0.3; 0.4 \) mm

Thickness of bed surface layer \( \delta = 0.2 \) m

Factor high-freq. susp. transport \( \gamma = 0.2 \)

Figure 5.2.26 shows the computed distribution of the \( d_{50} \) along the cross-shore profile after 7 days based on the multi fraction method (\( T_1 \) parameter according to method 7; Equation 4.3.3.3g) described in Section 4.3. The initial particle size distribution (model input) is
represented by a variation between 0.14 and 0.28 mm based on the trend of the measured $d_{50}$-values before the summer storm period, as shown in Figure 5.2.26.

The computed results are:

- during the storm event the computed suspended transport is dominant and offshore-directed in the swash zone (670 m < x < 720 m); on the landward flank of the inner bar (560 m < x < 620 m) and at the outer bar (x < 470 m), where the longshore and cross-shore currents are relatively large;
- bed-load transport is dominant and onshore-directed in the troughs landward of the inner and outer bars (470 m < x < 560 m; 620 m < x < 670 m);
- deposition can be observed in the swash zone (650 m < x < 690 m) where a swash bar is generated just seaward of the mean waterline (MSL-line); deposition also occurs around the crest of the inner bar (550 m < x < 580 m) and just landward of the outer bar (390 m < x < 420 m);
- erosion can be observed around the mean waterline (680 m < x < 720 m); in the trough landward of the inner bar (580 m < x < 650 m) and at the crest of the outer bar (330 m < x < 390 m);
- the changes in particle size ($d_{50}$) correspond strongly to the computed deposition and erosion patterns; coarsening of the bed surface does occur in the erosion zones where the finer particles are winnowed from the bed and are transported to the deposition zones resulting in fining of the bed surface; the beach shows a tendency for coarsening of the bed surface;
- the computed coarsening effects in the trough between the inner and outer bar are in reasonably good agreement with the observed pattern; the coarsening effect observed seaward of the outer bar is not represented by the model; the coarsening effect in the trough zone landward of the inner bar and the fining effect seaward of the mean waterline are in reasonable agreement with the observed patterns.
Figure 5.2.26  Effect of storm event on cross-shore particle size distribution, Katwijk-site, The Netherlands

Top: Wave heights and currents

Bottom: Particle size and bed profile
5.3 Conclusions

Sand transport at constant depth

- The total load transport (TRANSPOR2000) in conditions with a sand bed of 0.25 mm and a depth of 5 m strongly depends on the current velocity. Roughly, transport $\propto v^3$ for $H_s = 0$ m; transport $\propto v^{2.5}$ for $H_s = 1$ m and transport $\propto v^2$ for $H_s = 3$ m. Thus, the power of the current velocity decreases for increasing wave height. The total load transport in the main current direction mainly consists of suspended transport; the bed-load transport is negligibly small.

- The TRANSPOR1993 and TRANSPOR2000 models yield about the same transport rates for conditions without waves ($H_s = 0$ m). The TRANSPOR2000 model yields considerably larger (up to factor 5) total load transport rates for combined steady and oscillatory flow (current and wave) conditions compared to the results of the TRANSPOR1993 model, particularly for non-breaking wave conditions ($H_s/h < 0.4$). This is mainly caused by a much stronger effect of the ripples on the effective bed-shear stress and hence on the reference concentration in the TRANSPOR2000 model. For breaking wave conditions with $H_s/h > 0.4$ the TRANSPOR2000 model produces transport rates, which are about 30% to 50% larger than those of the TRANSPOR1993 model. This is mainly caused by a much stronger effect of wave breaking (through a wave breaking coefficient) on the sediment mixing coefficient and hence on the suspended concentrations in the TRANSPOR2000 model.

- For combined steady and oscillatory flow (current and wave) conditions the transport rates according to the BAGNOLD-BAILARD formula are significantly smaller (factor 10 to 15) than those of the TRANSPOR2000 model. The discrepancy increases with increasing wave height, because the wave breaking effect on the suspended transport is not explicitly taken into account by the B-B model.

- In conditions with steady flow (no waves) the BIJKER model yields almost the same current-related transport rates as the TRANSPOR2000 model for current velocities smaller than about 1.2 m/s, and smaller transport rates for current velocities between 1.2 and 2 m/s (maximum factor 3).

- For combined steady and oscillatory flow the transport rates according to the BIJKER model are about the same as those of the TRANSPOR2000 model for current velocities around 1 m/s; the transport rates according to BIJKER are significantly larger (maximum factor 5) than those of the TRANSPOR2000 model for current velocities smaller than about 0.8 m/s and significantly smaller (maximum factor 5) for current velocities larger than about 1.2 m/s.

- The results of the BIJKER model and the TRANSPOR2000 model show good agreement in comparison to the measured data for steady flow without waves.

- The wave-related transport according to the TRANSPOR2000 model is only weakly dependent on longshore current velocity for values smaller than 0.5 m/s. For larger current velocities there is a significant increase of transport with increasing current velocity due to the increase of the suspended sand concentrations. The bed-load transport is not more than about 30% of the total load transport in cross-shore direction.

- The wave-related transport according to the BAGNOLD-BAILARD model is almost independent of longshore current velocity. The results of the TRANSPOR2000 model and the B-B model show reasonably good agreement for current velocities smaller than about 0.5 m/s (conditions with relatively small suspended sand concentrations).
Sand transport along cross-shore coastal profiles

- The TRANSPOR2000 model has been implemented in the CROSMOR2000 model to compute the sand transport distribution and morphology along cross-shore profiles of uniform coasts. The CROSMOR2000-model simulates the propagation, refraction, shoaling and breaking of individual waves (wave by wave approach). Wave height decay after breaking is modelled by using an energy dissipation method. Wave-induced set-up and set-down and breaking-associated cross-shore and longshore currents are also modelled.

- The CROSMOR2000 model has been used to simulate the hydrodynamics and the morphodynamics for two experiments in the large-scale wave tank (LIP1B and 1C). The results show reasonable agreement of measured and computed significant wave height (within 10% error band), undertow (within 30% error band) and near-bed wave velocity asymmetry (within 20% error band). The computed sand transport and short-term bed-level evolution is in reasonable agreement with measured values, provided that the wave-related bed roughness variations along the profile (relatively large values in the trough zone landward of the bar crest) are taken into account.

- The CROSMOR2000 model has been used to simulate the hydrodynamics and the morphodynamics for two events (storm event and fairweather event) at the Duck beach site. The results show reasonable agreement of measured and computed significant wave height (generally within 20% to 30% error band). The measured and computed values of the longshore current generally show fairly good agreement seaward of the bar; the computed longshore current velocities show a distinct peak just landward of the bar crest, which is in fairly good agreement with measured values; generally the computed values are too small (up to 50%) in the bar crest zone and in the trough zone landward of the bar. The measured undertow current velocities generally show a strong peak up to -0.7 m/s (local rip current?) in the bar crest zone, which is absent in the computed values; the maximum computed undertow velocity is -0.25 m/s just seaward of the bar crest (wave breaking zone).

- The computed bed-level evolution for the bar at the Duck site during fairweather conditions shows slight onshore migration of the bar, but considerably smaller than the observed pattern.

- The computed bed-level evolution for the bar at the Duck site during storm conditions shows substantial offshore migration of the bar in reasonable agreement with the observed pattern, provided that the relatively large observed offshore-directed current at the bar crest is properly modelled.

- The Duck case is a clear example that short-term modelling on the event scale of days to weeks can not be applied on individual transects, because longshore variability generally is so large that bed-level changes of individual transects over short periods are not significant in statistical sense. Longshore averaging of individual transects should be applied to eliminate the effects of longshore variability. The longshore averaging distance should be so large that the longshore rithmicity including rip channels is fully covered.

- The CROSMOR2000 model has been used to simulate the cross-shore grain size variations during a storm event at the Katwijk site (about 20 km north of The Hague) along the North Sea coast of The Netherlands. The computed coarsening effects in the trough between the inner and outer bar are in reasonably good agreement with the observed pattern; the coarsening effect observed seaward of the outer bar is not represented by the model; the coarsening effect in the trough landward of the inner bar
and the fining effect seaward of the mean waterline are in reasonable agreement with the observed patterns. The changes in particle size (d$_{50}$) correspond strongly to the computed deposition and erosion patterns; coarsening of the bed surface does occur in the erosion zones where the finer particles are winnowed from the bed and are transported to the deposition zones resulting in fining of the bed surface; the beach shows a tendency for coarsening of the bed surface.

6 Overall conclusions and recommendations

6.1 Overall conclusions

Detailed conclusions are given in Sections 2.3, 3.4, 4.5 and 5.3.

The main findings of the present study can be summarized in the following conclusions.

**Sand transport processes; uniform bed material**
- The net bed-load transport due to oscillatory flow in the sheet flow regime has the same direction as the largest peak orbital velocity near the bed and the net bed-load transport rate is proportional to the power 3.5 of the peak forward (onshore) orbital velocity.
- 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.
- The net bed-load 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 results in a decrease of the transport rate by a factor of 2.
- The effect of water depth (between 1 and 15 m) on the suspended sand transport in steady flow (without waves) is of minor importance. The sand transport is strongly dependent on the current velocity; the sand transport increases from about 0.001 to 10 kg/s/m for velocities increasing from 0.4 to 2.2 m/s. Bed-load transport dominates at low velocities.
- The effect of particle size (between 0.14 and 0.6 mm) on suspended sand transport in steady flow (without waves) is rather strong for low velocities, but reduces gradually for larger velocities (larger than about 1.4 m/s).
- Analysis of the measured sand concentration profiles in combined oscillatory and steady flow shows that:
  - the near-bed concentrations increase with increasing relative wave height (H$_w$/h) and the concentration profile becomes more uniform for increasing relative wave height,
  - the near-bed concentrations (at 0.05 m above the bed) are between 0.2 and 5 kg/m$^3$ for relative wave heights of H$_w$/h=0.2 to 0.9,
  - the concentrations are confined to the near-bed region (z/h<0.1) for non-breaking waves
(H_2/h < 0.3),
- the concentration profile consists of a two-layer system (z/h < 0.1 and >0.1) for relative wave heights (H_2/h) between 0.3 and 0.5,
- the concentration profile is almost uniform for relatively large breaking waves (H_2/h between 0.5 and 1),
- the presence of relatively large current velocities (>0.5 m/s) have a strong effect on the concentration profile; both the near-bed concentrations and the concentrations in the outer layer are enhanced due to increased bed-shear stresses and mixing capacity,
- the presence of bed forms has a strong effect on the concentration profile.
  - The current-related suspended sand transport in the coastal zone is found to be strongly dependent on the relative wave height (H_2/h), particularly for current velocities in the range 0.2 to 0.5 m/s. The suspended transport in the size range 0.18-0.4 mm increases by a factor of 10 to 20 when waves with a relative wave height of about 0.2 are superimposed on a current of about 0.4 m/s. This factor may increase to about 50 for a relative wave height of about 0.33. The increase of the suspended transport due to the wave effect decreases with increasing current velocity, particularly for finer sand (0.14-0.18 mm). The increase of the suspended transport of 0.14-0.18 mm sand is not more than a factor of 2 to 3 when waves with a relative wave height of 0.2 are superimposed on a current of 0.8 m/s. The suspended transport rates show a considerable increase for breaking wave conditions with H_2/h larger than about 0.4.
  - The high-frequency wave-related suspended transport is found to be onshore-directed (in the wave direction) in conditions with irregular waves. This transport component increases with increasing significant wave height, but decreases with decreasing particle size. This latter effect is related to the ripple characteristics; the ripples are more pronounced in conditions with a relatively coarse sand bed (0.3 to 0.5 mm) resulting in stronger vortex motions and associated suspension processes and hence increased wave-related transport rates.

**Sand transport processes; graded bed material**

- 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.
- 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. The critical bed-shear stress for initiation of motion strongly depends on the degree of exposure of a grain with respect to surrounding grains. For steady flow in gravel-bed rivers it has been found that all sizes in a mixture begin to move at nearly the same bed-shear stress (equal mobility concept). For steady flow in graded sand-bed rivers the critical bed-shear stress of the finer fractions may be larger than those of the coarser fractions.
• The critical bed shear stress of the fractions of graded bed material can be reasonably well represented by the critical Shield’s value based on the median particle diameter $d_{50}$ and the hiding-exposure factor of Egiazaroff (1965).
**Sand transport models; uniform bed material (single fraction method)**

- An engineering sand transport model (TRANSPOR2000) has been formulated that can be used for the computation of sand transport in combined steady and oscillatory flow (waves and current), rippled and flat beds, uniform and graded bed materials with particle sizes between 0.1 and 2 mm.

- The calibration and the verification of the TRANSPOR2000 model is based on a large data base (see Appendix E) including about 1700 data sets for steady flow (rivers and estuaries) and about 120 data sets for combined steady and oscillatory flow (coastal seas) in depths up to 15 m and particle sizes between 0.1 and 2 mm.

- The net bed-load transport rate in conditions with combined steady and oscillatory flow over a sand bed can be reasonably well described (within factor 2 to 3) by time-averaging (over the wave period) of the instantaneous transport rates using a quasi-steady bed-load transport formula approach.

- The bed-load transport is mainly affected by the grain roughness. The overall bed-form roughness also has some (weak) influence on the bed-load transport in case of combined steady and oscillatory flow because of its effect on the near-bed velocity profile.

- The effect of particle size on bed-load transport can be reasonably well represented for particle sizes in the range of about 0.15 to 1 mm.

- The wave-related suspended transport component is modelled by an expression based on an instantaneous response of the suspended sediment concentrations and transport to the near-bed orbital velocity. Large-scale wave tank data have been used to calibrate the empirical coefficient involved. This coefficient is found to be constant for all test results considered (two grain sizes 0.16 and 0.33 mm).

- The current-related suspended transport is based on the modelling of the time-averaged velocity profile and the time-averaged sand concentration profile. The time-averaged (over the wave period) advection-diffusion equation is applied to compute the time-averaged sand concentration profile for combined current and wave conditions. Various empirical coefficients have been recalibrated using data from large-scale wave tank experiments. Important parameters for the suspended transport are the current-related and the wave-related bed-form roughness \((k_c, k_w)\). These parameters are directly related to the size and geometry of the bed forms (ripples). At present stage of research both parameters \((k_c, k_w)\) are used as input parameters.

- For conditions with steady flow (no waves) the computed suspended transport rates of sediments in the range of 0.14 to 0.6 mm show reasonably good agreement (within factor 2) with measured values for velocities in the range of 0.7 to 1.8 m/s. The computed values in the particle size range of 0.14 to 0.18 mm may be somewhat too small for velocities in the range of 0.4 to 0.6 m/s. This may be related to the effect of ripple type bed forms, which may be rather pronounced in conditions with fine sand bed of 0.14-0.18 mm. These ripple effects may not be properly taken into account by the model.

- Bed-roughness values in the range of \(k_c=0.03\) to 0.1 m have not much effect on the computed suspended transport rates for conditions with steady flow. The effect of the water depth on the suspended transport is rather significant at low velocities of 0.4 to 0.6 m/s. The suspended transport at a depth of 1 m is a factor 30 to 5 larger than the suspended transport at a depth of 10 m for velocities in the range of 0.4 to 0.6 m/s. This effect reduces to a factor 2 for a current velocity of 1.1 m/s and to a factor 1.1 for a current velocity of 2 m/s. The suspended size has a rather strong effect on the suspended transport, particularly at low current velocities.
• For conditions with combined steady and oscillatory flow (current and waves) the
computed sand transport rates are strongly dependent on the wave-related bed roughness
in the low velocity range of 0.2 to 0.6 m/s, for which the effect of wave-induced mixing
of sediment dominates over turbulence-induced mixing. The effect of bed roughness is
less important in conditions with dominating steady flow. The computed transport rates
are in reasonably good agreement with measured values, provided that the proper bed
roughness value is taken (in the range of 0.01 to 0.05 m). Generally, a bed roughness
value of 0.02 m yields the best results.

**Sand transport models; graded bed material (multi fraction method)**

• The sand transport for graded bed material can be computed by using a multi-fraction
method (MF-method); the sand transport rate of each size fraction of the bed material is
computed 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 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 between 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 bed-load transport and the suspended transport strongly depend on the
dimensionless bed-shear stress parameter. Sand concentration profiles measured in a
small flume for combined oscillatory and steady flow over a fine graded sediment bed
have been used for determination of the proper expression of the dimensionless bed-shear
stress parameter.

• The multi-fraction (MF) method yields somewhat smaller (maximum 50%) bed-load
transport rates than the single fraction (SF) method. The MF-method yields substantially
larger suspended transport rates than the SF-method, varying between a factor 3 for the
lower transport regime without waves to a factor of 1.5 for the upper transport regime
with waves. The increase of the suspended transport rate according to the MF-method is
caused by the relatively large contribution of the finer fractions to the total suspended
transport rate.

• In case of well-mixed bed material the application of a constant grain roughness related
to the dₙ₀ of the mixture seems to be the most logic approach. In case of bed material
consisting of segregated fractions it may be better to relate the grain roughness to the
grain size of the individual fractions of the bed material mixture.

• The suspended sand transport based on the SF-method can be adjusted to that of the MF-
method by using a smaller suspended sediment size dₛ compared to the dₙ₀ of the bed
material. The ratio of dₛ and dₙ₀,bed varies between 0.4 for a weak current over a coarse
graded sediment bed and 0.85 for a strong current over a fine graded sediment bed. The
ratio of dₛ and dₙ₀,bed varies between 0.7 and 1 for conditions with combined current and
waves.

• The application of the MF-method is found to be most appropriate for graded bed
material in conditions with weak currents and relatively low waves (Hₜ/h<0.2), because
of the relatively large contribution of the finer fractions in the transport process
(winnowing effects) resulting in relatively large suspended transport rates (larger than
those of the SF-method).
Model application: Sand transport at constant depth

- The TRANSPOR1993 and TRANSPOR2000 models yield about the same transport rates for conditions without waves (H_i= 0 m). The TRANSPOR2000 model yields considerably larger (up to factor 5) total load transport rates for combined steady and oscillatory flow (current and wave) conditions compared to the results of the TRANSPOR1993 model, particularly for non-breaking wave conditions (H_i/h<0.4). This is mainly caused by a much stronger effect of the ripples on the effective bed-shear stress and hence on the reference concentration in the TRANSPOR2000 model. For breaking wave conditions with H_i/h>0.4 the TRANSPOR2000 model produces transport rates, which are about 30% to 50% larger than those of the TRANSPOR1993 model. This is mainly caused by a much stronger effect of wave breaking (through a wave breaking coefficient) on the sediment mixing coefficient and hence on the suspended concentrations in the TRANSPOR2000 model.

- For combined steady and oscillatory flow (current and wave) conditions the transport rates according to the BAGNOLD-BAILARD formula are significantly smaller (factor 10 to 15) than those of the TRANSPOR2000 model. The discrepancy increases with increasing wave height, because the wave breaking effect on the suspended transport is not explicitly taken into account by the B-B model.

- In conditions with steady flow (no waves) the BIJKER model yields almost the same current-related transport rates as the TRANSPOR2000 model for current velocities smaller than about 1.2 m/s, and smaller transport rates for current velocities between 1.2 and 2 m/s (maximum factor 3).

- For combined steady and oscillatory flow the transport rates according to the BIJKER model are about the same as those of the TRANSPOR2000 model for current velocities around 1 m/s; the transport rates according to BIJKER are significantly larger (maximum factor 5) than those of the TRANSPOR2000 model for current velocities smaller than about 0.8 m/s and significantly smaller (maximum factor 5) for current velocities larger than about 1.2 m/s.

- The wave-related transport according to the TRANSPOR2000 model is only weakly dependent on longshore current velocity for values smaller than 0.5 m/s. For larger current velocities there is a significant increase of transport with increasing current velocity due to the increase of the suspended sand concentrations. The bed-load transport is not more than about 30% of the total load transport in cross-shore direction.

- The wave-related transport according to the BAGNOLD-BAILARD model is almost independent of longshore current velocity. The results of the TRANSPOR2000 model and the B-B model show reasonably good agreement for current velocities smaller than about 0.5 m/s (conditions with relatively small suspended sand concentrations).

Model application: Sand transport along cross-shore coastal profiles

- The TRANSPOR2000 model has been implemented in the CROSMOR2000 model to compute the sand transport distribution and morphology along cross-shore profiles of uniform coasts. The CROSMOR2000-model simulates the propagation, refraction, shoaling and breaking of individual waves (wave by wave approach). Wave height decay after breaking is modelled by using an energy dissipation method. Wave-induced set-up and set-down and breaking-associated cross-shore and longshore currents are also modelled.

- The CROSMOR2000 model has been used to compute the hydrodynamics and morphodynamics for two experiments in the large-scale wave tank (LIP1B and 1C). The
results show reasonable agreement of measured and computed significant wave height (within 10% error band), undertow (within 30% error band) and near-bed wave velocity asymmetry (within 20% error band). The computed sand transport and short-term bed-level evolution is in reasonable agreement with measured values, provided that the wave-related bed-roughness variations along the profile (relatively large values in the trough zone landward of the bar crest) are taken into account.

- The CROSMOR2000 model has been used to compute the hydrodynamics and morphodynamics for two events (storm event and fairweather event) at the Duck beach site. The results show reasonable agreement of measured and computed significant wave height (generally within 20% to 30% error band). The measured and computed values of the longshore current generally show fairly good agreement seaward of the bar; the computed longshore current velocities show a distinct peak just landward of the bar crest, which is in fairly good agreement with measured values; generally the computed values are too small (up to 50%) in the bar crest zone and in the trough zone landward of the bar. The measured undertow current velocities generally show a strong peak up to -0.7 m/s (local rip current?) in the bar crest zone, which is absent in the computed values; the maximum computed undertow velocity is -0.25 m/s just seaward of the bar crest (wave breaking zone).

- The computed bed-level evolution for the bar at the Duck site during fairweather conditions shows slight onshore migration of the bar, but considerably smaller than the observed pattern.

- The computed bed-level evolution for the bar at the Duck site during storm conditions shows substantial offshore migration of the bar in reasonable agreement with the observed pattern, provided that the relatively large observed offshore-directed current at the bar crest is properly modelled.

- The Duck case is a clear example that short-term modelling on the event scale of days to weeks can not be applied on individual transects, because longshore variability generally is so large that bed-level changes of individual transects over short periods are not significant in statistical sense. Longshore averaging of individual transects should be applied to eliminate the effects of longshore variability. The longshore averaging distance should be so large that the longshore rhythmicity including rip channels is fully covered.

- The CROSMOR2000 model has been used to simulate the cross-shore grain size variations during a storm event at the Katwijk site (about 20 km north of The Hague) along the North Sea coast of The Netherlands. The computed coarsening effects in the trough between the inner and outer bar are in reasonably good agreement with the observed pattern; the coarsening effect observed seaward of the outer bar is not represented by the model; the coarsening effect in the trough landward of the inner bar and the fining effect seaward of the mean waterline are in reasonable agreement with the observed patterns. The changes in particle size (d50) correspond strongly to the computed deposition and erosion patterns; coarsening of the bed surface does occur in the erosion zones where the finer particles are winnowed from the bed and are transported to the deposition zones resulting in fining of the bed surface; the beach shows a tendency for coarsening of the bed surface.
6.2 Recommendations

The limitations of the TRANSPOR2000 model are:

- the 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 (fine sand bed);
- the current-related and the wave-related suspended transport rates for combined steady and oscillatory flow are strongly dependent on the wave-related bed-form roughness; the latter parameter, which is an input value of the model, can not be estimated with sufficient accuracy; bed-form and bed-roughness predictors are missing;
- the high-frequency wave-related suspended transport is highly uncertain due to lack of sufficient field data for verification;
- the sand transport by low-frequency wave motion ($T > 20$ s) is not modelled.

The following recommendations are given:

- more field measurements of sand concentrations in the surf zone (bar crest zone) are required to study the wave-related mixing of sediment due to oscillatory flow for bed materials in the range of 0.2 to 1 mm;
- more field measurements of the high-frequency oscillatory suspended transport rates are required to better evaluate this transport component; it is of crucial importance for long-term profile development;
- bed roughness has a strong effect on the suspended transport in conditions with combined steady and oscillatory flow; field measurements of bed-form characteristics and associated effective bed-form roughness (based on analysis of the time-averaged velocity profiles) in the surf zone are required to include these effects in the transport models; measurements of sand transport in field conditions should always include ripple size measurements; the bed-form data should be used to develop bed-form and bed-roughness predictors;
- the bed-form dimensions and associated bed roughness are variable along the cross-shore profile (relatively large values in trough zone); these effects should be better modelled, because they are of crucial importance for bar development;
- field measurements of bed-load transport in the ripple regime during fairweather conditions are required, because this transport process may dominate during these conditions;
- the modified Isobe-Horikawa method is relatively simple and works reasonably well for the LIP tests; the method should be more extensively tested for field conditions, focussing on the effect of the wave period and the bed slope.
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B TRANSPOR-model 1993
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 \( \hat{\mathbf{u}} \) (defined as: \( u_{\text{sec}} + u_{\text{oef}} \)) is derived from deep water wave conditions as follows:

\[
\hat{\mathbf{u}} = 2.0 \cdot r \cdot u_{\text{linear}} \tag{C1}
\]

with:

\[
r_3 = -27.3 \log_{10} \left( \frac{H_0}{L_0} \right) - 16.3 \tag{C2}
\]

\[
r_2 = 1.28 \tag{C3}
\]

\[
r_1 = 1 \tag{C4}
\]

\[
r = r_1 - r_2 \exp \left\{ -r_3 \frac{h}{L_0} \right\} \tag{C5}
\]

\( u_{\text{linear}} \) = 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 \( \hat{\mathbf{u}} \). The \( r \)-factor was found by calibration using laboratory and field data with random waves (see Table C1). This resulted in:

\[
r = 1 - 3.2 \left( \frac{H}{L} \right) ^{0.65} \left( \frac{H}{L} \right) ^{3.4} L ^{3.4} \tag{C6}
\]

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 C1.

<table>
<thead>
<tr>
<th>Description</th>
<th>Testnumber</th>
<th>h (m)</th>
<th>Hₘ₀ (m)</th>
<th>Tₚ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Field: Terschelling, The Netherlands</td>
<td>Fop05330</td>
<td>9.0</td>
<td>1.89</td>
<td>8.0</td>
</tr>
<tr>
<td></td>
<td>Fop17352</td>
<td>9.4</td>
<td>0.37</td>
<td>17.1</td>
</tr>
<tr>
<td></td>
<td>Fop17576</td>
<td>10.5</td>
<td>4.20</td>
<td>10.2</td>
</tr>
<tr>
<td></td>
<td>Fra05330</td>
<td>5.6</td>
<td>1.87</td>
<td>8.0</td>
</tr>
<tr>
<td></td>
<td>Fra17576</td>
<td>7.7</td>
<td>3.42</td>
<td>9.7</td>
</tr>
<tr>
<td></td>
<td>Fre05330</td>
<td>4.1</td>
<td>1.70</td>
<td>8.1</td>
</tr>
<tr>
<td></td>
<td>Fre17352</td>
<td>5.4</td>
<td>0.37</td>
<td>16.5</td>
</tr>
<tr>
<td>Field: Egmond aan Zee, The Netherlands</td>
<td>1B_04430</td>
<td>1.9</td>
<td>0.76</td>
<td>6.1</td>
</tr>
<tr>
<td>Lab: small scale flume</td>
<td>TUDB2_01</td>
<td>0.60</td>
<td>0.18</td>
<td>2.2</td>
</tr>
<tr>
<td>Delft Univ. of Techn.</td>
<td>TUDB2_05</td>
<td>0.31</td>
<td>0.17</td>
<td>2.2</td>
</tr>
<tr>
<td></td>
<td>TUDB2_07</td>
<td>0.51</td>
<td>0.15</td>
<td>2.2</td>
</tr>
<tr>
<td>Field: Muriwai, New Zealand</td>
<td>Muriwai2</td>
<td>1.83</td>
<td>0.92</td>
<td>19.7</td>
</tr>
<tr>
<td>Field: Skallingen, Denmark</td>
<td>Sk304_08</td>
<td>1.8</td>
<td>0.80</td>
<td>11.0</td>
</tr>
<tr>
<td></td>
<td>Sk310_01</td>
<td>2.6</td>
<td>1.49</td>
<td>8.8</td>
</tr>
<tr>
<td>Lab: Delta Flume Lip11D</td>
<td>1A0203_02</td>
<td>2.31</td>
<td>0.92</td>
<td>5.0</td>
</tr>
<tr>
<td>WL Delft Hydraulics</td>
<td>1A0203_07</td>
<td>0.91</td>
<td>0.62</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>1B0213_02</td>
<td>2.30</td>
<td>1.19</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>1B0213_07</td>
<td>0.89</td>
<td>0.57</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>1C0204_02</td>
<td>2.25</td>
<td>0.63</td>
<td>8.0</td>
</tr>
<tr>
<td></td>
<td>1C0204_03</td>
<td>1.77</td>
<td>0.63</td>
<td>8.0</td>
</tr>
<tr>
<td></td>
<td>1C0204_05</td>
<td>1.16</td>
<td>0.63</td>
<td>8.0</td>
</tr>
<tr>
<td></td>
<td>1C0204_11</td>
<td>1.59</td>
<td>0.63</td>
<td>8.0</td>
</tr>
</tbody>
</table>

Table C1. 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, Tₚ. 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 ũ is presented in Figure C1. The broken lines indicate a 20% error band.
The following formulae, Eq.(C7)-Eq.(C14), were derived to account for the asymmetry of the velocity profile (Isobe and Horikawa, 1982). Eq.(C7)-Eq.(C12) is a parameterisation of fifth-order Stokes wave theory and third-order cnoidal wave theory. Eq.(C13) and Eq.(C14) were introduced to take into account the deformation of the velocity profile due to bottom slope.

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

(C7)

with:

\[
\lambda_1 = 0.5 - \lambda_3
\]

(C8)

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

(C9)

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

(C10)

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

(C11)
\[
\lambda_5 = \begin{cases} 
0.0032 \left( \frac{T \sqrt{g}}{h} \right)^2 + 0.000080 \left( \frac{T \sqrt{g}}{h} \right)^3, & T \frac{g}{\sqrt{h}} \leq 30 \\
0.0056 \left( \frac{T \sqrt{g}}{h} \right)^2 - 0.000040 \left( \frac{T \sqrt{g}}{h} \right)^3, & T \frac{g}{\sqrt{h}} > 30 
\end{cases} 
\]  
(C12)

\[
\left( \frac{u_{on}}{\hat{u}} \right)_{\text{modified}} = 0.5 + \left( \left( \frac{u_{on}}{\hat{u}} \right)_{\text{max}} - 0.5 \right) \tanh \left( \frac{\left( \frac{u_{on}}{\hat{u}} \right)_{a} - 0.5}{\left( \frac{u_{on}}{\hat{u}} \right)_{\text{max}} - 0.5} \right) 
\]  
(C13)

with:

\[
\left( \frac{u_{on}}{\hat{u}} \right)_{\text{max}} = 0.62 + \frac{0.003}{\text{bed slope}} 
\]  
(C14)

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:

\[
\left( \frac{u_{on}}{\hat{u}} \right)_{\text{max}} = 0.62 + \frac{0.001}{\text{bed slope}} 
\]  
(C15)

The offshore-directed peak orbital velocity follows from: \( u_{\text{off}} = \hat{u} - u_{\text{on}} \)

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).
D Velocity profile of undertow current
The undertow velocity profile is schematized into the following three layers.

**Near-bed layer** \((z_{o}<z<\delta_{m})\)

Velocity profile:  
\[
u_{z,z} = \alpha_{e} \left[ u_{\partial} \left( -1 + \ln \frac{\ln(z_{o})}{\ln(z_{a})} \right) \frac{\ln(z)}{\ln(z_{a})} \right] \quad (D1)
\]

with:
\[
z_{a} = k_{s,w}/30 \quad \text{= zero-velocity level,}
\]
\[
\delta_{m} = 0.216 A_{b} (A_{b}/k_{s,w})^{0.25} \quad \text{= thickness of mixing layer near bed}
\]
\[
u_{z,z} = u_{\partial} \left( -1 + \ln(30h/k_{a}) \right) \frac{\ln(30\delta_{m}/k_{a})}{\ln(z_{a})} \quad \text{= velocity at } z = \delta_{m}
\]
\[
A_{b} \quad \text{= amplitude of near-bed orbital excursion}
\]
\[
k_{a} \quad \text{= apparent bed roughness}
\]
\[
k_{s,w}, k_{s,w} \quad \text{= current and wave-related bed-roughness}
\]
\[
u_{a} \quad \text{= depth-averaged undertow vcl. (input)}
\]
\[
\alpha_{e} \quad \text{= correction factor}
\]

**Intermediate layer** \((\delta_{m}<z\leq 0.5h)\)

Velocity profile:  
\[
u_{z,z} = \alpha_{e} \left[ u_{\partial} \left( -1 + \ln \frac{h/z_{a}}{h/z_{a}} \right) \frac{\ln(z)}{\ln(z_{a})} \right] \quad (D2)
\]

with:
\[
z_{a} = k_{o}/30
\]
\[
h = \text{water depth}
\]

**Upper layer** \((0.5h<z\leq h)\)

Velocity profile:  
\[
u_{z,z} = u_{z,\text{mid}} \left[ 1 - \left( (z-0.5h)/(0.5h) \right)^{3} \right] \quad (D3)
\]

with:
\[
u_{z,\text{mid}} = \alpha_{e} \left[ u_{\partial} \left( -1 + \ln \frac{h}{z_{a}} \right) \right] \frac{\ln(0.5h/z_{a})}{\ln(z_{a})} \quad \text{= velocity at } z = 0.5h
\]

Equation D3 yields:  
\[u_{z,z} = u_{z,\text{mid}} \text{ for } z=0.5h \text{ and } u_{z,z}=0 \text{ for } z=h.
\]

The correction factor \(\alpha_{e}\) can be determined from:  
\[
q_{1} + q_{2} + q_{3} = u_{h}. \quad (D4)
\]

The \(q\)-values are:
\( q_1 = x_0 \beta u_{r,z} \, dz \geq 0 \)

\( q_2 = q_{0.5h}^{0.5h} u_{r,z} \, dz = \alpha_2 \left[ u_r / (-1 + \ln(h/z_a)) \right] \left[ (\delta_m - 0.5h) + 0.5h \ln(0.5h/z_a) - \delta_m \ln(\delta_m/z_a) \right] \)

\( q_3 = 0.5h \beta u_{r,z} \, dz = 0.375h \, u_{r,mid} \)

This yields:

\[ \alpha_r = C_1 / (C_1 + 0.375 \, C_2) \]  \hspace{1cm} (D5)

with:

\[ C_1 = -1 + \ln(h/z_a) \]

\[ C_2 = \ln (0.5h/z_a) \]

\[ C_3 = [(\delta_m h - 0.5) + 0.5 \ln(0.5h/z_a) - (\delta_m / h) \ln(\delta_m/z_a)] \equiv -0.5 + 0.5 \ln(0.5h/z_a) \]

**Example case**

<table>
<thead>
<tr>
<th>Input</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>( h )</td>
<td>5 m</td>
</tr>
<tr>
<td>( H_s )</td>
<td>2 m</td>
</tr>
<tr>
<td>( T_p )</td>
<td>7 s</td>
</tr>
<tr>
<td>( v_{long} )</td>
<td>1 m/s</td>
</tr>
<tr>
<td>( u_r )</td>
<td>-1 m/s</td>
</tr>
<tr>
<td>( \Phi_i )</td>
<td>90°</td>
</tr>
<tr>
<td>( k_s = k_{s,w} )</td>
<td>0.02 m</td>
</tr>
<tr>
<td>( \delta_m )</td>
<td>0.1 m</td>
</tr>
<tr>
<td>( k_a )</td>
<td>0.08 m</td>
</tr>
<tr>
<td>( z_a )</td>
<td>0.00267 m</td>
</tr>
</tbody>
</table>

This yields: \( \alpha_r = 1.19 \)

The undertow velocity profile is shown below and for reference a logarithmic velocity profile over the full depth is also shown. In both cases: \( u_r = 1 \) m/s.
E Summary of data used in calibration and verification of TRANSPOR 2000 model
A summary of all experimental data used for calibration and verification (time-averaged sand concentration, bed-load transport and suspended transport) of the TRANSPOR-model (1993, 2000) is given in the following tables.

**DATA SETS (1702) FOR CONDITIONS WITH CURRENTS ONLY**

<table>
<thead>
<tr>
<th>N</th>
<th>Sand</th>
<th>h (m)</th>
<th>V (m/s)</th>
<th>B.F.</th>
<th>Environment</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>dₜₐ (mm)</td>
<td>U/G</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>44</td>
<td>0.5-1</td>
<td>U</td>
<td>0.1-1</td>
<td>0.5-0.8</td>
<td>n.m.</td>
<td>BLT, rivers</td>
</tr>
<tr>
<td>12</td>
<td>1-2</td>
<td>U</td>
<td>0.2-0.7</td>
<td>0.6-0.9</td>
<td>n.m.</td>
<td>BLT, rivers</td>
</tr>
<tr>
<td>154</td>
<td>0.05-0.5</td>
<td>U</td>
<td>0.1-0.5</td>
<td>0.3-1.3</td>
<td>R/D</td>
<td>BLT, small flume</td>
</tr>
<tr>
<td>253</td>
<td>0.5-1</td>
<td>U</td>
<td>0.1-0.5</td>
<td>0.3-1.1</td>
<td>D/R</td>
<td>BLT, small flume</td>
</tr>
<tr>
<td>99</td>
<td>1-2</td>
<td>U</td>
<td>0.1-0.2</td>
<td>0.4-1</td>
<td>D/R</td>
<td>BLT, small flume</td>
</tr>
<tr>
<td>486</td>
<td>0.05-0.5</td>
<td>U</td>
<td>0.3-3.6</td>
<td>0.4-2.4</td>
<td>n.m.</td>
<td>SLT, rivers</td>
</tr>
<tr>
<td>297</td>
<td>0.05-0.5</td>
<td>U</td>
<td>0.1-0.4</td>
<td>0.4-1.3</td>
<td>R</td>
<td>SLT, small flume</td>
</tr>
<tr>
<td>32</td>
<td>0.24-0.6</td>
<td>G</td>
<td>2.8-6</td>
<td>0.3-0.9</td>
<td>D</td>
<td>BLT, Nile river, Egypt</td>
</tr>
<tr>
<td>3</td>
<td>0.5-1</td>
<td>G</td>
<td>0.5-1.5</td>
<td>0.5-1</td>
<td>n.m.</td>
<td>SC, Enoree river USA</td>
</tr>
<tr>
<td>120</td>
<td>0.2-0.35</td>
<td>U</td>
<td>3-11</td>
<td>1.2-2.1</td>
<td>D/R</td>
<td>SLT, Eastern and Western Scheldt Estuary</td>
</tr>
<tr>
<td>119</td>
<td>0.1-0.3</td>
<td>U</td>
<td>1-25</td>
<td>1.5-2.5</td>
<td>n.m.</td>
<td>SLT, USA-rivers</td>
</tr>
<tr>
<td>5</td>
<td>0.18-0.33</td>
<td>U</td>
<td>1-2</td>
<td>1.2-2</td>
<td>n.m.</td>
<td>SLT, Rio Grande, USA</td>
</tr>
<tr>
<td>6</td>
<td>0.4-0.5</td>
<td>U</td>
<td>4-6</td>
<td>0.8-0.9</td>
<td>D/R</td>
<td>BLT, Nile River, Egypt</td>
</tr>
<tr>
<td>12</td>
<td>0.5-1</td>
<td>G</td>
<td>4-8</td>
<td>0.4-1.3</td>
<td>D</td>
<td>BLT, Rhine River, The Netherlands</td>
</tr>
<tr>
<td>10</td>
<td>2.5-3</td>
<td>G</td>
<td>9-11</td>
<td>1.4-2</td>
<td>D</td>
<td>BLT, SLT, Rhine River, The Netherlands</td>
</tr>
<tr>
<td>10</td>
<td>0.25-0.31</td>
<td>U</td>
<td>4-5</td>
<td>0.6-0.8</td>
<td>D/R</td>
<td>SLT, Nile River, Egypt</td>
</tr>
<tr>
<td>5</td>
<td>0.27-0.36</td>
<td>U</td>
<td>4.7-5.5</td>
<td>0.7</td>
<td>D/R</td>
<td>SLT, Mississippi River, USA</td>
</tr>
<tr>
<td>6</td>
<td>0.12-0.18</td>
<td>U</td>
<td>14-16</td>
<td>0.3-1.2</td>
<td>R</td>
<td>SLT, Foulness, UK</td>
</tr>
<tr>
<td>24</td>
<td>0.11-0.17</td>
<td>U</td>
<td>4-6</td>
<td>0.3-0.7</td>
<td>R</td>
<td>SLT, Maplin Sands, UK</td>
</tr>
<tr>
<td>5</td>
<td>0.15</td>
<td>U</td>
<td>2-4</td>
<td>0.2-0.6</td>
<td>R</td>
<td>SLT, Galgeplaat, tidal flat, Eastern Scheldt Estuary, NL</td>
</tr>
</tbody>
</table>
### DATA SETS (121) FOR CONDITIONS WITH COMBINED CURRENT AND WAVES

<table>
<thead>
<tr>
<th>N</th>
<th>Sand</th>
<th>h</th>
<th>U&lt;sub&gt;s&lt;/sub&gt;</th>
<th>U&lt;sub&gt;max&lt;/sub&gt;</th>
<th>V</th>
<th>φ</th>
<th>B.F.</th>
<th>Environment</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>d&lt;sub&gt;s&lt;/sub&gt;</td>
<td>U/G</td>
<td>(m)</td>
<td>(m/s)</td>
<td>(m/s)</td>
<td>(deg)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>0.1-0.2</td>
<td>U</td>
<td>0.5</td>
<td>0.1-0.2</td>
<td>n.m.</td>
<td>0.1-0.4</td>
<td>0, 180</td>
<td>R</td>
<td>SLT, small scale flume</td>
</tr>
<tr>
<td>28</td>
<td>0.1</td>
<td>U</td>
<td>0.4</td>
<td>0.1-0.15</td>
<td>n.m.</td>
<td>0.1-0.3</td>
<td>60-120</td>
<td>R</td>
<td>SLT, small scale basin</td>
</tr>
<tr>
<td>4</td>
<td>0.15</td>
<td>U</td>
<td>2-4</td>
<td>1-2</td>
<td>n.m.</td>
<td>0-0.5</td>
<td>60-120</td>
<td>n.m.</td>
<td>SC, Duck surf zone, USA</td>
</tr>
<tr>
<td>10</td>
<td>0.3-0.4</td>
<td>U</td>
<td>1-1.6</td>
<td>0.2-0.9</td>
<td>n.m.</td>
<td>0.1-0.6</td>
<td>90</td>
<td>n.m.</td>
<td>SLT, SC, surf zone, Egmond, North Sea</td>
</tr>
<tr>
<td>11</td>
<td>0.16-0.4</td>
<td>U</td>
<td>1.1-4.5</td>
<td>0.2-1.5</td>
<td>n.m.</td>
<td>0</td>
<td>0</td>
<td>R/F</td>
<td>SC, large scale Wave Flume of DH</td>
</tr>
<tr>
<td>6</td>
<td>0.2</td>
<td>U</td>
<td>-</td>
<td>-</td>
<td>1-1.7, R</td>
<td>0</td>
<td>0</td>
<td>F</td>
<td>BLT, large scale Wave Tunnel of DH</td>
</tr>
<tr>
<td>6</td>
<td>0.2</td>
<td>U</td>
<td>-</td>
<td>-</td>
<td>0.9-1.5, R</td>
<td>0.1-0.4</td>
<td>0, 180</td>
<td>F</td>
<td>BLT, large scale Wave Tunnel of DH</td>
</tr>
<tr>
<td>3</td>
<td>0.24</td>
<td>G</td>
<td>-</td>
<td>-</td>
<td>1-1.6, R</td>
<td>0</td>
<td>0</td>
<td>F</td>
<td>BLT, large scale Wave Tunnel of DH</td>
</tr>
<tr>
<td>2</td>
<td>0.2</td>
<td>U</td>
<td>1-2</td>
<td>0.6-1.2</td>
<td>n.m.</td>
<td>-</td>
<td>0</td>
<td>R/F</td>
<td>TLT, large scale Wave Flume LIP1B/1C, surf zone</td>
</tr>
<tr>
<td>6</td>
<td>0.11-0.17</td>
<td>U</td>
<td>3-5</td>
<td>0.5-1</td>
<td>n.m.</td>
<td>0.4-0.8</td>
<td>0-30 150-180</td>
<td>R</td>
<td>SLT, Maplin Sands, UK</td>
</tr>
<tr>
<td>14</td>
<td>0.18-0.34</td>
<td>U</td>
<td>3-6</td>
<td>0.5-1</td>
<td>n.m.</td>
<td>0.2-0.4</td>
<td>150-180</td>
<td>R</td>
<td>Boscombe Bay, UK</td>
</tr>
<tr>
<td>2</td>
<td>0.15</td>
<td>U</td>
<td>2-4</td>
<td>0.7</td>
<td>n.m.</td>
<td>0.3-0.7</td>
<td>n.m.</td>
<td>R</td>
<td>Galgeplaat, Eastern Scheldt, Netherlands</td>
</tr>
<tr>
<td>6</td>
<td>0.2-0.3</td>
<td>U</td>
<td>1-5</td>
<td>0.2-1.5</td>
<td>n.m.</td>
<td>0.1-0.4</td>
<td>60-120</td>
<td>R/F</td>
<td>SLT, SC, surf zone, Egmond, North Sea</td>
</tr>
<tr>
<td>1</td>
<td>0.1</td>
<td>U</td>
<td>13</td>
<td>3.7</td>
<td>n.m.</td>
<td>0.6</td>
<td>80</td>
<td>F</td>
<td>SC, Duck Shelf, USA</td>
</tr>
</tbody>
</table>

- **N** = number data sets
- **U** = uniform sand, **G** = graded sand
- **h** = water depth; **H<sub>s</sub>** = significant wave height
- **U<sub>max</sub>** = peak near-bed orbital velocity near bed; **R** = regular waves, **IR** = irregular waves
- **V** = depth-averaged current velocity
- **φ** = angle between waves and current
- **B.F.** = bed forms (R=ripples, D=dunes and F= flat bed)
- **Envir.** = river, estuary, coastal sea
- **BLT** = bed load transport, **SLT**= suspended transport, **TLT**= total load transport,
- **SC** = sand concentration
- **Ref.** = reference of paper or report in which calibration/verification is described
- **n.m.** = not measured